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Rehabilitation of Mechanics Hall

A Major Qualifying Project By

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Abstract

Mechanics Hall is an historic building built in 1852 in Princeton, Massachusetts. The project goal was to develop a plan to bring Mechanics Hall into compliance with current building code requirements. A field inspection was performed to gather data used to develop a Building Information Model. Due to the extensive use of building code, a Code Review was developed, along with a design and cost estimate for two proposed uses. The results were presented to the Friends of Mechanics Hall.

Authorship

Matthew Cogswell, Nathan Jaworski, and Matthew Sprague all contributed to the writing of this report. Matthew Cogswell was responsible for the fire safety aspects of this report, including the egress requirements and sprinkler design, as well as accessibility solutions. Nathan Jaworski was responsible for the preparation of drawings and Building Information Modeling (BIM) of Mechanics Hall, as well as the design for drainage solutions. Matthew Sprague was responsible for the structural and foundation analysis and design.

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Capstone Design

Mechanics Hall is an historic building that was constructed in 1852 in Princeton, Massachusetts, and has been abandoned since the 1980s. The building was originally used as a school house, and then was used for a variety of town activities. The structure consists of a ground floor, a second floor, and a basement, as well as a two-story addition that is cantilevered at the rear of the building. In order to re-open this building to public use, repairs and renovations must be performed to ensure that the building meets modern building codes.

In order to complete this project, intensive field investigations were performed to gather data on the buildings geometry and features and to asses existing conditions. Using data collected during these inspections, computer models were generated using AutoCAD and Revit, and building code issues were identified. Structural concerns were identified and solutions were proposed along with measures to limit future damage to the structure. Matters of building code compliance were investigated in terms of egress and fire safety, as the building must be safe for public use. Using the 3D building information model, updated versions of the building were compiled and presented with the new intended uses for the building.

This project addressed six of the eight realistic constraints specified by the ASCE. First, like most engineering projects, this project was heavily influenced by economics. As limited funding is available for this project, steps were taken to reduce the cost of repair to the structure. A cost estimate was prepared that identifies and quantifies the materials and procedures needed to rehabilitate Mechanics Hall.

This project addressed health and safety issues through the consideration of building code compliance. The *International Building Code* presents requirements that buildings must meet depending on the use of the building. These requirements are designed to certify that all buildings will be safe for public use. This project addressed the fire protection, structural, accessibility, parking, and drainage requirements of *International Building Code* to ensure the health and safety of the users of Mechanics Hall.

This project addressed constructability through the proposal of practical design solutions. One example of this is with the design of the wood-steel composite beams. Because the joists in the original building connect flush to the carrier beams, simply replacing the carrier beam would undo every connection. Therefore, a steel plate was designed to connect to the bottom of the beam, which would add the required capacity without the need to reconstruct the basement.

This project was affected by sustainability in the form of historic context and drainage issues. The goal of this project is to renovate Mechanics Hall to modern building standards while maintaining the historical feel of the building. As much of the original building will be preserved, minimizing the impact of producing new materials. As part of the preservation of the structure, drainage solutions were proposed to limit water infiltration and protect the existing building.

This project dealt with ethical concerns due to the heavy burden of keeping the cost low. It can be difficult to properly design a safe restoration plan for buildings as performing the necessary work can be costly. While it is important to keep the design economical, the safety and integrity of the building design or construction cannot be compromised. At the same time, it is important to control the scope of the project to areas within our area of expertise. It is important to keep within our background and to keep from advising in areas where we are not qualified to do so.

This project also faced social issues due to the need to develop a use for the building. An appropriate use for the building is determined by recognizing what the needs are of the residents of Princeton and what the building is capable of providing.

Executive Summary

Mechanics Hall is a historic building located in Princeton, Massachusetts. The building was built in 1852 in East Princeton Village. Mechanics Hall functioned as a schoolhouse until 1945, when it became a general use building for the village. Mechanics Hall has been closed for some time, and the building has not been well maintained. The Friends of Mechanics Hall are a non-profit organization that is looking to rehabilitate Mechanics Hall, and open it for public use once more. While considering the desires of the Friends of the Mechanics Hall, two proposed uses were developed for the building. These intended uses determined the scope of the required renovations. The first use consists of an office on the first floor, and an assembly area on the second floor. The second proposal has an office on the first floor and a coffee house on the second floor. The two uses determined the necessary design solution and applicable code requirements. A cost estimate for both uses was also prepared and presented to the Friends of Mechanics Hall.

The goal of this project is to evaluate Mechanics Hall against the requirements specified in the *International Building Code* (*IBC*) in regards to structural strength and fire safety, as well as drainage concerns, accessibility, and parking issues. The first step in realizing this goal was to perform a thorough inspection of Mechanics Hall. During the inspection, data was gathered on the general dimensions of the building space, the dimensions of hallways and doorframes, and the dimensions and spacing of structural members. The condition of the building and structural members was also noted. Using this data, floor plans were created and Building Information Modeling (BIM) was used to convey information about the building into a 3D computer model to facilitate visualization and the communication and coordination of design idea.

The safety hazards present in Mechanics Hall were identified using the building requirements specified in the *International Existing Building Code* (*IEBC*) and *IBC*. The results of the building inspection were then compared to these requirements. A Code Review was prepared organizing the referenced code with the implications of that code on the project.

The width of the hallways and doorframes were recorded, as well as the steepness of the stairs and the heights of the stair railings. These were applied to the egress requirements specified in *IEBC* and *IBC*. *National Fire Protection Association* (*NFPA)* 13 was used to design a sprinkler system for Mechanics Hall to meet the requirements for one of the proposed uses. Requirements for smoke detectors were investigated with respect to *IBC.* The scope of this project also considered structural fire protection, which would require a certain fire resistance rating for interior and exterior walls.

