MQP LDA-1801

WPI Pedestrian Bridge and Athletic Building: Structural Design

Major Qualifying Project Report

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Presented to:

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Abstract

Currently, WPI does not fully utilize its property at the location of the A.J. Knight Field and tennis courts due to safety and accessibility concerns from the crossing of MA Route 122A. An indoor athletic and training facility was designed to fully utilize this property and expand the current athletic facilities. An enclosed pedestrian bridge was designed to increase foot traffic and accessibility to this area of campus by connecting it to the current Sports and Recreation Center. The scope of this project included architectural planning, site design, structural design, and cost analysis, and computer modelling for both structures.

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Capstone Design Statement

In order to successfully complete the requirements established by the Accreditation Board for Engineering and Technology (ABET) for the Capstone Design Project, the project team must properly consider a number of realistic constraints. This section will detail how the work done to complete this project addressed these constraints. Due to the complexity of the project, the majority of these constraints were considered without needing to make special considerations.

Constructability

It is important to consider constructability throughout the entire design of the project. If a structure works on paper but cannot be properly put together, then the design is inadequate. When considering the various loadings present throughout the life of the structure, it is important to factor in the construction loads (shoring, scaffolding, etc.). Our team made sure to always consult the Massachusetts State Building Code 8th Edition regarding various construction safety factors to ensure the safety of those working to build the facility, including but not limited to building zoning, regulations, design aspects, and structural analysis. Standard steel sections were taken from The American Institute of Steel Construction and observed dimensional standards for concrete construction were taken from The American Concrete Institute. Given the location of the project spanning over State Highway 122A, the process of constructing the bridge without obstructing traffic was taken into account with the designs.

Social

The social impact of any project determines its ultimate success. The new facility will alter the landscape of Worcester Polytechnic Institute (WPI). The athletic facility and bridge will increase the amount of recreational space for members of the WPI community by freeing up space in the Sports and Recreation Center, while providing varsity athletes with the space they need to train and improve their skills. Improving the quality of varsity athletics will be a source of pride for the Institution and providing more opportunities for health and fitness of the community will be beneficial for all at WPI.

Economic

In order to evaluate all economic constraints, a material and labor cost estimate was prepared. Given that this project will be funded by a private institution, every aspect from design through construction to operation and maintenance was evaluated. Costs were a major deciding factor throughout the design of both structures.

Health and Safety

Health and safety should be considered for all phases of a project's life, in this case both construction and occupancy were considered. By designing in accordance with Massachusetts State Building Code 8th Edition, *MassDOT LRFD Bridge Manual,* the Americans with Disabilities Act (ADA), the American Society of Civil Engineers (ASCE), and the AASHTO LRFD Bridge Design Specifications, the team ensured the safety of the construction process, the structure, and its occupants. However, the safety of the facility's occupants is also critical. By following the standards set in place by the ADA, the structures will be safe and accessible to all occupants. The team also made use of the Load and Resistance Factor Design Specifications for both the proposed building and pedestrian bridge when determining the loads and load factors. The location of both the structures subjects them to various environmental factors such as snow, wind, and earthquake loads. The designs of both structures were completed to safely account for these factors and the various usages of each space.

Environmental

Design decisions will be made with consideration to the impacts they could potentially have on the local environment. Excavation will be required for the construction of both the athletic facility and pedestrian bridge. Throughout this process it will be important to mitigate the inflow of any hazardous materials to the exposed soils. With the addition of the proposed athletic facility, the local terrain will be altered reducing the amount of impervious surface area available for water runoff to percolate and be absorbed. Consequently, the drainage and storm water collection systems on site will need to be reevaluated to ensure runoff is controlled.

By excavating soil and rock from the earth below the site, the construction process is disturbing the normally hidden and contained material. Once the excavation has begun, the soil and rock removed from the ground must be tested for environmental concerns such as toxins or pollutants. The result of these test will determine where and how the soil can be stored, transported, and reused. If the soil removed from the site does contain pollutants and it is not handled properly, it could have negative consequences on the environment. To reduce these risks, design decisions were made throughout the project, such as using shallow footings instead of a basement, to minimize the amount of excavation needed during construction.

Sustainability

Developing sustainable civil infrastructure and structures is critical to the success of a project. If structures do not properly stand the test of time, the owner will be liable for high maintenance and repair costs. In addition to their durability, new construction should also be built using sustainable materials and techniques. These materials are beneficial to the environment and allow for reductions in the aforementioned life-cycle costs.

To ensure that the proposed athletic building and pedestrian bridge are built sustainably, the design of the structures must be efficient. If the structure can be designed using less material, it will be inherently more environmentally friendly and sustainable. The structures of the building and bridge were designed to use the least amount of material and weight while also minimizing cost. Truss systems were used multiple times during the design of the project to take advantage of the high strength to weight ratio of truss systems. By using less steel to carry the same load, the structure is more sustainable.

Ethics

Considering the ethics behind each decision is vital because lives are always in consideration for all structural designs. In the design of both the proposed athletic building and the pedestrian bridge, the principles from The American Society of Civil Engineers (ASCE) were upheld. All risks and dangers involved in an infrastructure were considered and discussed, especially since both structures are intended to be used each day by students, faculty, and staff.

Professional Licensure Statement

Professional licensure in the state of Massachusetts allows individuals the freedom to consult and certify civil engineering documents. This certification is critical for those striving for upper-level engineering positions and increases their value to the companies that employ them. Professional Engineers (PE) are a vital piece of any successful engineering firm and are greatly responsible for designing the way society will interact with the infrastructure around them in the future.

A large portion of the importance of these individuals stems from the difficulty in obtaining their titles. In order to obtain this licensure, candidates must first graduate from a four-year college engineering program accredited by ABET. Following graduation, individuals must successfully pass the Fundamentals of Engineering Exam. This online examination is administered over a 355-minute period and spans across all areas of knowledge necessary to become an Engineer-in-Training (EIT). Topics include general mathematics, environmental engineering, structural analysis, engineering economics, and professional ethics. After passing the exam, EITs must complete four years of work under the supervision of a PE. Lastly, one must apply to sit for and pass the Principles and Practice in Engineering Exam, the composition of which varies from state to state.

Obtaining this licensure is a major milestone in the career of any engineer. It takes years of hard work and dedication to the profession. However, one’s work is not done following accreditation as PE’s are responsible for the safety of not only their designs, but the lives of those who interact with their designs as well. It also holds those with licensure accountable for the work performed by their subordinates.

It is important to note that multiple Professional Engineers from different disciplines, including civil engineering, electrical engineering, and mechanical engineering, would be required to successfully complete the project detailed within this report. Both the multi-story athletic facility and the pedestrian bridge spanning over a major state route have the potential to negatively affect the safety of those who interact with them if mistakes were to be made. For this reason, PE’s would be needed to approve the structural calculations and designs and ensure their safety.

Table of Contents

Abstract i
Authorship ii
Capstone Design Statement iii
Professional Licensure Statement vi
List of Figures xi
List of Tables xiii
1.0 Introduction 1
2.0 Background 3
2.1 Assessing the Need 3
2.2 Community Impact 4
2.2.1 Massachusetts Zoning Districts 4
2.2.2 Impact on the WPI Community 5
2.2.3 Impact on the Greater Worcester Community 5
2.2.4 Economics 5
2.3 Common Construction Materials 7
2.3.1 Concrete 7
2.3.2 Steel 8
2.3.3 Timber 9
2.3.4 Composites 10
2.4 Bridge and Building Systems 10
2.4.1 Suspension Bridges 10
2.4.2 Truss Bridges 11
2.4.3 Arch Bridges 12
2.5 Engineering Design Parameters 13
2.5.1 AASHTO Design Criteria 14
2.5.2 ADA Design Criteria 14
2.5.3 Massachusetts Building Code Design Criteria 17
2.5.4 International Code Council Wind and Seismic Design Code Master 17
2.5.5 LRFD Design Specifications 18
2.5.6 Ethics 19
2.6 Sustainability 19
2.7 Design Tools 21
2.7.1 RISA 2D 21
2.7.2 AutoDesk Revit 21
2.7.3 AutoDesk AutoCAD 21

2.7.4 Microsoft Excel 22
3.0 Methodology 23
3.1 Preliminary Design 23
3.1.1 Survey 23
3.1.2 Interview 24
3.1.3 Sources for Design and Calculations 24
3.2 Design of Athletic Facility 25
3.2.1 Building Roof System Analysis 25
3.2.2 Building Second Floor Design and Analysis 26
3.2.3 Building Column Design and Analysis 28
3.2.4 Building Wind and Seismic Analysis 29
3.2.5 Building Footing Design and Analysis 31
3.2.6 Building Elevator Hoist Beam Design 32
3.3 Design of Pedestrian Bridge 32
3.3.1 Bridge Through-Truss Analysis 33
3.3.2 Bridge Lateral Force Analysis 33
3.3.3 Bridge Roof Frame Analysis 33
3.3.4 Bridge Piers 34
3.3.5 Bridge Pier Caps 34
3.3.6 Bridge Elastomeric Bearings 34
3.3.7 Bridge Footings 35
3.4 3D Modeling and Cost Analysis 35
3.4.1 Building, Bridge, Merging 36
3.4.2 Renderings 36
3.4.3 Cost Analysis 36
4.0 Design Strategy 39
4.1 Current Site Details and Limitations 39
4.2 Athletic Facility Architectural Program and Design Strategy 41
4.3 Pedestrian Bridge Architectural Program and Design Strategy 43
4.4 Bridge Connection Design Strategy 44
5.0 Design of Athletic Facility Building 45
5.1 Design of Building Roof Truss 45
5.2 Design of Building Second Floor 48
5.3 Design of Building Columns 50
5.4 Design of Building Lateral Reinforcement System 51
5.5 Design of Building Footings 53

5.6 Design of Elevator Hoist Beam 55
6.0 Design of Pedestrian Bridge 56
6.1 Design of the Bridge Through-Truss 56
6.2 Design of the Bridge Roof Frame 58
6.3 Design of Bridge Lateral Truss 59
6.4 Design of the Bridge Pier Cap 60
6.5 Design of the Bridge Piers 62
6.6 Design of the Bridge Footings 65
6.7 Design of Bridge Earthquake Resisting System 68
6.8 Design of Bridge End Pier 69
7.0 Cost Analysis 71
8.0 Conclusion and Recommendations 72
8.1 Athletic Building Conclusions 72
8.2 Athletic Building Recommendations 73
8.3 Pedestrian Bridge Conclusions 73
8.4 Pedestrian Bridge Recommendations 74
9.0 Bibliography 76
Appendix A: Project Proposal 80
Appendix B: Site Maps and Images 101
Appendix C: Survey Data and Analysis 104
C.1 Site Cross-Section Diagram from Site Survey Data 104
C.2 Site Survey Raw Data 104
Appendix D: Design Specification Sheets 107
D.1 Building 2nd Floor Deck Spacing 108
Appendix E: Building Calculations 110
E.1 Roof Truss Calculations 111
E.2 Roof Beam/Girder Calculations 115
E.3 Building 2nd Floor Calculations 121
E.4 Building 2nd Floor Slab Calculations 128
E.5 Building Column Calculations 130
E.6 Building Lateral Reinforcement Calculations 133
E.7 Building Seismic Calculations 136
E.8 Building Footing Calculations 139
E.9 Elevator Hoist Beam Calculations 143
Appendix F: Pedestrian Bridge Calculations 145
F.1 Bridge Through-Truss Calculations 146

F.2 Bridge Roof Frame Calculations 155
F.3 Bridge Wind Distribution Calculations 157
F.4 Bridge Seismic Calculations 158
F.5 Bridge Elastomeric Bearing Calculations 161
F.6 Bridge Pier Calculations 164
F.7 Bridge Pier Cap Calculations 167
F.8 Bridge Footing Calculations 172
F.9 Bridge Final Pier Calculations 176
Appendix G: Cost Analysis 178
Appendix H: Connection Geometry Check 180

List of Figures

Figure 1: Suspension bridge diagram (Lamb, 2000) 11
Figure 2: Truss bridge diagram (Robb, 2016) 12
Figure 3: Arch bridge diagram (Shirley-Smith, 2017) 13
Figure 4: ADA Wheelchair Clearance Requirements (Department of Justice, 2010) 15
Figure 5: ADA Door Clearance Requirements (Department of Justice, 2010) 15
Figure 6: Handrail Clearance with Adjacent Surfaces (Department of Justice, 2010) 16
Figure 7: Handrail Height Clearance (Department of Justice, 2010) 16
Figure 8: Procedure Used to Design a Beam and Girder System. 25
Figure 9: Procedure Used to Design the Building Roof Truss System. 26
Figure 10: Procedure Used to Design the Building Second Floor. 27
Figure 11: Procedure Used to Design the Building Second Floor Solid Reinforced Concrete Slab. 27
Figure 12: Procedure Used to Design the Building Columns. 28
Figure 13: Procedure Used to Design Building Lateral Load System. 30
Figure 14: Procedure Used to Determine Wind Design Method. 30
Figure 15: Procedure used to Determine Seismic Loading on Building. 31
Figure 16: Procedure Used to Design Building Column Footings. 32
Figure 17: Procedure Used to Design the Bridge Footings. 35
Figure 18: Typical design fees for construction projects in Massachusetts (DCAM, 2015) 38
Figure 19: Aerial View of the Proposed Site. 39
Figure 20: Cross-Section of Current Site Ground Conditions. 40
Figure 21: Proposed Athletic Building Floor Plan, 2nd Floor 42
Figure 22: Proposed Athletic Building Floor Plan, 1st Floor 42
Figure 23: Pedestrian Bridge Cross-Section. .... **Error! Bookmark not defined.**
Figure 24: Proposed Building Alphanumeric Building Grid 45
Figure 25: Truss Panel Diagram 46
Figure 26: Second Floor Beam and Girder Layout 48
Figure 27: Column Layout 50
Figure 28: Lateral Load Resisting Frame for the Athletic Facility with Earthquake Loading 52
Figure 29: Lateral Load Resisting Frame for the Athletic Building with Wind Loads 53
Figure 30: Athletic facility typical footing and rebar cross-section 54
Figure 31: Building Footing Design 54
Figure 32: Typical Elevation View of Bridge Through Truss. 56
Figure 33: Bridge Roof Frame Section 58
Figure 34: Cross-section of Bridge Roof Frame 58
Figure 35: Lateral load resisting truss for half span of the pedestrian bridge with wind loads. 60
Figure 36: Alternative Pier Cap Design. (not used for final design) 62
Figure 37: Pedestrian Bridge Pier Cap and Rebar Cross-Section 62
Figure 38: Pedestrian Bridge Pier/Footing Layout 63
Figure 39: Pedestrian Bridge Pier and Rebar Cross-Section 64
Figure 40: Typical Bridge Pier, Pier Cap, and Footing. 65
Figure 41: Pedestrian Bridge Footing Design 1, Reinforcement Layout Parallel to Superstructure 67
Figure 43: Pedestrian Bridge Footing Design 2, Reinforcement Layout Parallel to Superstructure 68

Figure 44: Pedestrian Bridge Footing Design 2, Reinforcement Perpendicular to Superstructure 68
Figure 42: Pedestrian Bridge Footing Design 1, Reinforcement Perpendicular to Superstructure 68
Figure 45: Pedestrian Bridge End Pier and Rebar Cross-Section Plan View 70
Figure 46: Athletic Facility Material/Labor Cost Breakdown 71
Figure 47: Pedestrian Bridge Material/Labor Cost Breakdown 71
Figure 48: Athletic Facility REVIT Rendering 73
Figure 49: Pedestrian Bridge Rendering, View from Track 74
Figure 50: Side view of Parking Garage Field 101
Figure 51: Elevated view of proposed bridge span 101
Figure 52: Elevated View of Proposed Athletic Building Site Location 102
Figure 53: Aerial Map of Bridge Span and Building Site 102
Figure 54: Base of Sports and Recreation Center 103
Figure 55: Site survey diagram...104
Figure 56: Roof truss calculation spreadsheet..114
Figure 57: Building roof beam/girder calculation spreadsheet 120
Figure 58: Building 2nd floor calculation spreadsheet 127
Figure 59: Building Column Example 132
Figure 60: Building Column Loading and Descriptions 132
Figure 61: Building Footing Example 142
Figure 62: Building Footings Loading Conditions 142
Figure 63: Bridge Truss Calculations 154
Figure 64: Bridge Pier Calculations 166
Figure 65: Bridge Pier Loading Conditions..166
Figure 66: Athletic Facility Cost Estimate Overview...178
Figure 67: Pedestrian Bridge Cost Estimate Overview 179
Figure 68: Pedestrian Bridge, Superstructure Connection to Elastomeric Bearings.........................180
Figure 69: Athletic Facility, Interior Beam Connection to Column 180
Figure 70: Athletic Facility, Exterior Beam Connection to Column...181
Figure 71: Athletic Facility, Interior Girder Connection to Column...181
Figure 72: Athletic Facility, Interior Girder Connection to Interior Beam 181
Figure 73: Athletic Facility, Exterior Girder Connection to Interior Beam 182

List of Tables

Table 1: Various Zoning Types Present in Worcester, MA 4
Table 2: Pertinent Design Parameters 13
Table 3: ADA Design Parameters 14
Table 4: Sources Used to Complete the Following Procedures. 24
Table 5: Lateral Load Resisting System Evaluation 29
Table 6: Typical estimate contingency in Massachusetts (DCAM, 2006) 37
Table 7: Building Roof Truss Key Assumptions 47
Table 8: Truss Roof Member Schedule 47
Table 9: Beam-and-Girder Roof System Member Schedule 48
Table 10: Building Second Floor Beam and Girder Key Assumptions 49
Table 11: Second Floor Beam and Girder Schedule 50
Table 12: Building Columns Key Assumptions 51
Table 13: Schedule of Columns 51
Table 14: Building Seismic Calculation Key Assumptions 52
Table 15: Building Wind Design Key Assumptions 53
Table 16: Schedule of building lateral members 53
Table 17: Schedule of Footings 55
Table 18: Schedule of Footing Rebar 55
Table 19: Elevator Hoist Beam Key Assumptions 55
Table 20: Bridge Truss Key Assumptions 57
Table 21: Schedule of Bridge Through Truss Members 58
Table 22: Bridge Roof Frame Key Assumptions 59
Table 23: Schedule of Roof Frame Members 59
Table 24: Bridge Lateral Truss Key Assumptions 60
Table 25: Bridge Pier Cap Key Assumptions 61
Table 26: Bridge Pier Cap Design Results 61
Table 27: Pier Identification and Location. 63
Table 28: Bridge Pier Key Assumptions 64
Table 29: Bridge Pier Design Results 64
Table 30: Bridge Footing Locations 66
Table 31: Bridge Footings Design 1 Assumptions 66
Table 32: Bridge Footing Design 2 Assumptions 66
Table 33: Bridge Footing Design Results 67
Table 34: Bridge Elastomeric Bearing Design Results 69
Table 35: Bridge Elastomeric Bearing Assumptions 69
Table 36: Bridge End Pier Assumptions. 70
Table 37: Bridge End Pier Design Results 70
Table 38: Results of cost analysis 71
Table 38: Complete Survey Data 104

1.0 Introduction

A bridge can be defined as a structure spanning and providing passage over a river or road (Merriam Webster Dictionary, 2017). But a bridge can be more than that. A bridge can connect people, ideas, and cultures. Currently, part of our Worcester Polytechnic Institute (WPI) community and campus is disconnected from the rest. A state highway with limited pedestrian access divides the WPI campus into fragments. This is not only a culturally divisive barrier, but also a major safety concern. Students, faculty, and guests may have to navigate across this dangerous road in order to explore the full extent of the campus.

Our team is proposing and designing an enclosed pedestrian bridge that would span the state highway 122A, which is Park Avenue. The bridge will connect the current WPI Sports and Recreation Center 3rd floor to the proposed WPI athletic facilities, where the current A.J. Knight Field and tennis courts are located. Along with access to these facilities, the bridge would allow WPI students and faculty to travel more easily from the main campus to the Hughes House, Jeppson House, and the Alpha Tau Omega Fraternity house. We believe safe access to these campus buildings as well as the numerous off-campus residences is a priority for students and parents and should be a priority for the WPI community.

As part of our project we will also propose and design the construction of a new athletic building located along Park Ave near the location of the current WPI tennis courts and the A.J. Knight Field. The purpose of the proposed athletic building is to provide additional facilities for the WPI community as a whole. On the first floor of the facility, an indoor athletic training field was designed with elevated ceilings to be used for any indoor athletic drills/conditioning, training, and/or games. An open space area for strength training was also included on the first floor to accommodate strength training equipment along with men's and women's locker rooms and restrooms. This could be used by WPI's various division 3 and club athletic teams year-round. Due to the limited athletic facilities that are now available, many athletic teams must reserve or share current gymnasium space with the general population of the WPI community. This can create a restrictive environment for athletic teams that need space to train and condition. With the addition of a new indoor athletic training facility, sports teams would have additional designated space to train, especially during the winter months when outdoor fields are not available.

Varsity sports at WPI often require the use of conference rooms to hold team meetings. Currently the space designated for sports teams to hold team meetings is very limited, amounting to one dividable room in the Sports and Recreation Center. This space usually is shared with and used by the WPI administration and faculty for meetings. The proposed athletic building also includes space for offices and conference rooms on the second floor. These rooms can be used for team meetings, coaches' meetings, and study hall rooms for student-athletes.

2.0 Background

This project aimed to both provide access to a currently underutilized portion of campus and increase the amount of recreational space available to students. By developing this new athletic facility and enclosed pedestrian bridge, the Institution would be able to continue to grow in size and increase its sphere of influence to a larger number of current and prospective students. In order to properly deliver this project, certain background knowledge and understanding were required. This section provides the necessary information regarding the numerous factors that were taken into consideration within the design of these structures. Information regarding the current status of the project site, pertinent material properties, and the project's impact on the community can be found in this section. In addition, relevant regulatory provisions and design parameters necessary to deliver safe and constructible structures can also be found here.

2.1 Assessing the Need

The need for a bridge was assessed through the evaluation of the Worcester Polytechnic Institute (WPI) campus as a whole and through an interview with Dana Harmon, The Director of Physical Education, Recreation, and Athletics (Harmon, 2017). The initial thought of a pedestrian bridge came from the fact that the campus extends over Park Avenue and safety is always a concern. Park Avenue has four lanes of traffic (two in each direction), and there are only crosswalks on either side of campus. The bridge would connect the Alpha Tau Omega Fraternity House, The President's house, and a residential campus house to the rest of campus with little safety concern. Having a travel way that WPI's students and employees can safely access would decrease the likelihood of injury crossing Park Avenue. Also with a bridge located between the existing Salisbury Street and Institute Road intersections with Park Avenue, commuting time will be cut down by a few minutes because travelling to the perimeters of the campus wouldn't be necessary. Additionally, the tennis courts may be used more often and students accessing the courts would be in less danger crossing the street.

Dana Harmon spoke about the different athletic buildings she has seen on campus. With the growing success of the WPI athletics program, there is a need for more space for equipment, training facilities, and rehabilitation. In recent years, athletics has had a big impact on campus and continued support from the student body. Director Harmon mentioned that the new recreation center has helped maintain that continued support because student-athletes are able to have their own training space. With varsity sports, club sports, intramural sports, and physical education classes, WPI has developed a need for more space to host all of these programs. Director Harmon also mentioned that having multiple-use spaces would benefit the entire campus. With a surface that can withstand outdoor cleats, indoor shoes, and regular sneakers, all athletics can use the surface for practice and training. The office space would help accommodate meeting areas for teams thus allowing the campus to use classrooms for academic use.

2.2 Community Impact

The bridge and building will both positively impact the WPI community and the Worcester community. The pedestrian bridge would allow students and faculty to have a safer commute from the main campus to the new athletic facility and would allow drivers to be less interrupted by pedestrian traffic.

2.2.1 Massachusetts Zoning Districts

Across the State of Massachusetts, each city or town is required to have ordinances and regulations regarding the different zoning districts within the city or town. The defined zoning districts regulate the different types of land use that may occur. The districts included in Worcester, MA are listed in Table 1.

Table 1: Various Zoning Types Present in Worcester, MA

Worcester Zoning Districts	
Residential	Institutional
Industrial	Airport
Business	Open Space
Manufacturing	Overlay

Each of these districts have sub-areas which involve different permitting requirements for the types of land use. Permits within these zoning districts fall into four usage areas, each with a set of subsections: residential use, general use, business use, and manufacturing use. Usage is either permitted in the district, not permitted in the district, or requires a special permit. According to the Worcester, MA zoning map dated February 6th, 2017, the proposed site falls into the Institutional (Educational) district of the City of Worcester which is consistent with the rest of the WPI campus. In accordance with Article IV Section 2 Table 4.1, non-residential parking facilities, recreational/service facilities, and schools (non-profit) are permitted in the Institutional (Educational) district of Worcester. According to Article IV Section 4 Table 4.2, there is no minimum area or frontage; the front, side, and rear setbacks are 15 feet, 10 feet, and 10 feet respectively; there is no maximum number of floors or maximum height; and there is no floor-to-area ratio (City of Worcester, MA - Zoning Map; City of Worcester Zoning Ordinance).

2.2.2 Impact on the WPI Community

When the current Sports and Recreation Center was constructed, health insurance costs for WPI's faculty and staff were reduced (Harmon, 2017). This was because the new center opened up more space for the WPI employees to work out and gain the healthy benefits of exercise. A new athletic performance center would have additional space for offices and the athletic training staff, creating more recreation space in the current Sports and Recreation Center for students, faculty, and staff. As the number of students in the incoming classes continues to increase, it is important that the space provided can handle the student body. A new campus building will help WPI be successful as it continues to grow and expand in the future.

2.2.3 Impact on the Greater Worcester Community

The City of Worcester is heavily reliant on the students, faculty, and visitors of the twelve universities that make up the Worcester Consortium. Every year thousands of students move to Worcester to earn an education and grow as individuals. These students help drive the local economy by providing a steady flow of revenue and labor. Students also lead community service and social activism movements that help to improve the quality of life for the permanent residents of the city. Improving the quality of the facilities at one of Worcester's most prevalent universities will attract more highly-talented students to the city and help the local economy continue to grow. Outside of the increased student population, the proposed facility will provide a landmark for the city and generate numerous jobs during the construction phase of the project.

2.2.4 Economics

A project of this size is guaranteed to have a large initial cost associated with it. When considering this initial investment one must consider the major costs of engineering services, construction materials and building systems, project management, and long-term maintenance. A cost analysis was performed following the completion of the final structural design of both the bridge and the athletic building. The 2017 Building Construction Costs Book with RS Means (Plotner, 2017) was used to create this cost analysis. However, in order to provide a rough estimate for the new facility, similar facilities' costs can be utilized. The 78,000 ft^2. Foisie Innovation Studio and Messenger Residence Hall will cost Worcester Polytechnic Institute approximately $49 million (WPI 2017). While the Foisie facility provides living spaces and does not include a pedestrian bridge, it does provide insight into the cost of erecting a new building in Worcester, MA. The proposed athletic facility will provide approximately 51,000 ft^2 of extra space to WPI. By making a direct comparison to the Foisie Studio it can be inferred that the proposed athletic building would cost approximately $32 million.

Recently, the City of Worcester was ordered by a Superior Court to construct an elevated pedestrian bridge connecting the DCU Center to the Hilton Garden Inn and the Major Taylor Parking Garage in the City's downtown district (Moulton 2016). This mandate comes following a recent traffic accident in the area. The proposed bridge is to be 275 ft. long and 10ft. wide, and has an estimated cost of $10 million. The proposed pedestrian bridge on WPI's campus would span approximately 450 ft. and be 10 ft. wide. Using the same direct comparison method used above based on the cost per linear foot of the span, a rough cost estimate of the proposed bridge is $16.4 million. When added to the cost of the athletic building, a total project cost of $48.4 million can be derived.

It is important to note that the direct comparison method does provide good insight into construction costs in the Worcester area, but it does not provide exact values for the project. There are numerous differences between the proposed facility and the two projects used as reference. The new building will require a parking lot, field turf, and different finishes, etc. than the Foisie studio which will alter the final project cost. The pedestrian bridge also differs as it will be enclosed and have various security restrictions. The proposed building and bridge would also be built at a different time than the example projects, altering the cost estimates further. For this reason, the cost estimate that was prepared for the two proposed structures should be seen as a preliminary projection.

2.3 Common Construction Materials

In this section various construction materials are reviewed to assess whether or not they will be beneficial and feasible in the proposed athletic building and bridge. The current materials used on campus will also be taken into consideration in order to keep the similarities on campus and also due to some architectural constraints.

2.3.1 Concrete

Concrete is a composite material that consists of cement, water, aggregate, and sometimes admixtures. It is formed by a chemical reaction, called hydration; concrete forms a unique material that gets harder over time.

Aggregate is a granular material that is typically classified into two forms, fine aggregate and coarse aggregate. Various materials can be used for aggregate, including sand, gravel, crushed stone, or iron-blast furnace slag. Slag also has cementitious properties and may be used to reduce the cement content. Aggregate forms are determined through careful sieve analysis by passing through a set of sieves with progressively smaller mesh sizes. All material that is retained on the #4 sieve and larger is classified as coarse aggregate, and the material that passes through the #4 sieve is classified as fine aggregate. By grading the aggregate material an optimal particle size distribution can be determined, which results in the maximum packing density, where smaller aggregate particles can fill the spaces between the larger particles. This minimizes the amount of cement needed in the concrete and generally leads to improved mechanical and durability properties of the concrete.

In concrete, cement is the binding material for the aggregate. The most common cement used is Portland cement. Portland cement hardens through the chemical process of hydration, beginning as soon as the Portland cement touches water. This process produces a strong chemical bond, which makes the compressive strength of concrete possible. The material obtained immediately upon mixing of the various concrete ingredients is called fresh concrete, while hardened concrete results when the cement hydration process has advanced sufficiently to give the material mechanical strength. In the United States, the strength is determined 28 days after casting, but this only represents about 90% of the potential compressive strength and is usually sufficient to support the necessary loadings. Curing must be done in a controlled environment in order to ensure that none of the water needed for the hydration process is lost.

Abnormally fast drying can cause structural cracks or tensile failures due to the uneven nature of the curing process. This problem can be avoided by controlling the moisture content by covering curing surfaces with sheets of plastic or canvas or by periodically spraying them with water. Water is an essential ingredient in concrete because the water-to-cement ratio helps determine the final strength of concrete. The general rule is to add the minimum amount of water necessary to ensure that all of the aggregate is saturated and that the concrete remains fully workable until it is set in its forms. In most concrete mix designs, the water-to-cement ratio can range from 0.3 to 0.6. If there is not enough water present in the mix, the concrete may harden prematurely and leave voids in the finished product. On the contrary, too much water can weaken the compressive strength of the concrete and result in structural failures.

While aggregate, cement, and water are the main ingredients of concrete, various mineral and chemical admixtures can be added. These admixtures have been developed over time to allow concrete to be utilized on various projects with different needs and environments. For example, air-entraining agents are often used to improve the freeze-thaw resistance of concrete. Voids in concrete are often filled with water and by adding air bubbles there is space for the water to expand when it freezes, which reduces cracking. There are also water-reducing admixtures that increase the workability of concrete which allows for the use of less water in the mix design, resulting in increased strength and durability. Retarding admixtures are often used on projects when delays in concrete placement are expected because these admixtures shorten the period needed to commence the hydration process.

Concrete is one of the most versatile and most widely used construction material worldwide. Most commercially produced concrete has compressive strengths between 3,000 and 5,000 psi. If loaded in tension, the material fails at a stress much lower than that, typically of the order of 10% of the compressive strength. Because the tensile strength of concrete is much lower than its compressive strength, it is typically reinforced with steel bars. This creates an efficient composite material that is strong in both tension and compression (Meyer, 2016).

2.3.2 Steel

Steel is another widely used material in construction. It is able to be used to deliver cost effective and sustainable buildings. Off-site manufacturing improves safety and construction speed, reduces waste, and leads to better quality results. Steel construction is especially useful when creating educational buildings, which often need to be erected in a short period of time.

For university buildings, steel construction is very useful to due the speed of construction, adaptability and flexibility, safer construction, minimized disruption, and aesthetic capabilities. The use of components prefabricated off-site allows for construction periods to conform to the academic year. Health and safety is improved due to increased control over off-site manufacturing, which is important if construction is in parallel with educational activities. Long span steel construction allows for large column-free spaces and for rooms to be flexibly configured to meet changing educational needs. Furthermore, steel is a highly useful material for bridge design not only from a material standpoint, but also from an architectural standpoint. Steel can be fabricated into a wide variety of architectural shapes, which can allow for more architectural and aesthetically pleasing features.

In bridge design, steel offers many attractive advantages. One of the most important advantages steel offers is its high strength to weight ratio. This may be a crucial advantage when it comes to the design of the new pedestrian bridge and could have many positive impacts on the design of the bridge. One of the most important factors that the high strength to weight ratio could impact is that it will allow the bridge to carry a greater load for a shallower depth. Since the bridge design is constrained by clearance requirements over State Highway 122A, this would be an ideal material to utilize because it can carry a greater load for a shallower depth. Since steel has a high span-to-depth ratio, it would be useful when designing the pedestrian bridge. Additionally, the transportation and placement of the required beams may be easier due to their low self-weight. Steel may also contribute in the reduction of construction time. During the bridge construction the road may need to be closed temporarily, which is not favorable on a busy state route. It is easy to see why the closing of this area for an extended period of time would be unfavorable. With many of the components of the bridge being prefabricated, construction time would be greatly minimized.

2.3.3 Timber

Timber is another construction material used in various types of construction. The interest in green materials for construction and reduction of carbon footprints in urban development continues to grow, and it is important that building materials aid in sustainability and zero-waste usage practices. Among various construction materials, timber has the potential for use in green design. Currently, timber is primarily used in low-rise and small residential buildings.

Timber has a high strength-to-density ratio and is an extremely versatile and flexible material. These qualities make timber a compatible material with concrete and steel in certain building applications. Another important quality of timber is its resistance to fire. Timber is currently used in some low-rise building applications and has demonstrated favorable fire-resistance capabilities. Even though timber has several qualities that make it a favorable construction material in various applications, it was determined that it would not be explored further in the design process of the building and bridge. This decision was made primarily due to current campus aesthetics. The WPI campus currently consists mostly of buildings made from concrete or steel, with many having brick detailing on the outside as well. A timber building or bridge would not fit into this campus architecture as well as concrete or steel. Also, both the building and bridge need to have large open spans, and both concrete and steel provide greater span-to-depth ratios than timber (Mohammadi, 2017).

2.3.4 Composites

In addition to concrete and steel, composite construction is becoming increasingly popular in bridge design and building construction. They are most often used as an alternative to reinforced concrete decks, but there are some limitations of composite materials. Composite construction exists when two different materials are bound together so strongly that they act together as a single unit from a structural point of view. In bridge design, composite action means that the steel structure of a bridge is fixed to the concrete deck so the steel and concrete act together, helping to reduce deflections and increase strength.

2.4 Bridge and Building Systems

The structural performance of a structure is dependent upon a multitude of factors, such as materials, intended usage, and structural system. There are various types of bridge and building systems that all have different advantages and disadvantages. This section will discuss the function and characteristics of these systems.

2.4.1 Suspension Bridges

Suspension bridges provide structural support through high strength steel cables anchored in abutments on both ends of a bridge’s span that are strung over large pylons located at two equidistant points along the span. Suspenders (vertical steel connectors) attach the main cables to the bridge deck (Duan, 2015 B). A diagram of this bridge system can be found in Figure 1. Suspension bridges can be either self-anchored or externally anchored. Self-anchored bridges anchor the main cables into the bridge deck itself, while the externally anchored systems make use of large concrete abutments to provide anchorage.

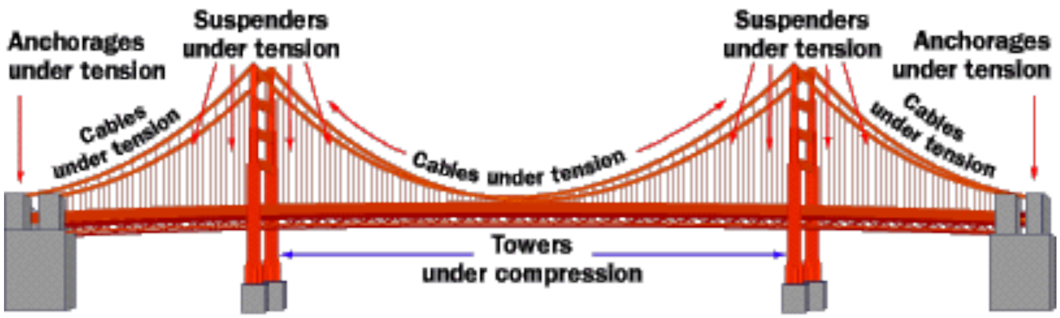

Figure 1: Suspension bridge diagram (Lamb, 2000)

There are numerous reasons to employ a suspension bridge system, the first being the economic savings. Suspension bridges require a very small amount of materials relative to the span lengths achievable by these systems. There is also a cost savings in the construction phase as there is no need to construct temporary span supports when installing the bridge deck because the steel cables accomplish this sufficiently (Duan, 2015 B). These bridges, when done properly, have the ability to stand as architectural statements such as the Golden Gate Bridge. However, there are also disadvantages to suspension bridge systems. These bridges are extremely flexible and can experience high deflection values caused by large gravitational or lateral loads. This can cause problems in applications that are subject to extreme weather conditions. In addition, these systems are highly reliant on the compressive strength provided by concrete piers, foundations, and abutments, while the anchorage relies on tension. In cases where the soil does not provide the proper strength or information is not fully known, these systems should not be utilized.

2.4.2 Truss Bridges

Truss bridge systems make use of horizontal, vertical, and angled members to provide structural support as seen in Figure 2. They can be seen as a flexural girder with the chords as flanges and a web of triangular member arrays. The top and bottom chords carry the majority of the moment while the vertical and angled members are responsible for shear forces (Duan, 2017). Various types of bridges make use of trusses as structural support; through-truss and deck truss bridges are the most typical configurations implemented. Through-trusses locate the deck along the bottom chord, while deck trusses make use of the top chord for the bridge deck. Depending on span length, high-strength steel can be used in the construction of trusses.

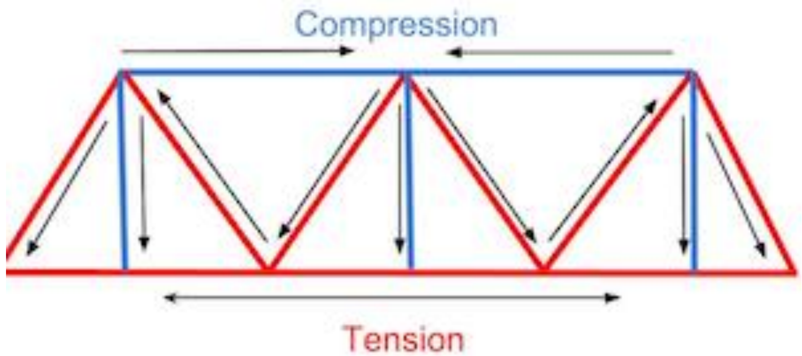

Figure 2: Truss bridge diagram (Robb, 2016)

Similar to suspension bridges, truss bridges have both advantages and disadvantages. Truss bridges behave well under dynamic loading, such as vehicles, and have high resistance against lateral forces. They are also built with smaller, light-weight members which makes transportation and erection much simpler. Due to their light weight, these bridges require less compressive strength from the soil than suspension bridges, making them a good alternative for applications where the soil is not strong enough for a suspension bridge. The primary disadvantage of truss bridges is the complexity of the construction phase. All of the steel members must be bolted or welded together which requires a large quantity of labor and funds.

2.4.3 Arch Bridges

Arch bridges are the oldest form of bridge system. They employ one vertically-curved compression member that transfer forces to foundations located at both ends of the span as seen in Figure 3. Materials for this system vary from timber to stone to steel, which provides flexibility in the design process. The arched member is responsible for the majority of the structural support, with vertical suspenders providing auxiliary support (Duan, 2015 A).

Like all bridge systems, the arch bridge is not applicable to all conditions. The major advantage to arch bridges is that all members are subject to compression which allow for a wider range of construction materials. This can decrease the overall cost of the bridge and make this a valid system option where materials are limited. However, as the span length of these bridges increases, tension can begin to propagate throughout the members, potentially causing failure. With larger spans, the middle members of an arch bridge develop tensile forces in the bottom which may cause cracking. This limits the usable span length of these bridges. Therefore, arch bridge systems are not optimal for large span applications.

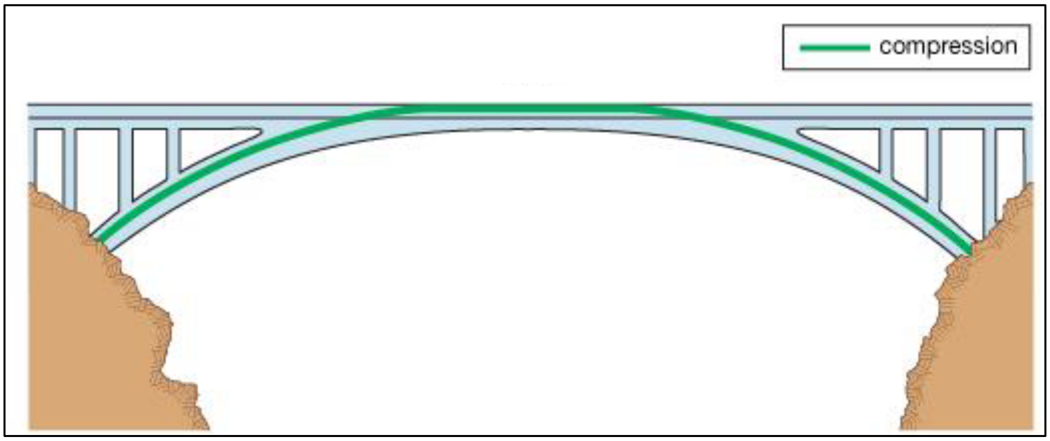

Figure 3: Arch bridge diagram (Shirley-Smith, 2017)

2.5 Engineering Design Parameters

Safety plays a major role in any design; for this reason it is important to comply with the regulatory agencies that govern the design and construction industries. For the proposed pedestrian bridge and building, the designs will be created in accordance with criteria in the design criteria documents displayed in Table 2. In the state of Massachusetts, it is critical that current and future transportation structures be in compliance with provisions of the American Association of State Highway and Transportation Officials (AASHTO), the Americans with Disabilities Act (ADA), and the Massachusetts State Building Code (780 CMR).

Table 2: Pertinent Design Parameters

Design Aspect	Regulatory Agency	Design Criteria
Pedestrian Bridge	American Association of State Highway and Transportation Officials (AASHTO)	LRFD Guide Specifications for the Design of Pedestrian Bridges
	Americans with Disabilities Act (ADA)	Standards for Accessible Design
	Massachusetts Department of Transportation (MassDOT)	Massachusetts LRFD Bridge Manual
Athletic Facility	State Board of Building Standards and Regulations	780 CMR: Massachusetts Amendments to the International Building Code 2009
	Americans with Disabilities Act (ADA)	Standards for Accessible Design

2.5.1 AASHTO Design Criteria

Due to the pedestrian bridge's location over MA Route 122A, the bridge must be constructed in accordance with the requirements of the Federal Highway Association and the US Department of Transportation. The Massachusetts Department of Transportation refers to the AASHTO design specifications for all of its bridge design criteria with some modifications. There are numerous parameters that must be considered when designing a structure of the magnitude of the proposed pedestrian bridge. One of which being the vertical clearance required over Park Avenue, which according to AASHTO is 17.5 ft. above the road surface (WSDOT, 2017). The manual also has requirements for allowable deflections, span-depth ratios, foundation parameters, drainage, and material requirements for structures passing over highways (AASHTO, 2014). This publication proved essential in the design of the pedestrian bridge to pass over Park Avenue.

2.5.2 ADA Design Criteria

It is essential that the both building and pedestrian bridge designs adhere to the American with Disabilities Act (ADA) standards (Department of Justice, 2010). Table 3 shows the corresponding reference sections and parts of the section that the criteria can be found in the ADA standards and design criteria. The table also states the design criteria that are relevant to the design of the pedestrian bridge, including slope requirements and handrail design requirements. These criteria were used during the design of the pedestrian bridge to ensure appropriate access to all facilities.

Table 3: ADA Design Parameters

Section	Design Criteria
302.3 & 3.3	Floor and ground surfaces shall be "stable, firm, and slip resistant."
302.3	If there are any openings in the surface the openings shall not exceed ½".
303	No vertical change in elevation greater than ¼" and if the surface is to be ramped.
402 & 403	Ramps with a rise of greater than 6" must have handrails installed.
405 & 406	Slope shall not exceed 1:20, cross slope shall not exceed 1:48, and the clear width for walking surfaces shall not be less than 36".
505	Handrails must be continuous along the entirety of the walking surfaces length. Handrails are not required on ramps with a running slope of 1:20. The gripping surface of the handrails must also be unobstructed for at least 80% of its length (with a 1-1/2".

Chapter 4 of the ADA Design Standards provides information regarding accessible routes. This chapter states that, in general, accessible routes must consist of one or more of the following components: walking surfaces with a running slope of 1:20 or less, doorways, ramps, elevators, or platform lifts. For walking surfaces, the clear width must be a minimum of 36 inches. However, the clear width is permitted to be reduced to 32 inches minimum for a length of 24 inches maximum provided that the reduced width segments are separated by segments that are 36 inches wide minimum and 48 inches long minimum. Door openings shall provide a clear width of 32 inches minimum. The clear width of an accessible route is shown below in Figure 4, and the clear width of door openings is also shown in Figure 5 (Department of Justice, 2010).

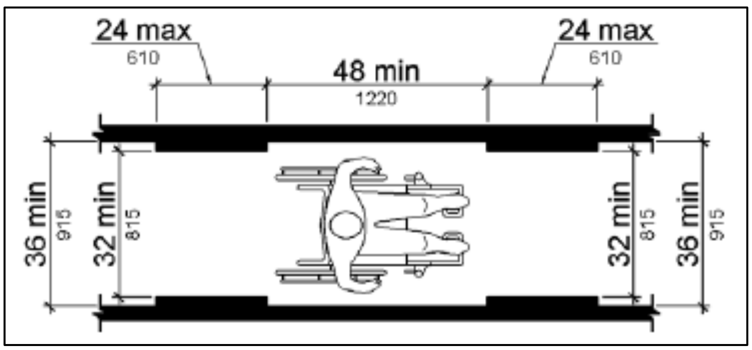

Figure 4: ADA Wheelchair Clearance Requirements (Department of Justice, 2010)

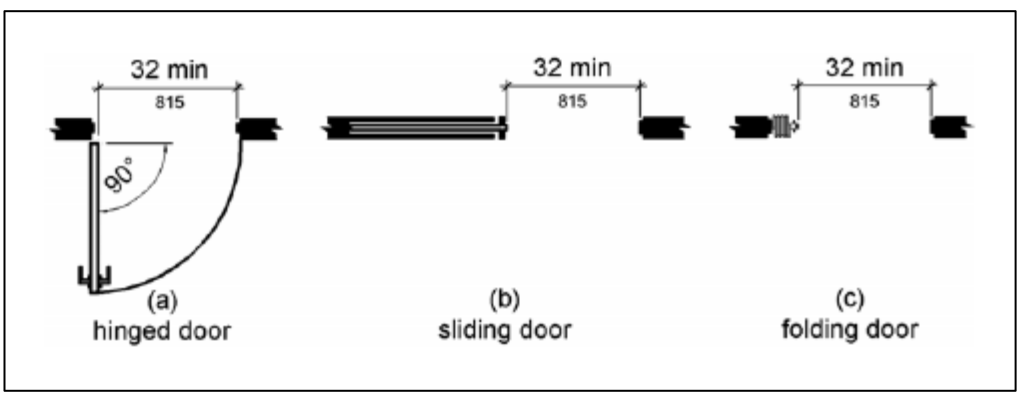

Figure 5: ADA Door Clearance Requirements (Department of Justice, 2010)

Both of these clear width standards for walkways and door openings, respectively, were critical to use during the design of the pedestrian bridge. These standards will also impact the entry from the pedestrian bridge to the proposed building. When looking at connection options, these standards were taken into consideration to ensure walkways and doorways comply with clearances and other design parameters (Department of Justice, 2010).

There is also an ADA Standards section regarding elevators. While the design of the proposed athletic facility does include an elevator, the specific design of the elevator itself was not completed for this project. However, it is still important to be aware of the elevator standards for potential constructability or other issues that may occur during the design process (Department of Justice, 2010).