A structural analysis of the gravity load-resisting floor assembly was guided by restrictions detailed in *IBC*. Because Mechanics Hall is a wood framed structure, the *National Design Specifications for Wood Construction* (*NDS*) was used as the design specifications for structural calculations. Allowable Stress Design was used to evaluate the structure. Mechanics Hall consists of the original building as well as an addition. The addition has a different framing plan than the original building, so two separate structural investigation were conducted. The results from both analyses were compared to requirements outlined in *IBC* for the structural frame and the foundation. Design solutions were proposed as necessary to add the required strength to a structural member.

Additional work included research into techniques to improve drainage systems, accessibility, and parking. Major issues were observed with Mechanics Hall regarding water infiltration, particularly through the fieldstone foundation wall. The building has poor accessibility due to excessive stairs and an awkward layout. Research into the Princeton zoning bylaws was also conducted to propose methods of increasing available parking.

The first proposed use required a sprinkler system due to occupancy requirements and the hazard. The sprinkler design was done in accordance with *NFPA 13.* Egress requirements were also remedied by installing new staircases in Mechanics Hall. The existing staircases posed the most significant concern for the compliance with the egress requirements specified by the *IEBC*. Upon investigating smoke detector requirements it was found that none were required for this situation. Structural fire protection needs were different for each proposed use. For the first use the requirements were stricter requiring the use of fire resistive paint. The second proposed use does not need any specific fire protection rating for any of its structural members.

The results of the structural analysis highlighted issues in the structural frame. It was found that the carrier beams and the columns in the original building had insufficient strength to carry the loads specified by *IBC*. There were also many joists that had been subjected to substantial water damage and had experienced mold and decay, and several of the joists were notched. The footings for the columns were inadequate to support the required loads, and the foundation walls contained holes that allowed water infiltration. Therefore, a wood-steel composite beam was designed to reinforce the existing carrier beams. Larger columns were designed to provide the surface area necessary to adequately resist the required loads. It was recommended that the damaged and notched joists be replaced. New concrete footings were designed to support the columns, and a concrete wall reinforced with steel-wire mesh was designed to seal and add strength to the existing foundation.

The joists in the addition were determined to have insufficient strength, as did the carrier beams. The foundation for the entire addition was also inadequate. It was recommended to replace each joist and carrier beam with larger structural members. A concrete strip footing and small foundation wall was designed to support the rear and sides of the addition.

It was found that the significant water damage found in the basement was due to poor drainage and holes in the foundation wall. A French Drain system was proposed to seal off the holes in the foundation wall and redirect the water out of the basement. To increase the accessibility of the building, it was recommended that a landing and a handicapped rail should be installed on the staircase. This will increase the accessibility of Mechanics Hall, but there will still be accessibility issues. For example, certain measures such as handicapped restrooms are impractical due to the nature of the building. While not every aspect of accessibility can be met, a building can still be approved by the code official if an effort is made to make the building as accessible as possible. The Princeton bylaws are vague regarding the requirements for parking. Building expansions in a business district in Princeton must be approved by the town planning board.

The above results were combined into two proposed uses for Mechanics Hall. A cost estimate was produced for each use based off of the required materials for each renovation. One solution cost significantly more expensive due to the inclusion of the sprinkler system. The results of this study were presented to the Friends of Mechanics Hall.

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1: Introduction

Mechanics Hall is situated in eastern Princeton, Massachusetts, a short distance from the WPI campus (Figure 1). It has a wealth of history, having been used for many different purposes over its 161-year lifetime. When Mechanics Hall was under construction in 1852, wood was salvaged from a collapsed building in Princeton and used for construction. This salvaged wood is dated to 1843, and is the reason for this date appearing on the front of the building (Figure 2). Originally it was used as a school and served that purpose until 1945. During this span it was also used for community gatherings, lectures, and banquets. Since then it has been used as everything from an extra library to an American Legion post. Because of the building's history, there are many groups that are extremely interested in its preservation and continued use. These include The Princeton Farmer and Mechanics Association, The East Princeton Village Improvement Society and The Friends of Mechanics Hall. All of these organizations have helped with the usage and upkeep of the building. Currently Mechanics Hall is listed on the National Register of Historic places adding to its importance in the history of Princeton.

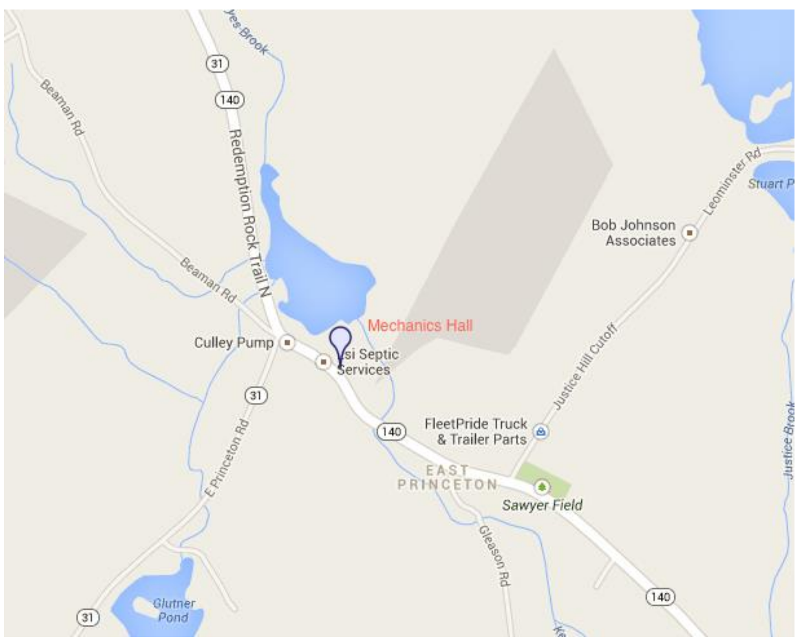

Figure 1: Mechanics Hall Locus Map

Mechanics Hall can be classified as a Greek-Revival building. The pillars and pediment out front can attest to this. It also has the classic look of a front gabled building (Figure 2). It rests on a

0.3-acre rectangular plot of land and has an approximate floor area of 3200 square feet (Figure 1). It is a two story building with a basement plus an addition. The first floor has 3 rooms, and two bathrooms. Because one of the rooms is part of the addition it can only be accessed from the basement and second floor. The second floor is a large open stage area. Behind the stage is a kitchen making the second floor great for functions. Because of its age many of the mechanical systems within the building need a lot of updating. Plumbing and heating are currently nonexistent. The electrical configuration in the building works, but is extremely outdated which can be seen by the outdated fixtures and hanging wires. Because of the building's age the structural integrity and code compliance need to be further investigated.

Figure 2: Mechanics Hall Front

Preservation Massachusetts listed Mechanics Hall as one of Massachusetts Most Endangered Historic Resources in 2012 ("Friends of mechanics," 2013). Mechanics Hall was assessed for some of the above problems along with others that were revealed during site investigations or subsequent engineering analyses. More background is presented in chapter 2 in order to give the reader a greater understanding of why different choices were made in regards to the building analysis. It was also important to confirm that the reader fully understood the process with which the project was carried out. For this reason a detailed methodology is supplied in chapter 3. Part of the project was coming up with potential uses for the building, and describing what had to be done in order to achieve these uses (chapter 5). However, before things could be changed the existing conditions had to be analyzed (chapter 4). Finally, cost estimates and design solution were furnished for the friends of Mechanics Hall.

2: Background

Before beginning work on renovating Mechanics Hall, it is important to research the issue at hand. An understanding of the historical aspect of the building as well as engineering is necessary to develop an appropriate restoration plan.

2.1: Historical Value

The current state of Mechanics Hall features many problems that threaten the future of the building, such as structural decomposition, faulty wiring, water infiltration and vandalism. The Friends of Mechanics Hall was established, in 2012 and set out a list of short and long-term goals for the preservation of Mechanics Hall. They hope that its listing as an endangered historical resource will help to educate the public about the importance of Mechanics Hall, what it means to their town, and their connections as a community.

In 2004, a committee surveying Princeton found 97% of residents were interested in seeing the building put to good use (UTAC, 2012). East Princeton Village, including Mechanics Hall, was accepted by the National Parks Service for inclusion on the National Register of Historic Places in September 2004. The National Register of Historic Places is an official listing of historically significant sites and properties throughout the country. It is maintained by the National Park Service, U.S. Department of the Interior. It includes districts, sites, buildings, structures, and objects that have been identified and documented as being significant in American history, architecture, archaeology, engineering or culture. These sites and properties reflect the prehistoric occupation and historical development of our nation, state, and local communities. It is a federal designation and is administered by the Secretary of the Interior through the Massachusetts Historical Commission as the State Historic Preservation Office (UTAC, 2012).

As a municipal building/non-profit property, Mechanics Hall qualifies for a 50/50 matching grant from the Massachusetts Preservation Projects for renovations (Galvin, 2012). Under the Massachusetts Historic Rehabilitation Tax Credit Program, historic properties that are income producing are eligible to receive up to 20 percent of the cost of certified rehabilitation expenditures in state income tax credits (Galvin, 2012).

The National Register does not restrict a property owner's private property rights. The owners of National Register properties are able to remodel, renovate, sell, or even demolish the property with no restrictions. However, significant modifications could result in removal from the National Register, before this occurs, the property owner should contact the State historic preservation office (SHPO.) The SHPO is the state agency that oversees historic preservation efforts in their state. There may be state or local preservation laws that the owner should be aware of before they undertake a project with a historic property Furthermore, the federal government does regulate alterations to historic properties where federal funds have been invested. If the owner has not received federal grant funds or federally sponsored tax benefits then there is no federal restriction on the property owner. The same applies at the state and local level. Recipients of state or local funds or tax benefits to preserve their historic property may be subject to design review for any alterations (Galvin, 2012).

2.2: Building Analysis

Before begin the building analysis on Mechanics Hall, it is important to research exactly what should be looked at in order to perform a proper analysis of the building.

2.2.1: Field Inspections

An integral part of the work that was performed on Mechanics Hall were on-site inspections. The purpose of the inspections is to gather data regarding the dimensions and conditions of the structural members that support the building, and to identify issues that must be fixed before the building can open to the public. The *ASCE* (American Society of Civil Engineers) *Guideline for Structural Condition Assessment* will be consulted as an aid to the inspection process, and the building will be analyzed according to the *International Building Code* (IBC).

2.2.2: Structural Systems

The *ASCE Guideline for Structural Condition Assessment* highlights potential structural issues that should be noted during a typical building inspection. The critical issues found in Mechanics Hall were damages due to moisture and load (ASCE 2000). As the moisture content of the wooden members change, the physical properties and dimensions of the wood will change. If the wood becomes too dry after installation, the beams will shrink and crack, resulting in a reduction in capacity. If the wood becomes too wet after installation, the wood may become soft and malleable. Water damage was inspected using a combination of visual inspection and physical testing of a member to identify soft areas. This was done by prodding structural members with a pencil to test its rigidity.