Chapter 5 of the ADA Standards provides criteria regarding General Site and Building Elements. The design of the pedestrian bridge will also require handrails, and due to this it is important to use the criteria in the ADA Standards during the design process. Along the entire length of the walkway, handrails shall be provided on both sides of the walking surface. The top gripping surface of handrails must be between 34 and 38 inches vertically above the walking surface, and this height must be consistent along the entire length of the surface. This can be seen in Figure 6. Clearance between handrail gripping surfaces and adjacent surfaces must be a minimum of 1.5 inches. This can be seen in Figure 7. The handrail gripping surfaces must be continuous along their length and not obstructed along the top or sides, and the bottom of the gripping surface shall not be obstructed for more than 20 percent of the entire handrail length (Department of Justice, 2010).

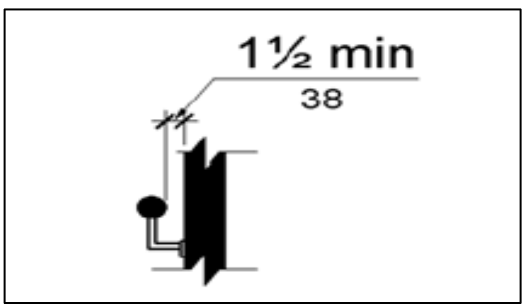

Figure 6: Handrail Clearance with Adjacent Surfaces (Department of Justice, 2010)

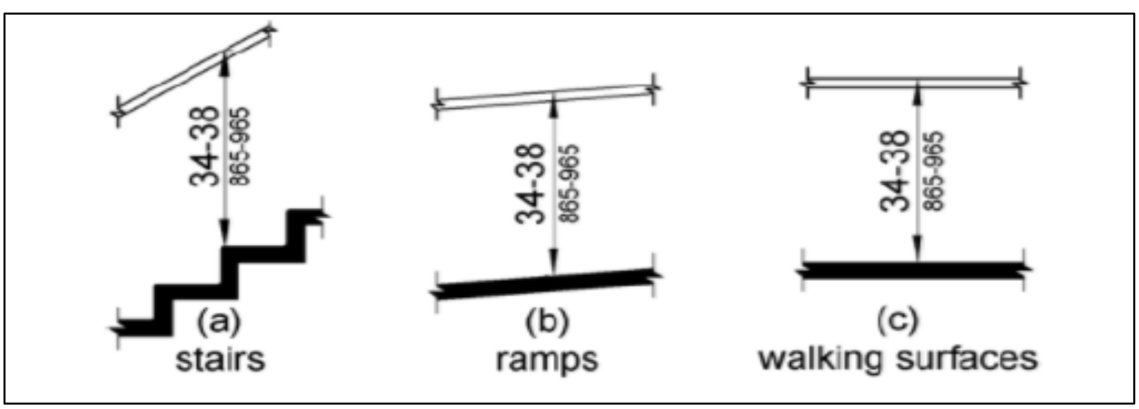

Figure 7: Handrail Height Clearance (Department of Justice, 2010)

2.5.3 Massachusetts Building Code Design Criteria

The parameters gathered from the Massachusetts State Building Code 8th Edition, which includes the 2009 International Building Codes and ASCE 7-05 were the primary provisions affecting the design of the proposed athletic facility. ASCE 7-05 was in effect at the start of the project in August 2017 but the 9th edition became in effect as of January 1st, 2018. The publication contains wind, snow, and seismic loads and load factors to assume, given the location of the project (2009, International Code Council). Sections taken into consideration for the scope of design include: the Use and Occupancy Classifications; the General Building Heights and Areas; Types of Construction; Means of Egress, Energy Efficiency; Structural Design; Structural Tests and Special Inspections; Soils and Foundations; Concrete; Steel; and Safeguards During Construction.

Under the Use and Occupancy Classifications, the theoretical building being designed falls into two groups: Assembly Group A-3 and Business Group B. A-3 describes larger function halls used for recreational use, which pertains to the athletic field portion of the building and the athletic equipment portion of the building where the weight training and athletic training machines would be located. B describes the office section of the building and also has the educational occupancies for students out of high school and in higher education. The potential building will have a one 30-ft story area of 17,500 ft^2 and a two-story area of 17,500 ft^2 limiting the overall type of construction for these two groups to Type I-B construction. Type I-B construction consists of construction elements where the primary structural frame, the interior and exterior bearing walls, the floor construction, and the roof construction are made of noncombustible materials which have 2 hour fire-resistance rating requirement.

The structural design followed the design conditions for LRFD which includes different load combinations, deflection limits for members, dead and live loads, snow loads, wind loads, soil loads, and seismic conditions. For the soil, concrete, and steel components of the design, provisions from the International Code Council governed the materials, quality control, design, and construction, as well as the fabrication and erection of steel members.

2.5.4 International Code Council Wind and Seismic Design Code Master

To assist in the use of the various structural code requirements, code masters were used. The code masters are used as guidelines for the calculation of certain forces, such as wind and seismic forces that act on structures. To calculate the wind pressure acting on the face of the building, the Wind Design Overview Codemaster (Structures and Code Institute, 2009) can be used which can have up to 12 steps that must be followed to determine the Net Design Wind Pressure. This code master breaks the process of calculating the wind pressure into multiple steps. Each step refers to a different calculation that references a different section to the code book, such as American Society of Civil Engineers (ASCE) 7-10 or the International Building Code (IBC). To calculate the seismic forces action on the building, the Seismic Design Codemaster (Structures and Code Institute, 2013) can be used which can have up to 11 steps that must be followed to determine the Seismic Base Shear and Seismic Load Effects. This code master breaks the process of calculating the wind pressure into multiple steps. The code master refers to the IBC 2012 and ASCE 7-10. These code masters can be helpful to standardize and streamline the process for structural engineers that are designing a building or other structure.

2.5.5 LRFD Design Specifications

Load and Resistance Factor Design (LRFD) is a limit state design method used in structural engineering. A structure designed using the LRFD method is proportioned to sustain all actions likely to occur during its design life and to remain usable. Previous to that, the design of steel structures was based solely on the Allowable Stress Design (ASD) or Working Stress Design (WSD) method. The LRFD method is used widely across the structural engineering industry, and was instituted by AASHTO in 1994. This method is based on reliability theory and statistics, and provides a uniform reliability for all limit states. Advantages of using the LRFD method on projects includes that it accounts for variability in load and resistance, achieves a uniform level of reliability, and it provides a uniform level of safety. For the design of building and pedestrian bridge, the LRFD design method was used.

For bridge design, AASHTO published the LRFD Bridge Specifications. The Federal Highway Administration mandated that as of October 1, 2007, all new bridges in the United States must be designed according to the LRFD code. The initial publication of the LRFD Code also succeeded in establishing a framework for introducing the bridge engineering community to the notion of a complete structural design specification on the basis of reliability theory while including a significant amount of recent engineering developments. For example, some of the most important were provisions for gravity loads, gravity load distribution, steel and concrete beam design, and concrete deck design. AASHTO has a variety of LRFD manuals dependent on bridge type, so the one specifically incorporated into the design of this project was the LRFD

Guide Specification for the Design of Pedestrian bridges (American Association of State Highway and Transportation Officials, 2009). This manual provided the necessary information on load combinations, formulas, and basic design methods. All three of these were important to learn about during the design of the pedestrian bridge in order to develop a design that satisfies all limit states.

2.5.6 Ethics

Many designers follow a number of codes that act as guiding principles for engineering, design, and construction decisions. Codes often protect both workers and clients from poor business practices. The American Society of Civil Engineers (ASCE) states that “ethics is integral to all decisions, designs, and services performed by civil engineers.” Not only the public trust but also their lives, safety, and welfare depend on professional engineers' efficient, safe, and economical performance of their duties. ASCE has programs, policies, and resources that are designed to help professionals understand their ethical obligations and how to incorporate them into their professional careers. For this project, designs for the pedestrian bridge and building were developed while upholding the principles stated by ASCE. These principles state that “engineers uphold and advance the integrity, honor, and dignity of the engineering profession by using their knowledge and skill for the enhancement of human welfare and the environment, being honest and impartial and serving with fidelity the public, their employers and clients, striving to increase the competence and prestige of the engineering profession, and supporting the professional and technical societies of their disciplines” (ASCE, 2017).

As an academic activity, it can be very easy to ignore or alter problems that are faced during the design of the pedestrian bridge or building. The same ethical policies and principles were upheld, however, as if this project was for actual delivery. This includes the risks and dangers involved in designing infrastructure that will be used and occupied by students and faculty. During the design process of the proposed structures, the governing regulatory requirements and design standards were used, and issues related to safety were not ignored. By doing this, the ethical standards of the ASCE and the engineering community were upheld.

2.6 Sustainability

Sustainability should be at the forefront of every engineer’s mind when designing or proposing a new structure. Designing structures to be sustainable not only makes economic sense, it also makes ethical sense. Being a technical and engineering school, WPI promotes sustainability with great significance. This means that any building or structure that WPI builds in the future will need to be sustainably designed and environmentally friendly (Ryan, 2017). As students of this Institution and future civil engineers, it is an ethical duty to ensure that this project is delivered in a manner that is both environmentally-friendly and sustainable for the generations of community members to come. Designing a structure for sustainability means that it has a smaller impact on the environment, whether that impact be immediate or in the future, and it ultimately means leaving a better planet for the next generation. Reducing the environmental impact of a construction project could include reducing the amount of energy used to build it, reducing the amount of greenhouse gases released from construction or materials, or reducing the amount of energy the building consumes over its lifetime.

Designing a bridge or building with sustainability in mind, requires conscience decisions on the building materials, design, and construction processes that will be used. “Humphreys and Mahasenan (2002) estimate that the cement industry is responsible for 3% of global anthropogenic greenhouse gas emissions and 5% of global anthropogenic CO2 emissions” (Noguchi, 2015). This shows that using cement to build a structure has environmental impacts that must be taken into account when designing sustainably. The amount used isn’t the only concern though. “Service life can be dramatically extended with little or no increase in – or even a reduction of – the environmental load” (Vanderly, 2003). If the service life of the cement and concrete structure can be extended less cement will be needed overall, saving life cycle costs and reducing emissions and energy usages.

When designing a structure, not only must the designer be critical of the global impact, but also of the local environmental impact. When a structure is built on a particular site, the properties of the location can change dramatically. For example, the area of permeable surface can decrease, causing an increase in rainwater runoff, and altering the current runoff and drainage characteristics. This can impact the local environment in many ways that are difficult to predict as is the case with erosion, flooding, and chemical dispersion. Depending on the site, a new structure may also alter or destroy animal and plant habitats, displacing or placing stress on the local community. For this reason it is important to asses each site and design ways to minimize the structure's impact on its surrounding area. Possible strategies that can be used or anticipated include minimizing asphalt parking, planting native trees and plants, using noise and dust mitigation techniques during construction, and minimizing external light emissions and pollution at night with smart lighting. When changes must be made to the local site, the impacts should be fully assessed and analyzed prior to construction in order to be prepared for potential complications.

2.7 Design Tools

In order to increase the efficiency of the structural design and analysis processes, various software and computer programs were utilized. This section discusses the uses of RISA 2D, AutoDesk Revit and AutoCAD, and Microsoft Excel within the framework of the project.

2.7.1 RISA 2D

RISA 2D is a structural analysis software that allows users to create computer models of the structural members they have designed. These members are then arranged into the required structural configuration. Loads and load factors can be applied to the structure in both the vertical and horizontal directions to simulate the various load combinations that must be considered for design. The software is capable of analyzing the effects of the loads on the given structure and determining moment, shear, and deflection values. These values can then be used to size structural members and components and to ensure that the structural design is within the requirements established by the pertinent regulatory body.

2.7.2 AutoDesk Revit

AutoDesk Revit is a 3D modeling software typically used for creating structural and architectural models of structures. The software allows the design to be created in 2D and develops that model into a 3D representation of the final product. Structural members such as trusses and columns can be created, as well as architectural finishes such as the façade and windows. This software was employed to create 3D renderings of the final structures to provide a proper visual representation of the final design.

2.7.3 AutoDesk AutoCAD

AutoDesk AutoCAD can be used for both 2D and 3D modeling. It is useful in creating floor plans, elevation views, and detail drawings. The software allows designers the freedom to create and edit their designs until a solution is established. This software was primarily used to develop 2D drawings for supporting final structural calculations and architectural plans.

2.7.4 Microsoft Excel

Due to the nature of designing structures with numerous different loads and loading situations, the hand-calculations can become repetitive. Microsoft Excel software provides the ability to create spreadsheets capable of performing the necessary calculations for multiple iterations of similar member types. The software makes use of data and formulas to output the necessary design values. The sheets can be repeated, increasing the speed and efficiency of the design process.

3.0 Methodology

The following section contains information regarding the methods used to determine design procedures, select structural systems, perform calculations, and design certain members and components. This section discusses the preliminary design methods, the design of the proposed athletic building, the design of the proposed pedestrian bridge, and the methods used to present the final results.

3.1 Preliminary Design

Prior to the structural design of the proposed athletic building and pedestrian bridge, some preliminary data and information was obtained. A site walk through and site survey were conducted by the team members in order to gather information about the site. This was done to define the area of interest and usable space for the designs. An interview was also conducted to gain perspective and additional feedback on what the WPI community would want if the proposed structures were to be built. This was done to define the occupant use and loading for certain areas of the structures.

3.1.1 Survey

In order to start the design of the two structures, an initial topographical survey was completed to help locate a solution and define the site geometry. Using the rod and level technique, elevations were gathered at key points running from the Sports and Recreation Center to A.J Knight Field. Measurements were taken at a maximum of 10 ft. increments in an attempt to increase the accuracy of the data, while some measurements were taken at smaller increments at locations with steep grade changes. It should be noted that due to the site's close proximity to a major roadway, high-visibility vests were worn to ensure that motorists were alerted to the field work.

Once elevations were obtained for the site, a topographical map and cross-section were created. These representations made it possible to develop a preliminary site layout and set geometric criteria of the site.

3.1.2 Interview

An interview was conducted on September 15th, 2017 with the Director of Physical Education, Recreation, and Athletics, Dana Harmon (Harmon, 2017). The interview provided additional insight for the usage and floor plan of the proposed athletic building. A list of questions was compiled to ensure that the necessary information was obtained. Director Harmon was able to provide additional aspects of the building which were not initially considered. The information obtained from this interview along with the site survey was used to develop the preliminary design for the building layout and bridge location.

3.1.3 Sources for Design and Calculations

This section summarizes the different sources that were used to design and perform calculations for the members of the pedestrian bridge and athletic building. Table 4 shows the reference building codes and design specifications used. The information taken from the resources are also indicated in the table.

Table 4: Sources Used to Complete the Following Procedures.

Design References	
AASHTO LRFD Bridge Design Specifications (Knovel, 2010)	• Bridge Pier Design • Bridge Footing Design
MA Building Code, 8th Edition (Riley, 2010)	• Design Wind Loads • Design Seismic loads
Design of Reinforced Concrete Structures (Subramanian, 2013)	• Design of bridge footings • Bridge pier interaction diagram
ASCE 7-10, 13 Edition (American, 2010)	• Building Wind Design Loads • Bridge Wind Design Loads
Wind Design Overview Codemaster (Structures and Codes Institute, 2009).	• Building Wind Design Loads • Bridge Wind Design Loads
International Building Code 2012 (ICC, 2012)	• Building Seismic Design Analysis
Seismic Design Overview Codemaster (Structures and Codes Institute, 2013).	• Building Seismic Design load • Bridge Seismic Design Load

3.2 Design of Athletic Facility

This section details the various steps associated with the planning, floor lay-out, structural design, and calculation of the new athletic facility. The building was designed from the top down starting with the building roof and ending with the support footings. All calculations were performed using LRFD methods.

3.2.1 Building Roof System Analysis

Two options were considered for the roof system of the proposed athletic building, a beam-and-girder system and a truss system. Load and Resistance Factor Design (LRFD) methods were used when considering the loads, load combinations, and design process. In Figure 8 and Figure 9, the steps for the design of the beam and girder system and truss system can be respectively seen. Due to the current standards, hot-rolled, wide-flange shapes, 60 ksi steel was used for the design of the beam-and-girder system. After both systems were designed and investigated, the truss system was chosen due to its relatively higher weight-to-strength efficiency when compared to the beam and girder system.

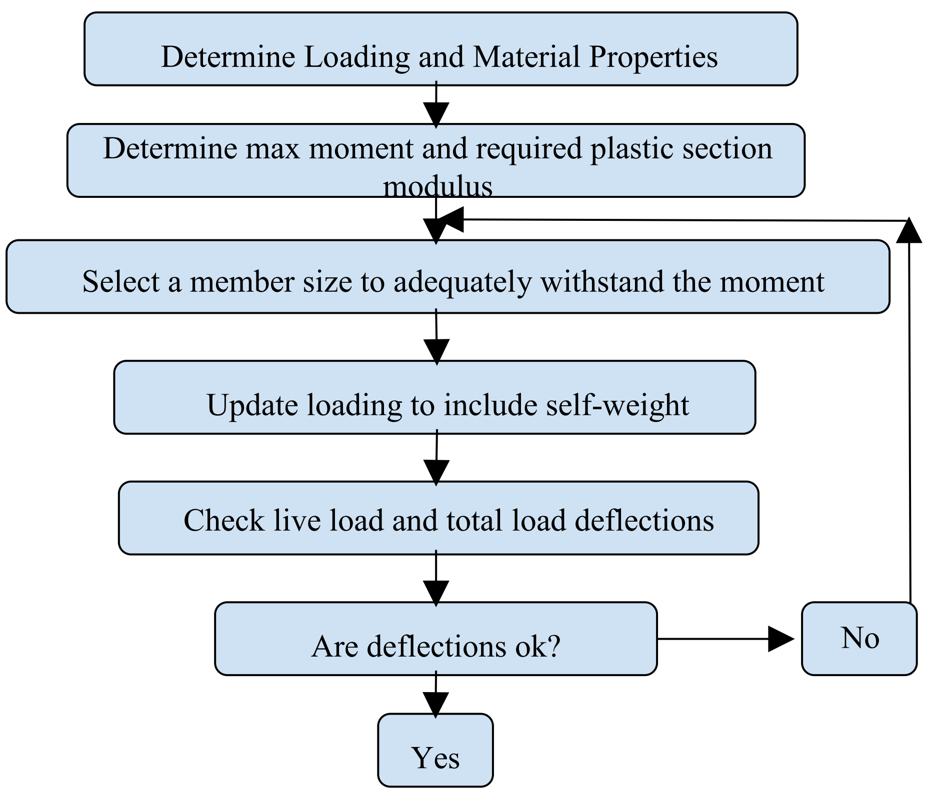

Figure 8: Procedure Used to Design a Beam and Girder System.

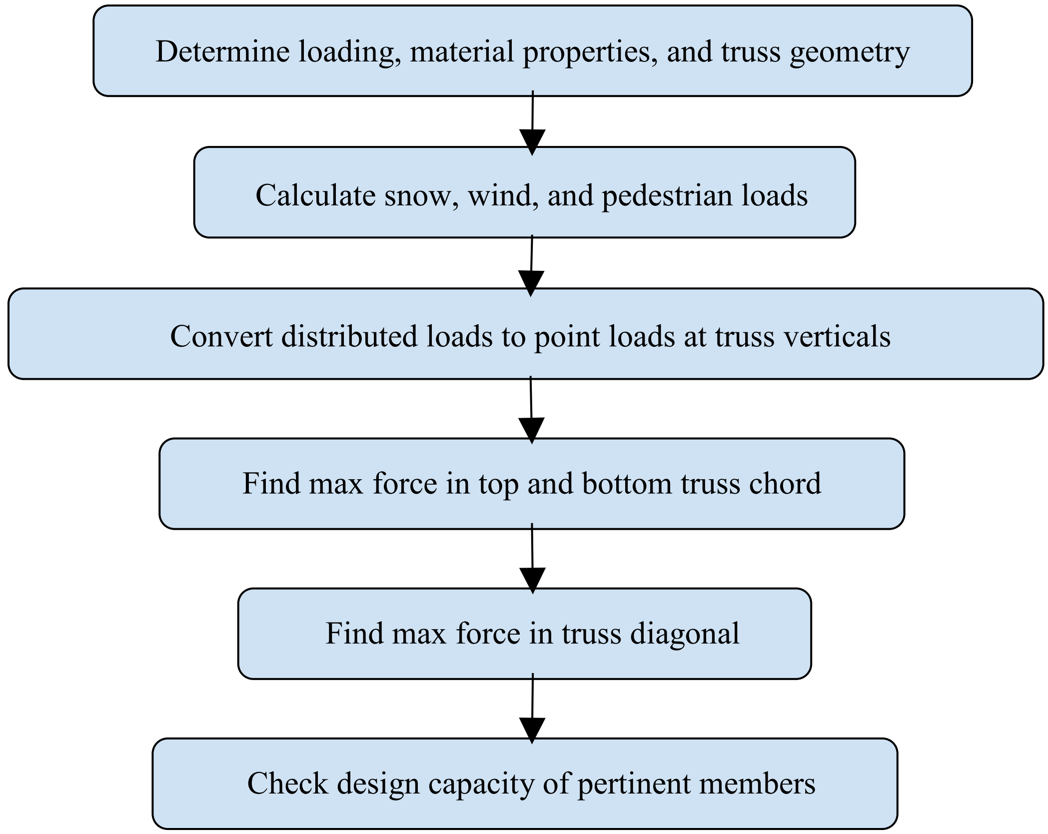

Figure 9: Procedure Used to Design the Building Roof Truss System.

3.2.2 Building Second Floor Design and Analysis

A beam and girder system was considered for the 2nd floor of the proposed athletic building and was designed using LRFD. This type of system was chosen due to its weight reduction and construction speed when compared to reinforced concrete as well as its height when compared to a truss system. The steps used to design the beams and girders can be seen in Figure 10.

Two different systems were analyzed for the floor decking on the second floor including a hollow-core precast concrete slab system and a solid reinforced concrete slab. Calculations were completed to determine the weight of each system in order to minimize the dead load on the second floor. Other considerations that were taken into account when selecting a flooring system included cost, time, serviceability, and aesthetics. The second-floor, slab design also followed LRFD design criteria and the steps used can be seen in Figure 11. For a more detailed process, consult Appendix E with the calculations. After both decking types were designed and analyzed, it was determined that the precast hollow-core planks would be lighter and would allow for faster construction since they would not have to be cast in place. After considering these factors, the pre-cast planks were chosen to be used on the second-floor of the proposed athletic building.

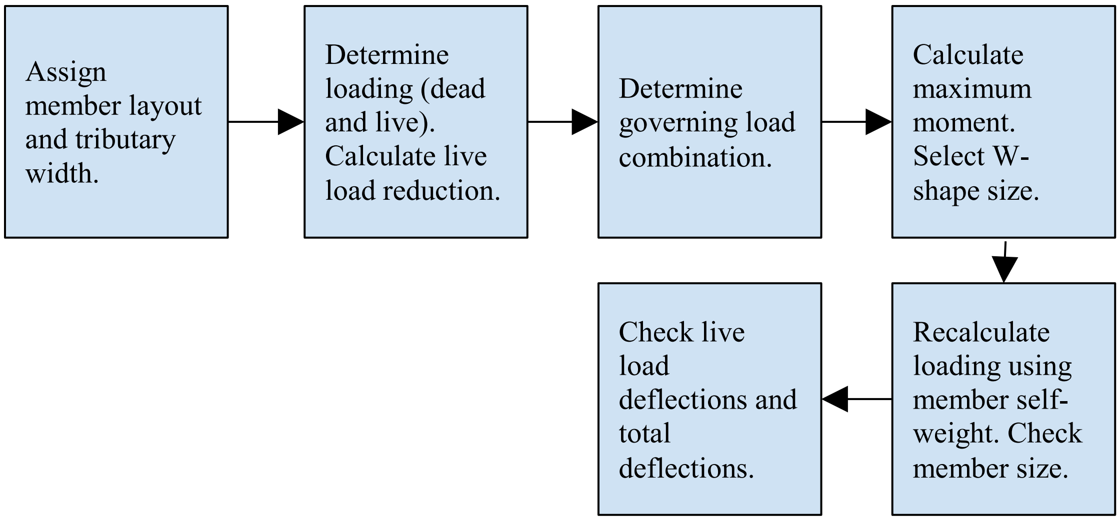

Figure 10: Procedure Used to Design the Building Second Floor.

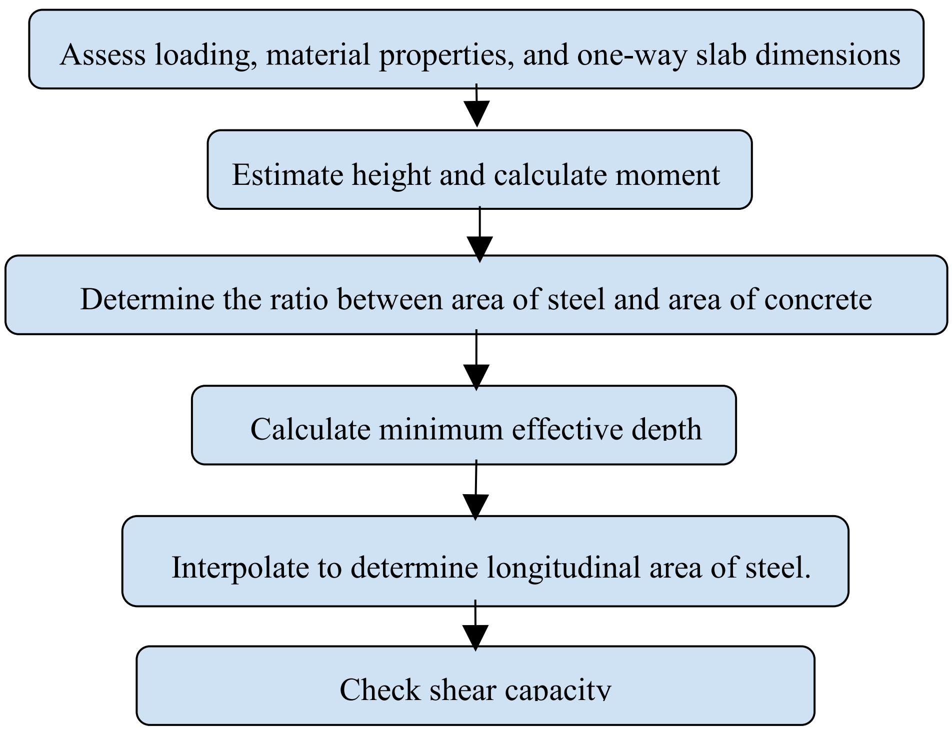

Figure 11: Procedure Used to Design the Building Second Floor Solid Reinforced Concrete Slab.

3.2.3 Building Column Design and Analysis

The steel columns were designed following LRFD methods for the proposed athletic building. W-sections were used for the columns and column segments were sized based on effective length. The columns were standardized to be only one size for ease in construction. All of the columns were designed using the procedure depicted in Figure 12.

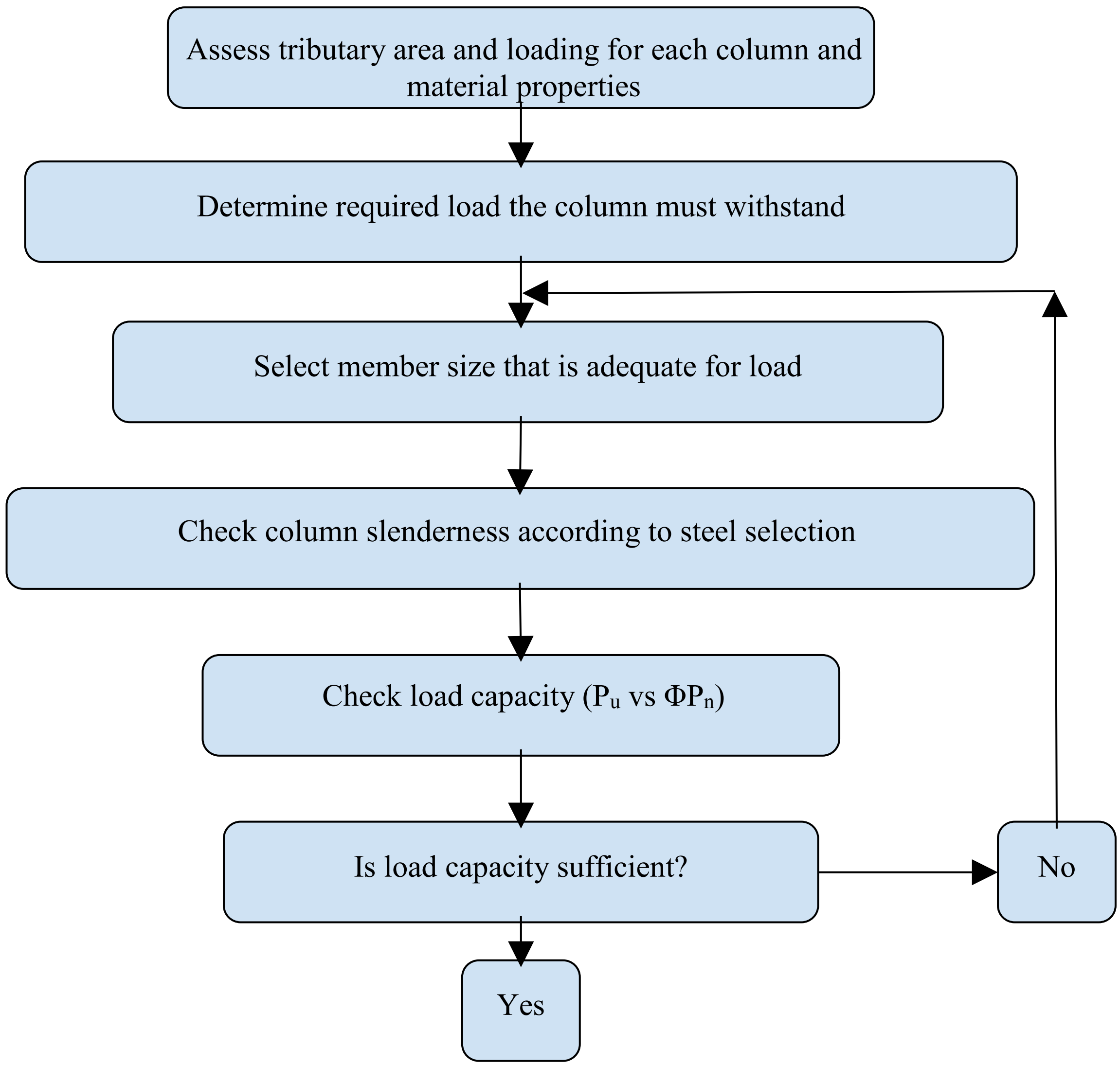

Figure 12: Procedure Used to Design the Building Columns.

3.2.4 Building Wind and Seismic Analysis

The proposed athletic building employs a symmetrical moment-resisting truss system to resist lateral loads. The process used to select this type of lateral force resisting system can be seen in Figure 13.

The Wind and Seismic load analysis for the proposed athletic building followed the requirements of the ASCE 7-05 with aid from the Wind Design Overview Codemaster (Structures and Codes Institute, 2009). The wind design loads were calculated using the ASCE 7 Simplified Procedure for the main wind force resisting system (MWFRS). The basic wind speed for Worcester, MA was found using the MA building code 8th edition Table 1604.11. The wind design load was applied to each vertical face of the proposed building to calculate the applied force on the building. The wind force was then transferred from the face of the building to the braced frames located at the four corners of the building. The use of the steel braced frame system was chosen based on the evaluation of three different systems. This evaluation can be found in Table 5. The braced frames were designed to resist the wind loads applied on the building and resist horizontal deflections at each story of less than 1 in. The calculation for the building wind load can be found in Appendix E. The process used to select a wind design method can be found in Figure 13. The process used to design the braced frame system can be found in Figure 14.

Table 5: Lateral Load Resisting System Evaluation.

Lateral Load Resisting Systems		
System Type	**Advantages**	**Disadvantages**
Concrete Shear Walls	High strength and lower cost.	Longer construction time.
Moment Resisting Frames	More architectural flexibility.	Higher cost of connections.
Steel Braced Frames	Lightweight and no need for moment resisting connections.	Less architectural flexibility.

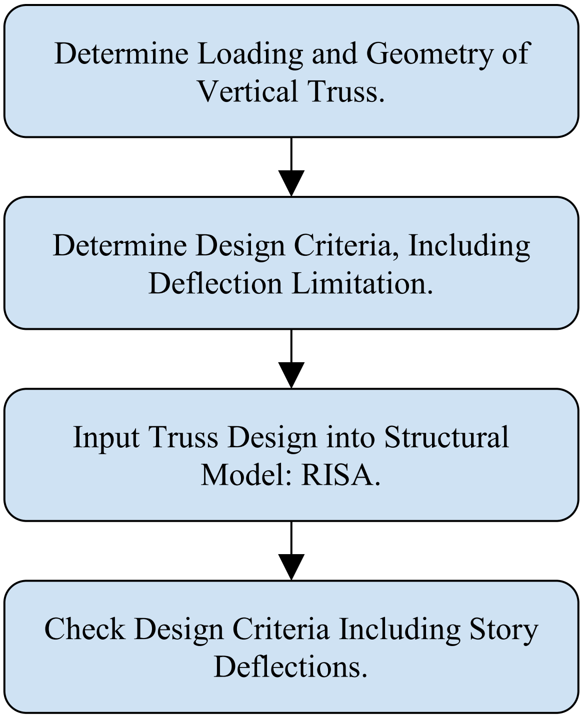

Figure 13: Procedure Used to Design Building Lateral Load System.

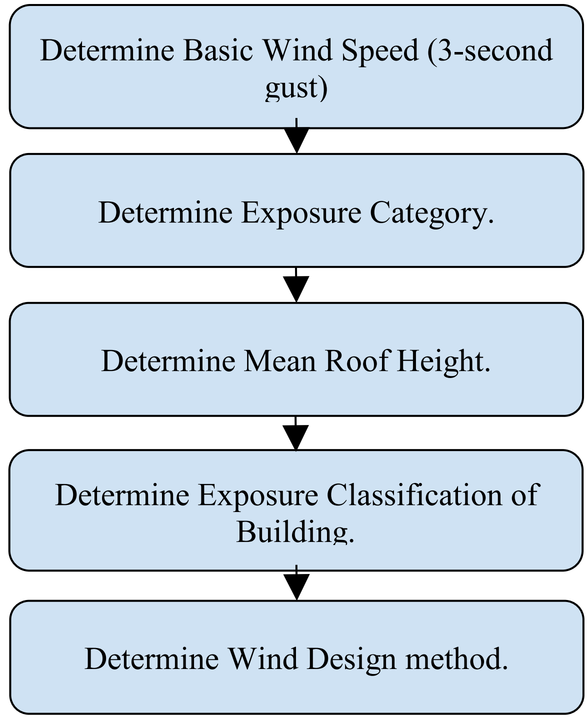

Figure 14: Procedure Used to Determine Wind Design Method.

The seismic load analysis for the proposed building followed the requirements in the ASCE 7-10 (American, 2010), the International Building Code (IBC) 2012 (ICC, 2012), and the Seismic Design Overview Codemaster (Structures and Codes Institute, 2013). The earthquake response accelerations, S_s and S_1, for the maximum considered earthquake for the town of Worcester were given using the MA building code 8th edition Table 1604.11 (Riley, 2010). Using these values, the factors and classifications determined in ASCE 7-10, and the associated loads of the building, the seismic forces were calculated and applied to the appropriate story on the braced frames located in the corners of the proposed athletic building. A response modification coefficient value of 3 was used for the building, meaning that no special seismic detailing was required. The loads included in the seismic calculation can be found in Appendix E. The Equivalent Lateral Force method was used for seismic loading. The braced frame system was then designed to resist the seismic forces and control deflections of the building of less than 1 in. The process used to determine the seismic loading can be found in Figure 15.

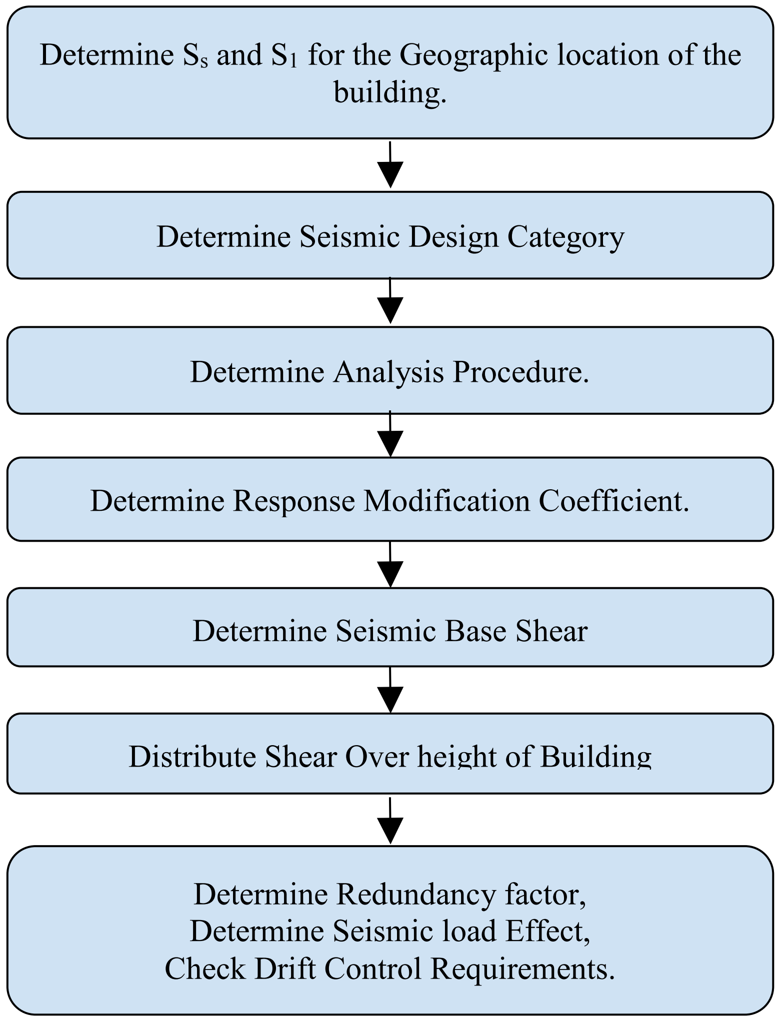

Figure 15: Procedure used to Determine Seismic Loading on Building.

3.2.5 Building Footing Design and Analysis

The footings for the columns for the proposed athletic building were designed using allowable bearing pressure method. The steps used to design the column footings can be seen in Figure 16. For a more detailed process, consult Appendix E with the calculations.

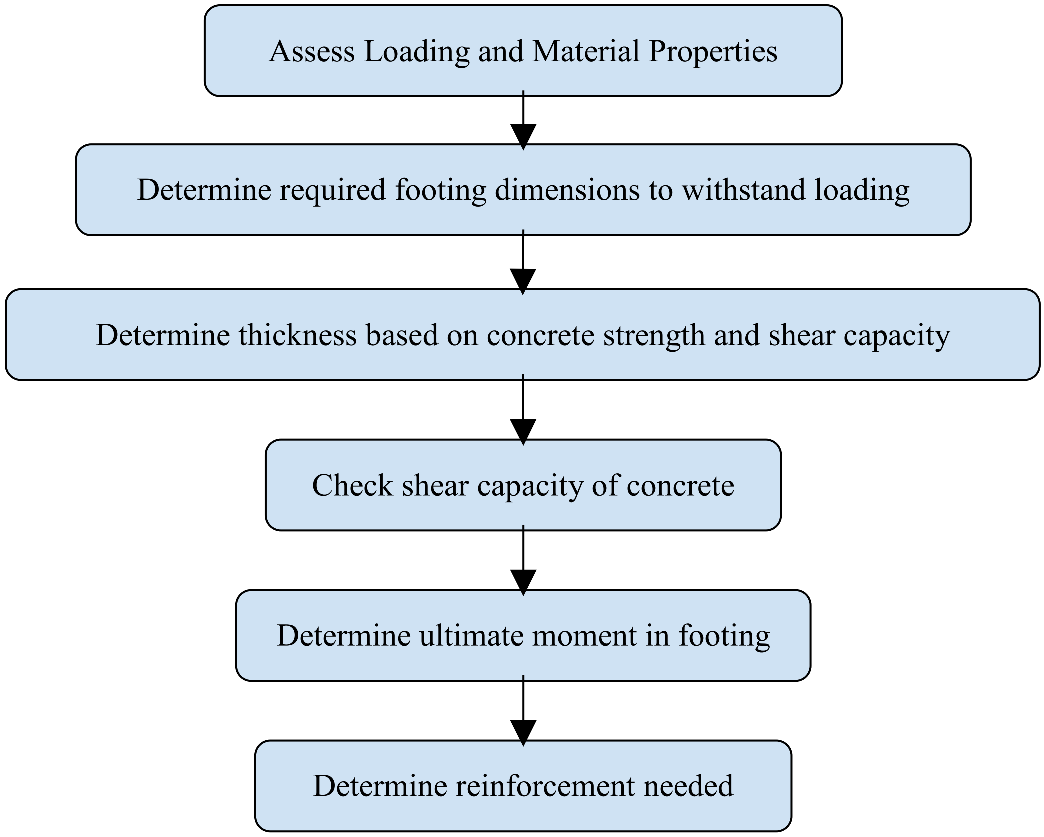

Figure 16: Procedure Used to Design Building Column Footings.

3.2.6 Building Elevator Hoist Beam Design

In order to improve the flow of traffic within the building and increase the ease with which equipment is moved, a hydraulic elevator was placed in the facility's southern-most corner. The primary calculation performed on the elevator was to design the hoist beam responsible for moving the elevator when the hydraulic piston is out of commission. The design was performed using the LRFD method for beam analysis. The full calculation for this beam can be found in Appendix E.

3.3 Design of Pedestrian Bridge

This section details the various steps associated with the planning, design, and calculation of the pedestrian bridge connecting the Sports and Recreation Center and the new athletic building. The bridge design began with the roof frame and ended with the support footings. Seismic and wind force calculations were also performed due to the geometry of the structure. Drawings of the proposed pedestrian bridge were done using software provided by WPI including Autodesk AutoCAD and SOLIDWORKS.

3.3.1 Bridge Through-Truss Analysis

Multiple bridge types were considered in the preliminary planning of the pedestrian bridge. However, given the required span and desire to maximize the clearance height of the structure, a through-truss bridge was selected. The use of a truss also provides advantages when compared to other options such as steel girders or pre-stressed concrete. The through truss uses small steel members making the bridge easier to transport, more efficient, and weigh less than the other options. The through truss is also unique because the structural members are located adjacent to the walking area, as opposed to below on the other options. This allows the bridge to maximize the clearance height below the bridge. The truss was designed using the same procedure as the building roof truss outlined in Figure 8. The complete calculations can be found in Appendix F.

3.3.2 Bridge Lateral Force Analysis

The bridge lateral force resisting system was designed to resist the lateral loads applied to the bridge superstructure. The lateral loads applied to the superstructure included the applied wind forces, but did not included lateral seismic forces because the proposed pedestrian bridge was determined to be classified in Seismic Design Category (SDC) A using the procedure outlined in Figure 15 and Appendix F. SDC A requirements for structures do not include a specific seismic design to be done. This means that the lateral force resisting system was not required to be designed to resist seismic loading.

The system chosen to resist the lateral loads and deflections was a horizontal truss system parallel to the length of the bridge. The procedure used to design this lateral truss can be found in Figure 13, as it was designed using the same methods as the building lateral load resisting system. The lateral trusses were placed at the top and bottom of the superstructure connecting the two through-trusses to minimize twisting under eccentric loading conditions. The calculations for the lateral truss can be found in Appendix F.

3.3.3 Bridge Roof Frame Analysis

The roof frame of the proposed pedestrian bridge was designed using LRFD. The lateral members of the frame were designed as steel beams following methods similar to that in Figure 10. The horizontal members of the roof frame were designed as short steel columns following methods similar to that in Figure 12. The roof frame was designed to slope in two directions as to allow for proper drainage of rain and snow melt. This was done by decreasing and increasing the length of the 2 column members and keeping the slope of the horizontal beam constant.

3.3.4 Bridge Piers

The pedestrian bridge superstructure is supported by the bridge piers. The piers transfer the live and dead loads of the superstructure to the bridge substructure. The piers were designed for the most critical loading and lengths and were applied to all piers for aesthetic and constructability reasons. Since the pedestrian bridge is exposed to wind and seismic forces in all directions, the piers were designed to be cylindrical which allows them to resist lateral loading symmetrically in all directions. Due to the unbraced length of the most critical pier, the piers were designed for the minimum size to ignore slenderness design concerns. Once this minimum size was determined, the piers were checked for their ability to resist the imposed superstructure forces, including live loads, dead loads, and wind loads. This was done using the column interaction diagram for spirally reinforced cylindrical columns with a concrete strength of 4 ksi and reinforcement strength of 60 ksi.

3.3.5 Bridge Pier Caps

The pedestrian bridge superstructure is connected to the concrete piers by a column pier cap. The caps are located at the top of the pier and were designed using LRFD. To design the reinforced concrete member, the caps were simplified as cantilever beam sections extending beyond the body of the pier and supporting the forces imposed onto it by the bridge superstructure. The connection between the pier cap and the bridge through-truss consists of an elastomeric bearing.

3.3.6 Bridge Elastomeric Bearings

The elastomeric bearings were designed using AASHTO LRFD Bridge Design Specifications 3rd edition with 2004 with 2005 interims, using method B. There are four elastomeric bearings located at the top of each bridge pier cap. They are designed to connect the pedestrian bridge truss chords to the pier caps. They were designed to be circular in order to provide symmetrical load resistance and dampening in all directions, similar to the bridge piers. The elastomeric bearings consist of stacked steel plates with alternating rubber dampening layers in between. The bearing layers are also encased in rubber to protect the steel from corrosion.

They were designed to support the bridge superstructure loading including the dead loads, live loads, and lateral loads.

3.3.7 Bridge Footings

The footings for the bridge piers were designed using LRFD and the Design of Concrete Structures (Subramanian, 2013). The footings were designed as rectangular spread footings in order to distribute the superstructure loading across the soil beneath. They were designed to minimize settling and resist overturning of the bridge superstructure. The process used to design the bridge footings can be found in Figure 17. This process was done for two different footing designs based on different columns heights and tributary widths. For footing calculations, see Appendix F.

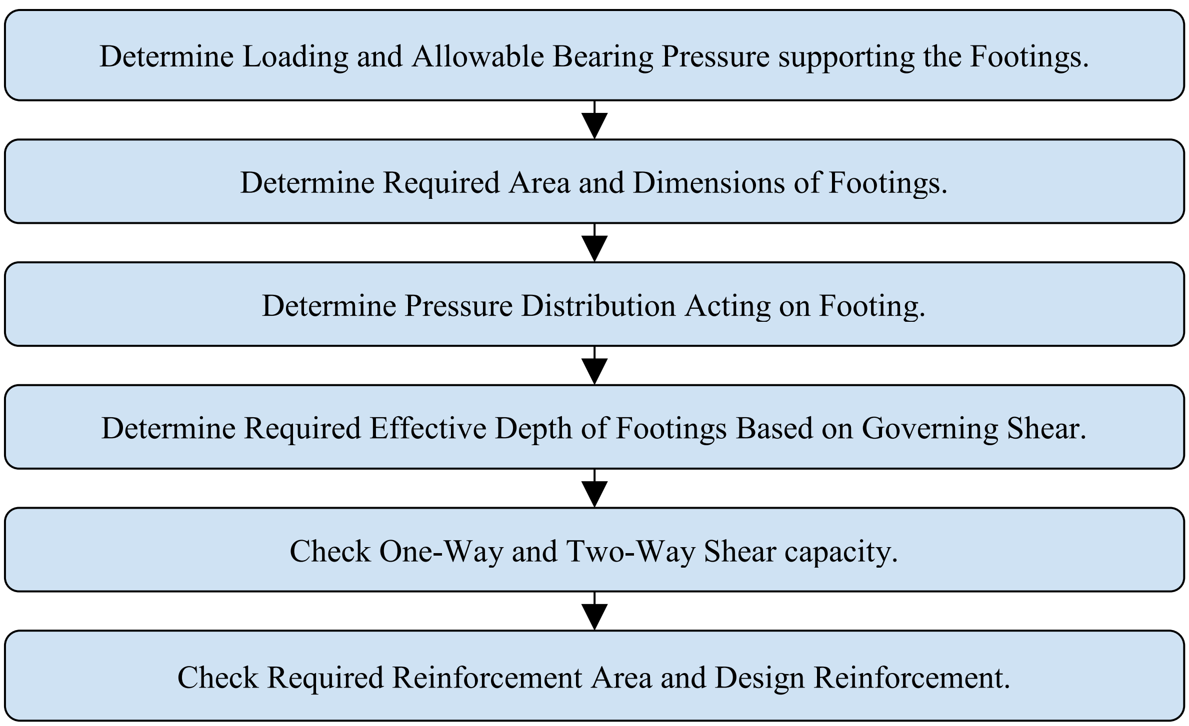

Figure 17: Procedure Used to Design the Bridge Footings.

3.4 3D Modeling and Cost Analysis

Both a cost analysis and 3D computer models were prepared to help present the final design to the public. The models and drawings created were done using AutoDesk Revit and AutoCAD. While the cost analysis was performed using Microsoft Excel.

3.4.1 Building, Bridge, Merging

Once the building and bridge designs were complete, separate 3D models were created in Revit. First, a structural model of the building was created displaying the designed beams, girders, columns, and roof truss. After this, architectural floor plans were created to depict the materials and facade of the proposed building. This helped to not only visualize the structural members of the building, but also to view how they connect and what the facade looks like and Revit was also used to generate schedules and help determine material types and amounts to be used in the cost analysis. The bridge model was created in a separate Revit file, depicting structural members and architectural components as well. Once both models were developed, they were merged into one Revit file to show how they connect together.

3.4.2 Renderings

Using the 3D model of the pedestrian bridge and athletic building, SightSpace Pro (SightSpace, n.d.), was used to generate an interactive rendering. SightSpace Pro is a paid application that converts .DWG files into virtual reality models that can be used interactively to display a 3D rendering. In addition, Google Sketch Up and Google Earth were used to create a realistic model of the structures superimposed on the site. These applications were used to convey design ideas in a visual and consolidated form.

3.4.3 Cost Analysis

The final step in the process of designing this major addition to WPI's campus was to develop an initial cost estimation for the project as a whole. Using various models and drawings, quantity take-offs were performed to establish the total quantity of materials required to complete the project. These values were input into a Microsoft Excel spreadsheet for ease of calculation. Once material quantities were established, R.S. Means Square Foot Costs 2016 and R.S. Means Building Construction Costs Data 2017 were used to determine the costs of materials and labor for the major components of the construction project. A square foot estimate was used to calculate the cost of installing MEP items rather than developing full piping and electrical schedules. The costs were then multiplied by a location factor to adjust for the increased cost of material and labor in Massachusetts. They were also adjusted for price inflation from the year 2016 to 2017. Lastly, engineering fees and project contingency were added to complete the cost estimate. The estimated contingency was determined using the data within Table 6, the project was determined to be in the preliminary study phase. While Figure 18 was used to determine an estimate for the anticipated design fees. Based on the nature of this project, it was classified as a Group II project.

Table 6: Typical estimate contingency in Massachusetts (DCAM, 2006)

Project Phase	**Restoration**	**New Construction**
Preliminary Study	20%	15%
Schematic Design	15%	10%
Design Development	10%	5%
Construction Documents	0%	0%

Building Types

FLCC*		I	II	III	IV	V
From	*to*					
	$149,999	14.0%	11.7%	10.0%	8.0%	10.6%
$150,000	$374,999	12.8%	10.8%	9.2%	7.5%	9.3%
$375,000	$749,999	11.9%	10.1%	8.5%	7.0%	7.7%
$750,000	$1,499,999	11.3%	9.5%	8.0%	6.6%	7.2%
$1,500,000	$3,749,999	11.0%	9.2%	7.7%	6.3%	6.7%
$3,750,000	$7,499,999	9.5%	8.0%	6.6%	5.3%	6.2%
$7,500,000	$14,999,999	8.5%	7.2%	5.9%	4.7%	5.9%
$15,000,000	$37,499,999	8.0%	6.7%	5.7%	4.5%	5.6%
$37,500,000	$149,999,999	7.5%	6.5%	5.5%	4.5%	5.3%
$150,000,000	or more	7.0%	6.0%	5.0%	4.0%	

* Note: The Fixed Limit Construction Cost (FLCC) is the Estimated Construction Cost (ECC) as established in the project study adjusted to the projected mid point of construction.

Add to Fee: - for Fixtures & Equipment (F&E) design and selection costs. (additional service)
- 0.5% for renovation projects

GROUP I Projects of above average complexity as for example: courthouses, college building with special facilities, extended care facilities, hospitals, laboratories, specialized portions of correction facilities, and mental institutions.

GROUP II Projects of average complexity for example: college classroom facilities, repetitive elements of correctional and detention facilities, dining halls (institutional), fire stations, gymnasiums, laundries and cleaning facilities, office buildings (for single occupancy), park, playgrounds and recreational facilities.

GROUP III Projects of less than average complexity as for example: armories, apartments, dormitories, exhibition halls, skating rinks, and service garages.

GROUP IV Utilitarian buildings as for example: parking structures and repetitive garages, simple loft-type structures (without special equipment), and warehouses.

GROUP V Repairs/renovations of limited complexity involving primarily a single discipline (engineering or architecture), i.e. roofs, masonry repairs, window replacement, mechanical/electrical plumbing work, etc.

Figure 18: Typical design fees for construction projects in Massachusetts (DCAM, 2015)

4.0 Design Strategy

This section contains information regarding the design objectives and restrictions created by the architectural program, current site conditions, and desired usage of the new athletic facility and pedestrian bridge.

4.1 Current Site Details and Limitations

The proposed athletic building is to sit on the plot of land directly across Massachusetts Route 122A (Park Avenue) from Alumni Field, on the campus of Worcester Polytechnic Institute (WPI) as seen in Figure 19. This site is currently underutilized by the Institution and is disconnected from the entirety of campus. In order to access the field, members of the WPI community must cross a hectic, four-lane state highway without the use of a convenient crosswalk that is readily accessible from the central campus. This creates an unsafe environment for pedestrians and should use of the location increase, a safer means of crossing is necessary.

The entirety of the proposed building site and current tennis courts is approximately 100,000 ft^2. As currently proposed, the project will leave the three tennis courts on the far northwest side of the site untouched. This will leave adequate space for parking and the new structure, as well as continue to provide tennis courts for students, the WPI community, and the club tennis team. The proposed building site has a relatively level topography and is elevated above Park Avenue by a distance of approximately 12 ft.

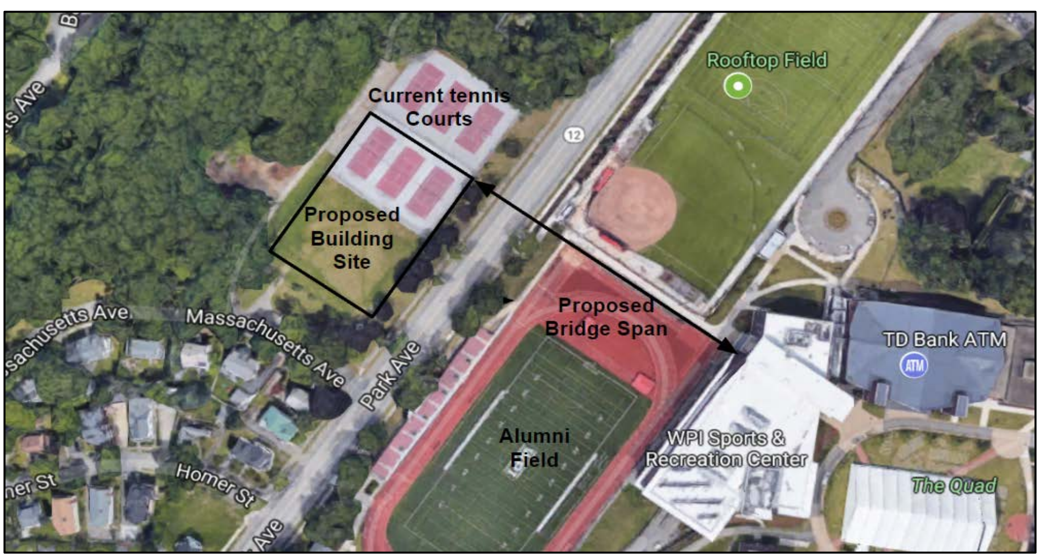

Figure 19: Aerial View of the Proposed Site.

The site's 12 ft. elevation above the sidewalk along Park Avenue does not allow sufficient clearance for a pedestrian bridge over a state highway. The required clearance, per *Massachusetts Department of Transportation (MassDOT) Load and Resistance Factor Design (LRFD) Bridge Specifications* (2013), is 17.5 feet, which can be seen in Figure 20. This requires the bridge to pass 5.5 feet over the current elevation of the athletic facility site. Therefore, it will be necessary to develop a connection between the bridge and building that meets American with Disabilities Act (ADA) regulations. Ground level and elevated connections will be considered.

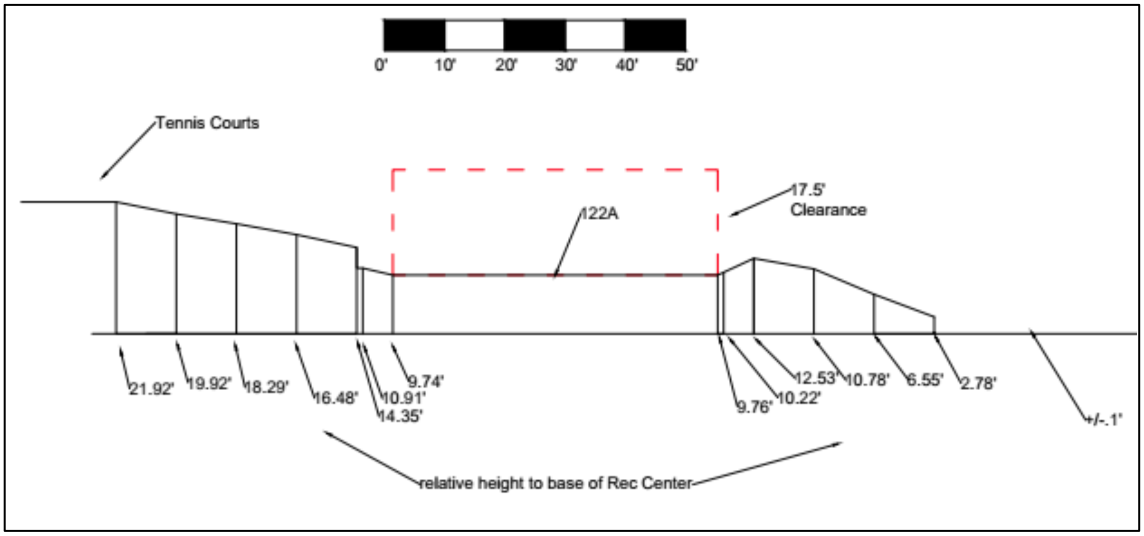

Figure 20: Cross-Section of Current Site Ground Conditions.

The space for the pedestrian connection bridge, as seen in Figure 19, is currently occupied by a concrete sidewalk, black chain-link fence, and a small, unused portion of the athletic track. The elevation of the track walkway remains approximately level throughout its entire length and elevates as it approaches Park Avenue at a retaining wall and sloped hill. The bridge height will be adequate for the usage of the walkway and track area to remain unchanged. However it will be necessary to place structural piers and footings along the walkway and hill to support the pedestrian bridge. These piers will be placed so as to minimize their impact.

According to the United States Department of Agriculture, the selected site sits on the border of two soil survey zones. One zone consists of 90% Paxton fine sandy loam, and the other is comprised of 80% udorthent soil over loamy basal till (Taylor 1985). Udorthent soils are a gravelly topsoil that have been placed back on site following an excavation. This site is more than 80 inches above the water table and is not flooding prone (Taylor, 1985). This information proved useful in determining the type, size, and design of foundation elements for both the athletic facility and pedestrian bridge.

While the site is adequate for the construction of the new facility, several potential alterations have been identified. One potential alteration to the site could include the addition of an access road and parking lot for the proposed building. The facility will be accessible via the pedestrian bridge, but it will also be necessary to provide parking for vehicles. This addition will require regrading of the site in order to tie in with Route 122A. Regrading the site could lead to potential issues with storm water runoff down the access road and onto Park Avenue. For this reason, during the development of the site plan for the facility, the drainage and runoff from the new facility was taken into consideration, but the design of these components was not in the scope of the project. Given the current location of existing buildings, their elevations, and the available space, the proposed site for the construction of the new athletic facility is a valuable opportunity to expand and connect WPI's campus with minimal required site alterations.

4.2 Athletic Facility Architectural Program and Design Strategy

An interview with WPI Athletic Director, Dana Harmon, was conducted in order to develop an architectural program and intended usage for the proposed building. The interview's primary take-away was the need for space to accommodate the growing student body and faculty. In order to solve this, the new facility will provide varsity athletes with the space they require for training, weight-lifting, and stretching thus freeing up space in the current Sports and Recreation Center for the rest of the WPI community. Director Harmon suggested relocating the athletic training room, providing meeting space for teams, and providing indoor training space. The layout of the building is provided in Figure 21 and Figure 22.

This project aimed to increase the training space available to WPI varsity athletes and free up space in the already-crowded Sports and Recreation Center. The new facility requires space for strength training, athletic training, team-specific meeting space, and a large open area to accommodate athletic events and practices. For this reason, the new structure will need a large open space without columns and minimal columns throughout the remainder of the building. The need for large spans creates large girder sizes. In order to reduce the self-weight of the roof members, a Warren truss was used to support the roof. While the truss is deeper than potential roof girders, the reduction in total weight allowed for smaller structural columns.

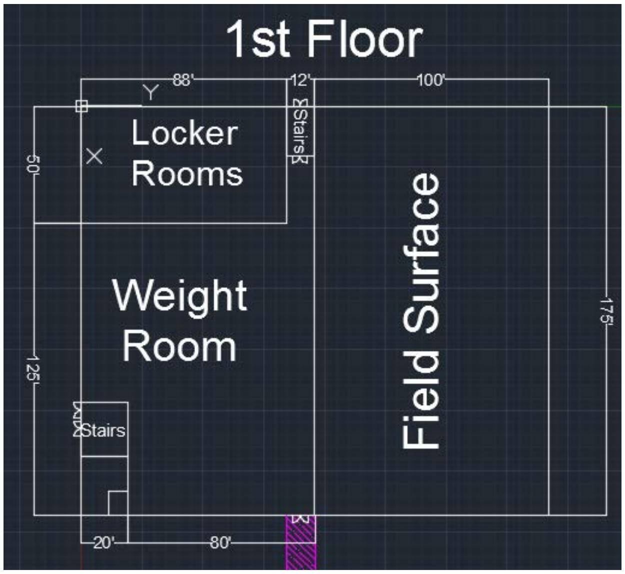

Figure 22: Proposed Athletic Building Floor Plan, 1st Floor

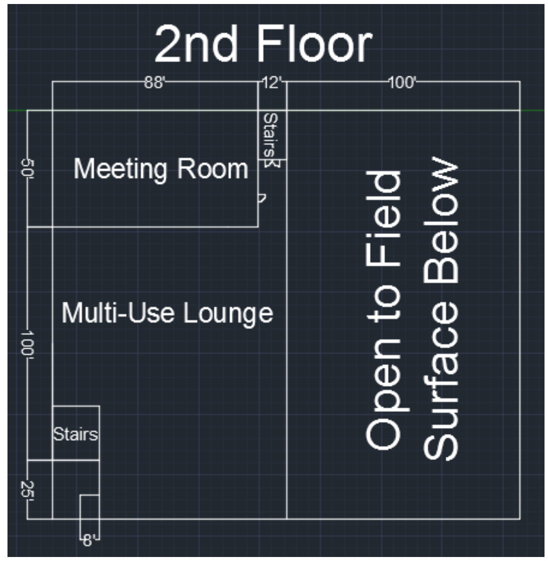

Figure 21: Proposed Athletic Building Floor Plan, 2nd Floor

The space layouts, shown in Figure 21 and Figure 22 allow for large structural columns directly down the centerline of the building and along the perimeter. The large open space must be two stories tall to accommodate athletic uses. The average ceiling height of an athletic gymnasium is 24 ft (Education Facilities and Specifications, 2012); however, in order to provide flexibility in the usage of the space, the large field space will have 30-ft ceilings. The structural columns in the facility will span from the foundation to the ceiling in order to simplify the erection process. Lateral bracing was specified at the end-bays along each side of the building. This will allow for the necessary lateral reinforcement and eliminate any diaphragm torsion caused by wind and seismic loads.

The usages of each room control the various design loads that were considered within the spaces. The weight room facility will be located on the 1st floor of the building to allow the added load from the weights and equipment to be supported by a slab on grade. This allows for the space on the 2nd floor to be utilized for lighter loading, such as athletic training and meeting space. A facility such as this should maximize the amount of functional floor space to increase the number of potential activities it can house, and the design decisions were made accordingly. Lastly, the architectural finishes of the building were defined to match those of the current Sports and Recreation Center, as seen in Appendix B.

4.3 Pedestrian Bridge Architectural Program and Design Strategy

The project aimed to expand and connect the WPI campus by improving accessibility and safety through the design of the pedestrian bridge. Given that this bridge expands the reach of the campus, it is critical that the final architectural layout blends the new athletic facility into the rest of campus. The architectural finishes for the bridge must also match the ones shown in Appendix B. A technical institution, such as WPI, requires that structures be modern, efficient, and be of high quality. For this reason, the bridge is sheathed with tempered glass and supports solar photovoltaic modules on its slanted roof. The roof is slanted in a manner that allows for maximum solar panel production, given the solar irradiance experienced by the location. However, given the bridge's location over a busy highway, its proximity to varsity athletic fields, and exposure to the harsh New England climate, it is important that the materials chosen for the design provide durability and longevity. The bridge is laid-out in order to accommodate two lanes of pedestrian traffic for individuals coming to and from the new athletic facility. The width of the bridge, per 2010 ADA specifications, must be a minimum of 7 ft. Space below the sloped roof as well as under the flooring is designated for the mechanical, electrical, and plumbing (MEP) required to make the transition between buildings seamless. A cross-section of the bridge superstructure can be found in Figure 23.

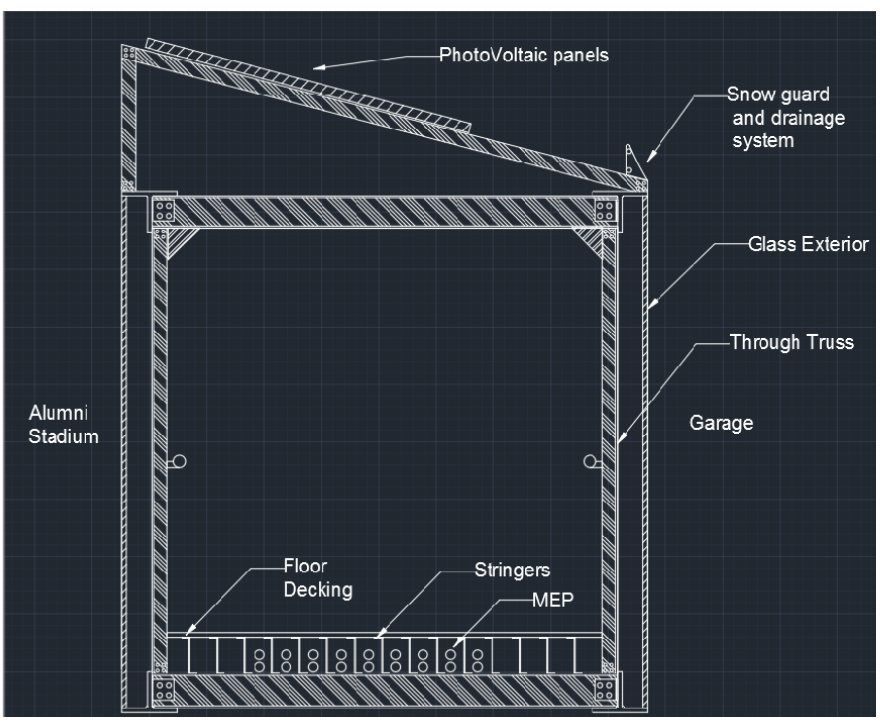

Figure 23: Proposed Pedestrian Bridge Cross-Section

The bridge was designed using a through truss because this allows for the bridge to be enclosed on all sides and supports the weight of both the pedestrian traffic and the roof. The through truss is connected to the lateral force resisting system, located at the top and bottom of the cross-section, using fixed connections to resist racking distortion, or lateral sway, of the superstructure. The truss design also allows for long spans, which is required in order to minimize the impact of structural piers on the area below. The final layout of the support piers can be found in Section 6.4. Additionally, roof drainage will be critical for this bridge given its location. To allow for proper drainage, the roof is angled in two directions. As it spans over a State Highway, mitigating rain and snow falling onto the road below is of the utmost importance. For this reason, drainage gutters and snow guards were defined along the roof of the bridge to allow for water runoff and prevent snow from falling from the bridge onto the cars below.

4.4 Bridge Connection Design Strategy

Both energy efficiency and student safety are major concerns of colleges and universities in current times. In order to ensure that both of these factors are properly addressed, vestibules that are accessed via a WPI ID card are provided at both ends of the bridge. However, in addition to safety and energy concerns, the connections between structures are critical because they are being tied into currently existing locations.

The connection of the pedestrian bridge to the new athletic facility is located on the first floor and was not connected structurally to eliminate additional loading on the athletic facility. The bridge is supported by a square concrete pier at the proposed athletic facility and structural piers were used throughout the rest of the bridge span. Factors such as cost, constructability, and the effect on the functionality of the facility were considered when making the design selection.

The bridge connects to the southeast side of the cantilevered viewing station attached to the Sports and Recreation Center. However, the cantilevered design of this structure raises concerns with making a load-bearing connection at this location. A structural pier was placed adjacent to the south wall of the Sports and Recreation Center. This pier carries the load of the bridge. The northwest side of the view station’s third-floor curtain wall must be removed to allow for access to the bridge. Matching glass is required to enclose the walls of the bridge to ensure continuity across the entire facility.

5.0 Design of Athletic Facility Building

This section contains the results of the structural design and analysis of the proposed WPI Athletic Building. The design was performed using hand calculations and Microsoft Excel spreadsheets, while referencing the *14th Edition of the AISC Steel Construction Manual, ASCE 7-10 standards*, and the *8th Edition of Massachusetts State Building Code*. The structural design of this building includes steel trusses, girders, beams, columns, lateral bracing systems, base plates, and concrete footings and pedestals. A general grid was created to help with structural member layout and calculations. The truss layout imposed on this grid can be found in Figure 24.

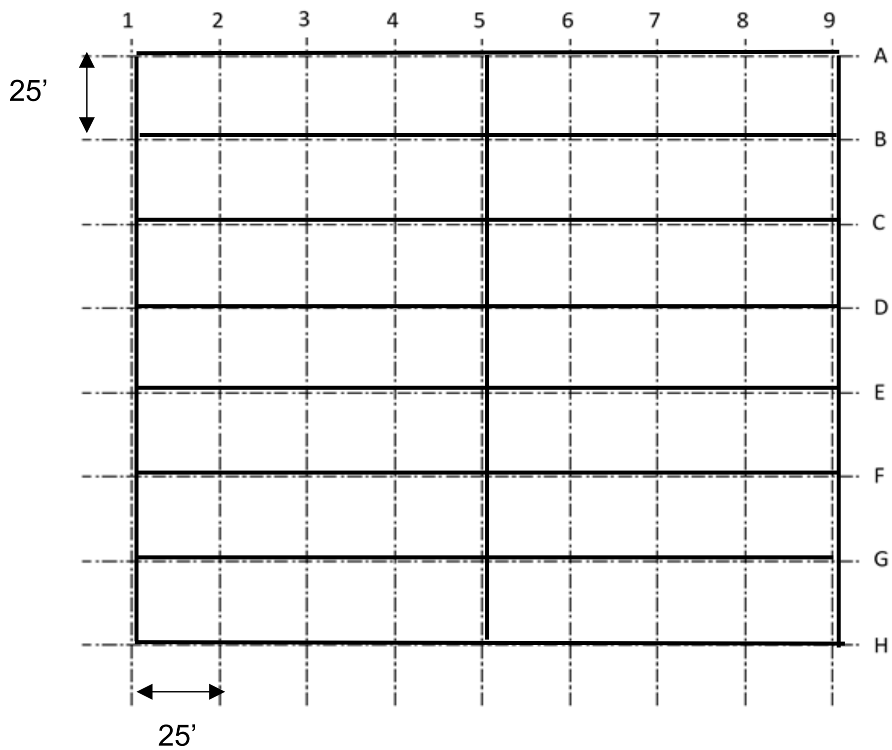

Figure 24: Proposed Building Alphanumeric Building Grid

5.1 Design of Building Roof Truss

The first structural component of the building designed was the roof system. Both a beam and girder system and roof truss system were considered, but due to cost and weight it was determined that a roof truss system was the better structural system for this facility. Appendix E contains the hand calculations for the roof truss design calculations and the beam-and-girder calculations. Each roof truss spans 100 ft in length with a height of four feet and ten panels that are each ten feet long. There are 16 trusses total in the roof system and each are 25 ft apart from one another. The vertical lines in the figure do not represent any structural members, they are used to depict where each truss begins and ends. At each end of each roof truss, there is a structural column that extends from the first floor to the roof that the roof truss connects to. Horizontal, vertical, and diagonal members were designed in order for the truss to support the various roof loadings. A diagram of one panel in the roof truss is shown in Figure 25. During the design of the roof truss system, key assumptions were made. These assumptions can be seen below in Table 7. The various member types and quantities for the roof truss can be found in Table 8 and the member schedule for the beam-and-girder roof system can be found in Table 9.

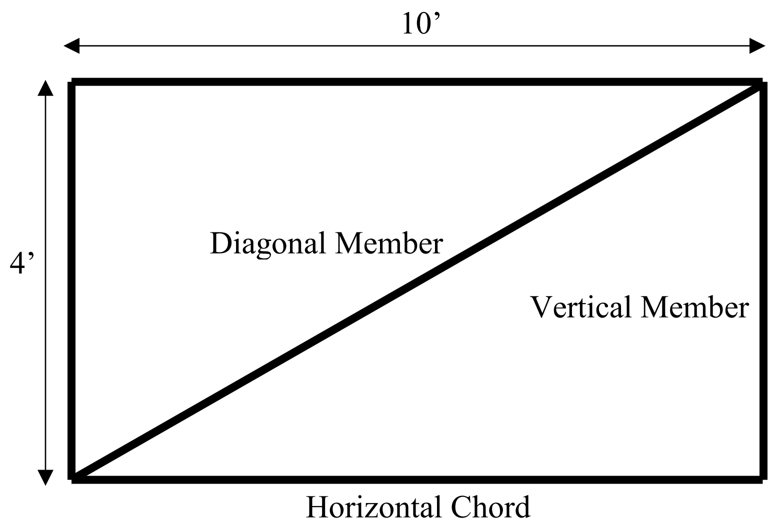

Figure 25: Truss Panel Diagram

Table 7: Building Roof Truss Key Assumptions

Key Building Calculation Assumptions	
Roof Truss	
Roof deck loading	10 psf
Insulation loading	2 psf
MEP loading	5 psf
Ceiling loading	3 psf
Snow loading	42.4 psf
Roof live loading	20 psf
Load Combination	1.2D + 1.6(Lr or S or R)
Truss	
Length	100 ft
Height	4 ft
Total Dead Load	20 psf + Truss self-weight
Diagonals	
Number per truss	10
Length	10.77 ft
Verticals	
Number per truss	11
Length	4 ft
Horizontal Chords	
Number per truss	20
Length	10 ft

Table 8: Truss Roof Member Schedule

Member Type	Size	Member Length	Quantity	Material
Horizontal Chord	WT 9 x 71.5	10 ft	20	A992 Steel
Vertical Member	LL 3 x 2.5 x 3/16	4 ft	11	A36 Steel
Diagonal Member	LL 4 x 4.0 x 3/4	10.77 ft	10	A36 Steel

Table 9: Beam-and-Girder Roof System Member Schedule

Member Type	Size	Member Length	Quantity	Material
Interior Beam	W12x16	25 ft	273	A992 Steel
Exterior Beam	W12x16	25 ft	14	A992 Steel
Interior Girder	W40x593	100 ft	12	A992 Steel
Exterior Girder	W12x22	25 ft	16	A992 Steel

5.2 Design of Building Second Floor

Following the design of the roof truss, the second floor beam and girder system was designed. This system is only present on one half of the building, for the other half is an open space that spans both levels. A portion of the beam and girder system layout is shown in Figure 26. Four types of members were designed, and the location of these typical members is shown in Figure 26 as well. Key assumptions made in the beam and girder system calculation can be seen in Table 10. The various member types and quantities can be found in Table 11.

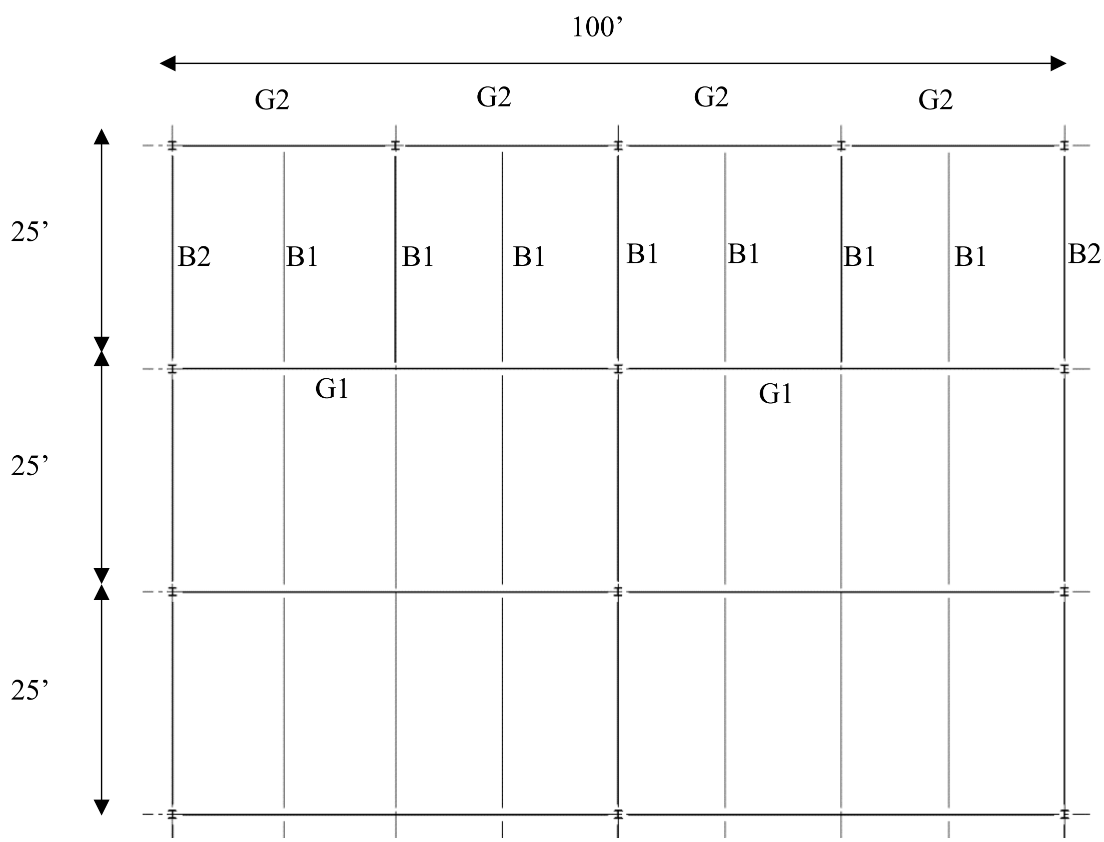

Figure 26: Second Floor Beam and Girder Layout (Figure Does Not Show Entire Building)

Table 10: Building Second Floor Beam and Girder Key Assumptions

Key Building Calculation Assumptions	
Second Floor Beam and Girder Framing	
Insulation loading	2 psf
MEP loading	5 psf
Ceiling loading	3 psf
Load combination	1.2D+1.6L
Resistance Factor (Φ)	0.9
Yield Strength of Steel (F_y)	50 ksi
Modulus of Elasticity (E)	29,000 ksi
Internal beam (B1)	
Tributary width	5 ft
Length	25 ft
External beam (B2)	
Total dead load	20 psf
Tributary width	2.5 ft
Length	25 ft
Internal Girder (G1)	
Length	100 ft
Tributary width	25 ft
Dead load	20 psf + 19 * B1 Self-weight
External Girder (G2)	
Length	25 ft
Tributary width	12.5 ft
Dead load	20 psf + 4 * B1 Self-weight

Table 11: Second Floor Beam and Girder Schedule

Member Type	Size	Member Length	Quantity	Material
B1	W 14x30	25 ft	49	A992 Steel
B2	W 12x19	25 ft	14	A992 Steel
G1	W 30x99	50 ft	12	60 ksi Steel
G2	W 14x30	25 ft	8	60 ksi Steel

5.3 Design of Building Columns

Following the design of the roof truss and second floor beams and girders, columns were designed to support the various loadings established by the usage of the facility and snow loadings as well. The layout of the columns is shown in Figure 27. In all, nine total column types were designed and analyzed, and all member sizes were relatively similar. The calculations for each of these members can be found in Appendix E. However, one typical column was utilized within the structure to make the erection process much simpler. While only one typical column size was used, there are varying lengths used throughout the building. Key assumptions for all column design calculations can be found in Table 12. The quantity of each length in Table 13. The loading cases vary between columns, these values can be found in Appendix E.

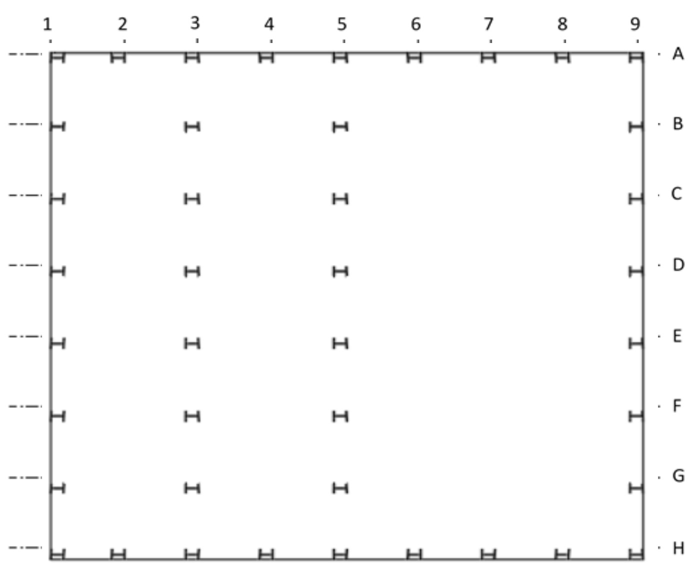

Figure 27: Column Layout

Table 12: Building Columns Key Assumptions

Key Building Calculation Assumptions	
Columns (Beam and Girder)	
Roof deck Loading	10 psf
Insulation Loading	2 psf
MEP Loading	5 psf
Ceiling Loading	3 psf
Bar Joist Loading	10 psf
Snow Roof Live Load	42.4 psf
Roof Live Load	20 psf
Load Combination	1.2D+1.6L+0.5(S or L_r or R)
Resistance Factor (Φ)	0.9
Yield Strength of Steel (F_y)	50 ksi
Modulus of Elasticity (E)	29,000 ksi

Table 13: Schedule of Columns

Member Size	Member Length	Quantity	Material
W 12 x 53	30 ft	36	A992 Steel
W 12 x 53	15 ft	6	A992 Steel

5.4 Design of Building Lateral Reinforcement System

The Building was designed to sustain and resist lateral and vertical wind and earthquake loading scenarios. The building resists lateral forces by utilizing lateral load resisting frame systems located symmetrically in its four corners. Each corner has two lateral load resisting frames, each resisting lateral loads in perpendicular directions to each other. The frames use diagonal and horizontal bracing elements to resist the lateral loads. Wind and seismic loads were calculated using ASCE 7-10 standards. The key assumptions for the seismic and wind load calculations can be seen in Tables 14 and 15, respectively. A typical frame was then input into RISA 2D to analyze the effects of the load combinations on the frame, including deflections, joint reactions, and story drift. The design of a typical lateral force resisting frame for earthquake and wind forces can be seen in Figure 28 and Figure 29, respectively, and member sizes can be found in Table 16.

Table 14: Building Seismic Calculation Key Assumptions

Key Building Calculation Assumptions	
Building Seismic Design	
Site Class	D
S_S	0.24 (Massachusetts State Building Code, 2010)
S_1	.067 (Massachusetts State Building Code, 2010)
Seismic Design Category	B
Risk Category	II
Seismic Force at Level 3 per frame	30.09 kips
Seismic Force at Level 2 per frame	18.84 kips

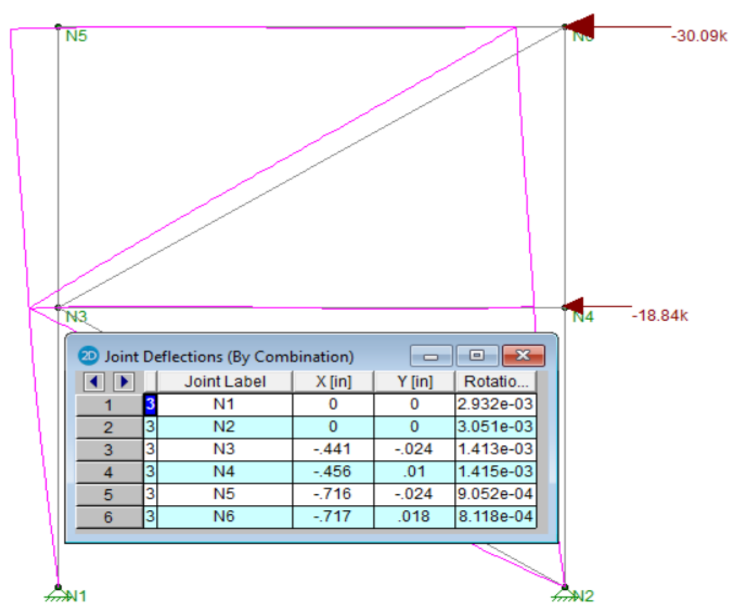

Figure 28: Lateral Load Resisting Frame for the Athletic Facility with Earthquake Loading

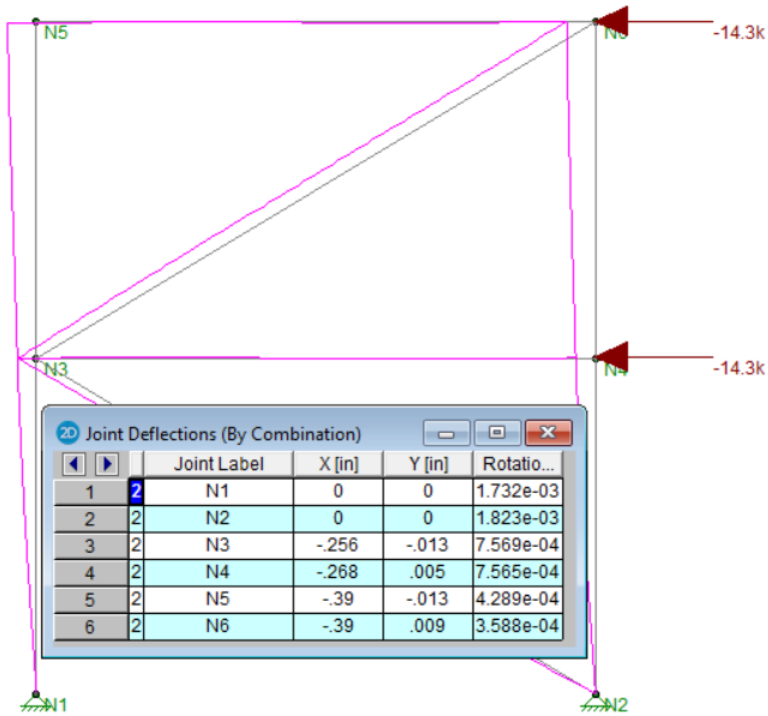

Figure 29: Lateral Load Resisting Frame for the Athletic Building with Wind Loads

Table 15: Building Wind Design Key Assumptions

Key Building Calculation Assumptions	
Building Wind Design	
Velocity Pressure Coefficient (q_z)	$0.00256k_zk_{zt}k_dV^2$
Exposure Category	B
Reference Wind Speed	100 mph

Table 16: Schedule of building lateral members

Member Type	Member Size	Quantity	Material	Length
Horizontal	L 3 x 2 x 1/2	16	A36 Steel	25 ft
Diagonal	L 3 x 2 x 1/2	16	A36 Steel	30 ft

5.5 Design of Building Footings

As a final step in the structural design all of the athletic facility, the baseplates, pedestals, and footings were designed to support the entirety of the loads carried throughout the building previously established. The layout and dimensions of the footing components can be seen in Figures 30 and 31. In all, nine different footings were designed for the nine different columns but some of the footings were similar in size due to the similarity in the column sizes and loads. The calculations for each of the different individual footings can be found in Appendix E.

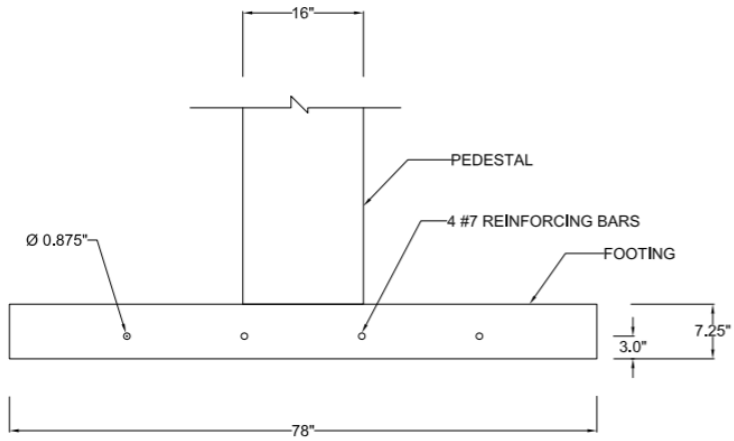

Figure 30: Athletic facility typical footing and rebar cross-section

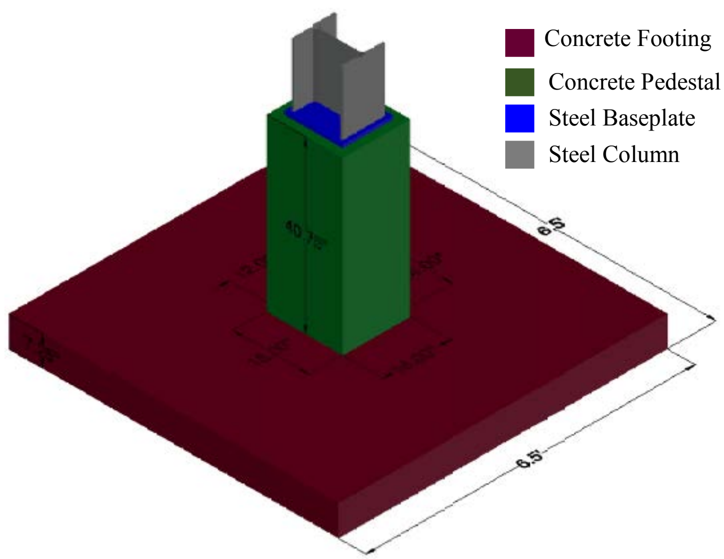

Figure 31: Building Footing Design

However, due to the variability in load for each column and the fact that each column will utilize a similar W12x53 section for simplicity in erection, footings were standardized for simplicity and efficiency in construction. As a result, there is only one footing design used in the complete design of the athletic building. The designed footings were the minimum size in order to be able to fully support the loading and column. Table 17 displays the dimensions of the standard footings, baseplates, and pedestals. Table 18 shows the design results of the concrete reinforcement in the building footing.

Table 17: Schedule of Footings

Component	Dimensions	Quantity	Material
Baseplate	14.00" x 10.00" x ¾"	42	A36 Steel
Pedestal	18.00" x 14.00" x 40.75"	42	4 ksi Concrete
Footing	6.50' x 6.50' x 7.25"	42	4 ksi Concrete

Table 18: Schedule of Footing Rebar

Component	Bar Area	Quantity	Yield Strength
Footing	2.4 in^2	4	50 ksi

5.6 Design of Elevator Hoist Beam

A multipurpose elevator capacity of 4500 lbs. and empty self-weight of 4500 lbs. were assumed for the design. In addition, the weight of a 500 lb. maintenance hoist was included in the design. The hoist way walls were assumed to be constructed of concrete masonry units with a 2-hr fire rating. These masonry walls surrounding the elevator are responsible for supporting the elevator hoist beam. Following completion of the design calculations, an A992 steel W8x15 beam was employed to support the elevator hoist. This beam does not connect to the building's roof truss and does not add to the overall load it carries. The key design assumptions for the elevator hoist beam can be found in Table 19.

Table 19: Elevator Hoist Beam Key Assumptions

Elevator Hoist Beam Key Assumptions	
Elevator Capacity	4,500 lb.
Elevator Weight	4,500 lb.
Hoist Weight	500 lb.
Yield Strength	50 ksi
Resistance factor (Φ)	0.9
Modulus of Elasticity	29,000 ksi

6.0 Design of Pedestrian Bridge

The following section describes the design of the pedestrian bridge, which spans from the current Worcester Polytechnic Institute (WPI) Sports and Recreation Center to the proposed athletic facility, spanning over a portion of the track and field area as well as Park Avenue. It discusses how the different structural members of the pedestrian bridge were designed using the Load and Resistance Factor Design (LRFD) Guide to the Design of Pedestrian Bridges as well as referencing The American Association of State Highway and Transportation Officials (AASHTO) LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals.

6.1 Design of the Bridge Through-Truss

The bridge utilizes a through-truss structural system to span the needed 75 ft between the supporting structural piers. The through-truss was designed to support the wind and seismic loads, live loads, and dead loads acting on the bridge members. The bridge is supported by a two trusses, one on the North face and one on the South face of the bridge. Each panel of the through-truss was designed to be 7.5 ft in length with 10 panels needed for each 75 ft bridge span. The full length of the bridge consists of five 75 ft spans, one span of 36.6 ft, and one span of 41.8 ft. The lateral loads due to wind and earthquake forces are supported by the lateral truss system detailed in Section 6.3. Key assumptions used in the design calculations of the through-truss bridge can be seen below in Table 20. A typical through-truss span can be seen in Figure 32 with Table 21 describing the member sizes and quantities. Hand calculations of the through-truss design can be seen in Appendix F.

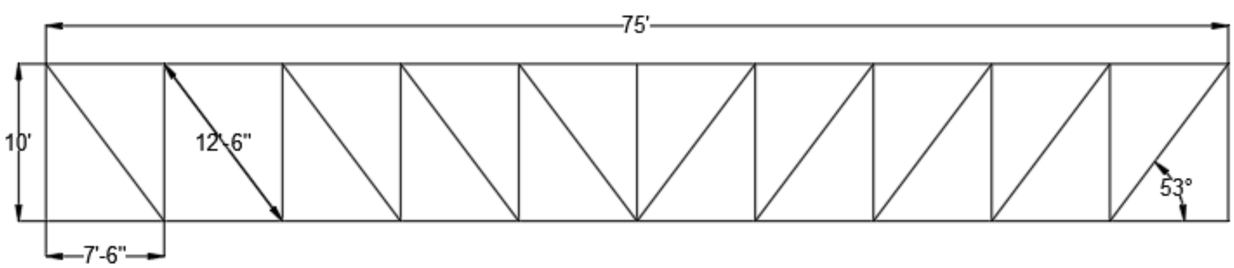

Figure 32: Typical Elevation View of Bridge Through Truss.

Table 20: Bridge Truss Key Assumptions

Key Bridge Calculation Assumptions	
Bridge Truss	
Roof frame loading	7.67 psf
MEP loading	5 psf
Solar Panel loading	4 psf
Roof loading	10 psf
Insulation loading	2 psf
Facade	125 lb/ft
Snow loading	42.4 psf
Pedestrian loading	90 psf
Gravity Load Combination	1.2D + 1.6L + 0.5S
Yield Strength (F_y)	36 ksi
Modulus of Elasticity (E)	29,000 ksi
Truss	
Height	10 ft
Span Length	75 ft
Deck Width	7 ft
Diagonals	
Number per truss	10
Length	12.6 ft
Verticals	
Number per truss	10
Length	10 ft
Horizontal Chords	
Number per truss	20
Length	7.5 ft

Table 21: Schedule of Bridge Through Truss Members

Member Type	Size	Quantity	Material
Chords	WT 9 x 25	120	A992 Steel
Diagonals	LL 5 x 3 x 3/16	59	A36 Steel
Verticals	LL 3 x 2.5 x 3/6	62	A36 Steel

6.2 Design of the Bridge Roof Frame

The roof frame of the bridge was designed to support the roof structure as well as transfer the roof live loads, including the snow live load, to the joints of the through-truss bridge. The roof frame is angled in two directions to allow for proper roof drainage of snow and rain, in addition to considering solar energy absorption. The roof will be angled at 15 degrees toward the South facing side and 2 degrees in the East-West direction. The roof frames are located with a spacing of 7.5 ft to allow for proper connection to the through-truss at truss panel points as detailed in section 6.1. Purlins lay on top of the frames and run longitudinally, spanning the frames. These purlins help to support the roof decking and the solar panels. Two cross-sections of the bridge roof frame can be seen in Figure 33 and Figure 34. Figure 33 is a typical section, each span contains sections with the peak located at mid-span. The lowest point of these sections is located at each support pier to allow storm water to drain from the bridge roof down to grade level. Key assumptions for roof frame calculations can be found in Table 22 and the member sizes can be found in Table 23.