Mechanics Hall also had a significant problem with mold growing on structural members. These organisms feed off the damp wood, which reduces the cross-sectional area of the structural member, reducing its capacity.

Load duration was a concern as the wood members were installed in the mid-1800s, and some members were not strong enough to satisfy modern building code requirements. A wooden beam's capacity decreases over time when subjected to consistent loading. Figure 3 displays how the load duration factor changes with regard to the duration of the load. These values are applied to the loads when analyzing a structural member. As these beams have been subjected to consistent load since 1852, this issue could be magnified by other issues such as moisture. Evidence of excessive load was identified through stress fractures observed in certain structural members.

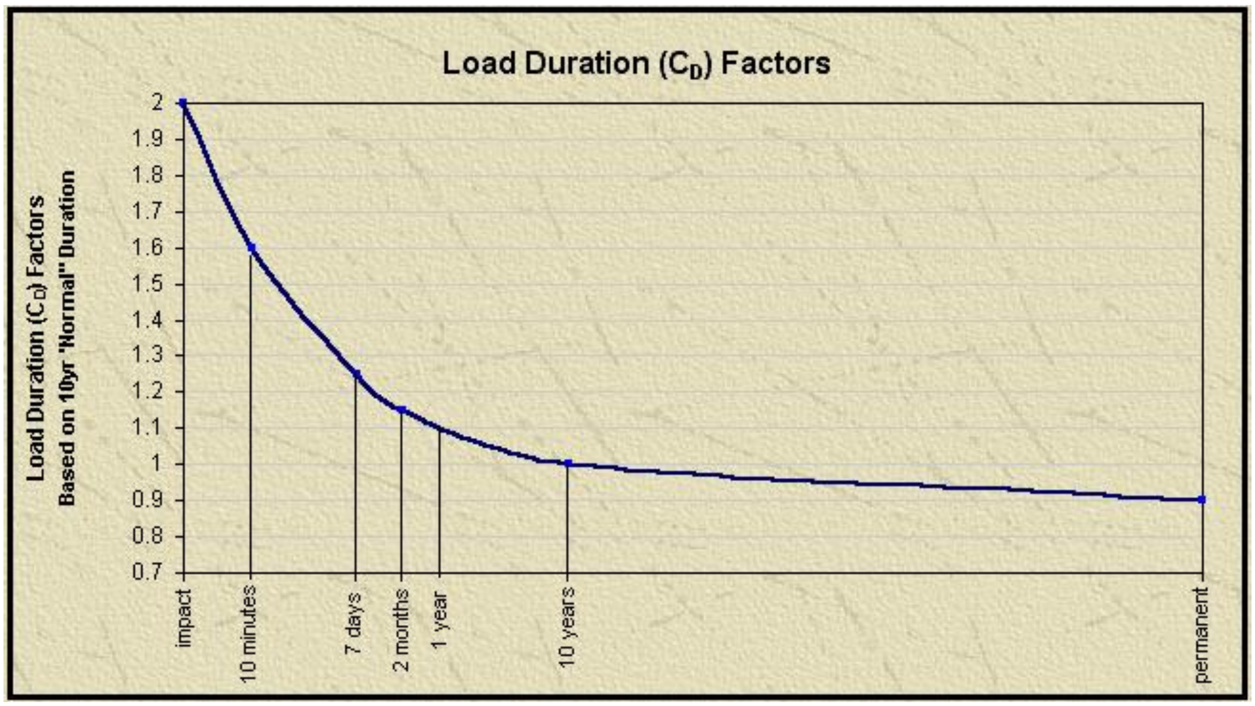

Figure 3: Load Duration Factor with respect to Duration of Applied Load (WSU, 2000)

Apart from these inspection techniques, a structural analysis was performed on the building frame and foundation.

2.2.2.1 Structural Analysis

The point of a structural analysis is to identify any structural members that cannot resist the required loads. Every structure has a load path that transfers forces from the source of the load to the earth. For buildings like Mechanics Hall, the load is applied to the floor, where it is distributed to the joists. The joists are supported by carrier beams, which are in turn supported by columns or a foundation wall. The columns and foundation walls are supported by footings, which spread the load evenly across a wider area to reduce settlement of the structure.

Different structural members were analyzed for different failure conditions. The horizontal floor deck, joists, and carrier beams were checked on their resistance to bending stress, shear stress, and deflection. The vertical columns were analyzed in axial compression. The foundation walls were designed to resist the required compressive loads created by the building as well as the lateral loads caused by the soil. The footings were dimensioned according to the strength of the soil, and analyzed in bending and in shear.

2.2.2.2 Method of Composite Sections

After the structural analysis of Mechanics Hall, it was determined that a set of beams were unable to support the required loads (see Section 5.2.1). Due to the construction of the building, it was impractical to remove and replace the existing beams, as the joists connected flush with the side of the beam. Therefore, a technique called the Method of Composite Sections was used to design reinforcing for the existing beam.

In the Method of Composite Sections, two different materials are connected to work together as one beam. In this case, a steel plate was bolted to the bottom of the wooden beam. This is called a Composite Beam. This beam was analyzed using a ratio between each material's Elastic Modulus. The geometry of the steel section was magnified horizontally until its properties represented and equivalent section of wood. The resulting theoretical shape was an upside-down T shape with very wide flanges. A new centroid of the shape and a new Moment of Inertia was determined using established mathematical techniques (see Section 3.6.3.2). Using the new section area, the beam was analyzed in bending, shear, and deflection.