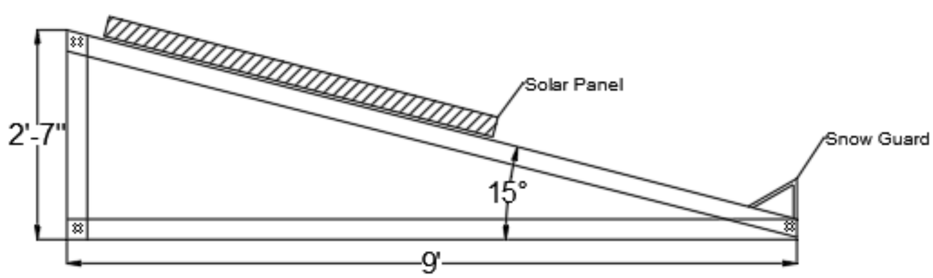

Figure 34: Cross-section of Bridge Roof Frame

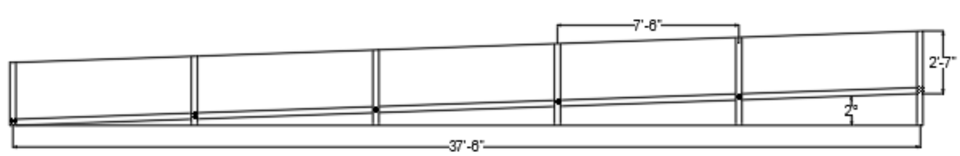

Figure 33: Bridge Roof Frame Section

Table 22: Bridge Roof Frame Key Assumptions

Key Bridge Calculation Assumptions	
Bridge Roof Frame Design	
Roof Sheathing Dead Load	10 psf
Solar Panel Dead Load	4 psf
Insulation Dead Load	2 psf
Snow Live Load	42.4 psf
Tributary Width	7.5 ft
Bridge Width	9 ft
Yield Strength (F_y)	46 ksi
Resistance Factor (ϕ)	0.9

Table 23: Schedule of Roof Frame Members

Member Type	Member Size	Quantity	Material
Vertical	HSS 5 x 4 x ½	62	A500 Steel
Diagonal	HSS 5 x 4 x ½	62	A500 Steel
Risers	HSS 5 x 4 x ½	37.84 ft	A500 Steel

6.3 Design of Bridge Lateral Truss

The bridge was designed to resist lateral loading due to wind forces with a lateral load resisting truss. The lateral truss was not designed to resist lateral loading due to earthquake forces due to the Seismic Design Category A classification. The procedure used to determine this can be found in Appendix F. This frame spans the entire length of the bridge and is mirrored on the top and bottom of the through-truss to prevent twisting and torsional forces on the bridge. The lateral truss connects to the through truss at each node with a fixed connection to resist torsional sway and to transfer the lateral loading to the bridge piers throughout the span at the end piers. The lateral load resisting truss was designed to limit the lateral deflections of the bridge between the pier spans to the deflection limit set by the *AASHTO LRFD Bridge Specification 5th Edition* Section 2.5.2.6.2 (AASHTO, 2014). Key assumptions for the lateral truss calculations can be found in Table 24 and the design of a typical lateral force resisting truss resisting wind forces can be seen in Figure 35. The model depicted displays only one half of the span. The half span still allows for maximum displacement to be calculated. Both ends are pinned using steel base plates bolted into the concrete pier caps.

Table 24: Bridge Lateral Truss Key Assumptions

Key Bridge Calculation Assumptions	
Bridge Lateral Truss Design	
Horizontal Wind Load	0.704 kips/foot of truss
Windward pressure distribution	75%
Leeward pressure distribution	25%
Lateral Truss Panel Length	7.5 ft
Wind Force Distribution	Loading occurs at truss nodes

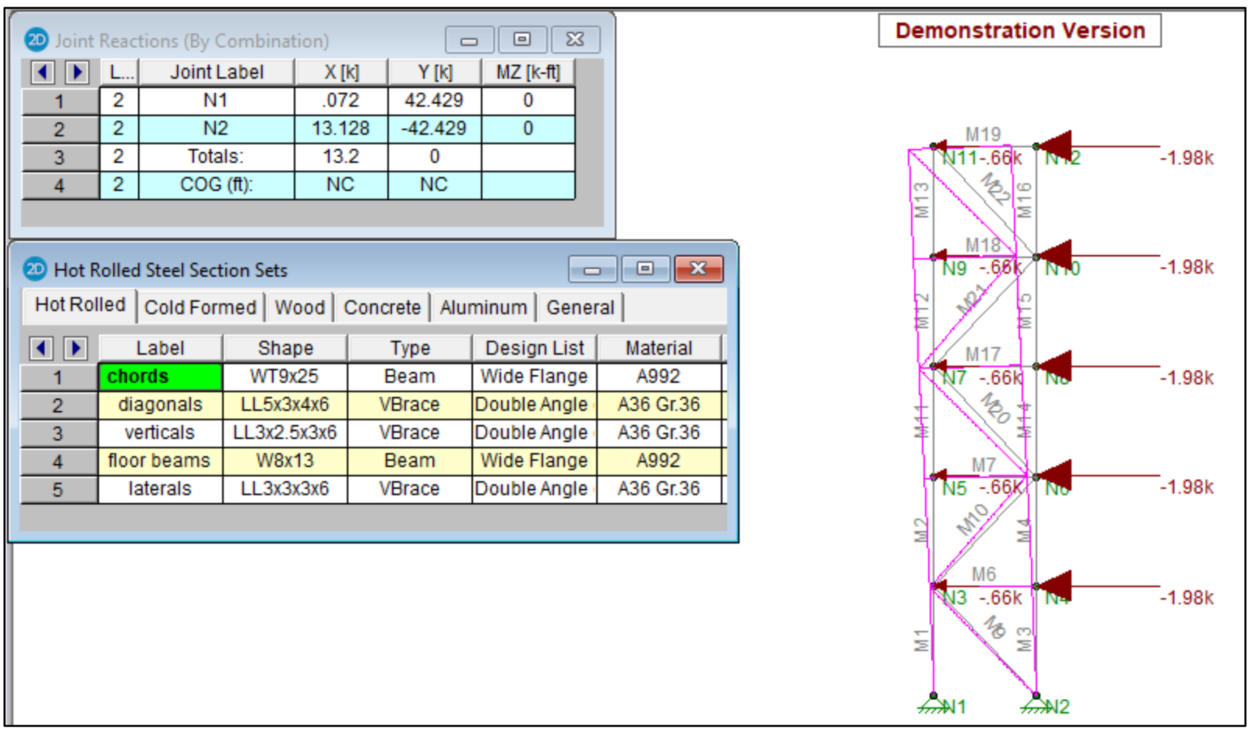

Figure 35: Lateral load resisting truss for half span of the pedestrian bridge with wind loads.

6.4 Design of the Bridge Pier Cap

At the top of the bridge piers is the pier cap. The cap connects the main bridge structure to the columns. The cap was designed to transfer gravity and lateral loads from the main bridge structure to the piers. It was designed as a reinforced concrete beam that cantilevers outward from the column and supports the bridge at the bottom chord of each of the two trusses. The reinforcement in the cap uses longitudinal bars and transverse stirrups to resist tension and shear respectively. The assumptions used to design the bridge pier caps can be seen in Table 25. The governing load combination makes use of only gravity loads. The results of the pier cap design can be seen in Table 26. The final design of the pier cap can be seen in Figure 36.

An alternative pier cap design was also considered. The pier cap was originally designed as cantilever beams extruding out radially from the pier. The beams were designed to support the load of the superstructure above at 4 points located in the 4 corners of the original pier at each bearing. An alternative design was considered that removed the concrete in between the 12 in cantilever beams that were supporting the bearings. This allows for less concrete and dead load acting on the pier and footing below. However, it was concluded that the cost benefits of saving the material and dead load would not outweigh the consequences associated with the time and money that would be needed to make this custom shaped formwork. This alternative design can be seen in Figure 37.

Table 25: Bridge Pier Cap Key Assumptions

Key Bridge Calculation Assumptions	
Bridge Pier Cap Assumptions	
Length	9 ft
Concrete Compressive Strength (f'c)	4 ksi
Reinforcing Bar Yield Strength (F_y)	60 ksi
Critical Load Combination	1.2D+1.6L+0.5S
Resistance Factor for Moment (ϕ)	0.9
Resistance Factor for Shear (ϕ)	0.75
Maximum Moment	738.91 kip-feet
Maximum Shear	165.51 kips

Table 26: Bridge Pier Cap Design Results

Key Bridge Calculation Results	
Bridge Pier Cap Design	
Length	120 in
Width	102 in
Depth	18 in
Height	21 in
Material	4 ksi Concrete
Radial Reinforcement	3 #10 bars per 12 in
Shear Reinforcement Size	# 4 stirrups
Shear Reinforcement spacing	1 stirrup @ 2.5 in 1 stirrup @ 18 in 3 stirrups @ 5.5 in

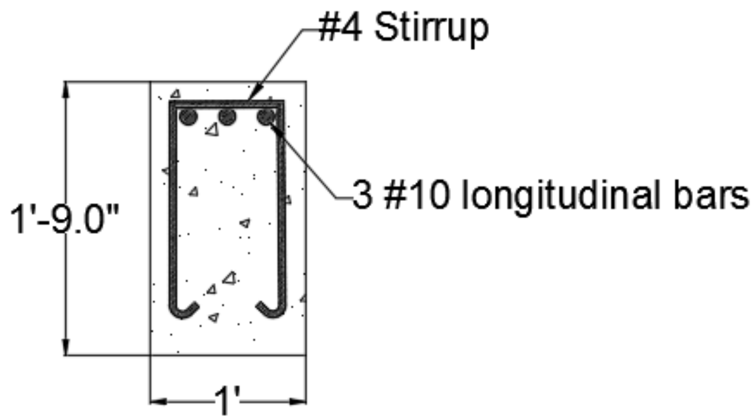

Figure 37: Pedestrian Bridge Pier Cap and Rebar Cross-Section

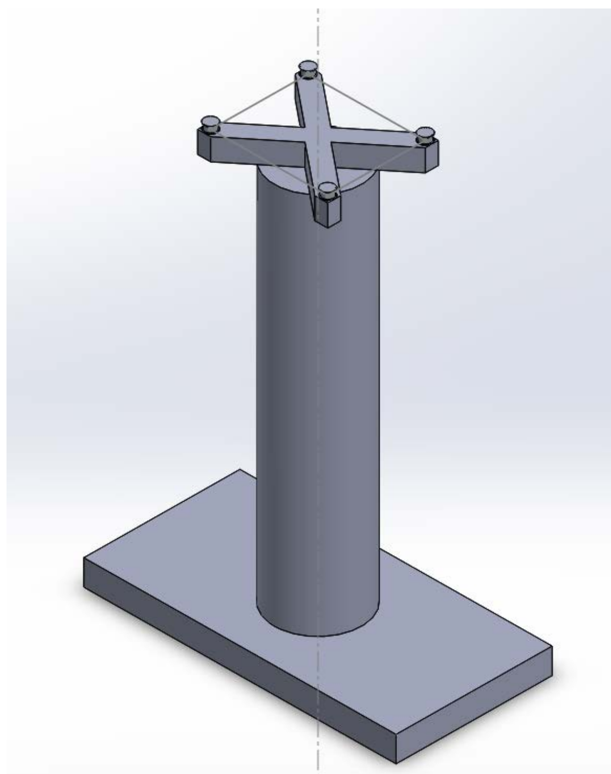

Figure 36: Alternative Pier Cap Design. (not used for final design)

6.5 Design of the Bridge Piers

The main bridge structure is supported by 7 reinforced concrete piers that are designed to resist the gravity loads of the pedestrian bridge as well as the lateral loads including wind and seismic, however, specific seismic provisions were not needed. The piers were designed to support a bridge span length of 75 ft. This represents the largest tributary length of the bridge that any single pier must support. To resist lateral loading in all directions, the piers were designed to be cylindrical. The symmetry of the piers ensures that load resistance in all directions is equal. The piers were designed using spiral lateral reinforcement and longitudinal bars to support the concrete in tension. The geometry of the bridge piers were determined by calculating the minimum pier radius to allow them to be designed as non-slender columns. Once the geometry was selected, the axial and moment capacity of the pier was checked, and the reinforcement was designed. The relative locations of the piers and their associated pier caps and pier footings can be found in Table 27 and Figure 38. The key assumptions used to design the bridge piers can be seen in Table 28. The results of the bridge piers can be found in Table 29. See Figure 39 for a typical bridge pier and reinforcement layout. A typical bridge pier, pier cap, and foundation can be seen in Figure 40.

Table 27: Pier Identification and Location.

Pier Identification	Pier Location	Pier Height (ft)
P1	Tennis Court side of 122A	11.3
P2	Garage side of 122A	15.9
P3	Tennis Court side of Alumni Field	27.8
P4	Middle of Track (75 ft from P3)	28.4
P5	Middle of Track (150 ft from P3)	28.8
P6	Recreation Center side of Track	29.5
P7	Adjacent to Recreation Center	29.7

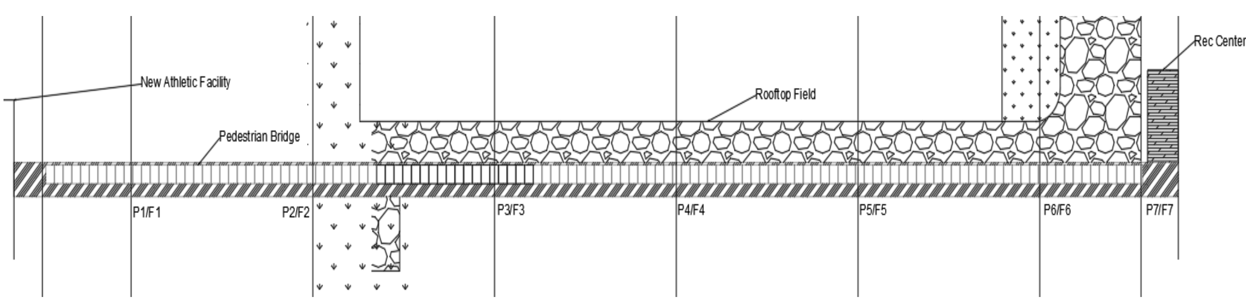

Figure 38: Pedestrian Bridge Pier/Footing Layout

Table 28: Bridge Pier Key Assumptions

Key Bridge Calculation Assumptions	
Bridge Pier Design Assumptions	
Length	30 ft
Concrete Compressive Strength (f'_c)	4 ksi
Reinforcing Bar Yield Strength (F_y)	60 ksi
Critical Load Combination	1.2D+1.6L+0.5S
Resistance Factor for Moment (ϕ)	0.9
Resistance Factor for Shear (ϕ)	0.75
Ultimate Compressive Strength (P_u)	320.4 kips
Ultimate Moment (M_u)	1520.6 kip-feet

Table 29: Bridge Pier Design Results

Key Bridge Calculation Results	
Bridge Pier Design Results	
Length (max)	30 ft
Diameter	80 in
Material	4 ksi Concrete
Longitudinal Reinforcement	19 #18 bars
Lateral Reinforcement	# 4 spiral @ S= 0.86
Clear Cover	4 in

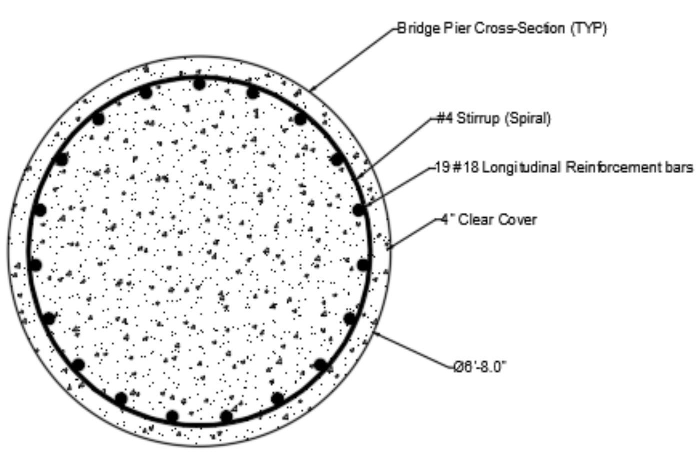

Figure 39: Pedestrian Bridge Pier and Rebar Cross-Section

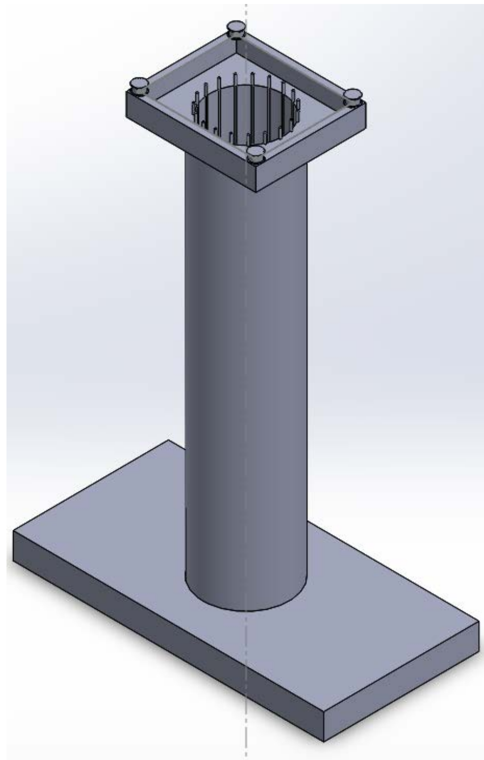

Figure 40: Typical Bridge Pier, Pier Cap, and Footing. (material removed to show reinforcement, actual pier cap and pier are solid concrete).

6.6 Design of the Bridge Footings

The bridge footings were designed to support the bridge piers and bridge superstructure, and to prevent the piers from settling into the soil. Spread pier footings were used to distribute the pier loads to the soil and limit settlement. Two footing designs were done using different assumptions based on the location of the footing and the loading acting on the footing. Footings designs were created using LRFD and methods from Reinforced Concrete Design. The locations of the bridge footings can be found in Table 30 and Figure 39. The assumptions used in the design of bridge footings 1 and 2 can be found in Table 31 and Table 32 respectively. The final results of the footing design can be found in Table 33. See Figures 41, 42, 43, and 44 for footing reinforcement layouts.

Table 30: Bridge Footing Locations

Bridge Footing Locations		
Footing Identification	Associated Footing Pier	Associated Footing Design
F1	P1	2
F2	P2	2
F3	P3	1
F4	P4	1
F5	P5	1
F6	P6	1
F7	P7	1

Table 31: Bridge Footings Design 1 Assumptions

Key Bridge Calculation Assumptions	
Bridge Footing Design 1 Assumptions	
Reinforcing Bar Yield Strength (F_y)	60 ksi
Concrete Compressive Strength (f'_c)	4 ksi
Axial Load	479.09 kips
Moment, M_1 (max)	1900.9 kips
Moment, M_2 (max)	960 kips
Pier Diameter	6.67 ft
Concrete Density	150 pcf
Pier Height	30 ft
Pier Tributary Width	75 ft

Table 32: Bridge Footing Design 2 Assumptions

Key Bridge Calculation Assumptions	
Bridge Footing Design 2 Assumptions	
Reinforcing Bar Yield Strength (F_y)	60 ksi
Concrete Compressive Strength (f'_c)	4 ksi
Axial Load	265.76 kips
Moment, M_1 (max)	1013.76 kips
Moment, M_2 (max)	506.88 kips
Pier Diameter	6.67 ft
Concrete Density	150 pcf
Pier Height	16 ft
Pier Tributary Width	40 ft

Table 33: Bridge Footing Design Results.

Key Bridge Calculation Results	
Bridge Footing Design Results	
Footing Design 1	
Length (Perpendicular to Superstructure)	22 ft
Base Width (Parallel to Superstructure)	14 ft
Depth	4.00 ft
Height	4.25 ft
Longitudinal Reinforcement (Parallel to Superstructure)	42 # 7 bars
Longitudinal Reinforcement (Perpendicular to Superstructure)	88 # 8 bars
Footing Design 2	
Length (Perpendicular to Superstructure)	20 ft
Base Width (Parallel to Superstructure)	12 ft
Depth	4.00 ft
Height	4.25 ft
Longitudinal Reinforcement (Parallel to Superstructure)	42 # 7 bars
Longitudinal Reinforcement (Perpendicular to Superstructure)	88 # 8 bars

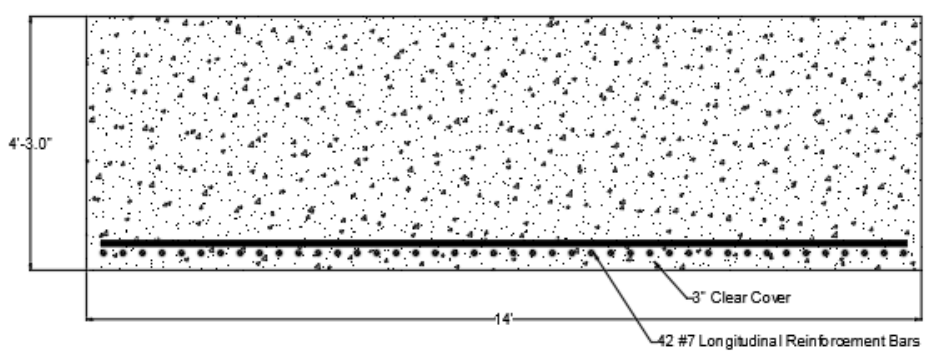

Figure 41: Pedestrian Bridge Footing Design 1, Reinforcement Layout Parallel to Superstructure

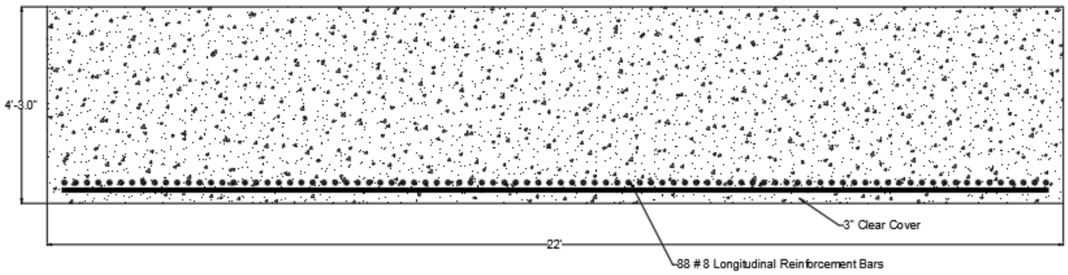

Figure 44: Pedestrian Bridge Footing Design 1, Reinforcement layout Perpendicular to Superstructure

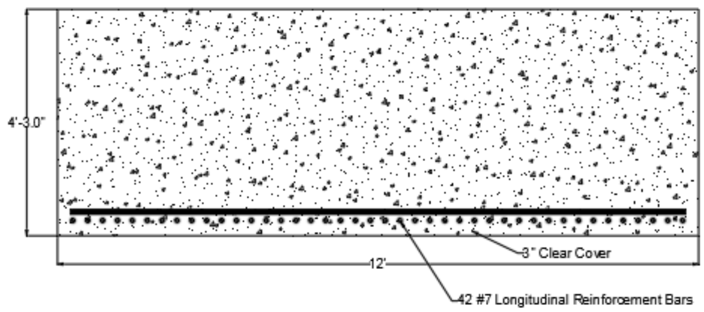

Figure 42: Pedestrian Bridge Footing Design 2, Reinforcement Layout Parallel to Superstructure

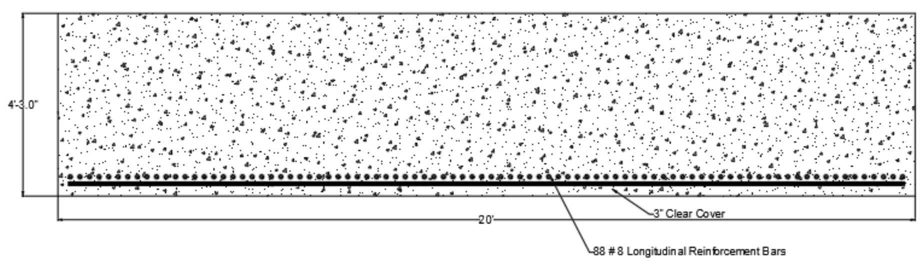

Figure 43: Pedestrian Bridge Footing Design 2, Reinforcement Layout Perpendicular to Superstructure

6.7 Design of Bridge Seismic System

Elastomeric bearings were designed to connect the bridge superstructure to the bridge pier caps. They are used to accommodate any rotation or movement the bridge may experience due to loading and/or thermal expansion. The bearings were designed to be circular to resist loading in all directions equally. To provide a stable connection between the pier caps and the bridge through truss, four bearings were placed on each pier. Each bearing connects to a bottom node of the bridge through truss so that each truss is supported by two bearings at each pier with a 7.5 ft spacing. The governing lateral load used to design the bearing was determined to be wind loading due the bridge Seismic Design Classification A. The results of the bearing design can be found in Table 34. The assumptions used to design the elastomeric bearings can be found in Table 35.

Table 34: Bridge Elastomeric Bearing Design Results

Key Bridge Calculation Results	
Bridge Elastomeric Bearing Design	
Total number of bearings	30
Bearing Shape	Circular
Bearing Flange	12 in
Bearing Diameter	10 in
Calculated Rotation	0.04 radians
Horizontal movement of bridge	1.0 in

Table 35: Bridge Elastomeric Bearing Assumptions

Key Bridge Calculation Assumptions	
Bridge Elastomeric Bearing Assumptions	
Dead load	63.7 kips
Live load	16.4 kips
Governing Lateral Load	Wind
Design method	B
Calculated Rotation	0.004 radians
Horizontal movement of bridge superstructure	1.0 in
Bridge deck fixed against horizontal translation	Yes
Bearing subject to shear deformation	Yes

6.8 Design of Bridge End Pier

A square concrete pier was designed for the end of the bridge closest to the new athletic facility. The purpose of this square pier is to resist the vertical and lateral loading on the bridge loading in the area where the bridge connects to the athletic building. It was a designed as a square, reinforced concrete column, and to ensure stability the pier was overdesigned to be a 9 ft by 9 ft square pier directly under the bridge. The assumptions used during the calculation of the bridge pier can be found in Table 36. The gravitational loads were determined to govern the design of this structure. For this reason, the load combination shown in Table 36 was chosen. The final results of the bridge end pier design can be seen in Table 37 and Figure 45.

Table 36: Bridge End Pier Assumptions.

Key Bridge Calculation Assumptions	
Bridge End Pier Assumptions	
Length	9 ft
Concrete Compressive Strength (f'_c)	4 ksi
Unit weight of concrete	150 lb/ft^3
Reinforcing Steel Yield Strength (F_y)	60 ksi
Minimum ratio of required steel to concrete (ρ_{min})	0.015
Critical Load Combination	1.2D+1.6L+0.5S
Resistance Factor (ϕ)	0.9
Ultimate Compressive Strength (P_u)	33.83 kips

Table 37: Bridge End Pier Design Results

Component	Dimensions/Bar Area	Quantity	Material
End Pier	9.00' x 9.00' x 8.00'	1	4 ksi concrete
Reinforcing Steel	4.00 in^2	44	A572 Grade 60 Steel

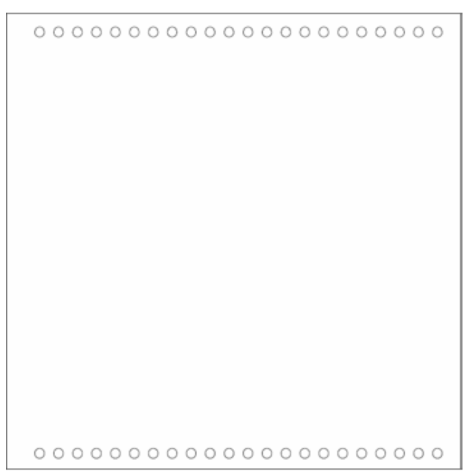

Figure 45: Pedestrian Bridge End Pier and Rebar Cross-Section Plan View

7.0 Cost Analysis

A basic cost analysis for the construction of both the pedestrian bridge and athletic facility was completed. The purpose of the analysis was to provide an estimate of the potential financial burden this project would place upon the Institution. This was done with the use of Building Construction Costs with RS Means Data as well as the Massachusetts Department of Capital Asset Management and Maintenance (DCAM) consultant estimating manual. The estimate includes both material and labor costs, as well as contingency and engineering fee considerations. An estimated contingency of 15% was applied to the final estimate, as well as a 7.2% design fee allowance. The results of the preliminary cost estimate are show in Table 38 and the complete analysis can be found in Appendix G.

Table 38: Results of cost analysis

Estimate Item	Cost
Athletic Facility (no design fees or contingency)	$7,700,000
Pedestrian Bridge (no design fees or contingency)	$2,500,000
Design Fees and Contingency	$2,400,000
TOTAL PROJECT COST	**$12,600,000**

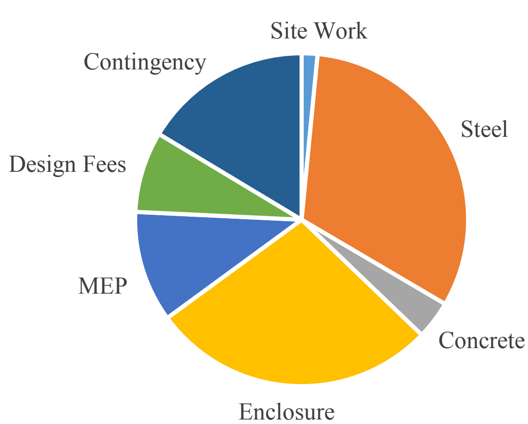

Figure 47: Pedestrian Bridge Material/Labor Cost Breakdown

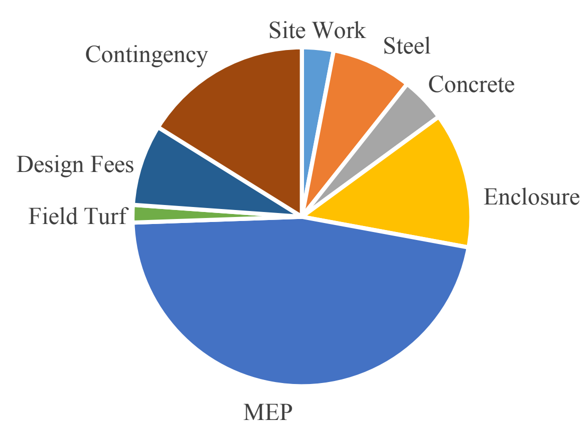

Figure 46: Athletic Facility Material/Labor Cost Breakdown

8.0 Conclusion and Recommendations

The following section discusses what was done over the course of the project to meet the goals originally set at the beginning of the project. This section will also discuss the recommendations that our group believes future groups could continue to work on. These recommendations are intended to offer future project ideas to students who are interested in advancing or improving the design set forth at the conclusion of this project.

8.1 Athletic Building Conclusions

The structural analysis, cost estimate, and 3-dimensional modelling of the facility was after considering initial design factors, site layout, and site criteria. After examining the criteria from the site survey, interview, and initial research, the building design followed Load and Resistance Factor Design methods. The structural analysis began with the roof members, moving down to the second floor, then to the columns, and finally to the base plates, pedestals, and footings. All members and components were designed as part of the larger system of the building.

Two different roof structures were considered during the design of the athletic facility: a typical beam-and-girder bay system and a roof truss system. The roof truss system was selected because it was a more efficient design due to weight considerations and its ability to have longer spans. Since the truss weighs less and uses less steel, it would have a lower material and shipping cost compared to the beam-and-girder system. The 2nd floor structural system was designed using a beam-and-girder bay system. Analysis included a comparison of a cast in place concrete slab and pre-cast hollow-core panks. This type of system was chosen due to the live loads on the 2nd floor and to decrease the depth of the floor system and maximize story height. The columns were individually designed for each different location and associated loading, but to facilitate construction and manufacturing, the column sizes were standardized based on the most critical case. The baseplates, pedestals, and footings were also designed in the same sense for ease in construction and standardization.

Once the individual components of the building were designed to be structurally sufficient, the components were compiled together to complete the design of the building. Using the results from the design of the components, a 3-dimensional rendering of the facility was created. Using this model, and some assumptions made about the architectural components, a cost estimate was performed. This cost estimate included considerations for materials (structural, architectural, MEP, and equipment) and construction of the facility. See Figure 44 for a rendering of the final Athletic facility Design.

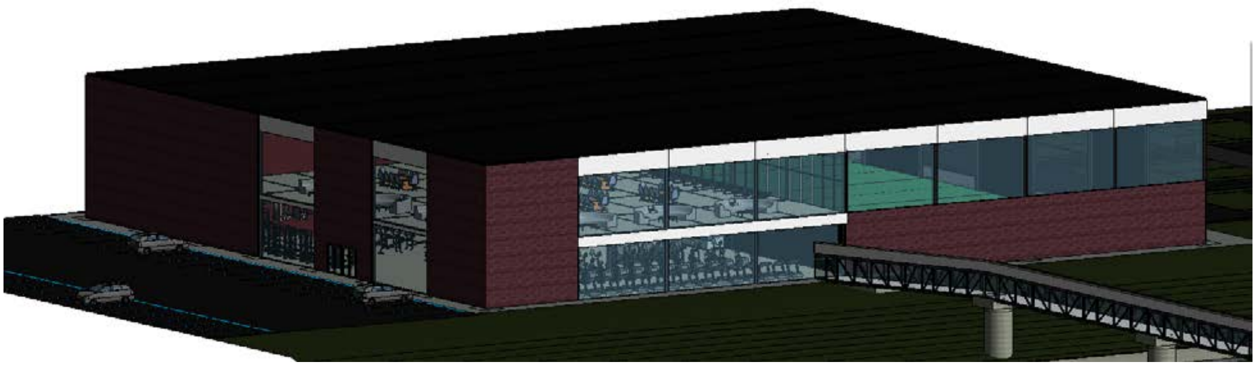

Figure 48: Athletic Facility REVIT Rendering

8.2 Athletic Building Recommendations

After completion of the project, we would recommend that this project be advanced. We believe that to advance the design of the proposed athletic facility, more focus should be placed on the design of member connections. The scope of our project verified that the geometry of the connections would be successful, as seen in Appendix H, however we did not consider the design of the angles, welds, or bolts for each individual connection. Also, another future project could advance the design of the facility by performing a fire safety analysis to explore the fire safety options or concerns that the current proposed design would entail. If the building was going to become reality it would also need to be subject to an architectural review including an energy analysis and the design of the façade, lighting, HVAC, heating, electrical, plumbing, and interior aesthetics. The facility would also need to have a parking lot and water runoff management system designed by a civil or environmental engineer.

8.3 Pedestrian Bridge Conclusions

The proposed pedestrian bridge was completed with an initial site survey and layout, determination of design criteria, structural analysis, cost analysis, and 3-dimensional modelling. The purpose of the pedestrian bridge was to provide a safe passage, across a state highway, from the main part of campus to the new proposed athletic facility. The bridge design followed both Load and Resistance Factor Design criteria and the AASHTO LRFD Pedestrian Bridge

Specifications design method. The structural analysis followed the load path and began with the roof frame, moving down to the through-truss, then to the bridge piers and pier caps, and finally to the footings.

A roof frame design for the pedestrian bridge was developed after the initial design was created. It was designed to slope in 2 directions to allow storm water and snow melt to drain effectively off of the bridge. Multiple bridge types and material options were researched originally, but due to aesthetic and economic reasons, a through-truss bridge design was used. The bridge piers, pier caps, and footings were individually designed, but similar to the building design they were standardized based on the most critical case to simplify construction and procurement. Once the final design of each bridge component was completed, the 3-dimensional model was created to show how each component worked together as a system. The 3-dimensional model was also used as a graphical representation during the presentation of our final design and results. See Figure 45 for the rendering of the pedestrian bridge.

Figure 49: Pedestrian Bridge Rendering, View from Track

8.4 Pedestrian Bridge Recommendations

For both the building and pedestrian bridge, design considerations were discussed in this project for the storm water runoff and drainage system. While the athletic facility clearly presents a larger storm water problem due to the size of the impermeable layer it creates, run-off from the Pedestrian Bridge should also be investigated. For a structure such as a bridge there are numerous means of structural design, we recommend that various bridge types be investigated for this application. Although we ultimately decided to use the through-truss bridge type for our design, it would be possible for other bridge types to be designed and compared in price and aesthetics. Based on the span of the bridge both a cable-stay and suspension bridge would be potential alternatives. Additionally, when designing the pedestrian bridge, some construction methods and constraints were explored, we recommend that a future project could design a fully encompassing construction and erection plan.

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Appendix A: Project Proposal

WPI Pedestrian Bridge and Athletic Building: Structural Design

Major Qualifying Project Proposal

Presented by:

Liam Beal, Joseph Berger, Conor Hoey, & Kaitlin Travers

Presented to:

Professor Leonard Albano and the Worcester Polytechnic Institute Civil Engineering Department

2017-2018

1.0 Introduction

A bridge can be defined as a structure spanning and providing passage over a river or road (Merriam Webster Dictionary, 2017). But a bridge can be more than that. A bridge can connect people, ideas, and cultures. Currently, part of our Worcester Polytechnic Institute (WPI) community and campus is disconnected from the rest. A state highway with limited pedestrian access divides the WPI campus into fragments. This is not only a culturally divisive barrier, but also a major safety concern. Students, faculty, and guests may have to navigate across this dangerous road in order to explore the full extent of the campus.

Our team is proposing and designing an enclosed pedestrian bridge that would span the state highway 122A. The bridge will connect the current WPI Sports and Recreation Center 3rd floor to the proposed WPI athletic facilities, where the current A.J. Knight Field and tennis courts are located. The bridge would also connect to the rooftop field/garage allowing for better access between this facility and the Sports and Recreation Center. Along with access to these facilities, the bridge would allow WPI students and faculty to travel more easily from the main campus to the Hughes House, Jeppson House, and the Alpha Tau Omega Fraternity house. We believe safe access to these campus buildings as well as the numerous off-campus housing is a priority for students and parents and should be a priority for the WPI community.

As part of our project we will also propose and design the construction of a new athletic building located along Park Ave near the location of the current WPI tennis courts and the A.J. Knight Field. The purpose of the proposed athletic building is to provide additional facilities for the WPI community as a whole. On the first floor of the facility, an indoor athletic training field would be designed with elevated ceilings to be used for any indoor athletic drills/conditioning, training, and/or games. An open space area for strength training would also be included on the first floor to accommodate strength training equipment along with men's and women's locker rooms and restrooms. This could be used by WPI's various division 3 and club athletic teams year-round. Due to the limited athletic facilities that are now available, many athletic teams must reserve or share current gymnasium space with the general population of the WPI community. This can create a restrictive environment for athletic teams that need space to train and condition. With the addition of a new indoor athletic training facility, sports teams would have additional designated space to train, especially during the winter months when outdoor fields are not available.

Varsity sports at WPI often require the use of conference rooms to hold team meetings. Currently the space designated for sports teams to hold team meetings is very limited, amounting to one dividable room in the Sports and Recreation Center. This space usually is shared with and used by the WPI faculty for staff meetings. Our proposed athletic building also includes space for offices and conference rooms on the second floor. These rooms can be used for team meetings, coaches' meetings, and study hall rooms for student-athletes.

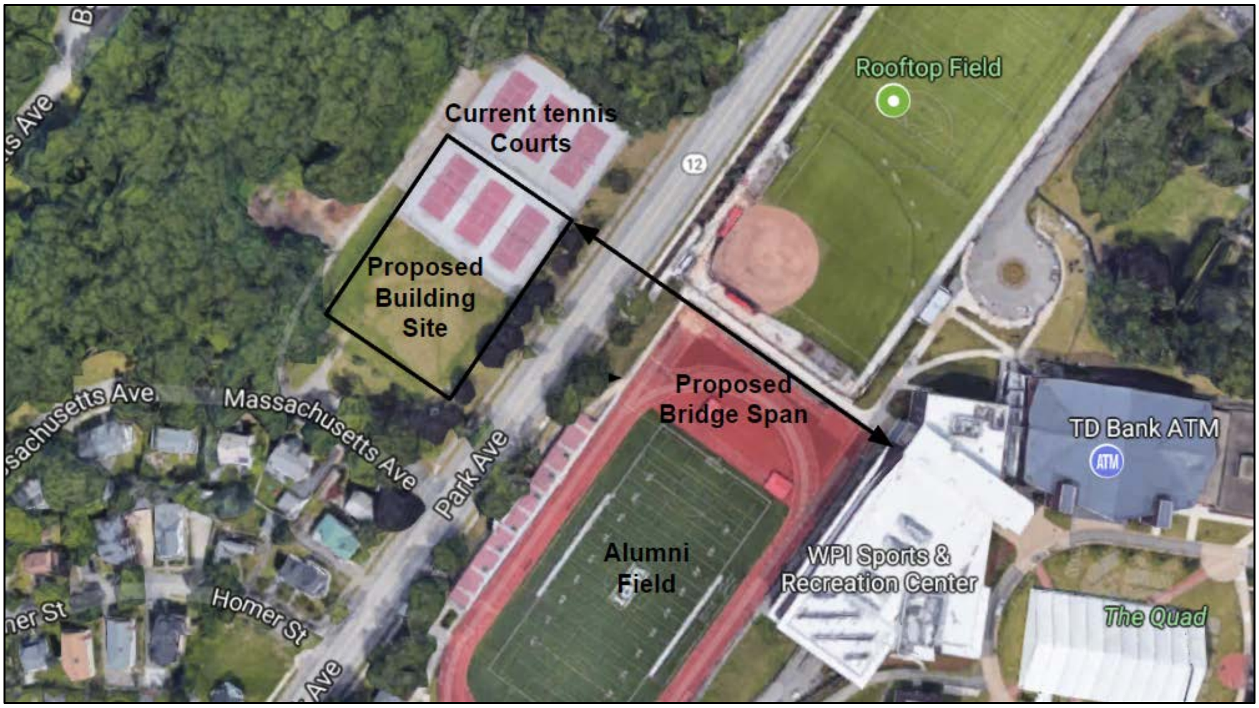
2.0 Background

Many design factors are taken into consideration for any new structure that is being designed or constructed. While considering the development of this project, first the site will be taken into consideration. Both the current conditions and proposed conditions will be assessed for sustainability and constructability. Materials and additional uses help keep the structures more economical and have less of an impact on the environment. Structures must follow design parameters in order to comply with regulatory requirements for accessibility and safety. In the case of the bridge, the American Association of State Highway and Transportation Officials (AASHTO) and the Americans with Disabilities Act (ADA) will be considered. While the building requires compliance the Massachusetts State Building Code. Both structures must conform to the regulations established by the City of Worcester zoning and permitting laws.

2.1 Current Site

The proposed building is to sit on the plot of land directly across Massachusetts Route 122A (Park Avenue) from Alumni Field, on the campus of Worcester Polytechnic Institute as seen in Figure 1. This site is currently underutilized by the University and disconnected from the entirety of campus. Since WPI currently has additional tennis courts in Institute Park, it can be inferred that there is a possibility for better utilization of this location. Additionally, in order to access the field, members of the WPI community must cross a hectic, four-lane state highway without the use of an easily-accessible crosswalk. This creates an unsafe environment for pedestrians and should use of the location increase, a safer means of crossing is necessary.

The entirety of the proposed building site and current tennis courts is approximately 100,000 ft^2. As currently proposed, the project will leave the three tennis courts on the far northwest side of the site untouched. This will leave adequate space for parking and the new structure, as well as continue to provide tennis courts for students and the WPI community and club team. The proposed building site has a relatively level topography and is elevated above Park Avenue by a distance of approximately 12 ft.



Proposal Figure 1: Aerial view of the proposed site

According to the United States Department of Agriculture, the selected site sits on the border of two soil survey zones. One zone consists of 90% Paxton fine sandy loam and the other is comprised of 80% udorthent soil over loamy basal till (Taylor 1985). Udorthent soils are gravelly topsoils that have been placed back on site following an excavation. This site is more than 80 inches above the water table, and is not flooding prone (Taylor 1985). This information will prove useful in determining the type, size, and design of foundation elements for both the athletic facility and pedestrian bridge.

While the site is adequate for the construction of the new facility, several potential alterations have been identified. One potential alteration to the site could include the addition of an access road and parking lot for the proposed building. The facility will be able to be accessed via the pedestrian bridge, but it will also be necessary to provide parking, especially handicapped parking, for vehicles as well. This addition will require regrading of the site in order to tie in with the existing Massachusetts Avenue. Regrading the site could lead to potential issues with storm water runoff down the access road and onto Park Avenue that may have to be assessed. For this reason, when the site plan for the facility is developed, the drainage and runoff from the new facility will be taken into consideration.

The space for the pedestrian connection bridge as seen in Figure 1 is currently occupied by a concrete sidewalk, black chain-link fence, and a small, unused portion of the track. This area will remain unchanged, as the bridge will span an adequate height above to still allow for the track and walkway to remain operational. The location of the cantilevered viewing station will allow for the pedestrian bridge to connect back to the southeast side of the viewing station. Given the current location of existing buildings, their elevations, and the available space, the proposed site for the construction of the new athletic facility is a valuable opportunity to expand and connect WPI's campus.

2.2 Engineering Design Parameters

Safety plays a major role in any design; for this reason it is important to comply with the regulatory agencies that govern the design and construction industries. For the proposed pedestrian bridge and building, the designs will be created in accordance with criteria in the design criteria documents displayed in Table 1. In the state of Massachusetts, it is critical that current and future structures be in compliance with both the American Association of State Highway, Transportation Officials (AASHTO) and the Americans with Disabilities Act (ADA), and the Massachusetts State Building Code (780 CMR).

Proposal Table 1: Pertinent design parameters

<table>
<tr><th>Design Aspect</th><th>Regulatory Agency</th><th>Design Criteria</th></tr>
<tr><td rowspan="3">Pedestrian Bridge</td><td rowspan="2">American Association of State Highway and Transportation Officials (AASHTO)</td><td>Pedestrian Bridge Manual</td></tr>
<tr><td>LRFD Guide Specifications for the Design of Pedestrian Bridges</td></tr>
<tr><td>Americans with Disabilities Act (ADA)</td><td>Standards for Accessible Design</td></tr>
<tr><td rowspan="2">Athletic Facility</td><td>State Board of Building Standards and Regulations</td><td>780 CMR: Massachusetts Amendments to the International Building Code 2009- Chapter 16: Structural Design</td></tr>
<tr><td>Americans with Disabilities Act (ADA)</td><td>Standards for Accessible Design</td></tr>
</table>

2.2.1 AASHTO Design Criteria

Due to the pedestrian bridge's location over MA Route 122A, the Massachusetts Department of Transportation refers to the AASHTO design specifications. There are numerous parameters that must be considered when designing a structure of the magnitude of the proposed pedestrian bridge. One of which being the vertical clearance required over Park Avenue, which according to AASHTO is 17.5 ft. above the road surface (WSDOT, 2017). The manual also has requirements for allowable deflections, span-depth ratios, foundation parameters, drainage, and material requirements for structures passing over highways (AASHTO, 2014). This publication will prove essential in the design of the pedestrian bridge to pass over Park Avenue.

2.2.2 ADA Design Criteria

The ADA design criteria and their corresponding reference sections can be seen below in Table 2. The table shows the section that the criteria can be found in the ADA regulations and design criteria. The table also states the design criteria that is relevant to the design of the pedestrian bridge including slope requirements and handrail design requirements. These criteria will be used during the design of the pedestrian bridge to ensure appropriate access to all facilities.

Proposal Table 2: ADA design parameters

ADA Section	Design Criteria
302.3 & 3.3	Floor and ground surfaces shall be "stable, firm, and slip resistant."
302.3	If there are any openings in the surface the openings shall not exceed ½".
303	There shall be no vertical change in elevation greater than ¼" and if the surface is to be ramped.
402 & 403	Ramps with a rise of greater than 6" must have handrails installed.
405 & 406	Running slope shall not exceed 1:20, the cross slope shall not exceed 1:48, and the clear width for walking surfaces shall not be less than 36 inches.
505	Handrails must be continuous along the entirety of the walking surfaces length. Handrails are not required on ramps with a running slope of 1:20, but when they are required they must be provided on both sides of the walkway. Additionally, the handrails must be 34-38" above the walking surface and be at a consistent height along the entire length of the walking surface. The gripping surface of the handrails must also be unobstructed for at least 80% of its length (with a 1-1/2".

2.2.3 Massachusetts Building Code Design Criteria

The parameters gathered from the Massachusetts State Building Code 8th Edition which includes the 2009 International Building Codes and ASCE 7-05 will primarily affect the design of the proposed athletic facility. The publication contains wind, snow, and seismic loads and load factors to assume given the location of the project (2009, International Code Council).

2.3 Community Impact

The bridge and building will both impact the surrounding community. After construction, both will provide a positive impact to not just the WPI community, but to the Worcester community as well.

2.3.1 Massachusetts Zoning Districts

Across the State of Massachusetts, each city or town is required to have ordinances and regulations regarding the different zoning districts within the city or town. The different zoning districts regulate the different types of land use that may occur. The districts in Worcester, MA include are listed in Table 3 below.

Proposal Table 3: Various zoning types present in Worcester, MA

Massachusetts Zoning Districts	
Residential	Institutional
Industrial	Airport
Business	Open Space
Manufacturing	Overlay

Each of these districts have sub-areas which fall into different permitting requirements for the types of land use. Permits fall into four usage areas, each with a set of subsections: residential use, general use, business use, and manufacturing use. Usage is either permitted in the district, not permitted in the district, or requires a special permit. According to the Worcester, MA zoning map dated February 6th, 2017, our proposed site falls into the Institutional (Educational) district of the City of Worcester which is consistent with the rest of the WPI campus. In accordance with Article IV Section 2 Table 4.1, non-residential parking facilities, recreational/service facilities, and schools (non-profit) are permitted in the Institutional (Educational) district of Worcester. According to Article IV Section 4 Table 4.2, there is no minimum area or frontage; the front, side, and rear setbacks are 15 feet, 10 feet, and 10 feet respectively; there is no maximum number of floors or maximum height; and there is no floor to area ratio (City of Worcester, MA - Zoning Map; City of Worcester Zoning Ordinance).

2.3.2 Impact on the WPI Community

When the current Sports and Recreation Center was constructed, insurance costs for insurance of WPI's faculty were reduced. This was because the new center opened up more space for the WPI faculty to work out and use the facility. A new athletic performance center would have additional space for offices and the athletic training staff, creating new recreation space in the current Sports and Recreation Center for students and faculty. As the number of students in the incoming graduating classes continues to increase, it is important that the space provided can handle the student body. A new campus building will help WPI be successful as it continues to grow and expand in the future.

2.3.3 Impact on the Greater Worcester Community

The City of Worcester is heavily reliant on the students, faculty, and visitors of the twelve universities that make up the Worcester Consortium. Every year thousands of students move to Worcester to earn an education and grow as individuals. These students help drive the local economy by providing a steady flow of revenue and labor. Students also lead community service and social activism movements that help to improve the quality of life for the permanent residents of the city. By improving the quality of the facilities at one of Worcester's most prevalent universities, it will attract more highly-skilled students to the city and help the local economy continue to grow. Outside of the increased student population, the proposed facility will provide a landmark for the city and generate numerous jobs during the construction phase of the project.

2.4 Sustainability

Sustainability should be at the forefront of every engineer's mind when designing or proposing a new structure. Designing structures to be sustainable not only makes economic sense, it also makes ethical sense. Being a technical and engineering school, WPI promotes sustainability with great significance. This means that any building or structure that WPI builds in the future will need to be sustainably designed and environmentally friendly (Ryan, 2017). As students of this Institution and future civil engineers, it is our ethical duty to ensure that this project is delivered in a manner that is both environmentally-friendly and sustainable for the generations of community members to come. Designing a structure sustainability means that it has a smaller impact on the environment, whether that impact be immediate or in the future and ultimately means leaving a better planet for the next generation. Reducing the environmental impact of a construction project could include reducing the amount of energy used to build it, reducing the amount of greenhouse gases released from construction or materials, or reducing the amount of energy the building consumes over its lifetime.

In order to design a bridge or building with sustainability in mind, we must be conscience of the building materials, design, and construction processes that will be used. “Humphreys and Mahasenan (2002) estimate that the cement industry is responsible for 3% of global anthropogenic greenhouse gas emissions and 5% of global anthropogenic $CO2$ emissions” (Noguchi, 2015). This shows that using cement to build a structure has environmental impacts that must be taken into account when designing sustainably. The amount we use isn’t the only concern though. “Service life can be dramatically extended with little or no increase in – or even a reduction of – the environmental load” (Vanderly, 2003). If we can make the same amount of cement last longer, it won’t need to be replaced as fast and will have a smaller environmental impact over its lifetime.

When designing a structure, we must not only be critical of the global impact, but also of the local environmental impact. When a structure is built on a particular site, the properties of the location can change dramatically. For example, the area of permeable surface can decrease, causing an increase in rainwater runoff, and altering the current runoff and drainage characteristics. This can impact the local environment in many ways that are difficult to predict as is the case with erosion, flooding, and chemical dispersion. Depending on the site, a new structure may also alter or destroy animal and plant habitats, displacing or placing stress on the local animal community. For this reason it is important to asses each site and design ways to minimize the structure's impact on its surrounding area. When changes must be made to the local site, the impacts should be fully assessed and analyzed prior to construction in order to be prepared for potential complications

2.5 Economics

A project of this size is guaranteed to have a large initial cost associated with it. When considering this initial investment one must consider the major costs of engineering services, construction materials and building systems, project management, and long-term maintenance. A complete cost analysis will be performed following the completion of the final structural design of both the bridge and the athletic building. The "2017 Building Construction Costs Book with RS Means" (Plotner, 2017) will be used to create this cost analysis. However, in order to provide a rough estimate for the new facility, similar facilities' costs can be utilized. The 78,000 ft^2. Foisie Innovation Studio and Messenger Residence Hall will cost Worcester Polytechnic Institute approximately $49 million (WPI 2017). While the Foisie facility provides living spaces and does not include a pedestrian bridge, it does provide insight into the cost of erecting a new building in Worcester, MA. The proposed athletic facility will provide approximately 51,000 ft^2 of extra space to WPI. By making a direct comparison to the Foisie Studio it can be inferred that the proposed athletic building would approximately cost $32 million.

Recently, the city of Worcester was ordered by a Superior Court to construct an elevated pedestrian bridge connecting the DCU Center to the Hilton Garden Inn and the Major Taylor Parking Garage in the city's downtown district (Moulton 2016). This mandate comes following a recent traffic accident in the area. The proposed bridge is to be 275 ft. long and 10ft. wide, and has an estimated cost of $10 million. The proposed pedestrian bridge on WPI's campus would span approximately 450 ft. and be 10 ft. wide. Using the same direct comparison method used above, a rough cost estimate of the proposed bridge is $16.4 million. When added to the cost of the athletic building, a total project cost of $48.4 million can be derived.

It is important to note that the direct comparison method does provide good insight into construction costs in the Worcester area, but it does not provide exact values for the project. There are numerous differences between the proposed facility and the two projects used as reference. The new building will require a parking lot, field turf, and different finishes, etc. than the Foisie studio which will alter the final project cost. The pedestrian bridge also differs as it will be enclosed and have various security restrictions. The proposed building and bridge would also be built at a different time than the example projects, altering the cost estimates further. For this reason, the initial cost of the project should be seen as a preliminary projection.

2.6 Ethics

Many designers follow a number of codes that act as guiding principles for engineering, design, and construction decisions. Codes often protect both workers and clients from poor business practices. The American Society of Civil Engineers (ASCE) states that “ethics is integral to all decisions, designs, and services performed by civil engineers.” Not only the public trust but also their lives, safety, and welfare depend on professional engineers' efficient, safe, and economical performance of their duties. ASCE has programs, policies, and resources that are designed to help professionals understand their ethical obligations and how to incorporate them into their professional careers. For this project, we plan to design our pedestrian bridge and building while upholding the principles stated by ASCE. These principles state that “engineers uphold and advance the integrity, honor, and dignity of the engineering profession by using their knowledge and skill for the enhancement of human welfare and the environment, being honest and impartial and serving with fidelity the public, their employers and clients, striving to increase the competence and prestige of the engineering profession, and supporting the professional and technical societies of their disciplines” (ASCE, 2017).

Since this project is entirely theoretical, it can be very easy to ignore or alter problems that are faced during the design of the pedestrian bridge or building. We will, however, uphold the same ethical policies and principles as if this project was for a real-world application. This includes the risks and dangers involved in designing infrastructure that will be used and occupied by students and faculty. During the design process of the proposed structures, the governing regulatory requirements and design standards will be used and issues related to safety will not be ignored. By doing this, the ethical standards of the ASCE and the engineering community will be upheld.

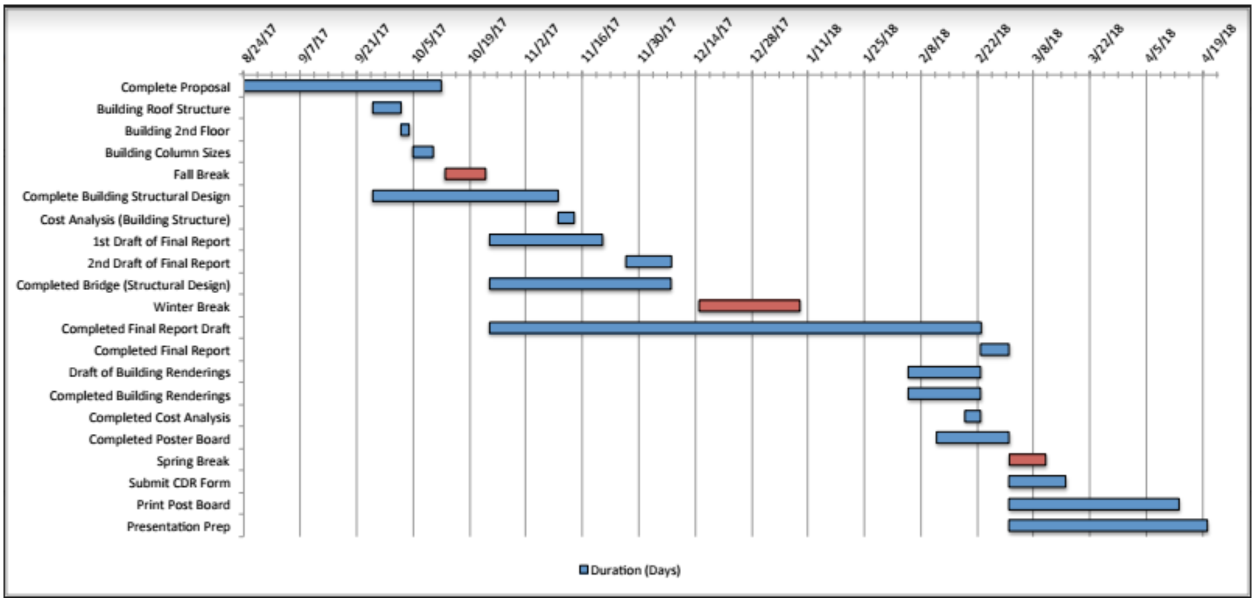
3.0 Methodology

This section presents the process of how the project is going to be completed. A project of this magnitude has many major phases and minor steps, and it is critical to the success of the project that it is completed in and organized and timely manner. Table 4 displays both the major phases and minor steps used to complete the scope of work. Figure 2 shows the schedule that will be followed to complete the scope of work.

Proposal Table 4: Proposed methodology breakdown

Project Methodology Summary
Site Survey
Site visit and evaluation
Obtain Sports and Recreation Center drawings with reference elevations
Topographic confirmation survey and cross-section topographic diagram
Establish Design Goals
Interview key stakeholders (Athletics Department and WPI Facilities)
Develop architectural program
Establish Design Parameters
Set functional restraints and requirements based upon the architectural program
Research permitting, ADA restrictions, AASHTO, and MA Building Code design criteria
Structural Analysis and Design of Athletic Facility
Establish structural grid and structural systems
Design calculations for structural system members, including beams, columns, and footings
Comparison of structural grids and systems
Structural Analysis and Design of Bridge Structure
Develop comparison criteria to structure and determine applicable materials: steel, concrete
Design calculations for bridge enclosure, spans, columns, and footings
Evaluate various design alternatives
Develop Final Structural Design of Pedestrian Bridge
Design bridge connections on both ends of the span
Computer simulations of final structural components in RISA
Identify potential structural issues and develop a plan to mitigate
Provide recommendations to WPI and future student projects
Cost Analysis

RS Means cost analysis
Total material quantities and associated costs
Estimate design and construction labor costs and calculate total project cost
Feasibility analysis
Deliverables
Final project report
Computer models: Revit renderings and AutoCAD floor plans and cross-sections
Structural calculations
Project cost estimate



Proposal Figure 2: MQP Methodology Schedule

3.1 Site Survey

In order to properly design both the athletic building and the pedestrian bridge, various site elevations are required. This will be accomplished by conducting an in-depth site survey of A.J. Knight Field and the proposed span location of the pedestrian bridge. It is crucial that the surveying equipment be properly leveled and operated. The equipment will be relocated and backsight as necessary to account for any visibility difficulties. The team will create a base point on top of a manhole cover located at the base of the Sports and Recreation Center and designate this as the datum surface. Following this, elevations will be taken in 10 ft. increments along the current walkway between Alumni Field and the parking garage. This elevation line will continue up the hill on the inside of the current fence and out onto the sidewalk along Park Avenue. Being mindful of the flow of traffic, elevations will be gathered for the road surface, as well as the sidewalk on the opposite side. Lastly, the line will be completed at the edge of the A.J. Knight tennis courts. This data will be compiled into a cross-sectional diagram detailing current site elevations and cross-referenced with construction documents for the Sports and Recreation Center and elevations available online.

3.2 Establish Design Goals and Develop Architectural Program

Once the site has been surveyed, it will be important to develop the required functionality and architectural program of the facility. Interviews with both the WPI Athletics and Facilities Departments will be conducted to determine their current spatial needs and how the proposed facility can solve them. The questions used to help guide the interviews will be made available within the Appendices of the final report. Following these interviews, various design requirements and constraints will be established based on the intended use of each space within the facility. This architectural program will be used to guide the structural system and layout of the facility.

3.2.1 Building Architectural Program

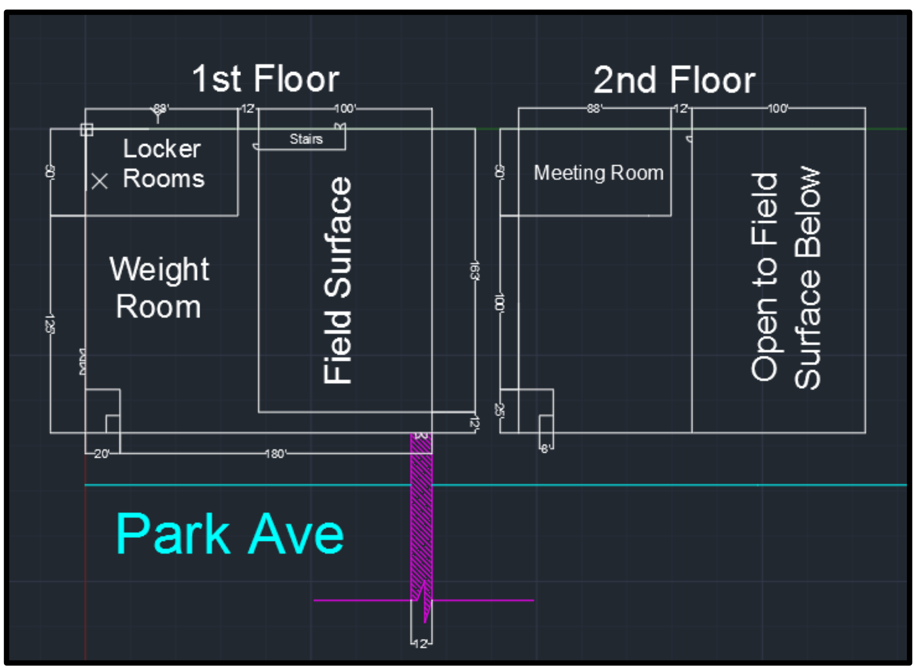
This project aims to increase the training space available to WPI varsity athletes and in turn free up space in the already-crowded Sports and Recreation Center. The layout for the new facility is shown in Figure #. The new facility requires space for strength training, athletic training, team-specific meeting space, and large open area to hold athletic events and practices. For this reason, the new structure will need a large open space without columns and minimal columns throughout the remainder of the building. The large open space must be two stories tall to allow for athletic events. The layout allows for large structural columns directly down the centerline of the building and along the perimeter.

The usages of each room shape the various loads that will be present within the space. The weight room facility will be located on the 1st floor of the building to allow the added load from the weights and equipment to not have to be carried by the structural columns. This allows for the space on the second floor to be utilized for uses that entail lighter loads, such as athletic training and meeting space, because these columns will be taller and responsible for supporting the roof, as well. A space such as this should optimize the amount of floor space possible to increase the number of potential activities it can hold, and the facility will be designed accordingly.

3.2.2 Bridge Architectural Program

The project aims to expand and connect the WPI campus by improving accessibility and safety through the design of a pedestrian bridge. Given that this bridge expands the reach of the campus, it is critical that the final architectural layout blend the new athletic facility into the rest of campus. A technical institution, such as WPI, requires that structures be modern, efficient, and be of high quality. For this reason, the bridge will be sheathed with tempered glass and will support solar photovoltaic modules on its slanted roof. However, given the bridge's location over a busy highway, its proximity to both the football and softball fields, and exposure to the harsh New England climate, it will be important that the materials chosen for the design provide durability and longevity.

The bridge will be laid-out in order to accommodate to lanes of pedestrian traffic for individuals coming to and from the new athletic facility. Space below the sloped roof will be delegated for the mechanical, electrical, and plumbing required to make the transition between buildings seamless. In order to improve the energy efficiency of the pedestrian bridge and increase the safety of its users, both ends of the bridge will be outfitted with vestibules that are accessed via WPI I.D. credentials. Lastly, the bridge will utilize minimal structural support columns to minimize the impact on the current athletic facilities and provide ramp access to and from the current Rooftop Field.



Proposal Figure 3: Proposed athletic facility floor plan

3.3 Establish Design Criteria

Following the completion of the architectural program, research must be performed to determine the pertinent design codes and restrictions on the proposed facility. Due to the broad scope of the project, both Massachusetts Department of Transportation (MassDOT) guidelines and Massachusetts State Building Code must be considered. In addition to adhering to AASHTO guidelines, MassDOT publishes an LRFD Bridge Design Manual which includes various loading and dimensional requirements. The Massachusetts State Building Code will provide the information necessary to design a building in Worcester, MA. This information will be critical in developing the facility's structural design.

3.4 Structural Design and Analysis

Structural design calculations are required for both the athletic building and pedestrian bridge. The design of the building will be completed prior to the start of the bridge design. Building calculations will be performed from the top down, starting with the roof system, then 2nd and 1st floor beams and girders, and lastly columns and footings. LRFD design will be used for the entirety of these calculations. The bridge design will be done in a similar manner. Calculations will start with bridge enclosure, followed by the bridge deck and support girders. The final step in the design of the facility is develop an effective method of connecting the pedestrian bridge to both the Sports and Recreation Center and new athletic building. Calculations will be performed on this connection to ensure the structural stability of the Sports and Recreation Center remains intact.

Structural analysis software, such as RISA 3D, will be used to simulate how the calculated members interact with one another. This will identify any potential design errors and areas for improvement. These errors will be corrected and the design will be reevaluated until it is considered satisfactory and capable of fulfilling all of its functional requirements. Recommendations to WPI and future student project groups will be established based on the results of this analysis.

3.5 Evaluation of Alternatives

During the structural design of both the athletic building and pedestrian bridge, various alternatives for structural systems and materials will be evaluated. For the athletic building, steel will be the design material used in the structural framing and system. Two different structural roof systems will be considered and then evaluated based on the final cost of steel. We want to ensure the structural system variations will safely resist the dead and live loads present, but also be as cost effective as possible. In our pedestrian bridge design process, we will begin with an evaluation of materials. The materials will be evaluated based on strength, serviceability, and cost. Not only will alternative materials be considered, but alternative structural systems will be evaluated as well. This will allow the project to most optimally meet the needs of WPI while still remaining cost-effective.

3.6 Cost Analysis

As cost is a parameter that a private client like WPI is especially concerned with, and a project cost analysis will be performed in order to ensure that the project is feasible. The 2017 R.S. Mean Building Construction Costs Book will be used to reference the current costs of materials and material quantities will be taken from the final structural design. Industry standards will be used to arrive at design service and construction labor costs. A total project cost will be established to allow for comparison to previous projects completed by WPI and provide insight to the feasibility of funding the project.

4.0 Deliverables

The completion of this project will provide a structural design of both the proposed athletic building and pedestrian bridge. This design will include analysis of structural members, a cost analysis of the overall project, and computer renderings of the final facility. A final report and write up of the hand-written calculations that have been checked using available software will be provided. Lastly, our team will present graphical representations of important data for better and easier understanding and presentation. These activities will culminate in a presentation of the work completed to WPI faculty, members of the Civil Engineering Department, and current students.

Deliverable	Primary Author(s)	Assistant Author(s)
Proposal	All	All
Building Structure	Liam	Elijah
Building Foundation	Elijah	Liam
Bridge Structure	Kaitlin	Conor
Bridge Columns/Foundation	Conor	Kaitlin
Cost Analysis	Elijah	All
Building Renderings	Liam	All
Bridge Renderings	Conor	All
Final Report	Kaitlin	All
Background Chapter	Elijah	Conor
Methodology Chapter	Liam	Kaitlin
Bridge Design Chapter	Conor	Kaitlin
Building Design Chapter	Elijah	Liam
Analysis	Kaitlin	Liam
Report Edits	All	All
Paper Formatting	Conor	All
Poster Design	Liam	Kaitlin

Proposal Table 5: Deliverable responsibilities

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Appendix B: Site Maps and Images

Figure 50: Side view of Parking Garage Field

Figure 51: Elevated view of proposed bridge span

Figure 52: Elevated View of Proposed Athletic Building Site Location

Figure 53: Aerial Map of Bridge Span and Building Site

Figure 54: Base of Sports and Recreation Center

Appendix C: Survey Data and Analysis

This section shows the raw data from the site survey that was conducted as well as the cross-section diagram of the site created using the survey data. All heights and elevations are shown relative to the base of the Sports and Recreation Center which was assumed to have an Elevation of 0’-0”.

C.1 Site Cross-Section Diagram from Site Survey Data

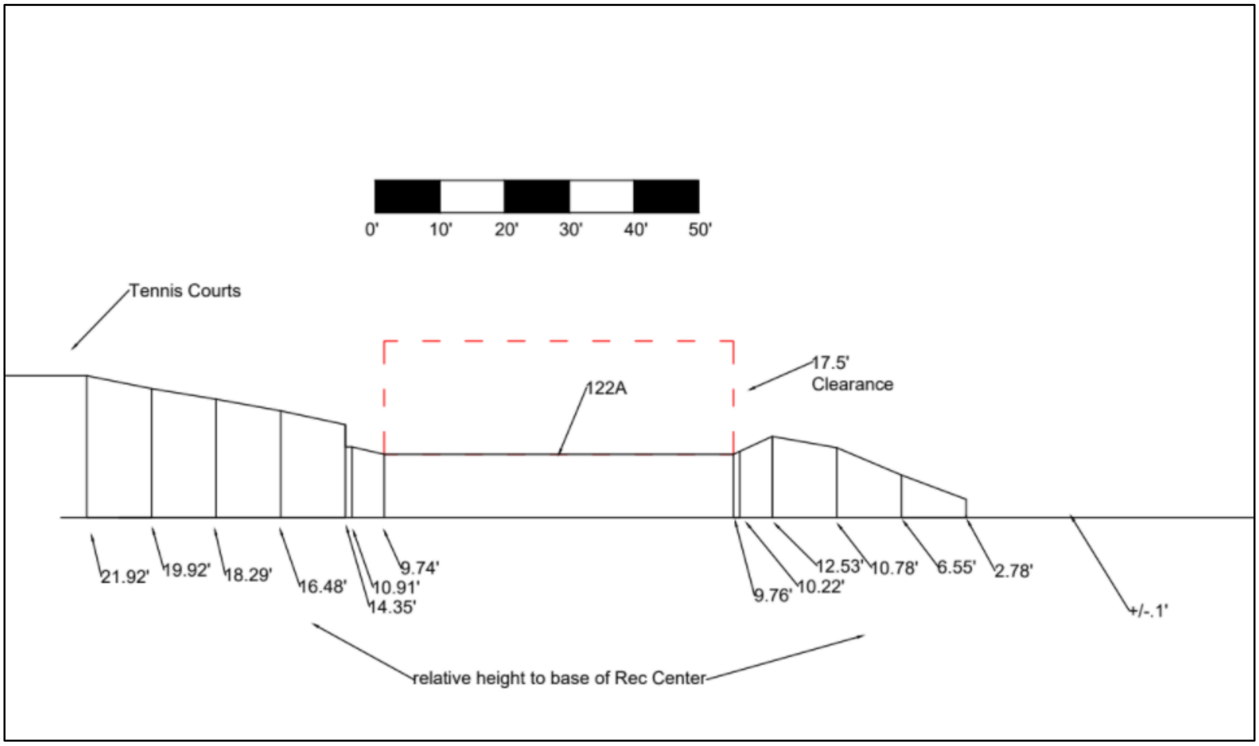


Figure 55: Site survey diagram

C.2 Site Survey Raw Data

Table 39: Complete Survey Data

Point # (10' apart)	Back Site (ft)	Height (ft)	Notes
1	4.81	0	Base of Rec Center assumed to be elevation 0'0"
2	4.97	-0.16	Walkway
3	5.025	-0.215	Walkway
4	5.13	-0.32	Walkway
5	5.2	-0.39	Walkway
6	4.93	-0.12	Walkway

7	4.71	0.1	Walkway
8	4.71	0.1	Walkway
9	4.71	0.1	Walkway
10	4.73	0.08	Walkway
11	4.73	0.08	Walkway
12	4.72	0.09	Walkway
13	4.72	0.09	Walkway
14	4.72	0.09	Walkway
15	4.71	0.1	Walkway
16	4.72	0.09	Walkway
17	4.71	0.1	Walkway
18	4.71	0.1	Walkway
19	4.71	0.1	Walkway
20	4.68	0.13	Walkway
21	4.72	0.09	Walkway
22	4.72	0.09	Walkway
23	4.72	0.09	Walkway
24	4.7	0.11	Walkway
25	14.69	0.09	Walkway
26	14.71	0.07	Walkway
27	14.7	0.08	Walkway
28	14.68	0.1	Walkway
29	14.71	0.07	Walkway
30	14.73	0.05	Walkway
31	14.71	0.07	Walkway
32	12	2.78	Retaining wall by entrance to Alumni Field
33	8.23	6.55	Hill at Alumni Field
34	4	10.78	Hill at Alumni Field
35	2.25	12.53	Top of hill at Alumni Field
36	2.34	12.44	Downslope of hill towards sidewalk
37	4.56	10.22	WPI side 122A sidewalk
38	5.02	9.76	WPI side of Route 122A
39	5.04	9.74	Field side of Route 122A
40	3.87	10.91	Field side 122A sidewalk
41	10.44	14.35	Retaining wall by A.J. Knight Field

42	8.31	16.48	Hill at A.J. Knight Field
43	6.5	18.29	Hill at A.J. Knight Field
44	4.87	19.92	Hill at A.J. Knight Field
45	2.87	21.92	Tennis Courts

Appendix D: Design Specification Sheets

This section contains the various published design aids that were utilized throughout the design of the proposed facility. Further information regarding their usage can be found within the Design Methodology Sections.

D.1 Building 2nd Floor Deck Spacing

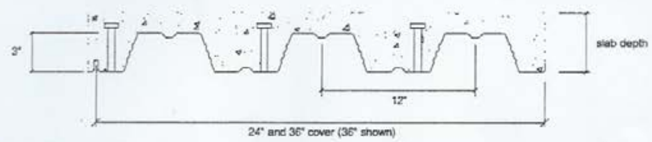
CMC Joist & Deck

FLOOR DECK

3" LOK-FLOOR

3" x 12" deck F_y = 40 ksi f'_c = 3 ksi   115 pcf concrete

Studs are not required for composite slab action. Studs on the cross-section indicate that it is possible to install studs at the beams.



DECK PROPERTIES

Gage	t	w	A_s	I_p	S_p	S_n	ϕR_{be}	ϕR_{bi}	ϕV_n	studs
22	0.0295	1.7	0.500	0.765	0.416	0.441	720	1270	2200	0.59
20	0.0358	2.1	0.610	0.953	0.548	0.577	820	1450	3420	0.71
19	0.0418	2.4	0.710	1.112	0.680	0.692	1100	1930	4660	0.83
18	0.0474	2.8	0.810	1.261	0.793	0.790	1380	2430	5900	0.94
16	0.0598	3.5	1.020	1.593	0.998	0.998	2120	3710	7420	0.94

COMPOSITE PROPERTIES

Gage	Slab Depth	ϕM_{nf} in.k	A_c in²	Vol. ft³/ft²	W psf	S_c in³	I_{av} in⁴	ϕM_{no} in.k	ϕV_{nt} lbs.	Max Unshored Span, ft. 1 span	2 span	3 span	A_{wwf} in²/ft
22 gage	5.50	62.44	37.6	0.333	38	1.25	7.5	42.68	4820	8.59	10.83	11.19	0.023
	6.00	70.94	42.0	0.375	43	1.44	9.6	49.11	5130	8.20	10.26	10.69	0.027
	6.25	75.19	44.3	0.396	46	1.54	10.8	52.43	5290	8.03	9.88	10.29	0.029
	6.50	79.44	46.6	0.417	48	1.64	12.1	55.80	5450	7.86	9.52	9.92	0.032
	7.00	87.94	51.3	0.458	53	1.84	14.9	62.72	5780	7.57	8.88	9.25	0.036
	7.25	92.19	53.8	0.479	55	1.94	16.5	66.24	5960	7.43	8.60	8.95	0.038
	7.50	96.44	56.3	0.500	58	2.05	18.2	69.80	6130	7.28	8.33	8.67	0.041
	8.00	104.94	61.3	0.542	62	2.26	21.9	77.02	6480	6.81	7.83	8.16	0.045
20 gage	5.50	74.69	37.6	0.333	38	1.50	8.1	51.02	5250	10.10	12.48	12.89	0.023
	6.00	85.06	42.0	0.375	43	1.72	10.4	58.71	5870	9.63	11.66	12.15	0.027
	6.25	90.25	44.3	0.396	46	1.84	11.6	62.69	6180	9.42	11.23	11.69	0.029
	6.50	95.43	46.6	0.417	48	1.96	13.0	66.74	6510	9.23	10.83	11.28	0.032
	7.00	105.80	51.3	0.458	53	2.20	16.0	75.06	7000	8.87	10.10	10.52	0.036
	7.25	110.99	53.8	0.479	55	2.33	17.7	79.30	7180	8.71	9.76	10.18	0.038
	7.50	116.17	56.3	0.500	58	2.46	19.5	83.59	7350	8.56	9.47	9.86	0.041
	8.00	126.54	61.3	0.542	62	2.71	[illegible]	[illegible]	[illegible]	8.12	8.91	9.28	0.045
19 gage	5.50	85.36	37.6	0.333	38	1.71	[illegible]	[illegible]	5250	11.44	13.69	14.15	0.023
	6.00	97.43	42.0	0.375	43	1.97	[illegible]	[illegible]	5870	10.90	13.14	13.58	0.027
	6.25	103.46	44.3	0.396	46	2.11	[illegible]	[illegible]	6180	10.66	12.89	13.33	0.029
	6.50	109.50	46.6	0.417	48	2.24	[illegible]	[illegible]	6510	10.44	12.66	13.08	0.032
	7.00	121.57	51.3	0.458	53	2.52	[illegible]	[illegible]	7170	10.03	12.22	12.63	0.036
	7.25	127.60	53.8	0.479	55	2.67	[illegible]	[illegible]	7510	9.84	12.02	12.42	0.038
	7.50	133.64	56.3	0.500	58	[illegible]	[illegible]	[illegible]	7860	9.67	11.83	12.22	0.041
	8.00	145.71	61.3	0.542	62	[illegible]	[illegible]	[illegible]	8570	9.39	11.47	11.85	0.045
18 gage	5.50	95.58	37.6	0.333	38	1.92	9.1	65.67	5250	12.48	14.62	15.11	0.023
	6.00	109.35	42.0	0.375	43	2.21	11.6	75.56	5870	11.89	14.05	14.52	0.027
	6.25	116.24	44.3	0.396	46	2.36	13.0	80.70	6180	11.62	13.78	14.24	0.029
	6.50	123.12	46.6	0.417	48	2.52	14.5	85.94	6510	11.38	13.53	13.99	0.032
	7.00	136.89	51.3	0.458	53	2.83	17.9	96.72	7170	10.93	13.07	13.51	0.036
	7.25	143.78	53.8	0.479	55	3.00	19.7	102.24	7510	10.72	12.86	13.29	0.038
	7.50	150.66	56.3	0.500	58	3.16	21.7	107.82	7860	10.53	12.65	13.08	0.041
	8.00	164.43	61.3	0.542	62	3.50	26.1	119.18	8570	10.23	12.27	12.68	0.045
16 gage	5.50	95.58	37.6	0.333	38	2.34	10.0	65.67	5250	13.92	16.38	16.31	0.023
	6.00	109.35	42.0	0.375	43	2.70	12.7	75.56	5870	13.50	15.74	15.87	0.027
	6.25	116.24	44.3	0.396	46	2.88	14.3	80.70	6180	13.19	15.44	15.67	0.029
	6.50	123.12	46.6	0.417	48	3.08	15.9	85.94	6510	12.91	15.17	15.49	0.032
	7.00	136.89	51.3	0.458	53	3.47	19.6	96.72	7170	12.40	14.66	15.15	0.036
	7.25	143.78	53.8	0.479	55	3.67	21.7	102.24	7510	12.16	14.42	14.90	0.038
	7.50	150.66	56.3	0.500	58	3.87	23.8	107.82	7860	11.94	14.19	14.67	0.041
	8.00	164.43	61.3	0.542	62	4.29	28.6	119.18	8570	11.60	13.77	14.23	0.045

Note:
50 ksi material is also available.
See website for load tables.

3" LOK-FLOOR - LW

54

3" LOK-FLOOR

3" x 12" deck F_y = 40 ksi f'_c = 3 ksi   115 pcf concrete

	Slab Depth	ϕMn in.k	Span "L" feet, Uniform Live Unfactored Service Loads, psf 9.00	9.50	10.00	10.50	11.00	11.50	12.00	12.50	13.00	13.50	14.00	14.50	15.00
22 gage	5.50	62.44	290	260	230	205	185	165	150	135	125	115	105	95	85
	6.00	70.94	330	295	260	235	210	190	170	155	140	130	115	105	100
	6.25	75.19	350	310	280	250	225	200	180	165	150	135	125	115	105
	6.50	79.44	370	330	295	265	235	215	195	175	160	145	130	120	110
	7.00	87.94	400	365	325	290	260	235	215	195	175	160	145	135	120
	7.25	92.19	400	385	340	305	275	250	225	205	185	170	155	140	130
	7.50	96.44	400	400	355	320	290	260	235	215	195	175	160	145	135
	8.00	104.94	400	400	390	350	315	285	255	230	210	190	175	160	145
20 gage	5.50	74.69	355	315	280	250	225	205	185	170	155	140	130	115	105
	6.00	85.06	400	360	320	290	260	235	210	195	175	160	145	135	125
	6.25	90.25	400	380	340	305	275	250	225	205	185	170	155	145	130
	6.50	95.43	400	400	360	325	290	265	240	215	200	180	165	150	140
	7.00	105.80	400	400	400	360	325	290	265	240	220	200	185	170	155
	7.25	110.99	400	400	400	375	340	305	280	255	230	210	195	175	165
	7.50	116.17	400	400	400	395	355	320	290	265	240	220	200	185	170
	8.00	126.54	400	400	400	400	385	350	320	290	265	240	220	205	185
19 gage	5.50	85.36	400	365	325	290	265	240	215	195	170	155	140	125	110
	6.00	97.43	400	400	370	335	300	275	250	225	205	190	175	160	145
	6.25	103.46	400	400	395	355	320	290	265	240	220	200	185	170	155
	6.50	109.50	400	400	400	375	340	305	280	255	230	215	195	180	165
	7.00	121.57	400	400	400	400	375	340	310	285	260	235	215	200	185
	7.25	127.60	400	400	400	400	395	360	325	295	270	250	230	210	195
	7.50	133.64	400	400	400	400	400	375	340	310	285	260	240	220	205
	8.00	145.71	400	400	400	400	400	400	375	340	310	285	260	240	220
18 gage	5.50	95.58	400	400	365	330	300	260	230	205	180	160	145	130	120
	6.00	109.35	400	400	400	380	340	310	280	255	230	205	185	165	150
	6.25	116.24	400	400	400	400	365	330	300	275	250	230	210	185	170
	6.50	123.12	400	400	400	400	385	350	320	290	265	245	225	205	190
	7.00	136.89	400	400	400	400	400	390	355	325	295	270	250	230	210
	7.25	143.78	400	400	400	400	400	400	375	340	310	285	260	240	225
	7.50	150.66	400	400	400	400	400	400	390	355	325	300	275	255	235
	8.00	164.43	400	400	400	400	400	400	400	390	355	325	300	275	255
16 gage	5.50	95.58	400	400	365	330	300	260	230	205	180	160	145	130	120
	6.00	109.35	400	400	400	380	340	310	280	255	230	205	185	165	150
	6.25	116.24	400	400	400	400	365	330	300	275	250	230	210	185	170
	6.50	123.12	400	400	400	400	385	350	320	290	265	245	225	205	190
	7.00	136.89	400	400	400	400	400	390	355	325	295	270	250	230	210
	7.25	143.78	400	400	400	400	400	400	375	340	310	285	260	240	225
	7.50	150.66	400	400	400	400	400	400	390	355	325	300	275	255	235
	8.00	164.43	400	400	400	400	400	400	400	390	355	325	300	275	255
22 gage	5.50	42.68	190	165	150	130	115	105	95	85	75	70	60	55	50
	6.00	49.11	220	195	170	150	135	120	110	95	85	80	70	65	55
	6.25	52.43	235	205	185	165	145	130	115	105	95	85	75	70	60
	6.50	55.80	250	220	195	175	155	140	125	110	100	90	80	75	65
	7.00	62.72	280	250	220	195	175	155	140	125	115	105	95	85	75
	7.25	66.24	300	265	235	210	185	165	150	135	120	110	100	90	80
	7.50	69.80	315	280	245	220	195	175	160	140	130	115	105	95	85
	8.00	77.02	350	310	275	245	215	195	175	155	140	130	115	105	95
20 gage	5.50	51.02	230	205	180	165	145	130	115	105	95	85	80	70	65
	6.00	58.71	270	235	210	190	170	150	135	125	110	100	90	80	75
	6.25	62.69	285	255	225	200	180	160	145	130	120	110	100	90	80
	6.50	66.74	305	270	240	215	190	175	155	140	125	115	105	95	85
	7.00	75.06	345	305	270	245	215	195	175	160	145	130	120	110	100
	7.25	79.30	365	325	290	255	230	205	185	170	155	140	125	115	105
	7.50	83.59	385	340	305	270	245	220	195	180	160	145	135	120	110
	8.00	92.31	400	380	335	300	270	245	220	200	180	165	150	135	125
19 gage	5.50	58.43	270	240	215	190	170	155	140	125	115	105	95	85	80
	6.00	67.24	310	275	245	220	195	180	160	145	130	120	110	100	90
	6.25	71.80	335	295	265	235	210	190	170	155	140	130	115	105	95
	6.50	76.46	355	315	280	250	225	205	185	165	150	135	125	115	105
	7.00	86.01	400	355	315	285	255	230	210	190	170	155	140	130	120
	7.25	90.90	400	375	335	300	270	245	220	200	180	165	150	135	125
	7.50	95.84	400	400	355	315	285	255	230	210	190	175	160	145	135
	8.00	105.89	400	400	395	350	315	285	260	235	215	195	175	160	150
18 gage	5.50	65.67	305	270	245	215	195	175	160	145	130	120	110	100	90
	6.00	75.56	355	315	280	250	225	205	185	165	150	140	125	115	105
	6.25	80.70	380	335	300	270	240	220	195	180	165	150	135	125	115
	6.50	85.94	400	360	320	285	260	235	210	190	175	160	145	130	120
	7.00	96.72	400	400	360	325	290	265	240	215	195	180	165	150	140
	7.25	102.24	400	400	385	345	310	280	250	230	210	190	175	160	145
	7.50	107.82	400	400	400	360	325	295	265	240	220	200	185	170	155
	8.00	119.18	400	400	400	400	360	325	295	270	245	225	205	185	170
16 gage	5.50	65.67	305	270	245	215	195	175	160	145	130	120	110	100	90
	6.00	75.56	355	315	280	250	225	205	185	165	150	140	125	115	105
	6.25	80.70	380	335	300	270	240	220	195	180	165	150	135	125	115
	6.50	85.94	400	360	320	285	260	235	210	190	175	160	145	130	120
	7.00	96.72	400	400	360	325	290	265	240	215	195	180	165	150	140
	7.25	102.24	400	400	385	345	310	280	250	230	210	190	175	160	145
	7.50	107.82	400	400	400	360	325	295	265	240	220	200	185	170	155
	8.00	119.18	400	400	400	400	360	325	295	270	245	225	205	185	170

Studs at 1 foot o.c. (unshaded)   No Studs (shaded)

FLOOR DECK

Appendix E: Building Calculations

This section contains the necessary hand-calculations required to properly design the proposed athletic building. Each section shows the typical methodology for determining the proper members for the structure. Microsoft Excel spreadsheets were then used to recreate these calculations multiple times to increase efficiency. Screenshots of these spreadsheets are included at the conclusion of their appropriate sections.