For a composite section to work, the bottom plate must be adequately secured to the existing member to transfer the stresses. The bolts transfer the load as shear between the two surfaces. Therefore, the bolt spacing must be based on the shear flow between the wooden beam and the steel plate.

2.2.2.3 Reinforcing Foundation Wall

The fieldstone foundation wall in Mechanics Hall had to be reinforced to provide adequate wall thickness to resist the soil loads specified in IBC 1610.1 and the loads caused by the structure itself. A reinforced concrete wall was designed using welded wire reinforcement. A typical reinforced concrete wall contains rebar of various size and spacing. To reduce the depth of the wall and for ease of installation, welded wire reinforcement was used. This type of reinforcement can be used in walls and slabs so long as the minimum required area of steel is still met. In the case of this wall, the minimum area of steel was determined based on the shear force applied by the lateral soil loads. The steel must also be covered with enough concrete to prevent water and other materials from infiltrating the porous concrete and reacting with the steel reinforcement to cause rust. To prevent water from seeping into the basement, every gap in the existing foundation wall was filled with concrete.

2.2.3: Fire Safety

A major consideration when opening a building for public use is fire safety and egress. As specified in Chapter 32 of the Massachusetts Building Code, existing buildings designed to hold 50 or more people must conform to most current fire safety and egress standards except for those, which are impractical, and those, which would remove the building from the National Registry of Historic Places (IBC, 2012). Important criteria to look for when considering egress are hallway dimensions, door dimensions, and door locations. The floor plan is also important to consider, as are aspects such as stair steepness and railing height.

When looking into a sprinkler design there are multiple ways to approach the problem. Two options are either a prescriptive or performance approach. In a prescriptive design the applicable code is used in order to obtain results. In a performance based design engineering analysis is used to solve the problem. For most sprinkler designs a prescriptive approach can be used. Therefore the general approach is outlined in a step-by-step procedure as follows. One of the first objectives is determining whether or not the building requires sprinklers. Massachusetts General Law Chapter 148 Section 26G states that sprinklers are required in an area that is over 7500 square feet. However, this cannot solely give a strong conclusion on whether sprinklers are necessary. The building's occupancy (use) might lead to hazardous condition that requires a sprinkler system regardless of size. The correct code needs to be consulted in order to determine the occupancy, and whether sprinklers are needed. In the case of Mechanics Hall the *International Existing Building Code* (IEBC) was used along with the *International Building Code* (IBC). Chapter 3 of the IBC presents occupancy information that can be used to determine what occupancy a building falls under. For example, if a group A assembly occupancy is being considered then added safety measures may have to be put into effect because of the associated volume of people.

Once the occupancy is found, the correct building code requirements can be referenced to determine whether or not sprinklers are necessary. Because the design does not focus on a new building, the IEBC was referenced, and the chapter on Historic Buildings was first consulted. When using the IEBC the level of alteration needs to be found. Chapter 5 of the IEBC describes the different levels based on how much work is going to be done to the building. It then can be found whether or not a sprinkler system is needed. Because this project deals with a renovation, the risk of changing the occupancy also has to be taken into consideration. Chapter 10 of the IEBC deals with this, and in some cases will refer readers to the IBC if the occupancy hazard is changed. In that situation the requirement for Automatic Sprinklers can be found in Chapter 9 of the IBC.

If the IEBC or IBC (if occupancy is changed) ultimately requires that sprinklers be installed then the next step is consulting NFPA 13. NFPA 13 is the standard on the design and installation of automatic sprinkler systems. The first think that needs to be determined is the buildings hazard. The hazard, not to be mistaken with the occupancy from IBC (described earlier), will determine how many sprinklers and what amount of water is required. NFPA 13 5.2 explains the different types of hazards (I-IV), and displays in charts what certain building uses fall under. Once the hazard is determined the type of sprinkler system, design area, and density need to be found.

Depending on the buildings use there are multiple types of sprinkler systems that can be used. These types of systems are described in NFPA chapter 6. If the building is serving a heated area then the most common type is a wet sprinkler system. Another example is a dry pipe sprinkler system, which is used when the area is not heated and there is fear of water freezing inside of the pipes. The design area determines the size area (square feet) that the sprinkler system must be able to protect. Lastly, the density (gallons per minute per square foot) determines the amount of water that is required per square foot of the design area. Using NFPA 13 Figure 11.2.3.1.1 the design area and density of a class I-IV building can be determined. Depending on certain aspects NFPA13 allows different reduction factors for the design area. For example if the ceiling height is not very high a formula can be applied using NFPA 13 Figure 11.2.3.2.3.1 that allows the design area to be reduced by a calculated percentage. In order to use this the building needs to have a wet sprinkler system, be light or ordinary hazard, and have a maximum ceiling height of 20 feet.

Next, the water demand needs to be determined. In order to get this we need to simply multiply the density times the area. NFPA 13 requires that light hazard occupancies must add an extra 100 gallons per minute of inside hose allowance. This all must be supplied for a minimum of 30 minutes per NFPA13 Table 11.2.3.1.2.

Equation 1

$$Density\left(\frac{gpm}{ft^2}\right) \times Area\ (ft^2) = Flow\ (gpm)$$

$$(Flow(gpm) + Hose\ Demand(gpm)) \times Duration(min) = Volume\ (gallons)$$

The supply must meet the demand, whether it is public water or a private well. Therefore if the demand is not met then a pump will have to be installed.