E.1 Roof Truss Calculations

Roof Truss Calculations

Dead Load

roof deck	10	psf
insulation	2	psf
MEP	5	psf
Ceiling	3	psf
Total Dead Load	20	psf

$\times 25'$

500 lb/ft

Live Load

Snow	42.4	psf
roof	20	psf

Tributary width = 25'
Truss length = 100'
Bar joist wt = 10 lb/ft
Bar joist spacing = 10'

Load Combinations

$1.4D = 350 \text{ lb/ft}$

$1.2D + 1.6(L_r \text{ or } S \text{ or } R) = 1148 \text{ lb/ft}$

$W_u = 1148 \text{ lb/ft}$

Truss Girder Components

Type	Shape	ϕP_n
Chords	WT 9x71 1/2	849 K
Diagonals	LL 4x4x3/4	35.3 K
Verticals	LL 3x2.5x3/16	K

Grade A992 steel

Chord area = 0.146 ft^2
γ = 490 lb/ft^3

Diagonal area = 0.076 ft^2
γ = 490 lb/ft^3

Verticals area = 0.013 ft^2
γ = 490 lb/ft^3

Number of diagonals = 10
Number of verticals = 11
Number of chords = 20

$$W_{self} = 20(0.146)(499.4 \text{ lb/ft}^3)(10') + 10(0.013)(499.4 \tfrac{lb}{ft^3})(4')$$

$$+ 11(0.076)(499.4 \tfrac{lb}{ft^3})(10.77') = 19338.6 \text{ lb}$$

$$19338.6/(100' \times 25') = 7.73 \text{ psf}$$

$$\text{Total dead load} = 7.7 + 20+1 = 28.7 \text{ psf}$$

ROOF TRUSS

Dead Load

$P^D_{int} = 27.7 \text{ psf} \times 25' \times 10' = 6925 \text{ lb}$

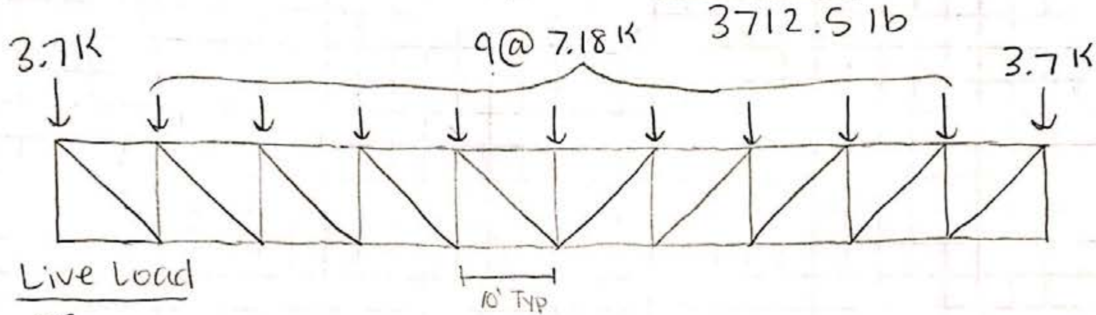
$10 \text{ plf} \times 25' = 250 \text{ lb}$

7175 lb

$P^D_{ext} = 27.7 \text{ psf} \times 25' \times 10/2' = 3462.5 \text{ lb}$

$10 \text{ plf} \times 25' = 250 \text{ lb}$

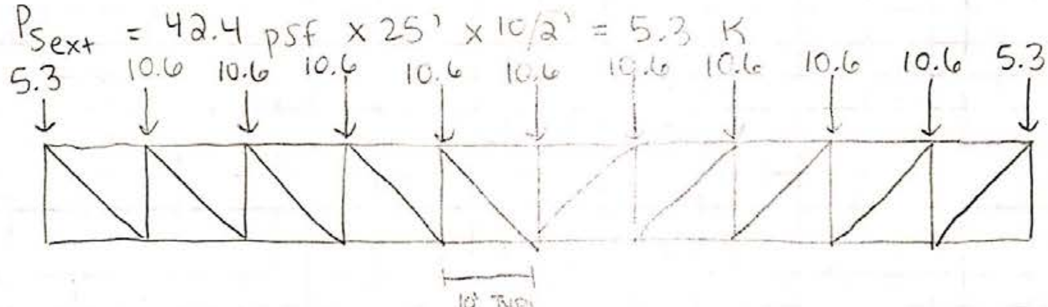
3712.5 lb



Live Load

$P^S_{int} = 42.4 \text{ psf} \times 25' \times 10' = 10.6 \text{ K}$

$P_{S\,ext} = 42.4 \text{ psf} \times 25' \times 10/2' = 5.3 \text{ K}$

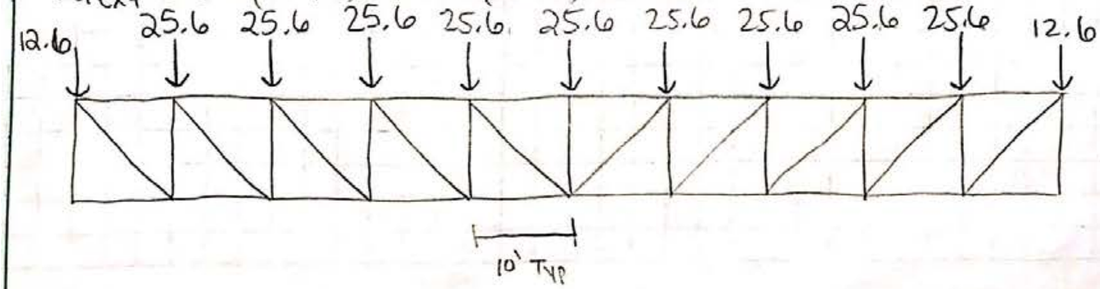


Factored Load

$P_u = 1.2D + 1.6S$

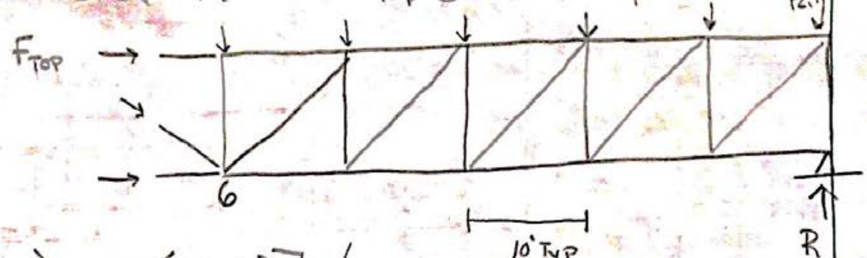
$P_{u\,int} = 1.2(7.18) + 1.6(10.6) = 25.6$

$P_{u\,ext} = 1.2(3.46) + 1.6(5.3) = 12.6^K$



Roof Truss

Calculate force in the top chord of panel #5



$R = [9(25.6) + 2(12.6)]/2$

$R = 127.8^k$

$\curvearrowleft + M_6 = 0$

$F_{Top}(4) - (127.8)(50) + 12.6(50) + 25.6(10) + 25.6(20) + 25.6(30) + 25.6(40)$

$F_{Top} = 200^k$

$\phi P_n = 309^k > 200^k$ ✓

AISC Steel Construction Manual Table 4-1

Diagonals

$\Sigma F_y = 0 = 127.8 - 12.6 - (4/10.77) F_D$

$F_D = 310.2^k$

$\phi P_n = 353^k > 310.2^k$ ✓

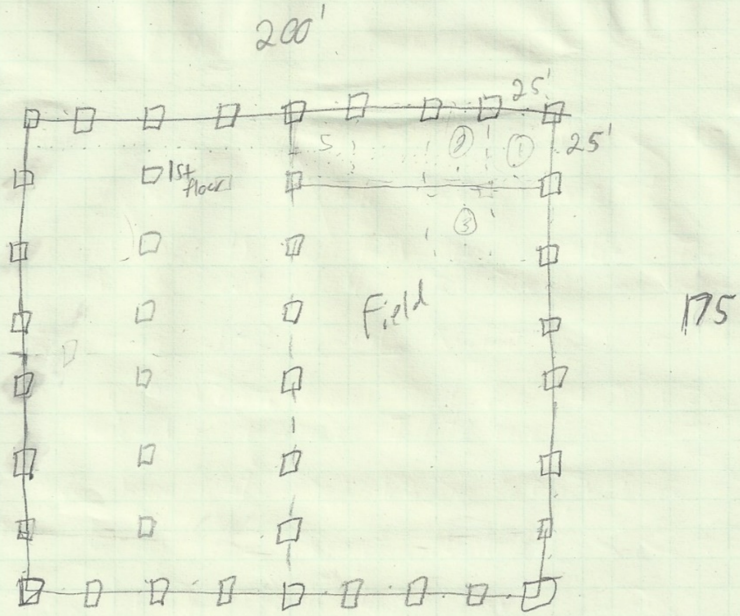
BAY PROPERTIES		UNITS
Truss Length (ft)	100.000	ft
Truss Spacing (ft)	25.000	ft
Vertical Member Spacing	10.000	ft
Number of Diagonals	10.000	Members
Number of Horizontal Members	20.000	Members
Number of Vertical Members	11.000	Members
Bar Joist Spacing	10.000	ft
Imposed Dead Loads		
roof deck (psf)	10.000	psf
insulation (psf)	2.000	psf
MEP (psf)	5.000	psf
Bar Joists	1.000	psf
ceiling (psf)	3.000	psf
Total Dead Load (psf)	21.000	psf
Live Loads		
snow (psf)	42.400	psf
roof (psf)	20.000	psf
Infuencial Live Load (psf)	42.400	psf
Member Properties		
Chord Shape	WT 9 x 71.5	
Chord Area	0.146	sf
Diagonal Shape	LL 4 x 4.0 x 3/4	
Diagonal Area	0.076	sf
Diagonal Length	10.770	ft
Vertical Shape	LL 3 x 2.5 x 3/16	
Vertical Area	0.013	sf
Vertical Length	4.000	ft
Truss Self-Weight	7.424	psf
Total Dead Load - Interior Panel	6881.040	lb
Total Dead Load - Exterior Panel	3453.020	lb
Total Live Load - Interior Panel	10600.000	lb
Total Live Laod - Exterior Panel	5300.000	lb

Factored Loads (Interior)		
1.4*D	9633.456	lb
1.2*D+1.6*(Lr or S or R)+(L or 0.5*W)	25217.248	lb
Use this W (sub) u	25217.248	lb
Factored Loads (Exterior)		
1.4*D	4834.228	lb
1.2*D+1.6*(Lr or S or R)+(L or 0.5*W)	12623.624	lb
Use this W (sub) u	12623.624	lb
Forces in Panel #5		
Vertical Reaction	126101.242	lb
Force in Top Chord	788.039	k
Available Strength	809.000	k
Force in Diagonals	305.548	k
Available Strength	353.000	k
Cost Analysis		
$/ton	$3,900.00	
Total Material Cost	$289,542.29	
OTHER PROPERTIES		**UNITS**
phi	0.90	(no units)
Fy	50.00	ksi
Es	29000.000	ksi
Steel Weight	490.000	pcf

Figure 56: Roof truss calculation spreadsheet

E.2 Roof Beam/Girder Calculations

Roof: Beam/Girder | Tributary Beam Width: 5' | 1/6



Roof System

flat vs. Angle

loading: Snow ASCE 7-10
wind-upload
Siesmic?
LL: ppl

Truss vs. Girder/beam

Exposure B, partially expose $C_e = 1.0$ ASCE 7-10 (pg 2.4)
risk category III ASCE 7-10 (pg 2)
$I_s = 1.1$ ASCE 7-10 (pg 4)
$I_i = 1.25$ $I_e = 1.25$
$I_w = 1.00$

$P_{flat} = .7\, C_e\, C_t\, I_s\, P_g$

$= .7(C_e = 1.0)(C_t = 1)(I_s = 1.1)(P_g = 55\text{ psf})$

$\boxed{P_f = 42.4\text{ psf}}$

LL reduction

$L = L_o\left(.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right)$

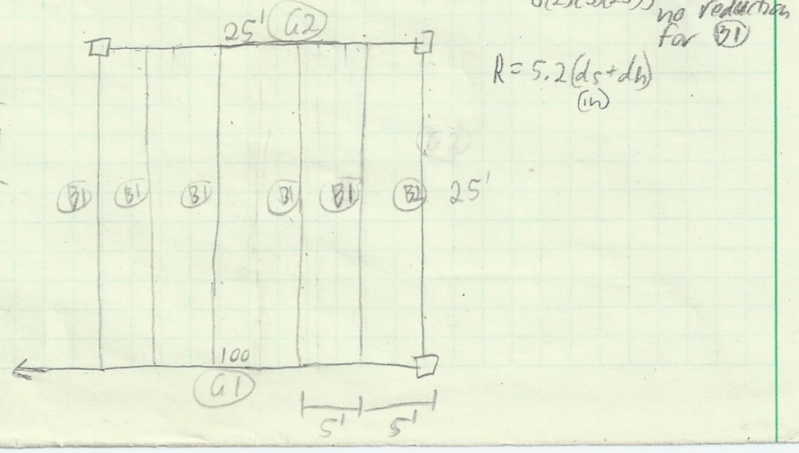
$= 20\text{psf}\left(.25 + \frac{15}{\sqrt{(2)(5')(25')}}\right) = 23.97\text{psf}$

no reduction for B1

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

internal beam = B1
external beam = B2

internal Girder = G1
external Girder = G2



Roof: Beam/Girder (B1) 2

dead load (psf)		live load (psf)	
roof deck	10	snow load (S)	Pf=42.4
insulation	2	roof LL (Lr)	20
MEP	5		
cieling	3		

total : 20psf

W_u

L = 25'

LC w/ W12x16 beam size

$W_u = 1.2D + 1.6S + (16\,lb/ft)(1.2)$
$= 478.4\ lb/ft$

$M_{max} = \frac{(478.4\,lb/ft)(25)^2}{8}$

$M_{max} = 37.38\ 'k$

$Z_x \geq \frac{(37.38\,'k)(12\,''/1)}{(\Phi=.9)(F_y = 50\,ksi)}$

$Z_x \geq 9.97$

beam $Z_x = 20.1 > 9.97$ ✓

(B1) tributary width : 5'
L = 25'

LC
$= 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L_{floor} \text{ or } .5W)$
$= 1.2(20psf)(5') + 1.6(42.4psf)(5')$
$= 459.2\ lb/ft$

$W_u = 459.2\ lb/ft$ factored load

$M_{max} = \frac{W_u L^2}{8} = \frac{(459.2\,lb/ft)(25)^2}{8}$

$M_{max} = 35875\ 'lk$ required capacity

$Z_x \geq \frac{M_u}{\Phi F_y} = \frac{35.88\,'k(12''/1)}{(\Phi=.9)(F_y = 50ksi)}$

$Z_x = 9.57\ in^3$

Selected beam: W12x16
$Z_x = 20.1\ in^3$
$\Phi_b M_{px} = 75.4\ k'$
weight = 16 lb/ft
$I = 103\ in^4$

deflection

$\Delta_{Lmax} = \frac{5(\frac{1}{2}LL)L^4}{384 E_s I_x} = \frac{5(\frac{1}{2})(42.4psf)(5')(25)^4(1728\,in^3/ft^3)}{384(E = 29\times10^6 psi)(I = 103\,in^4)}$

$\Delta_{Lmax} = .312\ in < \frac{L}{360} = \frac{25'(12)}{360} = \frac{300}{360} = .833 in$

$\Delta_{Tmax} = \frac{5(\frac{1}{2}LL + DL)L^4}{384 E_s I_x} \leq \frac{L}{240} = \frac{(25')(12''/1)}{240} = 1.25''$

$\Delta_{Tmax} = \frac{5[(\frac{1}{2})(42.4psf)(5') + (20psf)(5') + 16\,lb/ft](25')^4(1728\,in^3/ft^3)}{384(E_s = 29\times10^6 psi)(I_x = 103 in^4)}$

$\Delta_{Tmax} = .653\ in < 1.25''$

Roof: Beam/Girder | B2 | | 3

dead load: 20 psf

Live load: $S = 42.4$ psf ←

$LL_r = 20$ psf

Tributary width $= \frac{5'}{2} = 2.5'$

$L = 25'$ $E = 29\times10^6$ psi

$\phi = .9$

$F_y = 50$ ksi

LC

$W_u = 1.4D = (1.4)(20psf)(2.5') = 70^{lb}/ft$

$W_u = 1.2D + 1.6(S)$

$= (1.2)(20psf)(2.5') + 1.6(42.4psf)(2.5')$

$= 229.6^{lb}/ft$ ← Governing

$M_{max} = \frac{W_u L^2}{8} = \frac{(229.6^{lb}/ft)(25)^2}{8}$

$M_{max} = 17.94\ ^{\cdot k}$

$Z_x \geq \frac{M_u}{\phi F_y} = \frac{(17.94^{\cdot k})(12''/')}{(\phi = .9)(F_y = 50ksi)}$

$Z_x \geq 4.78\ in^3$

$$\boxed{\begin{array}{l}\text{Selected beam: } W12\times16 \\ Z_x = 20.1\ in^3\end{array}}$$

LC w/ beam

$W_u = 1.4D = 70^{lb}/ft + 1.4(16^{lb}/ft) = 92.4$

$W_u = 1.2D + 1.6(S) = 229.6^{lb}/ft + 1.2(16^{lb}/ft)$

$W_u = 248.8\ ^{lb}/ft$

$M_{max} = \frac{(248.8^{lb}/ft)(25')^2}{8} = 19.44\ ^{\cdot k}$

$Z_x = \frac{(19.44^{\cdot k})(12''/')}{(\phi = .9)(F_y = 50ksi)}$

$Z_x = 5.18 < Z_x = 20.1 in^3$ ✓

deflection

$\Delta_{max} = \frac{5(\frac{1}{2}LL)L^4}{384 E_s I_x} = \frac{(2.5')5)(\frac{1}{2})(42.4psf)(25)^4(1728\ in^3/ft^3)}{(384)(E = 29\times10^6 psi)(I_x = 103\ in^4)}$

$\Delta_{max} = .155''\ in < \frac{L}{360} = \frac{25'(12''/')}{360} = .833 in$ ✓

$\Delta_{T max} = \frac{5(\frac{1}{2}LL + DL)L^4}{384 E_s I_x}$

$= \frac{5[\frac{1}{2}(42.4psf)(2.5') + 20psf(2.5') + 16^{lb}/ft](25')^4(1728\ in^3/ft^3)}{384(29\times10^6 psi)(I_x = 103\ in^4)}$

$= .35_{in} < \frac{L}{240} = \frac{25'(12''/')}{240} = 1.25\ in$ ✓

Roof: Beam/Girder (G1) 4

dead load: $20psf + \frac{(19 \text{ beams})(16^{lb}/ft)(25')}{(25' \times 100')} = 23.04 psf$

$L = 100'$
$\phi = .9$
$F_y = 50ksi$
$E = 29 \times 10^6 psi$
Tributary width $= 25'$

Live load $= S = 42.4 psf$
$L_r = 20 psf$

LC

$W_u = 1.4D = 1.4(23.04psf)(25') = 806.4^{lb}/ft$

$W_u = 1.2D + 1.6(S) = 1.2(23.04psf)(25') + 1.6(42.4psf)(25')$

$W_u = 2387.2^{lb}/ft \leftarrow$

$M_{max} = \frac{W_u L^2}{8} = \frac{(2387.2^{lb}/ft)(100')^2}{8} = 2984'^k$

$Z_x \geq \frac{M_{max}}{\phi F_y} = \frac{2984'^k(12''/1)}{(\phi=.9)(F_y=50ksi)}$

$Z_x \geq = 795.73 in^3$

Selected beam: W40×593
$Z_x = 2760 in^3$
$I_x = 50400 in^4$

$\Delta_{Tmax} = \frac{5(\frac{1}{2}LL + DL)(L)^4}{384 E_s I_x} < \frac{L}{240} = \frac{100'(12)}{240}$

$\Delta_{Tmax} = \frac{5[(\frac{1}{2})(42.4)(25') + (23.04psf)(25') + (593^{lb}/ft)](100')^4 (1728^{in^3}/1)}{384(29 \times 10^6 psi)(I_x = 50400 in^4)}$

$\Delta_{Tmax} = 2.62in < \frac{L}{240} = 5in$

LC w/ beam

$W_u = 1.4D = 806.4^{lb}/ft + 1.4(593^{lb}/ft) = 1636.6^{lb}/ft$

$W_u = 1.2D + 1.6(S) = 2387.2^{lb}/ft + 1.2(593^{lb}/ft)$

$W_u = 3098^{lb}/ft$

$M_{max} = \frac{W_u L^2}{8} = \frac{(3098^{lb}/ft)(100')^2}{8} = 3873.5'^k$

$Z_x \geq \frac{M_{max}}{\phi F_y} = \frac{3873.5'^k(12''/1)}{(\phi=.9)(F_y=50ksi)}$

$Z_x \geq 1032.9 in^3 < Z_x = 2760 in^3$

deflection

$\Delta_{max} = \frac{5(\frac{1}{2}LL)(L)^4}{384 E_s I_x} = \frac{5(\frac{1}{2})(42.4psf)(100')^4(1728^{in^3}/ft^3)(25')}{384(E_s = 29 \times 10^6 psi)(I_x = 50400 in^4)}$

$\Delta_{max} = .82 in \qquad \frac{L}{360} = \frac{(100')(12''/1)}{360} = 3.33'' > 1''$

$\Delta_{max} = .82 in < 1 in$

Roof: beam/Girder (G2) 5

dead load: $20psf + \frac{(4 \text{ beams})(16^{lb}/ft)(12.5)}{(25' \times 12.5)} = 22.56 psf$

$L = 25'$
$\phi = .9$
$F_y = 50ksi$
$E = 29 \times 10^6 psi$
Tributary width $= \frac{25}{2} = 12.5'$

Live load $= S = 42.4 psf \leftarrow$
$L_r = 20 psf$

LC

$W_u = 1.4D = 1.4(22.56^{lb}/ft)(12.5') = 394.8^{lb}/ft$

$W_u = 1.2D + 1.6(S) = 1.2(22.56^{lb}/ft)(12.5') + 1.6(42.4psf)(12.5')$
$W_u = 1186.4^{lb}/ft$

$M_{max} = \frac{W_u L^2}{8} = \frac{(1186.4^{lb}/ft)(25')^2}{8} = 92.69^{'K}$

$Z_x \geq \frac{M_{max}}{\phi F_y} = \frac{(92.69^{'K})(12''/1)}{(\phi = .9)(F_y = 50ksi)}$

$Z_x \geq 24.72 in^3$

Selected beam: W12x22
$Z_x = 29.3 in^3$
$I_x = 156 in^4$

$\Delta_{T_{max}} = \frac{5(\frac{1}{2}LL + DL)L^4}{384 E_s I_x} < \frac{L}{240}$

$\Delta_{T_{max}} = \frac{5[(\frac{1}{2})(42.4psf)(12.5) + (22.56psf)(12.5') + 22^{lb}/ft)(25')^4 (1728 in^3/ft^3)}{384 (E_s = 29 \times 10^6 psi)(I_x = 156 in^4)}$

$\Delta_{T_{max}} = 1.74 in < \frac{L}{240} = \frac{25'(12''/1)}{240} = 1.25 in$

✓

LC w/ Girder

$W_u = 1.4D = 394.8 + 1.4(22^{lb}/ft) = 425.6^{lb}/ft$

$W_u = 1.2D + 1.6(S) = 1186.4^{lb}/ft + 1.2(22^{lb}/ft)$
$W_u = 1212.8^{lb}/ft$

$M_{max} = \frac{W_u L^2}{8} = \frac{1212.8^{lb}/ft (25')^2}{8} = 94.75^{'K}$

$Z_x \geq \frac{M_{max}}{\phi F_y} = \frac{94.75^{'K}(12''/1)}{(\phi = .9)(F_y = 50ksi)} = 25.27 in^3$

$Z_x = 25.27 in^3 < Z_x = 26.0 in^3$

deflection

$\Delta_{max} = \frac{5(\frac{1}{2}LL)(L^4)}{384 E_s I_x} = \frac{5(\frac{1}{2})(42.4psf)(25')^4(1728 in^3/ft^3)(12.5)}{384(E = 29 \times 10^6 psi)(I_x = 156 in^4)}$

$\Delta_{max} = .51 in < \frac{L}{360} = \frac{(25')(12''/1)}{360} = .833 in$ ✓

BAY PROPERTIES		**UNITS**
Beam Length (ft)	25.000	ft
Beam Spacing (ft)	2.500	ft
Girder Length (ft)	25.000	ft
Girder Spacing (ft)	25.000	ft
Dead Loads		
roof deck (psf)	10.000	psf
insulation (psf)	2.000	psf
MEP (psf)	5.000	psf
ceiling (psf)	3.000	psf
Total Dead Load (psf)	20.000	psf
Live Loads		
snow (psf)	42.400	psf
roof (psf)	20.000	psf
Infuencial Live Load (psf)	42.400	psf
OTHER PROPERTIES		**UNITS**
phi	0.90	(no units)
F (sub) y	50.00	ksi
E (sub) s	29000.000	ksi

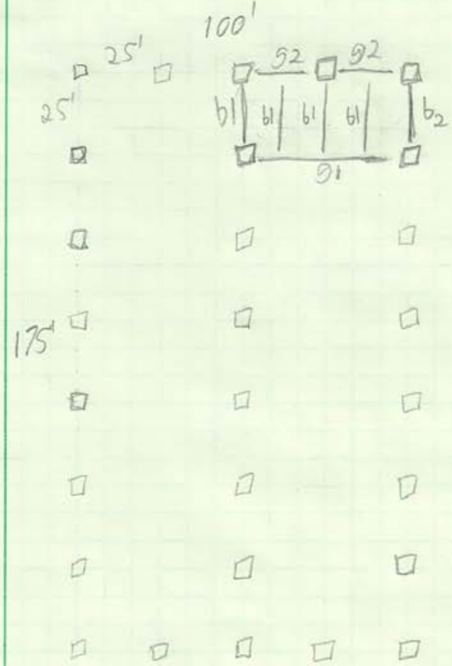
W (sub) u		
1.4*D	70.000	lb/ft
1.2*D+1.6*(Lr or S or R)+(I	229.600	lb/ft
Use this W (sub) u	229.600	lb/ft
M (sub) max	17.938	k-ft
Z (sub) x [greater than or	4.783	in^3
Find a Zx value that is greater than this in the steel manual		
Z (sub) x	4.783	in^3
W 12x16		
dead load of beam	16.000	lb/ft
Z (sub) x	20.100	in^3
I (sub) x	103.000	in^4
Updated loads		
W (sub) u	248.800	lb/ft
M (sub) max	19.438	k-ft
Z (sub) x	5.183	in^3
beam is ok?		
delta L	0.156	in
L/360	0.833	in
1 inch max	1.000	in
deflection restraint	0.833	in
is live load deflection ok?		
delta T	0.350	in
L/240	1.250	in
is total deflection ok?		

Figure 57: Building roof beam/girder calculation spreadsheet

E.3 Building 2nd Floor Calculations

2nd Floor

1



loading (LL)	load psf
School classroom	40psf
School corridors above 1st floor	80 psf
gym, main floors, balcony	100 psf

LL reduction

$$L = L_0\left(.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right)$$

K_{LL} values	
interior beam	2
edge beam w/out cantilever	2

2^{nd} Floor: b1

2

dead load	psf	Live load	psf
flooring	10	assembly	100
MEP	5	Area	
cieling	3		

Tributary width: 12.5'
$K_{LL} = 2$
length: 25'
$\Phi = .9$ $F_y = 50ksi$ $E_s = 29\times10^6 psi$

LL reduction

$L = L_0\left(.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right)$

$L = (100psf)\left(.25 + \frac{15}{\sqrt{(2)(25' \times 12.5')}}\right)$

$L = 85psf$

LC

$W_u = 1.4D = 1.4(18psf)(12.5') = 315\ lb/ft$

$W_u = 1.2D + 1.6L$

$= 1.2(18psf)(12.5') + 1.6(85psf)(12.5')$

$W_u = 1970\ lb/ft$ ←

$M_{max} = \frac{W_u L^2}{8} = \frac{(1970\ lb/ft)(25')^2}{8}$

$M_{max} = 153.91^{1k}$ required capacity

$Z_x \geq \frac{M_{max}}{\Phi F_y} = \frac{(153.91^{1k})(12''/1')}{(.9)(50ksi)}$

$Z_x \geq 41.04 in^3$

Selected beam: W14x30
$Z_x = 47.3\ in^3$
$I_x = 291\ in^4$

LC w/ beam

$W_u = 1.4D + 1.4(30\ lb/ft) = 357\ lb/ft$

$W_u = 1.2D + 1.6L = 1970 + 1.2(30\ lb/ft)$
$= 2006\ lb/ft$

$M_{max} = \frac{(2006\ lb/ft)(25')^2}{8}$

$M_{max} = 156.72^{1k}$

$Z_x = \frac{(156.72^{1k})(12''/1')}{(.9)(50ksi)}$

$Z_x = 41.79 < Z_x = 47.3 in^3$ ✓

Deflection

$\Delta_{max} = \frac{5(\frac{1}{2}LL)L^4}{384 E_s I_x} = \frac{5(\frac{1}{2})(85psf)(12.5')(25')^4(1728\ in^3/ft^3)}{(384)(E_s = 29\times10^6 psi)(I_x = 291\ in^4)}$

$= .55'' < \frac{L}{360} = \frac{25'(12''/1')}{360} = .83 in$ ✓

$\Delta_{Tmax} = \frac{5(\frac{1}{2}LL + DL)L^4}{384 E_s I_x} = \frac{5\left[\frac{1}{2}(85psf)(12.5') + (18psf)(12.5') + 30\ lb/ft\right](25')^4(1728\ in^3/ft^3)}{384\ (29\times10^6 psi)(I_x = 291 in^4)}$

$= .82 in < \frac{L}{240} = \frac{25'(12''/1')}{240} = 1.25''$ ✓

2^{nd} floor: b2 | 3

dead load

flooring 10 psf
MEP 5 psf
ceiling 3 psf

live load
assembly Area 100 psf

Tributary width = 6.25'
$K_{LL} = 2$
length = 25'
$\phi = .9$ $F_y = 50ksi$ $E_s = 29\times10^6 psi$

LL reduction

$L = L_o\left(.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) = (100psf)\left(.25 + \frac{15}{\sqrt{2(25'\times6.25')}}\right) = 109$ ⇐ No LL reduction

LC

$W_u = 1.4D = 1.4(18psf)(6.25') = 157.5\ ^{lb}/ft$

$W_u = 1.2D + 1.6L$
$= 1.2(18psf)(6.25') + 1.6(100psf)(6.25')$
$W_u = 1135\ ^{lb}/ft$ ←

$M_{max} = \frac{W_u L^2}{8} = \frac{(1135\ ^{lb}/ft)(25')^2}{8}$
$= 88.67^{1k}$

$Z_x \geq \frac{M_{max}}{\phi F_y} = \frac{88.67^{1k}(12"/')}{(\phi=.9)(F_y=50ksi)}$

$Z_x \geq 23.65\ in^3$

$\boxed{\text{Selected beam: W12x19} \quad Z_x = 24.7\ in^3 \quad I_x = 130\ in^4}$

LC w/ beam

$W_u = 1.4D$
$= 1.4(18psf)(6.25') + 1.4(19\ ^{lb}/ft)$
$= 188.3\ ^{lb}/ft$
$W_u = 1.2D + 1.6L$
$= 1135\ ^{lb}/ft + 1.2(19\ ^{lb}/ft)$
$= 1157.8\ ^{lb}/ft$ ←

$M_{max} = \frac{W_u L^2}{8} = \frac{(1157.8\ ^{lb}/ft)(25')^2}{8}$
$= 90.45^{1k}$

$Z_x = \frac{M_{max}}{\phi F_y} = \frac{(90.45^{1k})(12"/')}{(.9)(50ksi)}$

$Z_x = 24.12\ in^3 < Z_x = 24.7\ in^3$ ✓

Deflection

$\Delta_{Lmax} = \frac{5(\frac{1}{2}LL)L^4}{384E_sI_x} = \frac{5(\frac{1}{2})(100psf)(6.25')(25')^4(1728\ in^3/ft^3)}{384(29\times10^6psi)(I_x = 130in^4)}$

$= .73\ in < \frac{L}{360} = \frac{25'(12"/')}{360} = .83in$ ✓

$\Delta_{Tmax} = \frac{5(\frac{1}{2}LL + DL)(L^4)}{384E_s I_x} = \frac{5[(\frac{1}{2})(100psf)(6.25') + 18psf(6.25') + 19\ ^{lb}/ft](25')^4(1728\ in^3/ft^3)}{384(29\times10^6psi)(I_x = 130in^4)}$

$= 1.03in < \frac{L}{240} = \frac{25'(12"/')}{240} = 1.25in$ ✓

2^{nd} Floor: 91 | 4

dead loads	psf	live load	psf
flooring	10	Assembly Area	100
MEP	5		
cieling	3		

Tributary width = 25'
$K_{LL} = 2$
length = 50'
$\phi = .9$ $F_y = 50ksi$ $E_s = 29\times10^6 psi$

total dead load $= 18psf + \frac{(3\ 61)(30^{lb}/ft)(25')}{(50'\times25')} = 19.8psf$

LL reduction
$L = L_o\left(.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) = 100psf\left(.25 + \frac{15}{\sqrt{2(50'\times25')}}\right) = 55psf$

LC
$W_u = 1.4D = 1.4(19.8psf)(25')$
$= 693\ ^{lb}/ft$
$W_u = 1.2D + 1.6L$
$= 1.2(19.8psf)(25') + 1.6(55psf)(25')$
$= 2794\ ^{lb}/ft$

LC w/ beam
$W_u = 1.4D = 693^{lb}/ft + 1.4(99^{lb}/ft)$
$= 831.6^{lb}/ft$
$W_u = 1.2D + 1.6L$
$= 2794^{lb}/ft + 1.2(99^{lb}/ft)$
$= 2912.8^{lb}/ft$

$M_{max} = \frac{W_u L^2}{8} = \frac{(2794^{lb}/ft)(50')^2}{8}$
$= 873.13^{1k}$

$M_{max} = \frac{(2912.8^{lb}/ft)(50')^2}{8}$
$= 910.25^{1k}$

$Z_x = \frac{M_{max}}{\phi F_y} = \frac{(873.13^{1k})(12''/1')}{(.9)(50ksi)}$

$Z_x = 232.83 in^3$

$Z_x = \frac{M_{max}}{\phi F_y} = \frac{(910.25^{1k})(12''/1')}{(.9)(50ksi)}$

$Z_x = 242.73 in^3 < Z_x = 312\ in^3$ ✓

$$\boxed{\begin{array}{l}\text{Selected beam: W30x99}\\ Z_x = 312\ in^3\\ I_x = 3990 in^4\end{array}}$$

Deflection
$\Delta_{Lmax} = \frac{5(\frac{1}{2}LL)L^4}{384 E_s I_x} = \frac{5(\frac{1}{2})(55psf)(25')(50)^4(1728\ ^{in^3}/ft^3)}{384(29\times10^6 psi)(3990 in^4)}$

$= .83\ in < \frac{L}{360} = \frac{50(12')}{360} = 1.67 in$ or $1 in$ ✓

$\Delta_{Tmax} = \frac{5[\frac{1}{2}(L+D)L]L^4}{384 E_s I_x}$

$= \frac{5[(\frac{1}{2})(55psf)(25') + 19.8psf(25') + 99^{lb}/ft](50')^4(1728^{lb}/ft)}{384(29\times10^6 psi)(I_x = 3990 in^4)}$

$= 1.56\ in < \frac{L}{240} = \frac{50'(12''/1')}{240} = 2.5 in$ ✓

2^{nd} floor: 92 5

dead load	psf	live load	psf
flooring	10	assembly Area	100
MEP	5		
ceiling	3		

Tributary width = 12.5'
$K_{LL} = 2$
length = 25'
$\Phi = .9$ $F_y = 50ksi$ $E = 29\times10^6 psi$

total dead load $= (18psf) + \frac{(1 beam)(30^{lb}/ft)(12.5')}{(25' \times 12.5')} = 19.2psf$

LL reduction

$L = L_0\left(.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) = 100psf\left(.25 + \frac{15}{\sqrt{(2)(12.5\times25')}}\right) = \underline{85psf}$

LC

$W_u = 1.4D = 1.4(19.2psf)(12.5')$
$= 336\ ^{lb}/ft$
$W_u = 1.2D + 1.6L$
$= 1.2(19.2psf)(12.5') + 1.6(85psf)(12.5')$
$= 1988\ ^{lb}/ft$

$M_{max} = \frac{W_u L^2}{8} = \frac{(1988\ ^{lb}/ft)(25')^2}{8}$

$M_{max} = 155.31\ ^{1k}$

$Z_x \geq \frac{M_{max}}{\phi F_y} = \frac{(155.31\ ^{1k})(12^{in}/_1)}{(.9)(50ksi)}$

$Z_x \geq 41.42\ in^3$

$\boxed{\begin{array}{l}\text{Selected beam: W14x30}\\ Z_x = 47.3\ in^3\\ I_x = 291\ in^4\end{array}}$

LC w/beam

$W_u = 1.4D = 336\ ^{lb}/ft + 1.4(30\ ^{lb}/ft)$
$= 378\ ^{lb}/ft$
$W_u = 1.2D + 1.6L$
$= 1988\ ^{lb}/ft + 1.2(30\ ^{lb}/ft)$
$= 2024\ ^{lb}/ft$

$M_{max} = \frac{(2024\ ^{lb}/ft)(25')^2}{8}$
$= 158.13\ ^{1k}$

$Z_x = \frac{M_{max}}{\phi F_y} = \frac{(158.13\ ^{1k})(12^{in}/_1)}{(.9)(50ksi)}$

$Z_x = 42.17\ in^3 > Z_x = 47.3\ in^3$ ✓

$\Delta_{Lmax} = \frac{5(\frac{1}{2}LL)L^4}{384 E_s I_x} = \frac{5(\frac{1}{2})(85psf)(12.5')(25')^4(1728\ in^3/ft^3)}{384(29\times10^6 psi)(291\ in^4)}$

$\underline{= .55in} < \frac{L}{360} = \frac{25'(12^{in}/_1)}{360} = .83in$ ✓

$\Delta_{Tmax} = \frac{5[\frac{1}{2}LL + DL]L^4}{384 E_s I_x}$

$= \frac{5[(\frac{1}{2})(85psf)(12.5') + (19.2psf)(12.5') + 30\ ^{lb}/ft](25'^4)(1728\ in^3/ft^3)}{384(29\times10^6 psi)(291\ in^4)}$

$\underline{= .83in} < \frac{L}{240} = \frac{25'(12^{in}/_1)}{240} = 1.25in$ ✓

2nd Floor: Cost 6

member	size	# of members	total length	total weight	total cost
b1	W14x30	49	1225'	36,750 lb	$71,662.5
b2	W12x19	14	350'	6650 lb	$12,967.5
g1	W30x99	12	600'	59,400 lb	$115,830
g2	W14x30	8	200'	6000 lb	$11,700

Σ $212,160 cost for 2nd floor structural members

Assumed cost per ton of Steel = 3900 $/ton

example calculation

b1 W14x30 $L = 25'$

$$\text{total length} = \#\text{ of members} \times L = 49 \times 25' = 1225'$$

$$\text{total weight} = \text{total length} \times \text{weight per ft} = 1225' \times 30\,\text{lb/ft} = 36{,}750\,\text{lb}$$

$$\text{total cost} = \text{total weight} \times \text{cost of material} = 36{,}750\,\text{lb}\left(\frac{1\,\text{ton}}{2000\,\text{lb}}\right)\left(3900\,\$/\text{ton}\right)$$

$$= \$71{,}162.5$$

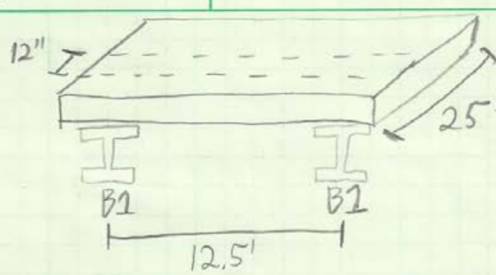
BAY PROPERTIES		**UNITS**	**W (sub) u**		
Beam Length (ft)	25.000	ft	1.4*D	927.500	lb/ft
Beam Spacing (ft)	12.500	ft	1.2*D+1.6*(Lr or S or R)+(L or 0.5*W)	2495.000	lb/ft
Girder Length (ft)	25.000	ft	Use this W (sub) u	2495.000	lb/ft
Girder Spacing (ft)	25.000	ft			
Dead Loads			**M (sub) max**	194.922	k-ft
flooring	45.000	psf	**Z (sub) x [greater than or equal to]**	51.979	in^3
MEP (psf)	5.000	psf			
ceiling (psf)	3.000	psf	Find a Zx value that is greater than this in the steel manual		
Total Dead Load (psf)	53.000	psf	Z (sub) x	51.979	in^3
Live Loads					
assembly area	100.000	psf	**W 14x34**		
Infuencial Live Load (psf)	100.000	psf	dead load of beam	34.000	lb/ft
			Z (sub) x	54.600	in^3
K (sub) LL	2.000		I (sub) x	340.000	in^4
Reduced Live Load	85.000	psf			
Use this live load	85.000	psf	**Updated loads**		
			W (sub) u	2535.800	lb/ft
OTHER PROPERTIES		**UNITS**	M (sub) max	198.109	k-ft
phi	0.90	(no units)	Z (sub) x	52.829	in^3
F (sub) y	50.00	ksi	beam is ok?		
E (sub) s	29000.000	ksi			
			delta L	0.474	in
			L/360	0.833	in
			1 inch max	1.000	in
			deflection restraint	0.833	in
			is live load deflection ok?		
			delta T	1.094	in
			L/240	1.250	in
			is total deflection ok?		

Figure 58: Building 2nd floor calculation spreadsheet

E.4 Building 2nd Floor Slab Calculations

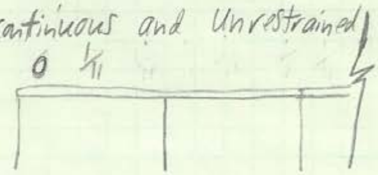
Building: Slab | 1/17/18 | 1/2

One-way slab
$f'_c = 4$ ksi
$f_y = 60$ ksi
$\phi = .9$
Spandrel
LL = 100psf
DL = MEP = 5psf
ceiling = 3psf



estimate h
one-end continuous
$L/24 = 12.5/24\,(12''/1') = 6.25''$
$L = 12.5'$

Discontinuous and Unrestrained



Wu
$DL = (h = 6.25'')(1'/12'')(150pcf) = 78.125$ psf

$W_u = 1.2D + 1.6L = 1.2(86.13psf) + 1.6(100psf)$
$W_u = 263.36$ psf
$W_u = .2634$ ksf $(1') = .2634$ k/ft
Mu

$M_u = (\frac{1}{11})(.2634\ k/ft)(12.5')^2 = 3.74$ k-ft

$\rho_{max} = (.85)(B_1 = .85)\frac{(f'_c = 4ksi)}{(f_y = 60ksi)}\frac{(\varepsilon_u = .003)}{(.003 + .004)} = .021$

$\rho @ .005 = .018$

$M_u = \phi \rho f_y b d^2 (1 - .59 \rho\, f_y/f'_c)$
3.74 k-ft $(12''/1) = (.9)(\rho = .018)(f_y = 60ksi)(b = 12'')\, d^2 (1 - .59(.018)(60/4))$
44.88 k-in $= 11.66\, d^2 (1 - .1593)$
$d^2 = 4.58$
$d \geq 2.14''$

$h = 6.25''$ min slab cover $= .75''$
$d = h - c_c - \frac{1}{2}(d_b) = 6.25'' - .75'' - \frac{1}{2}(.5'') = 5.25''$

Building: Slab | 1/17/18 | | 2/2

$a = 1$

$A_s = \frac{M_u}{\phi f_y (d - \frac{a}{2})} = \frac{3.74^{k\text{-}ft}(12''/_1)}{(.9)(60ksi)(5.25'' - \frac{1}{2})} = .175$

$a = \frac{A_s f_y}{.85 f'_c b} = \frac{(.175'')(60ksi)}{(.85)(4ksi)(12'')} = .257''$

$A_s = \frac{3.74^{k\text{-}ft}(12''/_1)}{(.9)(60ksi)(5.25 - \frac{.257''}{2})} = .16$

$a = \frac{.16(60ksi)}{.85(4ksi)12''} = .235''$

$A_s = \frac{3.74^{k\text{-}ft}(12''/_1)}{(.9)(60ksi)(5.25 - \frac{.235}{2})} = .16$

use $a = .235''$ $A_s = .16$

$A_{s\,min} = (\rho_{min} = .018)(b = 12'')(h = 6.25'') = 1.35$

3#6 bars $A_s = 1.32 in^2$

$V_u = 1.15 \frac{WL}{2} = 1.15\left[\frac{(.2634^{k}/ft)(12.5')}{2} - (.2634^{k}/ft)(\overset{d=}{5.25''})(1'/12'')\right]$

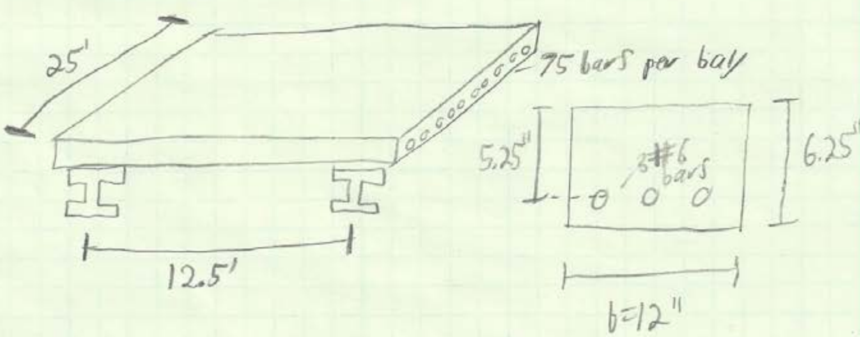
$V_u = 1.15[1.646 - .115)$

$V_u = 1.76^k$

$\phi V_c = \phi 2\sqrt{f'_c}\,bd = (.9)(2)\frac{\sqrt{4000psi}}{1000}(12'')(5.25'') = 7.17^k$

$\phi V_c > V_u$

no shear reinforcement required

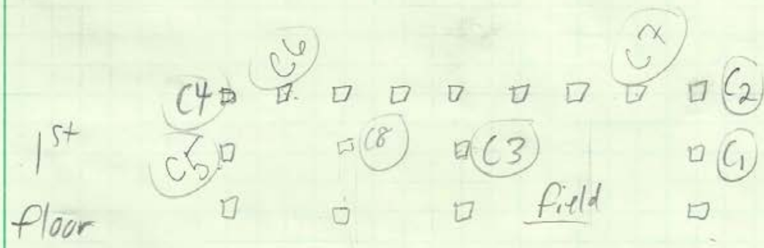


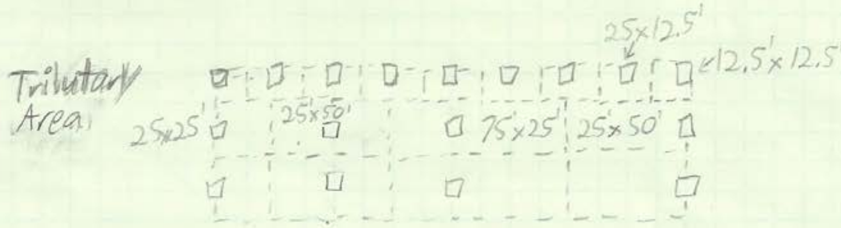
weight $= (150pcf)(6.25'')(1'/12'')$

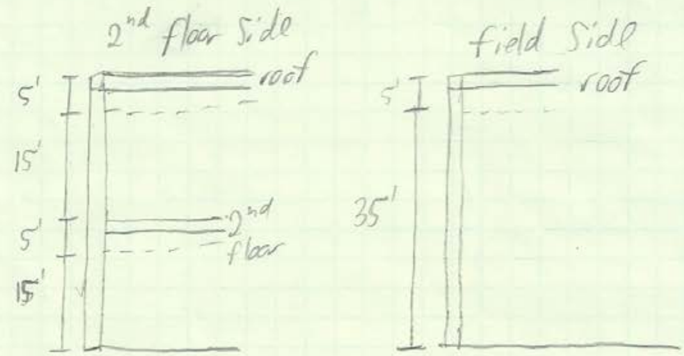
$= 78psf$

E.5 Building Column Calculations

4 Columns layout 1/







Columns: Field Side | (C1) | | 1/

Dead loads	psf	Live loads	psf
roof deck	10	Snow	42.4
insulation	2	roof	20
MEP	5		
bar joists	10		
ceiling	3		
	30 psf		

Truss $\frac{7.424^k}{2} = 3.712^k$

LRFD
Tributary Area $= 25' \times 50' = 1250 ft^2$

$L = 40'$

$F_y = 50 ksi$

$P_D = (30 psf)(1250 ft^2) + 3.712^k$
$= 41,212$ lbs

$P_L = (42.4 psf)(1250 ft^2)$
$= 53,000$ lbs

LC

$w_u = 1.4D = 1.4(41.212^k) = 57.69^k$

$w_u = 1.2D + 1.6L = 1.2(41.212^k) + 1.6(53^k)$
$= 134.24^k \leftarrow$

$W_u = 1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$

$w = 1.2(41.212) + 1.6(0) + 0.5(42.4)$

$w = 75.94$

Table 4-1 $L = 30'$ $P_u = 134.25$ kips

$\boxed{W12 \times 53}$ $\phi P_n = 167^k$ $I_x = 425 in^3$
$r_y = 2.48 in$ $r_x/r_y = 2.11$ $I_y = 95.8$

$L_y/r_y = \frac{30'(12''/')}{2.48 in} = 145.16 \leftarrow$ Governs

$L_x/r_x = \frac{30'(12''/')}{5.23 in} = 68.12$

$L_y/r_y = 145.16 \leq 4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29 \times 10^6 psi}{50,000 psi}} = 113.43$

$F_{cr} = .877 F_e = 0.877(13.58) = 11.91$ ksi

$F_e = \frac{\pi^2 E}{(L/r)^2} = \frac{\pi^2(29 \times 10^3 ksi)}{(145.16)^2} = 13.58$ ksi

$\phi P_n = (\phi = .9)(F_{cr} = 11.91 ksi)(A_g = 15.6 in^2)$

$\phi P_n = 167.25^k > 134.25^k$ ✓

Dead Load		units		Live Load		units
Roof Decking	10.0	psf		Floor Load		psf
Insulation	2.0	psf		Snow Load	42.4	psf
MEP	5.0	psf		Roof Load	20	psf
Bar Joists	1.0	psf				
Ceiling	1.0	psf		Influential Live Load		psf
	19.0	psf				
Truss Dead Load	7.4	kips	per truss per 200'			
Interior Beam	34.0	lb/ft				
Exterior Beam	26.0	lb/ft				
Interior Girder	108.0	lb/ft				
Exterior Girder	34.0	lb/ft				
Column ID	Column Description	Column Size	lumber of Columns			
C1	Exterior field side of building, on the 175' length	W 12x53	6			
C2	Exterior fiield side of building, on the corners	W 12x53	2			
C3	Interior middle of the entire building	W 12x53	6			
C4	Exterior office side of building, on the corners	W 12x53	2			
C5	Exterior office side of building, on the 175' length	W 12x53	6			
C6	Exterior office side of building, on the 200' length	W 12x53	6			
C7	Exterior field side of building, on the 200' length	W 12x53	6			
C8	Interior middle of the office side of the building	W 12x53	6			
C9	Exterior middle of the entire building	W 12x53	2			
		total	42			

Figure 60: Building Column Loading and Descriptions

DIMENSIONS			**LOAD CASE**			
Column Length (Lx)	30.0	ft	1.4*D	41.93	kips	
Column Length (Ly)	30.0	ft	1.2*D + 1.6*L + 0.5*(Lr or S or R)	62.44	kips	
Tributary Area	1250.0	ft^2	1.2*D + 1.6*(Lr or S or R) + (0.5*L or 0.8*W)	120.74	kips	
			P (sub) u (Use this load case)	**120.74**	**kips**	
LOADING						
Roof Decking	10.0	psf	Look at Table 4-1 for a phi Pn value greater than		120.74	kips
Insulation	2.0	psf	**W 12x53**			
MEP	5.0	psf	dead load of beam	53.00	lb/ft	
Bar joists	1.0	psf	phi P(sub)n	167.00	kips	
Ceiling	3.0	psf	I (sub) x	425.00	in^4	
Truss Load	3.7	kips	I (sub) y	95.80	in^4	
Total Dead Load (D)	**30.0**	**kips**	r (sub) x	5.23	in	
Floor Live Load (L)	0.0	psf	r (sub) y	2.48	in	
Snow Load	42.4	psf	r (sub) x / r (sub) y	2.11	no units	
Roof Load	20.0	psf	A (sub) g	15.60	in^2	
Influential Roof Live Load (Lr or S or R)	**42.4**	**psf**				
			CHECK			
MATERIAL PROPERTIES			Lx/rx	68.80	no units	
Yield Strength (F (sub) y)	50.0	ksi	Ly/ry	145.16	no units	
Modulus of Elasticity	29000.0	ksi	governing value (larger) of two above	145.16	no units	
phi	0.9	no units	check (4.71*square root of E/Fy) "reference"	113.43	no units	
			F (sub) e	13.58	ksi	
			F (sub) cr if less than	10.71	ksi	
			F (sub) cr if greater than	11.91	ksi	
			Use this Fcr	**11.91**	**ksi**	
			Phi * P (sub) n	167.25	kips	
			Phi Pn is greater than Pu			

Figure 59: Building Column Example

E.6 Building Lateral Reinforcement Calculations

WIND LOAD | Building | 1/2

ASCE 7-10 wind design loads

$q_z = .00256\, k_z\, k_{zt}\, k_d\, V^2 = .00256\,(.7)(1.045)(.85)(100mph)^2$
$= 15.92\ psi$

$V^2 = 100mph$

$k_z = .7$ exposure B $z = 30'$ table 27.3-1 pg 261

$k_{zt} = (1 + k_1 k_2 k_3)^2$
$= 1.045$

$x/L = 50'/50' = 1.0$ $z/L = 42/50 = .84$ Figure 26.8-1 page 252
$H/L = 12'/50' = .24$ 2D escarpment
$k_1 = .21$ $k_2 = .75$ $k_3 = .14$

$K_d = .85$ Table 26.6-1
pg 250

$A_T = 25' \times 50' = 1250ft^2$

live load roof : 42.4 psf $= 53000 lb = 53^k$
2nd floor : 100 psf $= 125{,}000 lb = 125^k$
dead load roof : 7.4^k
2nd floor : $30 lb/ft = .03^k/ft\,(25') = .75^k$