Next the system has to be laid out using design software. AutoCAD was used to lay out the sprinkler system for Mechanics Hall. The head spacing requirements, found in NFPA Table 8.6.2.2.1(a), need to be determined in order to lay out a design. Using these requirements sprinkler head spacing can be done making sure to stay under the maximum spacing. Once the heads are mapped out sprinkler line and mains can connect the heads. In most scenarios black steel pipe will be used because it is the lowest cost to install and easiest to maintain. When laying out the piping arrangement, the mains need to be run perpendicular to the trusses so there is something to hang them on. Mains are what supplies water, and lines are what run from the mains to the sprinklers.

Lastly a pump needs to be considered. A pump could be necessary if the public water supply does not create enough pressure in order to supply the system. In order to figure out the size of the pump a hydraulic calculation needs to be calculated to find the pipe sizes. This will show how much pressure and flow is required in order for water to sufficiently supply the sprinkler system. Another method to determining pipe sizes is by using the prescriptive tables presented in NFPA13. These allow for a very quick estimate of pipe sizes, but they are not as accurate or cost effective as a hydraulic approach. If a pump is supplied a pump house that will be outside of the building will also have to be built. A pump house is a heated enclosure that will protect the pump, and allow for easy maintenance.

Structural fire protection also needs to be considered. This consists of the fire resistance rating of different structural members. In order to determine what the fire rating will be the construction type needs to be found. Construction type deals with the different types of materials that the buildings is made of. The reason that fire resistance deals with this is because depending on the material used a general assumption can be made on how long something will take to burn. Based upon the maximum area and height the construction type can be determined from table 503 in the IBC. The construction type can then be used with table 601 in the IBC to determine the different fire resistance requirements for specific structural elements. It is also important to note than for some construction types there may be allowable reductions if a sprinkler system is present. Section 704.13 also allows painting on a fire resistant material in order to meet fire resistance requirements. There are also other structural fire considerations for buildings depending on how high risk the scenarios are. For, example if there is a sleeping area next to a high-risk situation a firewall may be required.

This basically lets the designer consider the two spaces as two separate buildings in regards to fire protection.

2.2.4: Accessibility

When investigation both the IBC and the IEBC existing building accessibility requirements are not as stringent as new buildings. The IEBC doesn't specifically say any requirements are needed for renovated building, it states that the building has to be made as compliant as possible. When looking into a historical building the requirements are very similar for a regular existing building. The reason for this could be that many of the accessibility features are not possible in an existing building. For example, it would not be very easy to put a wheel chair lift into mechanics hall to get to the second floor. In the end it all comes down to the discretion of the code official. If he/she says that there was a conscious effort to make the building accessible then it will be considered compliant.

2.2.5 Parking

Adequate parking is critical to the accessibility of a public building. The number of required parking spaces is dependent on the intended use, the occupancy of the structure, and the amount of land available to be developed into a parking area. The parking requirements are usually determined using local zoning ordinances, as the IBC only regulates the amount of required accessible parking spaces based on the total number of spaces.

2.2.6: Rising Damp

From our site visits we saw the northwest side of the building is where the worst conditions are located; due to the beams being heavily saturated and covered in mold. At first we thought that this might have been a result of "rising damp", but after some research we found that it was not a case of rising damp. Rising damp is a major cause of decay to masonry materials such as stone, brick and mortar. It may also cause musty smells in poorly ventilated rooms. Rising damp occurs as a result of capillary suction of moisture from the ground into porous masonry building materials. The moisture evaporates from either face of the wall (inside or outside), allowing more to be drawn from below. The height to which the moisture will rise is determined by the evaporation rate and the nature of the wall. Rising damp may show as a high-tide-like stain on wallpaper and other interior finishes, and, when more severe, as blistering of paint and loss of plaster (Wise, 2013).

2.2.7: Cost Estimate

Part of preparing a design solution to a problem is acknowledging the cost of the project. Until the project is complete, it is impossible to know the actual price. The costs can be estimated once the design is completed based on the amount of materials required and the type of work being performed. As part of the design, an itemized list is prepared with the every material required for the job, the amount required, and the unit price. This allows for a relatively accurate cost estimate

2.2.8: Renovation Plan

Through the use of CAD models it is important to show the client the potential of what could be expected after a renovation has been completed on the building.

2.2.9: Code Review

In the case of Mechanics Hall a code review was a necessity. Most of the literature that was reviewed for this project was different forms of building codes. One way to keep track of all of the different referenced codes is through a code review. For this case the code review looks at every referenced code and states the implications of that code. This also aids in give a further definition of why each code was referenced and what it means. Using the code review the actual section in each building code can be referenced as well. One of the main reasons for a code review is to aid in organization.

3: Methodology

This section is to describe the work that was completed to meet the Scope of Work as discussed in Section 3.

3.1 Scope

The purpose of this project was to propose a plan to renovate Mechanics Hall to meet modern building code while maintaining the historical significance of the structure. For this to be practical, the building had to have a purpose. Two different potential uses for the structure based on the needs of the community and cost were developed. The benefits and drawbacks of each purpose were also compared.

Once two purposes for Mechanics Hall were identified, the building was analyzed in its current condition. The focus was on concerns regarding structural integrity, egress, fire safety, drainage, handicapped access, and parking. The causes of the current issues with the building were identified and preventative measures for the future structure were considered. Plumbing and electrical work were also considered, but was not the focus of our investigation.

Results obtained during analysis of the structure will be used to identify the major issues obstructing current use. Using the current issues and potential issues a set of solutions to these problems was developed. These solutions were intended to resolve deficiencies in the areas mentioned in the above paragraph, as well as attempt to address any concerns that may result in future damage to the building. These ideas, as well as computer models of the entire building, were combined into a Restoration Plan. This Restoration Plan included the results of our analysis, calculations, computer models, a cost estimate, and an estimate of potential revenue for the two purposes developed for the building.