$W = 28.7^k$

LC ASCE 7-10

$= 1.4D = 11.41^k$

$= 1.2D + 1.6L + .5(Lr\ or\ S\ or\ R) = 233.16^k$

$= 1.2D + 1.6(Lr\ or\ S\ or\ R) + (L\ or\ .5W) = 216.46^k$

$= 1.2D + W + L + .5(Lr\ or\ S\ or\ R) = 213.36^k$

$= 1.2D + E + .2S = ?$

$= .9D + W = 33.69^k$

$= .9D + E = ?$

story drift $< h/500 = 360''/500 = .72''$

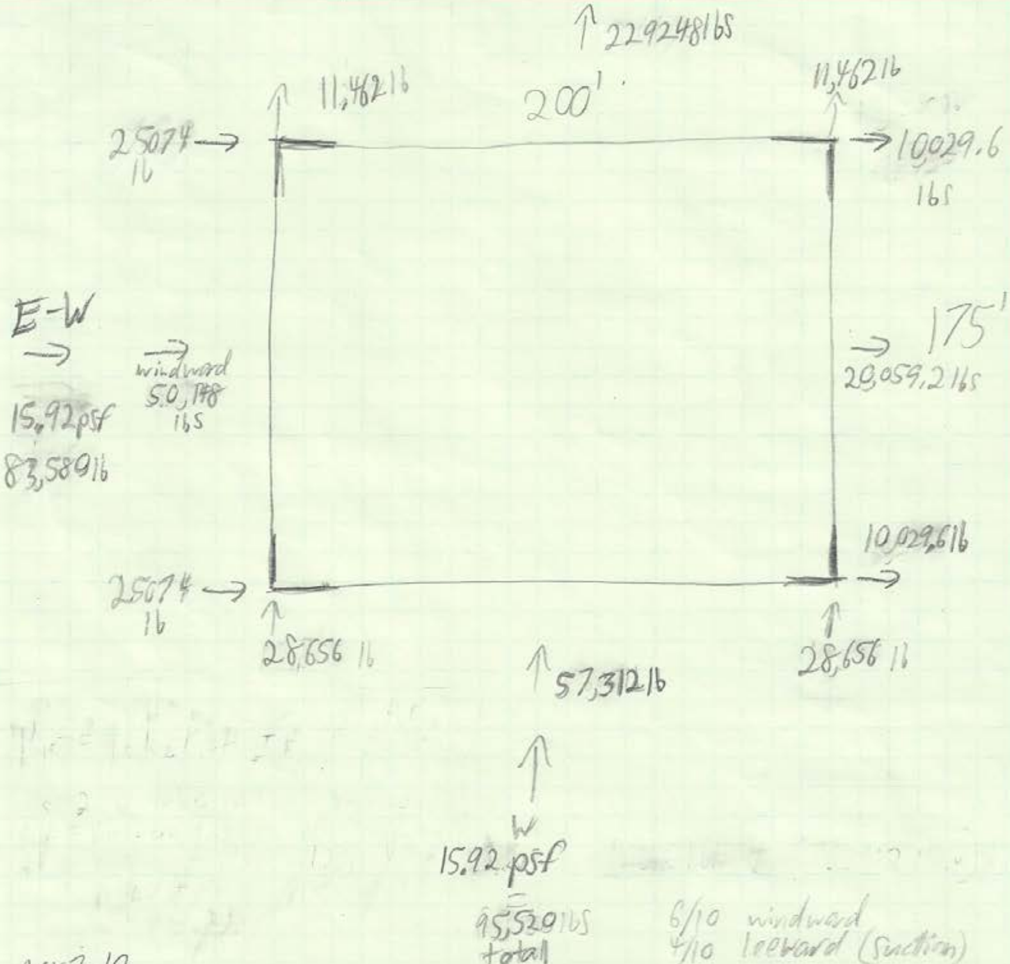
ASCE 7-05

$= 1.2D + 1.6W + .5(Lr\ or\ S\ or\ R)$

$= 1.2D + 1.6W + 1.6H$

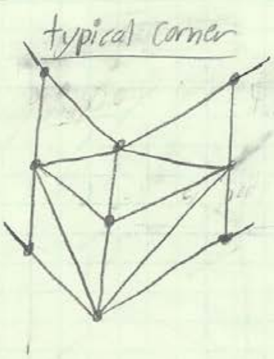
Wind load

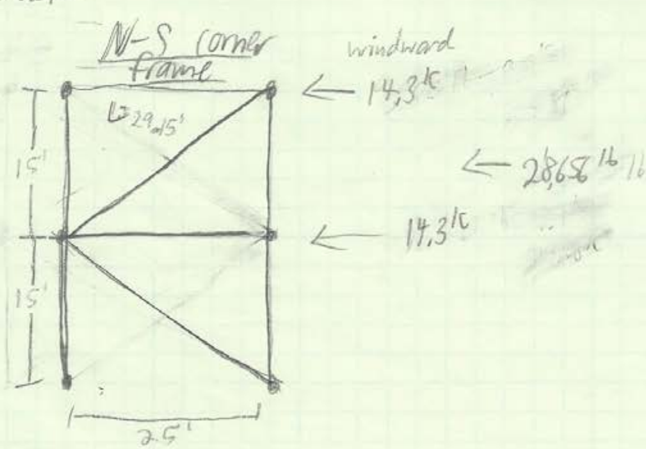
2/2



ACE7-10

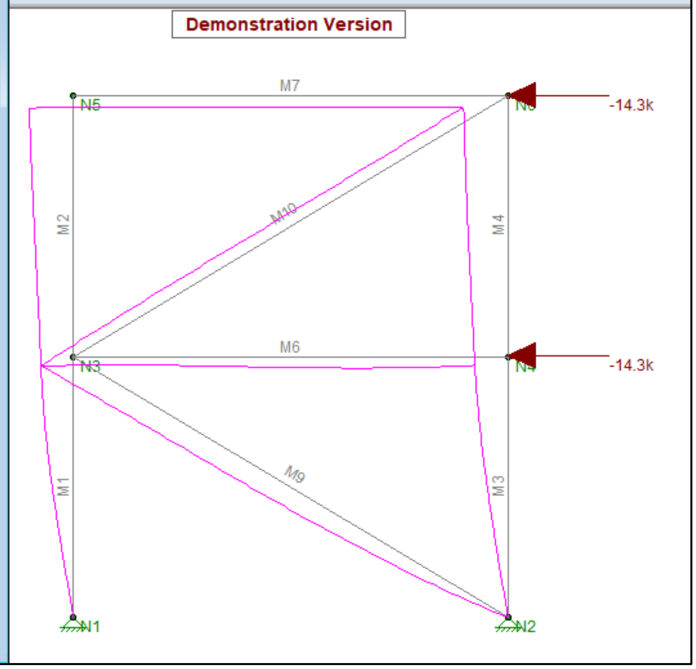
$q_z = .00256 K_z K_d K_{zt} V^2 \text{ psf} = 15.92 \text{ psf}$





Member Section Deflections (By Combination)

	LC	Member Label	Sec	x [in]	y [in]	(n) L/y ...
14			4	-.107	.486	3270.257
15			5	-.142	.575	NC
16	1	M4	1	-.142	.575	NC
17			2	-.153	.634	NC
18			3	-.164	.689	9943.128
19			4	-.175	.736	NC
20			5	-.186	.768	NC
21	1	M6	1	-.566	-.157	NC
22			2	-.568	-.138	NC
23			3	-.57	-.17	NC
24			4	-.573	-.189	6927.644
25			5	-.575	-.142	NC
26	1	M7	1	-.767	-.203	NC
27			2	-.767	-.179	NC
28			3	-.767	-.188	NC
29			4	-.768	-.199	NC
30			5	-.768	-.186	NC
31	1	M9	1	0	0	NC
32			2	.101	.253	2389.661
33			3	.202	.351	2525.032
34			4	.303	.38	5695.331
35			5	.405	.425	NC
36	1	M10	1	-.566	.157	NC
37			2	-.613	.207	NC
38			3	-.66	.215	NC
39			4	-.707	.214	NC
40			5	-.754	.236	NC



E.7 Building Seismic Calculations

building: Siesmic load | | | 1/

using Seismic Design CodeMaster ASCE 7-10, IBC 2012

① $S_s = .24$
$S_1 = .067$ } 780 CMR: MA amendment to IBC 2009

② exemption → none

③ Siesmic Design Category

Site Class D → Soil classification unknown

$S_{DS} = (2/3) F_a S_s = (2/3)(1.6)(.24) = .256$

F_a → ASCE 7-10 table 11.4-1
$F_a = 1.6$

$S_{D1} = (2/3) F_v S_1 = (2/3)(\cancel{1.5}\ 2.4)(.067) = \cancel{.067}\ .107$

F_v → ASCE 7-10 table 11.4-2
$F_v = \cancel{1.5}\ 2.4$

risk category → II - ASCE 7-10 table 1.5-1

Siesmic design Category → B ASCE 7-10 table 11.6-1

④ determine Analysis procedure
Simplified design Procedure → ASCE 7-10 12.14

⑤ $R^* = 3$ → ASCE 12.2-1 part H

⑥ $I_e = 1.0$ → Codemaster

⑦ Siesmic Base Shear

$V = C_s W$ → ASCE 7-10 12.8.1

$T_s = S_{D1}/S_{DS}$ → codemaster
$= \cancel{.067/.256} = .107/.256$
$T_s = .418$
$T_L = 6$ → ASCE 7-10 22-12

$T = T_a = C_t h_a^x = .02(15')^{.75}$ → ASCE 7-10 12.8-2
$T = .15$

$T < T_s$

$V = \left[\frac{S_{DS}}{R/I_e}\right] W$ → Code Master

$V = \left[\frac{.256}{(3)/(1.0)}\right] W = .085 W$

2/

$V = .085\,W$

$W =$ Steel $= 128$ tons $= 256^k$

concrete $= 216\ yd^3 = (5832\ ft^3)(150\ lb/ft^3)(\frac{1k}{1000 lbs}) = 874.8^k$

MEP $= 5psf\,(175')(200') + 5psf\,(100')(175') = 262.5^k$

insulation $= 2psf\,(175')(200') = 70^k$

roofing $= 10psf\,(175')(200') = 350^k$

ceiling $= 3psf\,(175')(200') + 3psf\,(100')(175') = 157.5^k$

flooring $= 10psf\,(175')(100') = 175^k$

Snow (20%) $= (.2)(42.4psf)(175')(200') = 296.8^k$

bar joists $= 1\,psf\,(175')(200') = 35^k$

Columns $= 36(7.5')(53\,lb/ft)(1/1000) + 42(15')(53\,lb/ft)(1/1000) = 47.7^k$

total $W = 2525.3^k$

$V = 195.73^k$

(8) distribute V over height of building

$F_x = C_{vx} V$ ASCE 7-10 12.8.3

$C_{vx} = \dfrac{W_x h_x^k}{\sum_{i=1}^{n} W_i h_i^k}$ $T = .15 \leq .5 \rightarrow k = 1$

$F_3 = C_{v3} V = (.615)(195.73) = 120.37^k$

$W_3 h_3 = (1064.5^k)(30') = 31{,}935.3^{ft\text{-}k}$

$W_2 h_2 = (1331.9^k)(15') = 19\,979.9^{ft\text{-}k}$

(excludes columns)

$C_{v3} = \dfrac{W_3 h_3^1}{W_3 h_3 + W_2 h_2}$

$= \dfrac{31935.3}{31935.3 + 19979.9}$

$= .615$

$F_2 = C_{v2} V = (.385)(195.73) = 75.36^k$

$C_{v2} = \dfrac{W_2 h_2^1}{W_3 h_3 + W_2 h_2} = \dfrac{19979.9}{31935.3 + 19979.9} = .385$

(9) redundancy factor ρ

$\rho = 1.0$ ASCE 12.3.4.2

(10) Seismic load effects E, E_M

$E = \rho Q_E \pm .2 S_{DS} D$ ASCE 7-10 12.4.2

$= (1.0)(V = 195.73^k) \pm (.2)(S_{DS} = .256)(D = 2525.3^k)$

$E = 195.73^k \pm 129.29^k = 325.03^k,\ 66.43^k$

31

LC

$1.2(D+F)+1.0E+f_1L+1.6H+f_2S$ 2012 IBC eq16-5 additive

$(1.2+.2S_{DS})D+f_1L+f_2S+\rho Q_E+1.2F+1.6H$

$(.9)(D+F)+1.0E+1.6H$ 2012 IBC eq16-7 counteractive

$(.9-.2S_{DS})D+\rho Q_E+.9F+1.6H$

E_m max seismic load effect including overstrength factor

$E_m = \Omega_o Q_E \pm .2S_{DS}D$ ASCE 12.4.3

$E_{mh} = \Omega_o Q_E = (2)(195.73^k) = 391.46^k$

$\Omega_o = 3$ ASCE 7-10 12.2-1

not needed

(11) Drift control requirements

$\delta_x = \dfrac{C_d\,\delta_{xe}}{I_e}$ ASCE 7-10 12.12.1

$C_d = 3 \rightarrow$ from R=3

allowable story drift

$\Delta_a^{ab} = .025h_{sx}{}^c$ ASCE7-10 12.12-1

$= .025(15')(12''/')$

$\Delta_a = 4.5''$

$\Delta_b = .025(30')(12''/')$

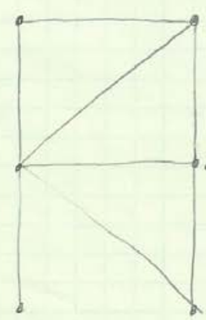
$\Delta_b = 9''$

$\frac{F_3}{4} = \frac{120.37^k}{4}$

$\boxed{30.09^k}$

$\frac{F_2}{4} = \frac{75.36^k}{2}$

$\boxed{18.84^k}$



E.8 Building Footing Calculations

Footing Design | C_1 as Example | 1/3

Use column W12x53 From Excel sheet $\phi_c = 0.65$

$P_u = 134.24 \text{ kips} + 1.59 + \frac{53 \times 30}{1000}$

$A_F = \frac{P}{S}$ $P = 95.79$ kips, $S = 3$ k/ft^2

$A_F = \frac{95.79^k}{3^k/ft} = 31.93$ ft^2 = dimensions = 5.65 ft × 5.65 ft so 6.0 ft × 6.0 ft

① Assume $\sqrt{\frac{A_2}{A_1}} = 1.5$

② $A_1 = \frac{P_u}{\phi_c\, 0.85 f'_c \sqrt{A_2/A_1}} = \frac{134.24^k}{0.65(0.85)(4)(1.5)}$

$A_1 = 40.494$ in^2

③ $\Delta = 0.5(0.95d - 0.8b_f)$ $\Delta = 0.5(0.95(12.1) - 0.8(12))$

$\Delta = 0.9475$

$N = \sqrt{A_1} + \Delta$ $N = \sqrt{40.494} + 0.9475 = \cancel{5.96} \; 7.311$ in

$B = \frac{A_1}{N} = \frac{40.494}{7.311} = 5.539$ in

round up to even inch $N = 8$ in $B = 6$ in

④ $A_1 = BN = 8 \times 6 = 48$ in^2

⑤ Base plate = 8 in × 6 in
Pedestal = 12 in × 10 in

⑥ $A_2 = 12 \times 10 = 120$ in^2

⑦ $\sqrt{A_2/A_1} = \sqrt{\frac{120}{48}} = 1.58$

need to assume $\sqrt{A_2/A_1}$ to be larger

① Assume $\sqrt{A_2/A_1} = 1.58$

② $A_1 = \frac{134.24}{0.65(0.85)(4)(1.58)} = 38.44$ in^2

③ $\Delta = 0.5(0.95(12.1) - 0.8(12)) = 0.9475$

$N = 6.200 + 0.9478 = 7.148$

$B = \frac{38.44}{7.148} = 5.38$ in $N = 8$ in, $B = 6$ in

④ $A_1 = 8.00 \times 6.00 = 48$ in

⑤ Base plate = 8 × 6 in Pedestal = 12 in × 10 in

2/3

⑥ $A_2 = 12 \times 10 = 120\ in^2$

⑦ $\sqrt{A_2/A_1} = \sqrt{\frac{120}{48}} = 1.581$

minimum dimensions of 8in × 6in
and 12in × 10in

so use 14in × 12in for baseplate and
18in × 16in for pedestal for
12 × 53 column.

Check bearing strength of concrete

$\phi P_p = \phi_c\ 0.85\ f'_c\ A_1 \sqrt{A_2/A_1}$

$= 0.65\ (0.85)\ (4ksi)\ (168\ in^2)\ (\sqrt{288/168}) = 486.12\ k$

$\phi P_p > P_u \Rightarrow 486.12k > 134.24^k$ ✓

Compute baseplate thickness

$m = \frac{N - 0.95d}{2} = \frac{14 - 0.95(12.1)}{2} = 1.2525\ in$

$n = \frac{B - 0.8b_f}{2} = \frac{12 - 0.8(10)}{2} = 1.2\ in$

$n' = \frac{\sqrt{d b_f}}{4} = \frac{\sqrt{(12.1)(10)}}{4} = 2.75\ in$

$l =$ largest

$l = 2.75\ in$

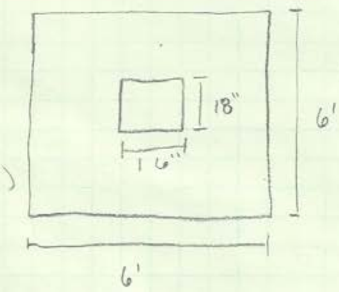
$t_{req} = l\sqrt{\frac{2P_u}{0.9\ F_y\ BN}} = 2.75\sqrt{\frac{2(134.24)}{0.9(36ksi)(14)(12)}}$

$= 0.6107 \approx 0.75\ in$

Baseplate Dimensions $= B \times N \times l$
$= 12 \times 14 \times 2.75in$

Footing design | 3/3

one-way shear design



$\phi V_n \geq V_u$

$V_n = V_c + V_s$ $\quad V_s = 0$ (no shear reinforce)

$V_n = V_c$

$V_c = 2\sqrt{f'_c}\, b_w d$

$f'_c = 4000$ psi

$b_w = 6$ ft

$d = 28 - 3\text{in} - 1\text{in} - 1/2 = 23.5"$

cover on bottom reinforcement

$\phi V_n \geq V_u = \left(\frac{l}{2} - \frac{c}{2} - d\right) b q_u$

$= \left(\frac{6\text{ft}}{2} - \frac{16}{2} - \frac{23.5}{}\right)(6\text{ft})(134.24\text{k})$

$V_u = 7.33$ kips

$\phi V_c = 13.376$ kips

$\phi V_c > V_u$ ✓

Two-way shear design

$b_o = 4(c+d) = 4(16 + 23.5) = 158$ in

$V_c = 4(1.0)(\sqrt{4000}) = 253$ psi

$V_c = (253)(158)(23.5) = 939.3$ kips

$\phi V_c = 704.5$ kips

$V_u = 3.25\text{ ksf}\left((6\text{ ft})^2 - \left(\frac{16 + 23.5}{12}\right)\right) = 81.98$

$\phi V_c > V_u$ ✓

Local Column Name	Column Size	Length (ft)	Column Self Weight (lbs/ft)	Factored Load (Pu in kips)
C1	W 12x53	30.0	53.0	134.24
C2	W 12x53	30.0	53.0	17.34
C3	W 12x53	30.0	53.0	34.67
C4	W 12x53	30.0	53.0	43.15
C5	W 12x53	30.0	53.0	140.78
C6	W 12x53	30.0	53.0	76.44
C7	W 12x53	30.0	53.0	34.67
C8	W 12x53	15.0	53.0	185.91
C9	W 12x53	30.0	53.0	76.27

COLUMN PROPERTIES (W 12x53)		
d	12.1	in
b (sub) f	10	in

LOADING					
Total Roof Dead Load	30.00	psf			
Truss Dead Load (Roof)	7.40	kips	per 100' of truss		
Snow Load (Roof LL)	42.40	psf			
2nd Floor Total Dead Load	18.00	psf			
2nd Floor Live Load	100.00	psf			
Interior Beam Load (DL)	34.00	lb/ft	Contributing Length		ft
Exterior Beam Load (DL)	26.00	lb/ft	Contributing Length		ft
Interior Girder Load (DL)	108.00	lb/ft	Contributing Length		ft
Exterior Girder Load (DL)	34.00	lb/ft	Contributing Length		ft
Phi (sub) c	0.65				
F (sub) y	36.00	ksi			

Figure 62: Building Footings Loading Conditions

LOADING PROPERTIES AND LOAD CASES		
Tributariy Area	156.25	ft^2
Column Length	30.00	ft
Total Load (P)	13.83	kips
Soil Capacity	3.00	k/ft
Factored Load (Pu)	17.34	kips
Phi (sub) c	0.65	no units
F'c	4.00	ksi
Phi (sub) shear	0.75	no units
Lambda (normal weight of concrete)	1.00	no units

**will be linked from column sheet

Interior Beam Length	0.00	ft
Exterior Beam Length	0.00	ft
Interior Girder Length	0.00	ft
Exterior Girder Length	0.00	ft
Truss Portion	0.13	

			Rounded Dimensions	
A (sub) footing	4.61	ft^2		
Footing Dimensions	2.15	ft	2.50	ft
	2.15	ft	2.50	ft

			Base Plate Dimensions		Pedestal Dimensions	
Assumed Value of SQRT (A2/A1)	2.45000	no units				
A1	3.20	in^2				
delta	1.75	in				
N	3.54	in	4.00	in	8.00	in
B	0.91	in	2.00	in	6.00	in
A1	8.00	in^2				
A2	48.00	in^2				
SQRT(A2/A1)	2.44949	no units				
assumed SQRT(A2/A1) is within 1% of actual SQRT(A2/A1)						

	Base Plate Dimensions		Pedestal Dimensions	
	14.00	in	18.00	in
	12.00	in	16.00	in
A1	168.00	in^2	-	-
A2	-	-	288	in^2

Check Bearing Strength of Concrete		
Phi P (sub) p	486.1196297	kips
Greater than Pu		
m	1.25	in
n	2	in
n'	2.75	in
ℓ	2.75	in
t (sub) req	0.219	in
Use this t req	0.25	in

Shear Design		
Footing Depth	28	inch
d	23.5	inch
Reinforcement Cover on Bottom	3	inch
No. 8 Bar Diame	1	inch
c	16.00	inch
Service Dead Lo	5.6	kip
Service Live Loa	42.4	kip
Seismic Load	9.023	kip
q (sub) u	11.932	ksf
Beta 1	1.166666667	no units
alpha s	20	no units

One Way Shear		
V (sub) c	7.431352501	kip
V (sub) u	-41.01625	kip
phi V (sub) c	5.573514376	kip
Phi Vc > Vu		

Two Way Shear		
b (sub) o	111	in
v (sub) c	252.9822128	psi
V (sub) c	659.9041021	kips
phi V (sub) c	494.9280766	kips
V (sub) u	-54.70904861	kips
Phi Vc > Vu		

Assumptions		
Footing Depth	4.0	ft
q (sub) a	3.0	ksf
f'c	4.0	ksi
fy	60.0	ksi
gamma conc	150.0	pcf
gamma soil	100.0	pcf
gamma avg	125.0	pcf
Unfactored Load	13.8	kips
q (sub) e	2.5	k/ft^2
A req	5.5	ft^2
b	2.4	ft
Assumed b	6.5	ft
Factored Load	17.3	kips
q (sub) u	0.4	k/ft^2

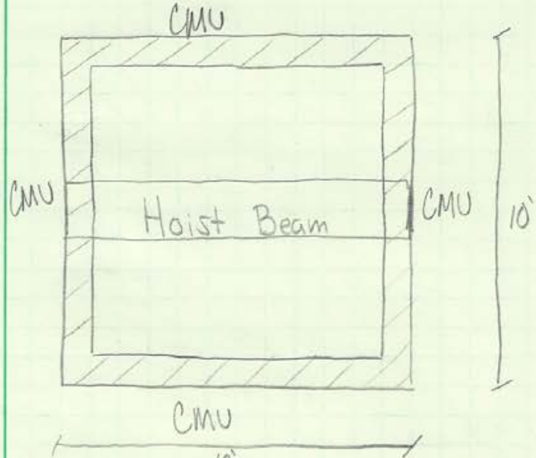
Punching Shear		
Beta c	1.1	
phi	0.75	no units
min	4.0	no units
Sqrt(f'c)	0.1	
b^2	42.3	ft^2

QUADFORM		
A	0.8	
B	-12.2	
C	16.6	
Root 1	-17.3	
Root 2	1.3	
Use this for d	7.250	in

Figure 61: Building Footing Example

E.9 Elevator Hoist Beam Calculations

Elevator Support Beam



↳ Capacity: 4500 lbs

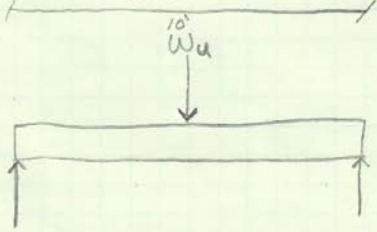
↳ Empty Weight: 4500 lbs

↳ Hoist Weight: 500 lbs

↳ Self weight: ?

↳ $f_y = 50$ ksi

↳ E =



↳ $\omega_u = 1.2D + 1.6L$

$= 1.2(\omega_E + \omega_H) + 1.6(\text{Capacity})$

$= 1.2(4500 + 500) + 1.6(4500)$

$\omega_u = 13.2$ kips

↳ $M_u = \frac{\omega l}{4} = \frac{(13.2^k)(10')}{4} = 33^{k\text{-}ft}$

↳ $\phi M_n = M_u$

$\phi M_n = 33^k$] Required

∴ W8 x 15 is chosen →

↳ $\phi M_n = 40.8^k$

Check Self weight

↳ $W_U = 1.2(W_E + W_H + W_S) + 1.6\,(\text{Capacity})$

$= 1.2\left(4500\text{lb} + 500\text{lb} + 15\text{lb}/\text{ft}(10')\right) + 1.6(4500\text{ lb})$

$W_U = 13.38^{k}$

↳ $M_U = \dfrac{W_U\, l}{4}$

$= \dfrac{(13.38)^{k}(10')}{4}$

$M_U = 33.45 \text{ K-ft}$

↳ $\phi M_n = 40.8^{\text{k-ft}} > 33.45^{\text{k-ft}}$ ✓

Check Deflection

↳ $\Delta_{max} = \dfrac{W_U\, l^3}{45\, E\, I}$

$= \dfrac{(13.2^{k})(10')^3(12''/1)^3}{45(29{,}000\text{Ksi})(48\text{ in}^4)}$

$\Delta_{max} = .364''$

↳ $\Delta_{Allowable} = L/300 = \dfrac{120''}{300} = .4''$

$\Delta_{Allow} > \Delta_{max}$ ✓

W8x15 is acceptable for application

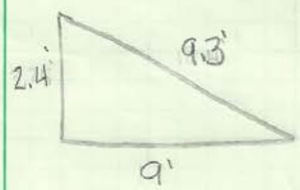
Appendix F: Pedestrian Bridge Calculations

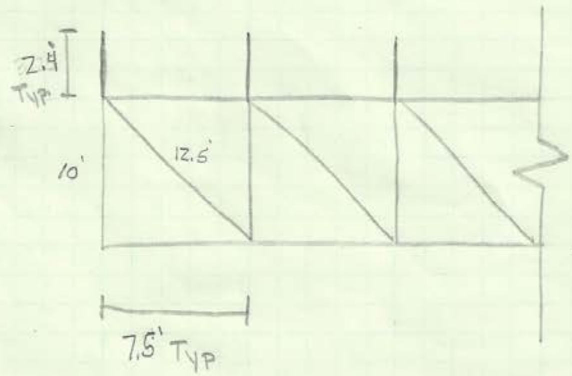
This section contains the necessary hand-calculations required to properly design the proposed pedestrian bridge. Each section shows the typical methodology for determining the proper members for the structure. Microsoft Excel spreadsheets were then used to recreate these calculations multiple times to increase efficiency. Screenshots of these spreadsheets are included at the conclusion of their appropriate sections.

F.1 Bridge Through-Truss Calculations

Bridge Truss

Roof





↳ HSS 5x4x1/2

↳ 25 lb/ft ; A500 Steel

↳ span = 75'

↳ Weight of Roof Frame

$$W_r = \frac{(2.4' + 9.3' + 9')(25\ \text{lb/ft})}{(9' \cdot 7.5')} = 7.67\ \text{psf}$$

↳ Snow Load

RISK CAT. #3

ASCE 7-10 7-2, 7-3

ASCE 7-10 Table 1.5-2

$P_s = C_s P_f$

ASCE 7-4

↳ $P_f = .7 C_e C_t I_s P_g$ ← 780 CMR

$= .7(1)(1)(1.1)(55\ \text{psf})$

$P_f = 42.4\ \text{psf}$

$P_s = (1)(42.4\ \text{psf})$

$$\boxed{P_s = 42.4\ \text{psf}}$$

Imposed
DEAD

MEP = 5 psf
SOLAR = 4 psf
ROOF = 10 psf
FRAME = 7.67 psf
INSULATION = 2 psf

479.02 lb/ft

Facade = 125 lb/ft

LIVE

SNOW = 42.4 psf

*PEDESTRIAN LOAD = 90 psf

Geometry

↳ Fy = 50 ksi
↳ Panels @ 7.5'
↳ Height = 10'
↳ SPAN = 5 @ 75' A
2 @ 35' B

SPAN "A" IS LARGEST, AND SMALLER SPANS WILL BE TREATED THE SAME AS SPAN A.

↳ Deck width = 7'
↳ Truss Height = 10'

3

Truss Components → Self Weight

Type	Shape	Total #	A	w	Material
Chords	WT 9x25	20	$21.0 in^2$	25 lb/ft	A992
Diagonals	LL 5x3x3/16	10	$7.5 in^2$	27.3 lb/ft	A36
Verticals	LL 3x2.5x3/16	10	$2.0 in^2$	9.04 lb/ft	A36
Floor Beams	W 8 x 13	10	$3.84 in^2$	12.1 lb/ft	A992
Laterals	LL 3x3x1/2	10	$2.76 in^2$	12.9 lb/ft	A36

Total = 8350.5 lb/truss

↳ Pedestrian Loads

↳ 90 PSF → LRFD Pedestrian Article 3.1

↳ $\frac{90\,PSF(7')}{2} = 315\ lb/ft$

↳ No Vehicular Load (Article 3.2)

↳ Wind Load

↳ Assume 100 mph

↳ Horizontal (AASHTO Signs)

↳ $P_z = .00265 K_z G V^2 K_d C_d$

↳ $K_z = 1.09$ (Table C3.8.4-1 → height = 44')

↳ $G = 1.14$ (Article 3.8.6)

↳ $V = 100$ mph (AASHTO Signs)

↳ $C_d = 2.0$ (Table 3.8.7-1)

↳ $K_d = .85$ (Table 3.8.5-1)

↳ $P_z = 55.98$ psf

↳ Projected Vertical Area / foot

↳ Chords = $\dfrac{2 @ 9" \cdot \frac{1'}{12"} \times 7.5'}{7.5'} = 1.5 \text{ ft}^2$

↳ Diagonals = $\dfrac{\frac{3" \cdot 1'}{12"} \times 12.6'}{7.5'} = .42 \text{ ft}^2$

↳ Verticals = $\dfrac{\frac{2.5" \cdot 1'}{12"} \times 10'}{7.5'} = .28 \text{ ft}^2$

5

Bride max height = 12.58' × 1'

* Glass enclosure causes ft²/ft to increase

Total ft^2 / linear foot = 12.58 ft^2

↳ Horizontal Wind Load = $(12.58\,ft^2)\,55.98\,psf$

$$\boxed{W_H = 704.23\ lb/ft}$$

↳ Vertical Wind Loading

$W_v = P_v(w_{deck})$

↳ P_v = .02 KSF (Article 3.8.2)

↳ w_{deck} = 9'

$\therefore \boxed{211.2\ lb/ft = W_v}$

Load on Leeward = $(211.2\ lb/ft)(.75)$ = 158.4 lb/ft

Load on windward = $(211.2\ lb/ft)(.25)$ = 52.8 lb/ft → Uplift

6

Weight of Floor deck

↳ assume 20 psf

↳ $\omega_F = \frac{(20psf)(9')}{2} = 90 \text{ lb/ft/Truss}$

Total Vertical Loads

Dead = $\omega_F + \omega_{self} + \omega_{other}$

Dead = 455.3 lb/ft

Floor Live = 405 lb/ft

Roof Live/Snow = 190.8 lb/ft

Wind (overturning) = 135 lb/ft

Service I

Strength I

Strength III

Extreme II

SERV I

$1(D) + 1(L_F) + .3(W) = 900.8 \text{ lb/ft}$

STR I

$1.25(D) + 1.75(L_F) + 0(W) = 1277.9 \text{ lb/ft}$ → governs

↑

Table 3.4.1-2

7

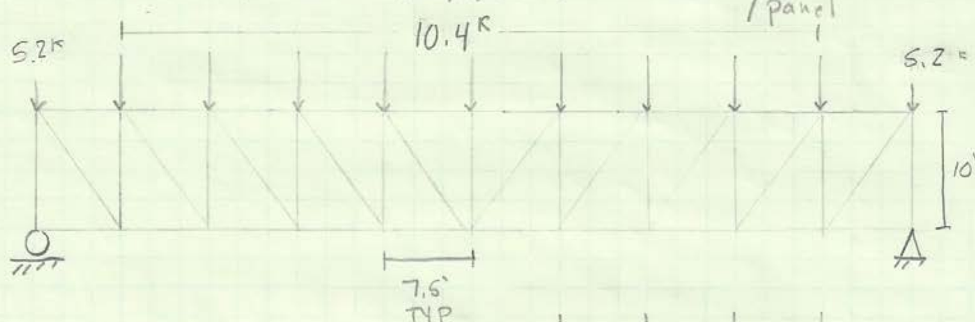
STR III

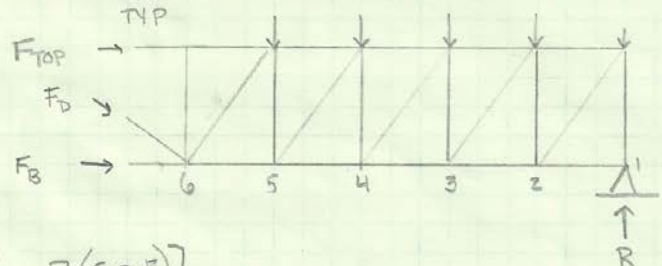
$$1.25(D) + 0(L_F) + 1.4(w) = 758.2 \text{ lb/ft}$$

EXT II

$$1.25(D) + .5(L_F) + 1(w) + 1(S) = 1,097.3 \text{ lb/ft}$$

Point Load = (1277.9 lb/ft)(7.5') = [illegible] k/panel





$$\hookrightarrow R = \frac{\left[9(10.4^k) + 2(5.2^k)\right]}{2}$$

$$R = 52^k$$

8

$\hookrightarrow \Sigma \circlearrowright^{+} M_6 = 0$

$$F_T(10') - 52(37.5') + [10.4^k(7.5' + 15' + 22.5' + 30')]$$
$$+ 5.2^k(37.5') = 0$$

$$\boxed{F_T = 89.9^k}$$

$\therefore$ Change Member to WT 9x17.5

$\hookrightarrow \phi P_n = 100$

$\hookrightarrow \phi P_n > F_T$ ✓

$\hookrightarrow \Sigma F_{Y_1} = 0$

$$52 - 5.2^k - F_D(10/12.5) = 0$$

$$\boxed{F_D = 53.9^k}$$

$\therefore$ Change Member to LL 3x2½x¼

$\hookrightarrow \phi P_n = 85.5^k$

$\phi P_n > F_D$ ✓

Assumptions:

Roof Frame Members		
Type	HSS 5x4x1/2	
Material	A500 Steel	
Weight	25	lb/ft
Height	2.4	ft
Length	9	ft
Diagonal	9.3	ft
Spacing	7.5	ft
Truss Chords		
Type	WT 9x25	
Material	A992 Steel	
Weight	25	lb/ft
Area	21	si
Total #	20	
Strength	100	kips
Truss Diagonals		
Type	LL 5x3x3/16	
Material	A36 Steel	
Weight	27.3	lb/ft
Area	7.5	si
Total #	10	
Strength	85.5	kips
Truss Verticals		
Type	LL 3x2.5x3/16	
Material	A36 Steel	
Weight	9.04	lb/ft
Area	2	si
Total #	10	
Floor Beams		
Type	W 8x13	
Material	A992 Steel	
Weight	12.1	lb/ft
Area	3.84	si
Total #	10	
Floor Laterals		
Type	LL 3x3x1/2	
Material	A36 Steel	
Weight	12.9	lb/ft
Area	2.78	si
Total #	10	

Dimensions

Deck Width	9	ft
Truss Height	10	ft
Panel Spacing	7.5	ft
Span Length	75	ft
Diagonal Length	12.5	ft
Roof Frame Angle	0.26	radians
Roof Frame Length	9.32	ft
Roof Frame Max Height	2.42	ft

Loads

Dead Loads		
Roof Frame	7.7	psf
MEP	5	psf
Roof Deck	10	psf
Insulation	2	psf
Facade	125	lb/ft
Solar	4	psf
Self-weight	111.3	lb/ft
Total Truss Weight	8350.5	lb
Floor Deck	90	lb/ft/truss
Live Loads		
Pedestrian	90	psf
Vehicular	0	psf
Snow Load	42.4	psf
Wind Loads		
Wind Speed	100	mph
Horizontal Wind	694.14	lb/ft
Leeward Wind	144.7	psf
Windward Wind	48.2	psf
Total Loads		
Dead	455.3	lb/ft
Floor Live	405	lb/ft
Roof Live/Snow	190.6	lb/ft
Wind (overturning)	144.7	lb/ft

Snow Load Calculation

From ASCE 7-10			Calculations		
Ce	1				
Ct	1		Snow Load	42.4	psf
Is	1.1				
Pg	55				
Cs	1				
Pf	42.4				

Horizontal Wind Calculation

Assumptions			Calculations		
Kz	1.09		Pz	55.98	psf
G	1.14		Total effect area	12.4	sf/ft
V	100		Horizontal Wind Load	694.14	lb/ft
Cd	2				
Kd	0.85				

Vertical Wind Calculation

Assumptions			Calculations		
Pv	0.02	ksf	Vertical Wind Load	193.0	lb/ft
			Leeeward Load	144.7	lb/ft
			Windward Load	48.2	lb/ft

Loading Conditions

Service I	903.8	lb/ft
Strength I	1277.9	lb/ft
Strength III	771.8	lb/ft
Extreme II	1107.0	lb/ft

Load per Joint

Midspan Joints	9.6	kips
End Joints	4.8	kips

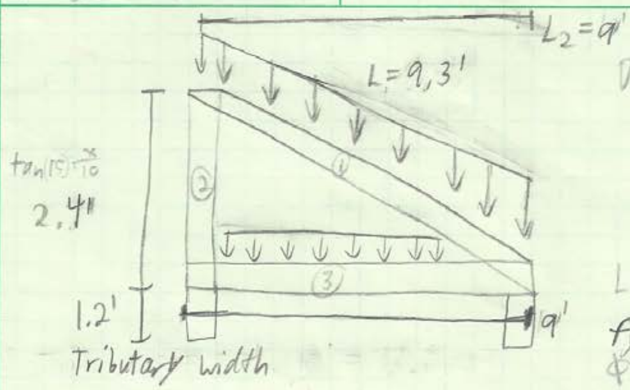
Truss Analysis

End Reaction	47.9	kips		
Chord Forces	89.9	kips		Indicates sufficient Member Selection
Diagonal Forces	53.9	kips		

Figure 63: Bridge Truss Calculations

F.2 Bridge Roof Frame Calculations

Bridge: Roof frame 1 | | | 1/2

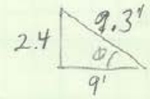


DL = member = ? ①
Sheathing = 10psf ①
insulation = 2 psf ①

DL = 12 psf

$LL_r = S = 42.4$ psf

$f_y = 36$ ksi
$\phi = .9$



$\phi = 15°$

$\cos 15 = \frac{9}{x}$

① L = 9.3'

LC

$1.4D = 1.4(12\,psf)(7.5\,ft) = 126$ lb/ft

$1.2D + 1.6(L_r \text{ or } S) + .5L = (1.2)(12psf)(7.5ft) + 1.6(42.4psf)(7.5ft) = \underline{616.8 \text{ lb/ft}}$

$1.2D + 1.6L + .5(L_r \text{ or } S) = (1.2)(12psf)(7.5ft) + 1.6(0) + .5(42.4)(7.5ft)$
$= 267$ lb/ft

$W_u = 616.8 \text{ lb/ft} = \overset{DL}{108} + \overset{S}{508.8}$

$M_u = \frac{WL^2}{8} = \frac{(108 \text{ lb/ft})(L_1 = 9.3')^2}{8} + \frac{(.509 \text{ k/ft})(L_2 = 9')^2}{8} = 1.16 + 5.15 = 6.31$ k-ft

$\boxed{\text{Selected Beam} \quad \text{HSS } 3 \times 3 \times 3/8}$

$LC = 1.2(\overset{DL}{12.2 \text{ lb/ft}}) + \overset{DL}{108 \text{ lb/ft}} + \overset{S}{508.8 \text{ lb/ft}} = \overset{DL}{122.64 \text{ lb/ft}} + 508.8$ k/ft

$M_u = \frac{wL^2}{8} = \frac{(.12264 \text{ k/ft})(9.3')^2}{8} + \frac{(.509 \text{ k/ft})(9')^2}{8} = 6.48$ ft-K $< \phi M_n = 8.34$ ft-K

Self-weight $= (12.2 \text{ lb/ft})(9.3') = 113.46$ lb

Bridge: Roof frame 1 | 2/2

② $L = 2.4' + 1.2' = 3.6'$ trib Area $= \frac{7.5' \times 9'}{2} = 33.75\,ft^2$

$DL = \frac{1}{2}(113.46\,lb) + (10\,psf)(33.75\,ft) + (2\,psf)(33.75\,ft^2) = 461.7\,lb$

LC

$1.4D = 1.4(461.7\,lb) = 646.4\,lb$

$1.2D + 1.6(S \text{ or } L_r) + .5L = 1.2(461.71\,lb) + 1.6(42.4\,psf)(33.75\,ft) = \underline{2843.65\,lb}$

$1.2D + 1.6L + .5(L_r \text{ or } S) = 1.2(461.71\,lb) + .5(42.4\,psf)(33.75\,ft) = 1269.6\,lb$

$P_u = 2843.65\,lb = 2.84\,k$

Selected column

$\boxed{HSS\ 3 \times 3 \times 3/8}$

lb/ft = 12.2

$t_{design} = 0.349$ in

$\phi_c P_n = 122^K$

$A_g = 3.39\,in^2$

$I_x = I_y = 3.78\,in^4$

$r_x = r_y = 1.06$ in

$\phi_c = 0.90$

$f_y = 46$ ksi

$L/r = 3.6'/1.06''\left(\frac{12''}{1'}\right) = 40.75$

$L/r < 4.71\sqrt{E/F_y} = 118.3 \rightarrow$ inelastic

$F_{cr} = 0.658^{(F_y/F_e)} F_y$

$F_{cr} = 0.658^{(46/172.36)}(46) = 41.14$ ksi

$F_e = \frac{\pi^2 E}{(L/r)^2} = \frac{\pi^2(29000\,ksi)}{(40.75)^2} = 172.36$

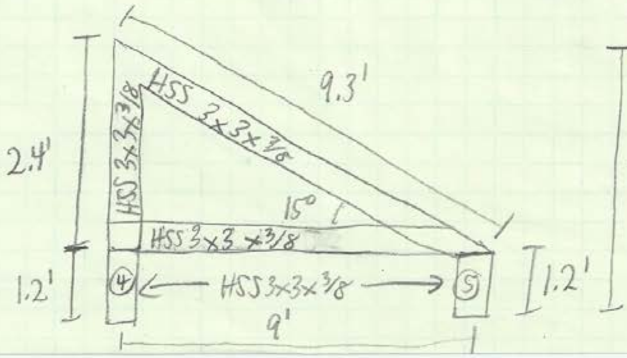
$\phi P_n = (\phi = .9)(F_{cr} = 41.14\,ksi)(A_g = 3.39\,in^2)$

$\phi P_n = 125.5^K > P_u = 2.88^K$ ✓

Self-weight in one frame $= (12.2\,lb/ft)(2.4') = 29.28\,lb$

① = 113.46 lb
② = 29.28 lb
③ = 109.8 lb
④ = 14.64 lb
⑤ = 14.64 lb

total roof frame self-weight = 281.82 lb
= .282 kips per largest frames



F.3 Bridge Wind Distribution Calculations

Bridge: Wind dist.

Horizontal wind load: $W_H = .704\ ^k/ft$

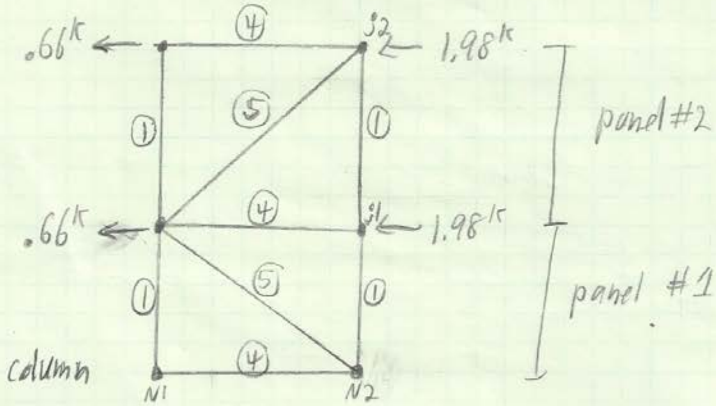
pressure on windward side: $(.75)(1.409\ ^k/ft) = .528\ ^k/ft$

pressure on leeward side: $(.25)(1.409\ ^k/ft) = .176\ ^k/ft$

Wind load per lateral truss panel: (2 lateral trusses)

windward $= (.528\ ^k/ft)(7.5')(\frac{1}{2}) = 1.98^k$

leeward $= (.176\ ^k/ft)(7.5')(\frac{1}{2}) = .66^k$



Member	Shape	material
① chords	WT 9x25	A992
② Diagonals	LL 5x3x 3/16	A36
③ verticals	LL 3x2.5x 3/16	A36
④ floor/ceiling beams	W 8 x 13	A992
⑤ laterals	LL 3x3x½	A36

?

lateral joint (J#)	deflection deflection (in)	opp. wind direction deflection
1	-.094	.052
2	-.2	.13
3	-.316	.22
4	-.425	.316
5	-.527 ← max	.408
6	-.425	.316
7	-.316	.22
8	-.2	.13
9	-.094	.052

lateral force on columns due to wind

x reaction force

$N1 = .732^k$ $\quad N1 + N_2 = 15.84^k$

$N2 = 15.108^k$

$= (15.84^k)(4\ \text{lat trusses}) = 63.36^k$

per column $\quad$ opposite wind direction

F.4 Bridge Seismic Calculations

Bridge: Siesmic load | Inverted Pendulum | 1/2

① $S_S = .24$
$S_1 = .067$ / 780 CMR: MA amendment to IBC 2009

② exception ⇒ none

③ Siesmic Design category
site class D → Soil Classification unknown
$S_{DS} = \frac{2}{3} F_a S_S = (\frac{2}{3})(1.6)(.24) = .256$ $\quad S_{DS} = .256$
F_a → ASCE 7-10 table 11.4-1
$F_a = 1.6$
$S_{D1} = \frac{2}{3} F_v S_1 = (\frac{2}{3})(2.4)(.067) = .107$ $\quad S_{D1} = .107$
$F_v = 2.4$ ASCE 7-10 table 11.4-2
risk category → III ASCE 7-10 table 1.5-1
Siesmic design category → B ASCE 7-10 table 11.6-1

④ determine analysis Procedure
Structure type: Inverted Pendulum-Type Structure ASCE 7-10 12.2.5.3
$M_{top} = \frac{1}{2} M_{bottom}$
$R = 2$
$\Omega_0 = 2$
$C_d = 2$
h_n = not limited
ASCE 7-10 Table 15.4-2
Equivalent lateral Force Procedure ASCE 7-10 [12.2.5.3, 12.8]

⑤ $V_1 = C_s W_1$ ASCE 7-10 12.8-1 (Siesmic Base Shear)
$V_1 = (.116)(351.99^k) = 40.83^k$

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{(.256)}{\left(\frac{2}{1.25}\right)} = .16 \leq C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = .116$$
ASCE 7-10 12.8.1.1

$I_e = 1.25$ ASCE 7-10 table 1.5-2

assumed Bridge width = 9'

W for 1 column (75' Span)
See assoc. calc.:
Bridge Span = 2.5408 k/ft (75') = 190.56 k
hammerhead = 4.347 k
column = 5.236 k/ft (30') = 157.08 k

$W_1 = 351.99^k$

2/2

$T \le C_u T_a$ $\underline{T \le}$ ASCE 7-10 12.8.2

$T \le (1.7)(.34) = \underline{.578\,s}$

$C_u = 1.7$ ASCE 7-10 table 12.8-1

$T_a = C_t h_n^x$ ASCE 7-10 12.8.2.1

$= .016(30')^{.9}$

$\underline{T_a = .34}$

$C_t = .016$

$x = .9$

ASCE 7-10 table 12.8-2

$T_L = 6\,s$ ASCE 7-10 Figure 22-12

$T < T_L$

$C_s \le \dfrac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \dfrac{.107}{(.578)\left(\frac{2}{1.25}\right)} = \boxed{C_s \le .116}$ ASCE 7-10 12.8.1.1

$C_s \ge .044\, S_{DS}\, I_e \ge .01 = .044(.256)(1.25) = .0141 < C_s = .116$

ASCE 7-10 12.8.1.1

Vertical distribution of lateral forces ASCE 7-10 12.8.3

$F_x = C_{vx} V$

$C_{vx} = \dfrac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$

$\underline{k}$

$T \le .5,\ k = 1$

$T \ge 2.5,\ k = 2.5$

$T = .578\,s$ $\underline{k_1 = 1.039}$ → linear interpolation

$C_{v1} = \dfrac{w_1 h_1^{k_1}}{w_1 h_1^{k_1}} = 1$ $h_1 = 30'$

$F_1 = (1)(V_1 = 40.83^k) = \underline{40.83^k}$

1/23/18

1/

Bridge Seismic : AASHTO

Site classification "C" "very Dense Soil and Soft Rock"
- Rec Center Structural plans

risk category → III ASCE7-10 table 1.5-1

S_S = .237g USGS Design Map
S_1 = .066g ref: ASCE7-10 Standard
coordinates: 42.27641°N, 71.81185°W

F_v = 1.7 AASHTO table 3.4.2.3-2

$S_{D1} = F_v S_1 = (1.7)(.066g) = .1122$ AASHTO table 3.5-1

Seismic Design Category → A AASHTO table 3.5-1

AASHTO 3.5

SDC A → No identification of ERS
according to article 3.3
→ No Demand Analysis
→ No implicit capacity check needed
→ No capacity design required

F.5 Bridge Elastomeric Bearing Calculations

NSBA ELASTOMERIC BEARING DESIGN (ENGLISH UNITS)

AASHTO LRFD, 3RD ED., 2004 WITH 2005 INTERIMS

ETHOD B - STEEL-REINFORCED ELASTOMERIC BEARINGS - SECTION 14.7.