3.2 Initial Meeting and Proposed Uses

Before beginning the engineering work associated with the project, it was important to meet the Friends of Mechanics Hall and to visit the building. The purpose of the initial meeting was to get a sense of both the nature of the building and the vision of the Friends of Mechanics Hall. It was important to understand the desires of the Friends of Mechanics Hall to focus the engineering work to their specific needs.

After the initial meeting two uses for the building were proposed. Two uses were chosen because they give the Friends of Mechanics Hall a degree of variability instead of locking them into one option. By comparing each proposed design scheme it allowed for explanation on the strengths and weaknesses of certain designs. The proposed uses catered to what was discussed with the Friends of Mechanics Hall at the initial meeting.

3.3 Building Inspection

Subsequent site visits were taken to perform an extensive inspection of Mechanics Hall. The purpose of the inspection was to gather measurements to accurately analyze the building for Fire Protection and Structural safety concerns, as well as to create CAD models of the structure. Measurements were taken of ceiling height, room dimensions, exterior dimensions, window locations, doorway height and width, staircase steepness and width, and the location of structural members such as columns. Beam spacing and dimensions were observed and measured in the basement. The condition of the building was assessed visually and, in some cases, by prodding the material to test the durability, water damage, and rotting. The measurements were compiled in AutoCAD drawings and Revit models, and used for the analysis of existing conditions and the investigation of two proposed uses.