The following design program was developed based upon the above-referenced AASHTO LRFD code. The program is applicable to the design of steel-reinforced elastomeric bearings, both rectangular and circular in shape. The program is not applicable to design of rectangular bearings subject to combined rotation about the transverse and longitudinal axes. The program assumes that interior elastomeric layers are of equal thickness, as are the two exterior elastomeric layers.

I. INITIAL DESIGN INPUTS

Dead Load = P_D =	63.7	kips	
Live Load = P_{LL} =	16.4	kips	
Horizontal Movement of Bridge Superstructure = Δ_T =	1.0	in.	
Axis of Pad Rotation:	Transverse		
Calculated Rotation =	0.004	Radians	
Rotation Construction Tolerance =	0.005	Radians	(14.4.2.1)
Design Rotation = θ_s =	0.009	Radians	
Bearing Shape:	Circular		
Bearing Subject to Shear Deformation?	yes		
Bridge Deck Fixed Against Horizontal Translation?	yes		

II. BEARING GEOMETRY

Flange Width =	12	in.	
Bearing Width = W =	15	in.	
Flange Width ≥ W			
N/A ≥ N/A		N/	N/A
Total Unfactored Compressive Load = P_T =	80	kips	
Minimum Required Area of Bearing = A_{min} =	50.1	in.	Based on service limit (14.7.5.3.2)
Minimum Bearing Diameter = D_{min}	7.98	in.	
Bearing Diameter = D =	10	in.	
D ≥ D_{min}			
10.0 ≥ 7.98		in.	OK
Flange Width ≥ Bearing Diameter			
12 ≥ 10		in.	OK
Bearing Area = A =	78.5	$in.^2$	

III. SHEAR DEFORMATION (AASHTO LRFD 14.7.5.3.4)

ximum Total Shear Deformation of Elastomer at Service Limit = Δ_s = Δ_T =	1.000	in.	
$2\Delta_s$ =	2.000	in.	
Elastomeric Layer Thickness = h_{ri} =	0.375	in.	
Thickness of top and Bottom Cover Layers (each) = h_{cover} =	0.250	in.	
h_{cover} ≤ 0.7 h_{ri}			(14.7.5.1)
0.250 ≤ 0.263		in.	OK
terior Elastomeric Layers (Excluding Exterior Layer Allowance) = n_{int} =	10		
Total Elastomer Thickness = h_{rt} = 2 h_{cover} + n_{int} h_{ri} =	4.250	in.	
h_{rt} ≥ $2\Delta_s$			(14.7.5.3.4-1)
4.250 ≥ 2.000		in.	OK

IV. COMPRESSIVE STRESS (AASHTO LRFD 14.7.5.3.2)

Service Average Compressive Stress (Tot $\sigma_s = \frac{P_T}{A}$ =	1.02	ksi	
Service Average Compressive Stress (Live L $\sigma_L = \frac{P_{LL}}{A}$ =	0.21	ksi	
Rectangular Shape = $\frac{LW}{2h_{ri}(L+W)}$ =	N/A		(14.7.5.1-1)
Circular Sha = $\frac{D}{4h_{ri}}$ = S_i	6.67		(14.7.5.1-2)
Shear Modulus of Elastomer = G =	0.100	ksi	
0.080 ≤ G ≤ 0.175		ksi	(14.7.5.2)
0.080 ≤ ### ≤ 0.175		ksi	OK

For Bearings Subject to Shear Deformation:

$\sigma_s \leq 1.66\,GS$			(14.7.5.3.2-1)
## ≤ 1.11	ksi	OK	
$\sigma_s \leq 1.6$ ksi			(14.7.5.3.2-1)
## ≤ 1.6	ksi	OK	
$\sigma_L \leq 0.66\,GS$			(14.7.5.3.2-2)
## ≤ ##	ksi	OK	

For Bearings Fixed Against Shear Deformation:

$\sigma_s \leq 2.00\,GS$			(14.7.5.3.2-3)
N/A ≤ N/A		N/A	
$\sigma_s \leq 1.75$ ksi			(14.7.5.3.2-3)
N/A ≤ N/A		N/A	
$\sigma_L \leq 1.00\,GS$			(14.7.5.3.2-4)
N/A ≤ N/A		N/A	

V. COMBINED COMPRESSION AND ROTATION (AASHTO LRFD 14.7.5.3.5)

RECTANGULAR BEARINGS:

B = Length of Pad = N/A

Exterior Layer Allowance = n_{ext} = 1.0 (14.7.5.3.5)

Equivalent Number of Interior Elastomeric Layers = $n = n_{int} + n_{ext}$ = 11

$$\sigma_s > 1.0GS\left(\frac{\theta_s}{n}\right)\left(\frac{B}{h_{ri}}\right)^2$$ (14.7.5.3.5-1)

1.02 > N/A N/A

Subject to shear deformation:

$$\sigma_s < 1.875GS\left[1-0.200\left(\frac{\theta_s}{n}\right)\left(\frac{B}{h_{ri}}\right)^2\right]$$ (14.7.5.3.5-2)

1.02 < N/A N/A

Fixed against shear deformation:

$$\sigma_s < 2.25GS\left[1-0.167\left(\frac{\theta_s}{n}\right)\left(\frac{B}{h_{ri}}\right)^2\right]$$ (14.7.5.3.5-3)

1.02 < N/A N/A

CIRCULAR BEARINGS:

$$\sigma_s > 0.75GS\left(\frac{\theta_s}{n}\right)\left(\frac{D}{h_{ri}}\right)^2$$ (14.7.5.3.5-4)

1.02 > 0.29 ksi OK

Subject to Shear Deformation:

$$\sigma_s < 2.5GS\left[1-0.15\left(\frac{\theta_s}{n}\right)\left(\frac{D}{h_{ri}}\right)^2\right]$$ (14.7.5.3.5-5)

1.02 < 1.52 ksi OK

Fixed Against Shear Deformation:

$$\sigma_s < 3.0GS\left[1-0.125\left(\frac{\theta_s}{n}\right)\left(\frac{D}{h_{ri}}\right)^2\right]$$ (14.7.5.3.5-6)

1.02 < N/A N/A

VI. STABILITY (AASHTO LRFD 14.7.5.3.6)

For free horizontal translation:*

2A ≤ B (14.7.5.3.6-1)

$$A = \frac{1.92\frac{h_{rt}}{L}}{\sqrt{1+\frac{2L}{W}}} = 0.589$$ (14.7.5.3.6-2)

2A = 1.178

$$B = \frac{2.67}{(S+2.0)\left(1+\frac{L}{4.0W}\right)} = 0.246$$ (14.7.5.3.6-3)

2A ≤ B

≤ ## NG - SEE EQ. 5

*Notes - For rectangular bearings where L > W, L and W are interchanged. (14.7.5.3.6)

- For circular bearings, W = L = 0.8D. (14.7.5.3.6)

Bridge Deck Free to Translate Horizontally:

$$\sigma_s \le \frac{GS}{2A - B}$$ (14.7.5.3.6-4)

1.02 ≤ N/A N/A

Bridge Deck Fixed Against Horizontal Translation:

$$\sigma_s \le \frac{GS}{A - B}$$ (14.7.5.3.6-5)

1.02 ≤ 1.95 ksi OK

VII. REINFORCEMENT (AASHTO LRFD 14.7.5.3.7)

Service Limit State:

Min. Yield Strength of Steel Reinforcement = F_y = 36 ksi

Thickness of Steel Reinforcement = h_s

$$h_{s\,min} = \frac{3.0\,h_{max}\,\sigma_s}{F_y} = 0.032$$ in. *Controls* (14.7.5.3.7-1)

Fatigue Limit State:

Constant Amplitude Fatigue Threshold = ΔF_{TH} = 24.0 ksi (Table 6.6.1.2.5-3)

$$h_{s\,min} = \frac{2.0\,h_{max}\,\sigma_L}{\Delta F_{TH}} = 0.007$$ in. (14.7.5.3.7-2)

Required Minimum Reinforcement Thickness = 0.032 in.

Reinforcement Thickness = h_s = 0.1250 in.

$h_s \ge h_{s\,min}$

0.125 ≥ 0.032 in. OK

VIII. FINAL DESIGN SUMMARY

Bearing Width = W =	N/A	
Bearing Diameter = D =	10	in.
Elastomeric Layer Thickness = h_{ri} =	0.375	in.
Thickness of top and Bottom Cover Layers (each) = h_{cover} =	0.250	in.
terior Elastomeric Layers (Excluding Exterior Layer Allowance) = n_{int} =	10	
Total Elastomer Thickness = h_{rt} =	4.250	in.
Reinforcement Thickness = h_s =	0.1250	in.
Total Bearing Thickness = $h_{rt} + h_s\,(n_{int}+1)$ =	5.6250	in.

F.6 Bridge Pier Calculations

MQP: Pier Design | Pedestrian Bridge | 1/28/18 | 1/2

LC

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

$P_u = 320.4^k$, $M_u = 1520.6^{k\text{-}ft}$] loading at to of each pier w/ 75' Span

max pier height = 30'

$f'_c = 4$ ksi

$f_y = 60$ ksi

$\phi = .7$

Shape: circular

reinforcement: Spiral

$\rho_g = .015$

$C_c = 4''$

$h = 80''$ (non-Slender req. AASHTO C4.6.2.5-1)

$e = M_u/P_u = 1520.6^{k\text{-}ft} / 320.4^k$

$= 4.75' = 56.95''$

$e/h = 56.95''/80'' = .712$

$h - 2C_c = 80'' - 2(4'') = 72''$

$\gamma = 72''/h = 72''/80'' = .9$

Interaction Diagram

$K_n(.9) = .24 = \dfrac{P_u}{\phi f'_c A_g} = \dfrac{320.4^k}{(.7)(4\text{ksi})(\pi(40'')^2)} = .023 < .24 \checkmark$

$R_n(.9) = .2 = \dfrac{P_u e}{\phi f'_c A_g h} = \dfrac{320.4^k(56.95'')}{(.7)(4\text{ksi})(\pi(40'')^2)(80'')} = .016 < .2 \checkmark$

Longitudinal Reinforcement

$A_s = \rho A_g = (.015)(\pi(40'')^2) = 75.39$

19 #18 bars

Transverse Reinforcement

$A_c = \dfrac{\pi D_c^2}{4} = \dfrac{\pi(72'')^2}{4} = 4071.50\text{ in}^2$

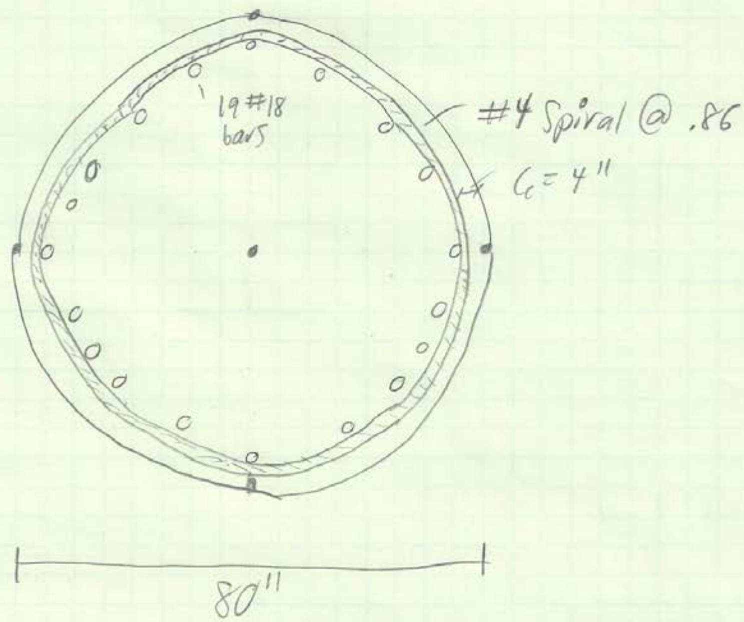
$A_g = \dfrac{\pi h^2}{4} = \dfrac{\pi(80'')^2}{4} = 5026.55\text{ in}^2$

$\rho_s = .45\left(\dfrac{A_g}{A_c} - 1\right)\left(\dfrac{f'_c}{f_y}\right) = .45(1.23 - 1)\left(\dfrac{4\text{ksi}}{60\text{ksi}}\right) = .0069$

#4 Spiral, $a_s = .11\text{ in}^2$

$S = \dfrac{4a_s(D_c - d_b)}{D^2 \rho_s} = \dfrac{4(.11\text{in}^2)(72'' - 18/10'')}{(72'')^2(.0069)} = .86$

2/2



<u>Weight of pier</u>

$\gamma_c = 150\ pcf$

$\gamma_c A_g = (150pcf)(\pi\ (3.33'')^2) = \underline{5.24\ k/ft}$

$5.24k/ft\ (30') = \underline{157.08\ kips}$

DEAD LOADS		
MEP	5.00	lb/ft
Solar	4.00	lb/ft
Purlins/bar joists	10.00	lb/ft
Roof Sheating	10.00	lb/ft
Roof Frame	23.00	lb/ft
Flooring	10.00	lb/ft
Lat/Through-Truss	2226.80	lb/ft
Facade	250.00	lb/ft
Insulation	2.00	lb/ft
Total	2540.80	lb/ft
Hammerhead	21.78	kips per column

LIVE LOADS				
Floor Live Load	90.00	psf	810	lb/ft
Snow/Roof Live Load	42.40	psf	440.96	lb/ft
Wind (downward)		psf	211.2	lb/ft

lateral force @top of column		
Siesmic	22.90	kips
Wind	63.36	kips

LC			
1	1.4D	3557.12	lb/ft
	1.4D	30.492	kips
2	1.2D+1.6Lr+(.5L or .8W)	4159.496	lb/ft
	1.2D	26.136	kips
3	1.2D+1.6L+.5S	4565.44	lb/ft
	1.2D	26.136	kips
2W	1.2D+1.6Lr	3754.496	lb/ft
	.8W horizontal	1520.64	kip-ft
	1.2D	26.136	Kips
	.8W downward	168.96	lb/ft

Pu for 75' span		
1	297.276	kips
2	338.0982	kips
3	368.544	kips

Pu and Mu for 75' span		
Mu	1520.64	kip-ft
Pu	320.3952	kips
Pu (LL)	65.5872	kips
Pu (DL)	254.808	kips

Elastomeric Bearings loading	
16.3968	kips per bearing
63.702	kips per bearing

Figure 65: Bridge Pier Loading Conditions

DIMENSIONS		
Column base (b)	8.0	in
Column depth (h)	8.0	in
effective depth (d)		in
Reinforcement		
rho g	0.030	no units
Ag	64.0	in^2
As	1.92	in^2
Use 4 #6 bars		
New As	1.77	in^2
Stirrup Spacing		
Use #3 ties		
48*stirrup diameter	18.0	inches
16*bar diameter	12.0	inches
Smallest Dimension	8.0	inches
USE THIS SPACING	8.0	inches

Pu max	315.07	kips
Ag	73.96	in^2

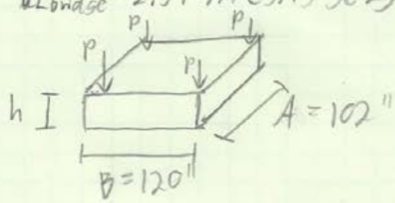
Figure 64: Bridge Pier Calculations

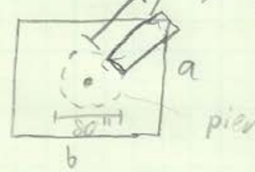
F.7 Bridge Pier Cap Calculations

MQP: Pier Cap Design | Pedestrian Bridge | 1/25/18 | 1/4

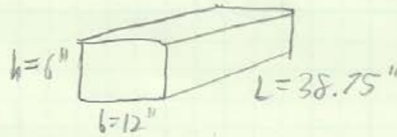
assumptions

$f'_c = 4$ ksi, Shape: rectangular prism
$f_y = 60$ ksi design method: cantilever beam
pier diameter = 80" $LL_{bridge} = S = 42.4$ psf
$\gamma_c = 150$ pcf $L_f = .09$ ksf
$DL_{bridge} = 2.54^k/ft(37.5')(\frac{1}{2}) = 47.63^k$ L = 38.75" Typical cantilever beam



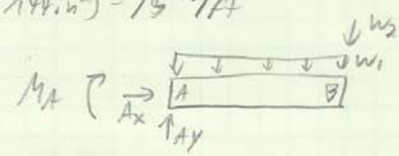


estimate $h \approx 6"$
$b \approx 12"$



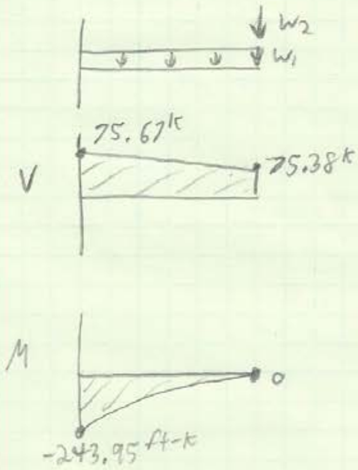
beam DL
$\gamma_c h b = (150pcf)(12")(6")(1ft^2/144in^2) = 75\ lb/ft$

LC
1.2D + 1.6L



$w_1 = 1.2D = 1.2(75\ lb/ft)(1^k/1000lb) = .09^k/ft$

$w_2 = 1.2D + 1.6L = 1.2(47.63^k) + 1.6(.09ksf)(\frac{9'\ \text{bridge width}}{2})(37.5')$
$= 57.156^k + 18.225^k$
$= 75.38^k$



$F_x = 0 = A_x$

$F_y = 0 = A_y - 75.38^k - .09^k/ft(3.23')$

$A_y = 75.67^k$

$\curvearrowleft M_A = M_A + 75.38^k(3.23') + (.09^k/ft)(3.23')(\frac{3.23'}{2})$

$M_A = -243.95^{k-ft}$

2/4

$M_{max} = 243.95^{ft\text{-}k}$

$\rho_{max} = .85(\beta_1 = .85)\left(\frac{f'_c = 4ksi}{f_y = 60ksi}\right)\left(\frac{\varepsilon_u = .003}{(\varepsilon_u = .003) + (\varepsilon_T = .005)}\right) = .0181$

$M_u = \phi \rho f_y bd^2 (1 - .59\rho \, f_y/f'_c)$

$243.95^{ft\text{-}k}(12''/1) = (.9)(.0181)(60ksi)(12'')d^2(1 - .59(.0181)(\frac{60ksi}{4ksi}))$

$297.19 = d^2$

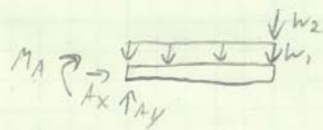
$d \geq 17.24''$

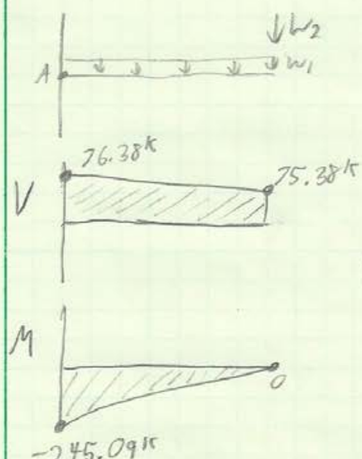
use $d = 18''$ $h = 21''$

beam DL $= (150pcf)(h = 21'')(12'')(1'/144'') = 262\ ^{lb}/ft = .262\ ^{k}/ft$

$w_1 = 1.2D = 1.2(.262\ ^{k}/ft) = .31\ ^{k}/ft$

$w_2 = 75.38^k$





$F_x = 0 = A_x$

$F_y = 0 = A_y - 75.38^k - .31\ ^{k}/ft\,(3.23')$

$\underline{A_y = 76.38^k}$

$\circlearrowleft + M_A = 0 = M_A + 75.38^k(3.23') + .31\ ^{k}/ft\,(3.23')(\frac{3.23'}{2})$

$\underline{M_A = -245.09^{k\text{-}ft}}$

$M_{max} = 245.09^{k\text{-}ft}$

$M_u = \phi \rho f_y bd^2(1 - .59\rho \, f_y/f'_c)$

$245.09^{k\text{-}ft}(12''/1) = (60ksi)(.9)(.0181)(12'')d^2(1 - .59(.0181)(\frac{60ksi}{4ksi}))$

$2941.08 = 9.85d^2$

$d \geq 17.28 < d = 18''$ ✓

3/4

longitudinal reinforcement

$M_u = \phi \rho f_y b d^2 (1 - .59 \rho \, f_y/f'_c)$

$244.89^{k\text{-}ft}(12''/_1) = (.9)\rho(60ksi)(12'')(18'')^2(1 - .59\rho(60ksi/4ksi))$

$2938.68 = 209952\rho(1 - 8.85\rho)$

$2938.68 = 209952\rho - 1858075.2\rho^2$

$.001582 = .11299\rho - \rho^2$

$\rho = .0966, \boxed{.0164}$

$\rho_{max} = .0181$

$A_s = \rho b d = (.0164)(12'')(18'') = 3.54 in^2$

use 3 #10 bars

check fit

#4 Stirrups

$b = 12'' \geq 2(1'') + 3(d_b = 10/8'') + 2(d_s = 4/8'') + 2(C_c = 1.5'')$

$b = 12'' \geq 9.75''$

✓

Shear reinforcement S_{max}

$4\sqrt{f'_c}\, b_w d = 4 \frac{\sqrt{4000psi}}{1000}(12'')(18'') = 54.64^k$

$l = d = 18''$

$V(18'') = 76.25^k - .31^{k/ft}(l = 18'')(1/12'') = 75.79^k$

$V_s = \frac{V - \phi V_c}{\phi} = \frac{75.79^k - 20.49^k}{.75} = 73.73^k$

$S = \frac{A_v f_y d}{V_s} = \frac{(.39 in^2)(60ksi)(18'')}{73.73^k} = 5.71''$

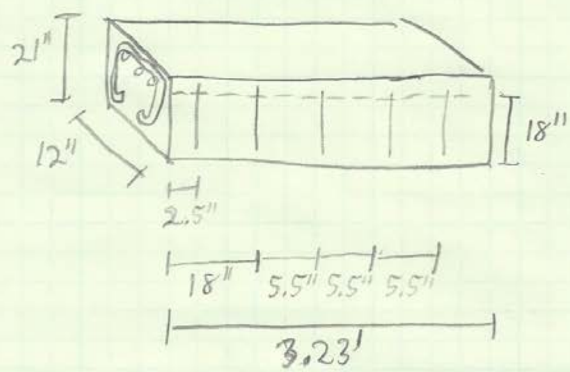
max stirrup spacing = 5.5''

4/4

Shear
$\phi = .75$

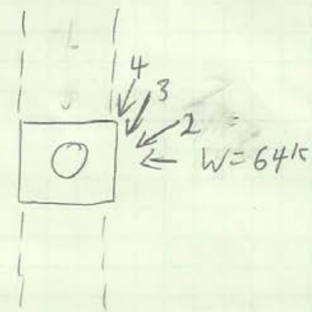
$V_{max} = 76.25^k$
$\phi V_c = \phi 2\sqrt{f'_c}\, bd = (.75)(2)\left(\frac{\sqrt{4000}}{1000}\right)(12")(18") = 20.49^k$
$\frac{\phi V_c}{2} = 10.25^k$

Results



1 #4 Stirrup @ 2.5"
1 #4 Stirrup @ 15.5"
3 #4 Stirrup @ 5.5"

Weight
$\gamma_c = 150pcf$
$V = (120"/12)(102"/12)(21"/12) = 148.75ft^3$
$W = (150pcf)(145.21 ft^3)$
$W = 22$ kips



W_2 @ 30°

$\cos(30) = \frac{\#}{64}$

$W_2 = 55.43^k$

$M = 55.43^k(30') = 1662.77$

$e = 1662.77^{k\text{-}ft} / 479.08^k = 3.47'$ req $L = 20.8'$

W_3 @ 45°

$\cos(45) = \frac{\#}{64}$

$W_3 = 45.25^k$

$M = 45.25^k(30') = 1357.65^{k\text{-}ft}$

$e = 2.83'$ req $L = 16.9'$

W_4 @ 60°

$\cos(60) = \#/64$

$W_4 = 32^k$

$M = 32^k(30') = 960^{k\text{-}ft}$

$e = 2.00'$

req $L = 12'$

W_5 @ 75°

$\cos(75) = \#/64$

$W_4 = 16.56^k$

$M = 16.56^k(30') = 496.9^{k\text{-}ft}$

$e = 1.04'$

req $L = 6.24'$

F.8 Bridge Footing Calculations

Bridge Footings

Loading (per peir calculation done on excel sheet)

DL: Superstructure: $2.5408 \, ^k/_{ft} \cdot (75') = 190.56 \, k$

Cap: 21.78^k

Peir: 157.08^k

Total: 369.42^k

LL: $S/_{Lv} = .441 \, ^k/_{ft} \cdot (75') = 33.08^k$

$L_v = .81 \, ^k/_{ft} (75') = 60.75^k$

$W_v = .211 \, ^k/_{ft} (75') = 15.83^k$

$W_L = 63.36 \, k$

$q_a = 5 \, ^k/_{ft^2}$ depth = 5'

$f'_c = 4$ ksi height = 30'

f_y: 60 ksi $\omega = 1.3$ (Design of concrete structures, ch.16)

$\rho_s = 125$ pcf

// Park Ave

$P = 479.08^k$

$M = 1900.3^{k\text{-}ft}$

$e = M/P = 3.97'$

$e < L/6$

$L > 6e = 23.82'$

$L = 24'$

⊥ Park Ave

$P = 479.08^k$

$M = 960$ k-ft

$e = M/P = 2.00'$

$L > 6e = 12'$

$L = 12'$

↳ $A: 24' \times 12' = 288 \, ft^2$

$$q_{max/min} = \frac{P}{A} \pm \frac{6M}{BL^2} = 1.66 \pm 1.65 \quad q_{max} = 3.31, \; q_{min} = .01$$

$$\boxed{\begin{array}{ll} L = 24' & d = 24'' \\ b = 12' & H = 27'' \end{array}}$$

Pier Footing reinforcement

// to Superstructure

case #5

$q_{max} = 4.34\ ksf$

$b = 14'$ $d = 4'$

$M_u = q_{max} b \frac{\left(\frac{b-a}{2}\right)^2}{2} = (4.34 ksf)(14') \frac{\left(\frac{14'-6.67'}{2}\right)^2}{2}$

$M_u = 408.07^{k\text{-}ft}$

$M_u = \Phi \rho f_y b d^2 (1 - .59 \rho\, f_y/f'_c)$

$(1000)\ 408.07^{k\text{-}ft}(12''/') = (.9)(60000 psi)(168'')(48'')^2 (1 - .59\rho (\frac{60000 psi}{4000 psi}))$

$4896840^{lb\text{-}in} = 2.09 \times 10^{10} \rho\ (1 - 8.85\rho)$

$2.34 \times 10^{-4} = \rho - 8.85 \rho^2$

$\rho = .1127$
2.34×10^{-4}

$\rho_{min} = \frac{3\sqrt{f'c}}{f_y} \geq \frac{200}{f_y} = \frac{3\sqrt{4000 psi}}{60000 psi} \geq \frac{200}{60000 psi}$

$\rho_{min} = .00316 \geq .0033$

$\rho_{min} = .0033$

$\beta_1 = .85 - .05 \left(\frac{f'_c - 4000}{1000}\right) = .85$

$\rho_{max} = .85 \beta_1 (f'_c/f_y)\left(\frac{\epsilon_u}{\epsilon_u + .004}\right) = .85(.85)\left(\frac{4 ksi}{60 ksi}\right)\left(\frac{.003}{.003+.004}\right)$

$\rho_{max} = .02064$

$\rho = .0033$

$A_s = \rho b d = (.0033)(14')(4')\left(\frac{144 in^2}{ft^2}\right) = 26.61 in^2$

$A_s = 1.9\ in^2/ft$ of b

3 #7 bars

$A_s = 1.8\ in^2$

pier Footing reinforcement

$\perp$ to Superstructure

Case #5
$q_{max} = 4.34$ ksf
$D = 22'$ $d = 4'$
$a = 6.67'$

$$M_u = q_{max} D \frac{\left(\frac{D-a}{2}\right)^2}{2} = (4.34\text{ ksf})(22')\frac{\left(\frac{22'-6.67'}{2}\right)^2}{2}$$

$M_u = 2804.83$ k-ft

$$M_u = \phi \rho f_y D d^2 (1 - .59\rho (f_y/f'_c))$$

$$2804.83^{k\text{-}ft}(12''/1) = (.9)(60\text{ksi})(264'')(48'')^2 \rho (1 - .59(60\text{ksi}/4\text{ksi})\rho)$$

$$33657.96 = 32845824\rho(1 - 8.85\rho)$$

$$33657.96 = 32845824\rho - 290685542.4\rho^2$$

$$1.158 \times 10^{-4} = .1129943503\rho - \rho^2$$

$\rho = .11196,\ .00103$

$$\rho_{min} = \frac{3\sqrt{f'_c}}{f_y} > \frac{200}{f_y} = .00316 \geq .0033$$

$\rho_{min} = .0033$

$$\rho_{max} = .85\beta_1 (f'_c/f_y)\left(\frac{\epsilon_u}{\epsilon_u + .004}\right)$$

$\rho_{max} = .02064$

$\rho = .0033$

$$A_s = \rho D d = (.0033)(22')(4')(144\text{ in}^2/\text{ft}^2) = 41.82\text{ in}^2$$

$A_s = 3.48$ in^2/foot of b

use
4 #8 bars
$A_s = 3.16$ in^2

Short Pier Footings

pier height = 16'
pier tributary width = 40'

DL
Superstructure = 2.5408 k/ft (40') = 101.63 kips
pier cap = 21.78 kips
pier = 5.24 k/ft (16') = 83.84 kips

LL = S = .441 k/ft (40') = 17.64 kips
L_p = .81 k/ft (40') = 32.4 kips

w_{vert} = .211 k/ft (40') = 8.44

w_{lat} = 63.36 kips

$P = DL + LL + w_{vert}$ = 101.63 + 21.78 + 83.84 + 17.64 + 32.4 + 8.44

P = 265.73

$M = w_{lat}$ (pier height) = 63.36 k (16') = 1013.76 kips

F.9 Bridge Final Pier Calculations

Final Pier Calculations

Dead Loads	
Item	Value (lb/ft)
MEP	5.00
Solar	4.00
Purlins/bar joists	10.00
Roof sheathing	10.00
Roof frame	23.00
Flooring	10.00
Lat/through-truss	2226.80
Facade	250.00
Insulation	2.00
Total	2540.80

Live Loads	
Item	Value (lb/ft)
Floor	810.00
Snow/Roof	440.96
Wind (downward)	211.20

DC, LL, EH, WS

Soil bearing pressure 5 Ksf

Bridge section is 36' 6.9" long, designing abutment to hold 9' worth of the bridge loads.

Dead load = 2540.80 lb/ft (9') /1000 = 22.9 Kips

Live Load = 440.96 lb/ft (9') /1000 = 3.97 Kips

$P_u = 1.2D + 1.6L = 33.83$ Kips $\qquad A_{st} = 0.02A_g$

$\phi P_u = \phi\alpha[0.85f'_c(A_g - A_{st}) + f_y A_{st}]$

$33.83 = 0.65(0.80)[0.85(4)(A_g - 0.02A_g) + (60 \times 0.02A_g)]$

$65.06 = 3.4A_g - 0.068A_g + 1.2A_g$

$65.06 = 4.53A_g$

$14.36 \text{ in}^2 = A_g$ req.

Final Pier Calcs

Due to the width of our bridge, the peir will be a 9' by 9' square tie column.

$A_g = 11664 \text{ in}^2$

Required steel area - minimum $\rho = 0.015$ (like in column calculation)

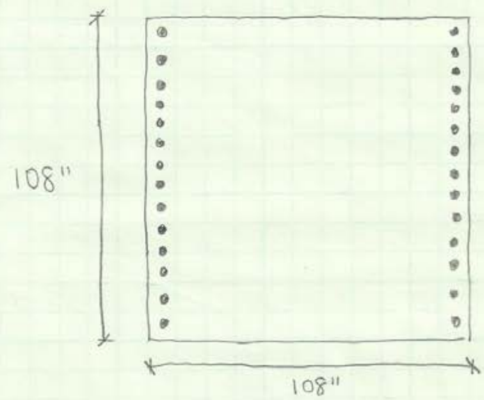
$A_{st} = 0.015\,(11664 \text{ in}^2) = 174.96 \text{ in}^2$

Use #18 bars

$d = 2.26"$
$A = 4 \text{ in}^2$

$\#\text{ of bars} = \frac{A_{st}}{\text{Area}} = \frac{174.96}{4} = 43.74 \text{ bars}$

Use 44 bars with 2" space in between



Peir will extend to 4 feet below the ground at highest point to satisfy the bearing capacity.

Due to this, the height will be 8' feet.

Weight of peir

$(9' \times 9' \times 8') + 47.25 \text{ ft}^3 = \quad \times \frac{150 \text{ lb}}{\text{ft}^3} = 104.3^{K}$

Appendix G: Cost Analysis

Sitework & Labor						
Description	Qty.	Units	Unit Cost	Total Cost	RS Means Cost Code	Secondary Cost Code
Excavation	2,991	cy	$ 14.05	$ 42,020.95	31 23 16.46 2420	
Backfill	25	cy	$ 7.65	$ 190.40	31 23 23.14 1000	
Parking Lot	11,666.67	sf	$ 17.95	$ 209,416.67	32 12 16.13 0200	
Walkways	55.56	cy	$ 39.00	$ 2,166.67	03 31 13.70 4350	
Subgrade	2,837.96	cy	3.79	$ 10,755.88	31 23 23.14 2400	
Materials & Labor						
Description	Qty.	Units	Unit Cost	Total Cost		
Structural Steel	128.97	tons	$ 5,225.00	$ 673,858.07	05 12 23.77 3100	05 12 23.77 5390
Concrete: Cast in Place	691.23	cy	$ 226.86	$ 158,104.48	03 30 53.40 4700	03 30 53.40 3850
Concrete: Hollow Core Planks	17,500.00	planks	$ 12.50	$ 218,750.00	03 41 13.50 0100	
Brick Siding	89.94	1000 bricks	$ 2,500.00	$ 224,857.14	04 21 13.13 0020	
Roof Deck	35,000.00	sf	$ 2.44	$ 85,400.00	05 31 23.50 2400	
Waterproofing: Roof	35,000.00	sf	$ 3.71	$ 129,850.00	07 13 53.10 0100	
Waterproofing: Walls	15,740.00	sf	$ 3.74	$ 58,867.60	07 13 53.10 0300	
Insulation	50,740.00	sf	$ 1.18	$ 59,873.20	07 22 16.10 0030	
Flooring	35,000.00	sf	$ 4.74	$ 165,900.00	09 65 66.10 1000	
Acoustic Ceiling	17,500.00	sf	$ 3.50	$ 61,250.00	09 51 23.30 0600	
Exterior Doors	3.00	doors	$ 1,175.00	$ 3,525.00	08 11 16.10 0020	
Interior Doors	8.00	doors	$ 240.00	$ 1,920.00	08 12 13.13 0025	
Curtain Walls	4,760.00	sf	$ 74.00	$ 352,240.00	08 44 13.10 0050	
HVAC	52,500.00	sf	$ 40.00	$ 2,100,000.00		
Plumbing	52,500.00	sf	$ 10.00	$ 525,000.00	DCAM Cost	
Electrical	52,500.00	sf	$ 22.00	$ 1,155,000.00	Estimating Manual	
Communications and Security	52,500.00	sf	$ 2.00	$ 105,000.00	(4.2.1.4)	
Fire Suppression	52,500.00	sf	$ 4.00	$ 210,000.00		
Field Turf	17,500.00	sf	$ 8.40	$ 147,000.00	32 18 13.10 0600	
Elevator	1.00	lump sum	$ 122,900.00	$ 122,900.00	14 24 13.10 1150	
Elevator Shaft	1,200.00	sf	$ 9.10	$ 10,920.00	04 22 10.11 0080	
Multipliers						
Location: Worcester, MA	1.077					
Time: 2019**	1.042					
Design Fees	7.2%					
Estimate Contingency	15%					
Total Cost				$ 9,459,832.94		

*Concrete estimates vary based on the type of concrete being installed. For this reason, a weighted average unit cost was used in this estimate.

** An inflation rate of 2.1% was used for analysis

Figure 66: Athletic Facility Cost Estimate Overview

Description	Qty.	Units	Unit Cost	Total Cost	RS Means Cost Code	Secondary Cost Code
Excavation	2,990.81	cy	14.05	$ 42,020.95	31 23 16.46 2420	
Backfill	24.89	cy	7.65	$ 190.40	31 23 23.14 1000	
Subgrade	10.37	cy	3.79	$ 39.30	31 23 23.14 2400	
Structural Steel	151.45	tons	$ 5,225.00	$ 791,310.83	05 12 23.77 3100	05 12 23.77 5390
Reinforcing Bars	38.26	tons	$ 2,200.00	$ 84,167.60	03 21 11.60 0250	
Concrete: Cast in Place	448.44	cy	$ 226.86	$ 101,733.94	03 30 53.40 4700	
Roof Deck	4,000.00	sf	$ 2.44	$ 9,760.00	05 31 23.50 2400	
Waterproofing	4,000.00	sf	$ 3.71	$ 14,840.00	07 13 53.10 0300	
Insulation	46,484.00	sf	$ 1.18	$ 54,851.12	07 22 16.10 0030	
Interior Doors	2.00	doors	$ 240.00	$ 480.00	08 12 13.13 0025	
Glass Façade	8,800.00	sf	$ 74.00	$ 651,200.00	08 44 13.10 0050	
HVAC	3,800.00	sf	$ 40.00	$ 152,000.00	DCAM Cost Estimating Manual (4.2.1.4)	
Plumbing	3,800.00	sf	$ 10.00	$ 38,000.00		
Electric	3,800.00	sf	$ 22.00	$ 83,600.00		
Communications and Security	3,800.00	sf	$ 2.00	$ 7,600.00		
Fire Suppression	3,800.00	sf	$ 4.00	$ 15,200.00		
Flooring	3,800.00	sf	$ 4.74	$ 18,012.00	09 65 66.10 1000	
Acoustic Ceiling	3,800.00	sf	$ 3.50	$ 13,300.00	09 51 23.30 0600	
Hand Rails	800.00	lf	$ 76.00	$ 60,800.00	05 52 13.50 0015	
Solar Panels*	150.00	panels	$ 25,600.00	$ 25,600.00	DCAM Cost Estimating Manual	
Multipliers						
Location: Worcester, MA	1.077					
Time: 2019**	1.04					
Design Fees	7.2%					
Estimate Contingency	15%					
Total Cost				$ 2,996,116.97		

* A lump sum was assumed for the Solar Panel cost, and estimated as a 10 kW system

** An inflation rate of 2.1% was used for analysis

Figure 67: Pedestrian Bridge Cost Estimate Overview

Appendix H: Connection Geometry Check

The following section shows how key connections for the proposed athletic building and pedestrian bridge structural system would look. Each connection was checked for geometric constraints to ensure that connections could be designed. Note that the connections were not structural designed of checked for failure conditions.

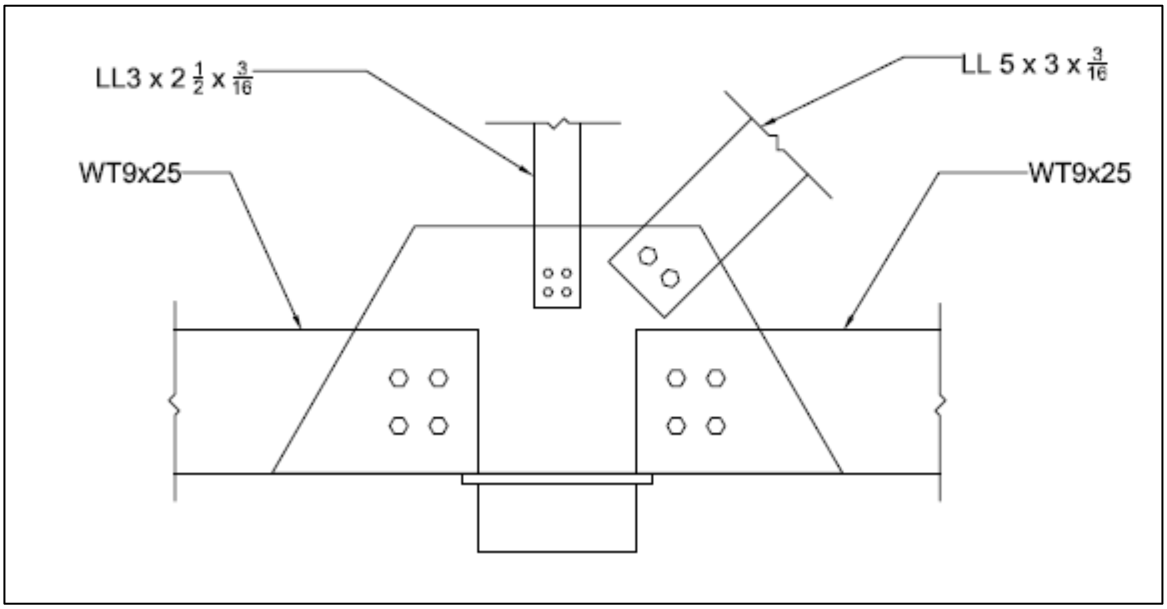


Figure 68: Pedestrian Bridge, Superstructure Connection to Elastomeric Bearings

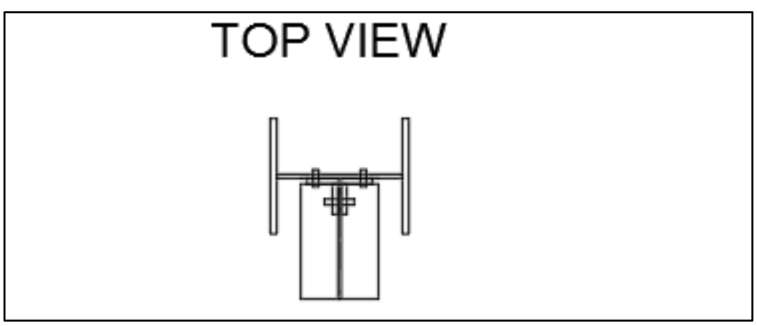


Figure 69: Athletic Facility, Interior Beam Connection to Column

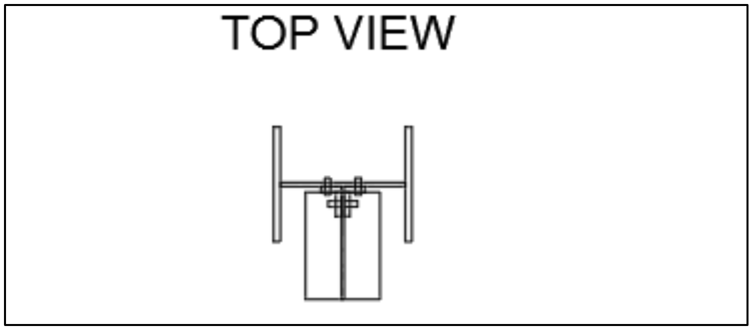


Figure 70: Athletic Facility, Exterior Beam Connection to Column

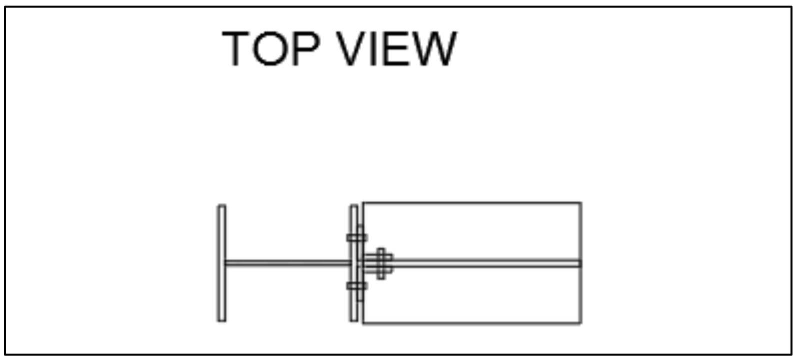


Figure 71: Athletic Facility, Interior Girder Connection to Column

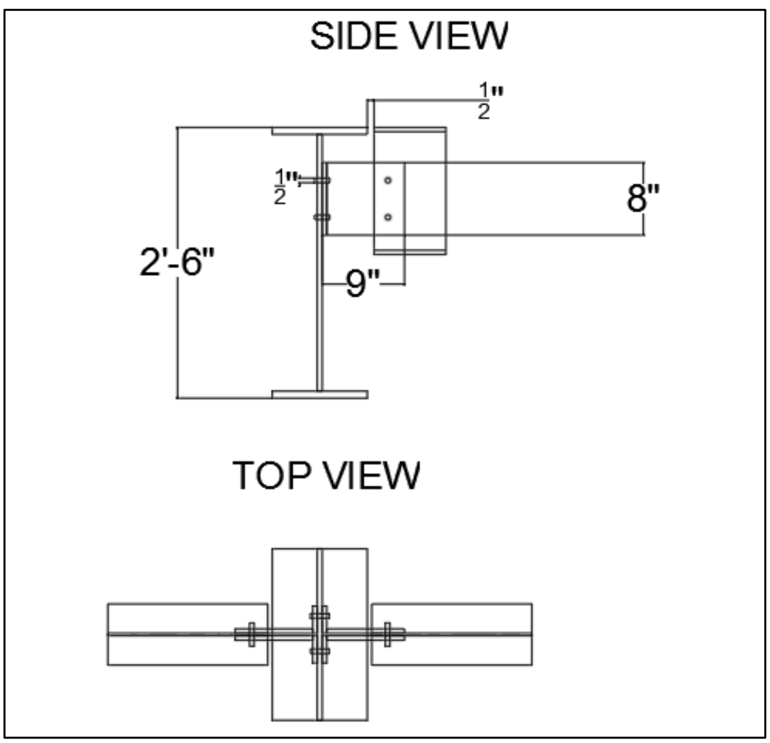


Figure 72: Athletic Facility, Interior Girder Connection to Interior Beam

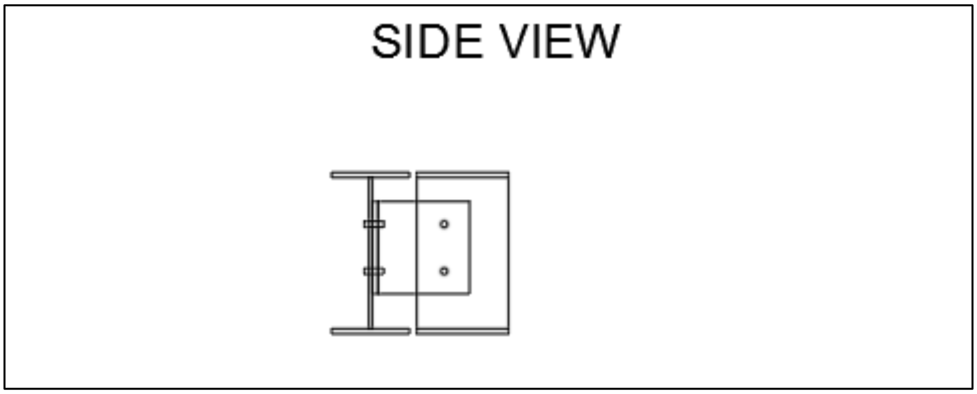


Figure 73: Athletic Facility, Exterior Girder Connection to Interior Beam