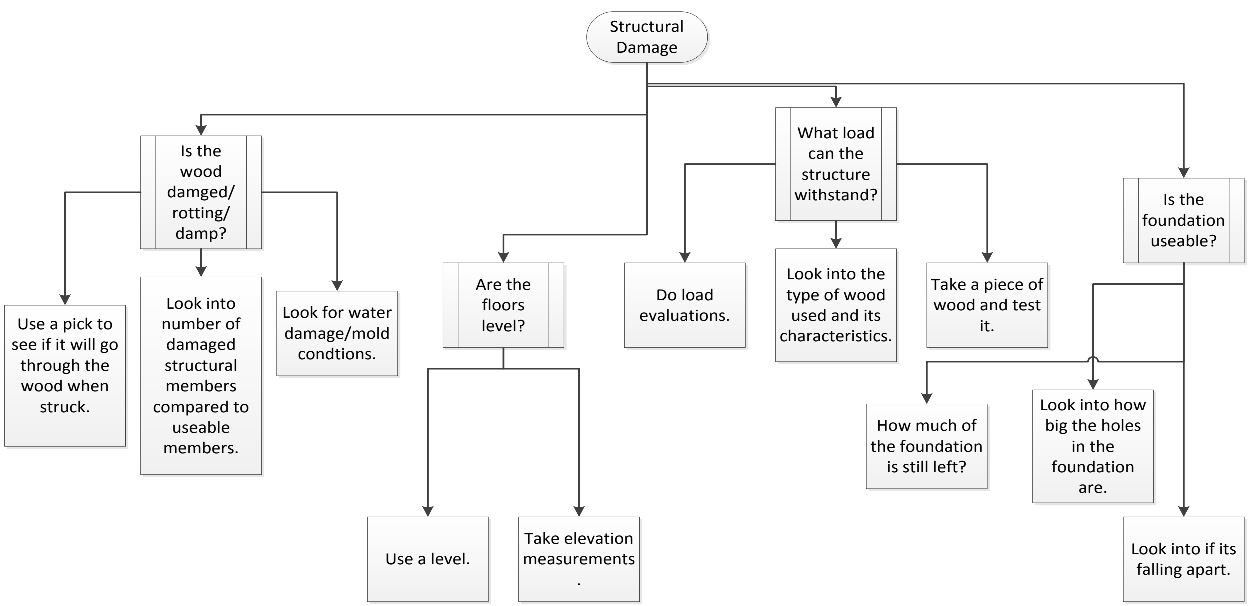

Figure 4: Structural Damage Assessment Procedure

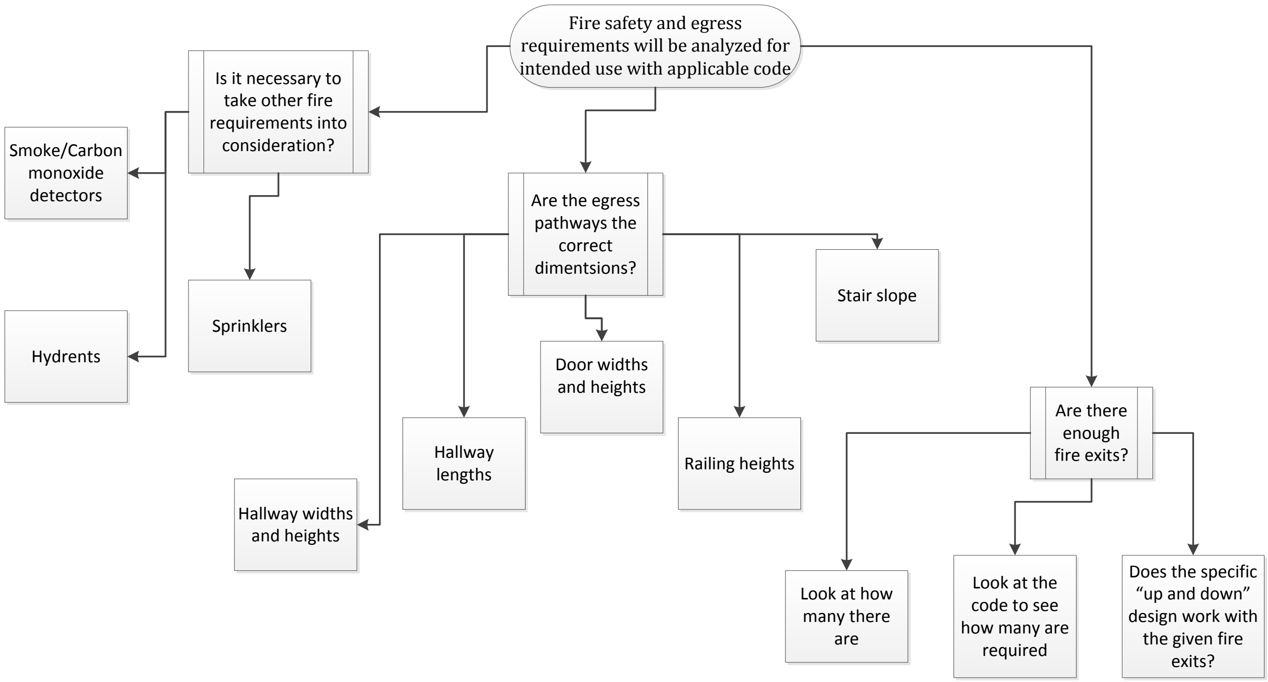

Figure 5: Fire Safety and Egress Assessment Procedure

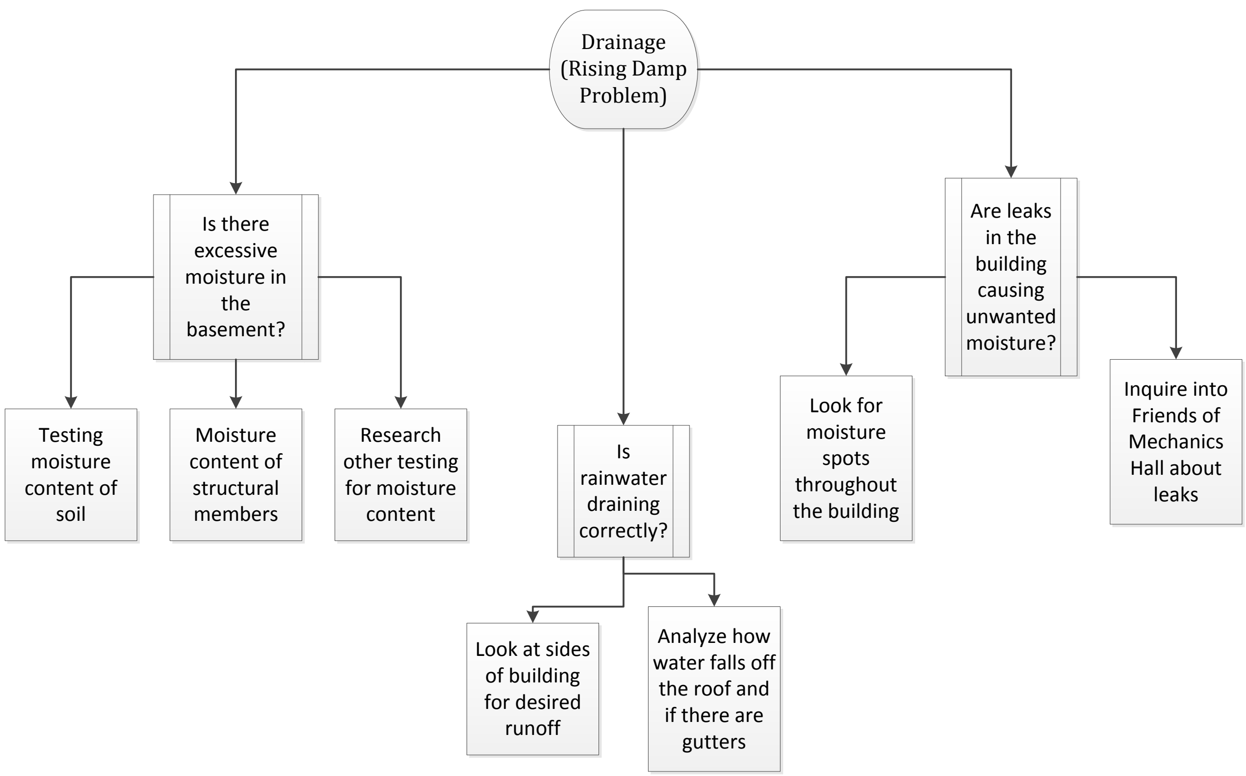

Figure 6: Drainage Assessment Procedure

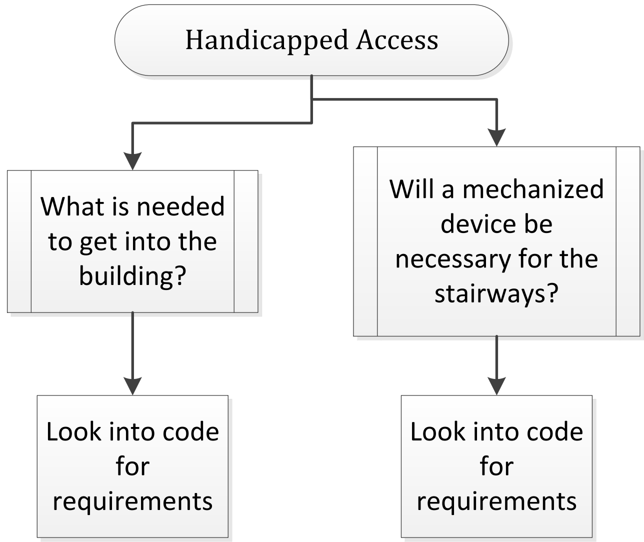

Figure 7: Handicapped Access Assessment Procedure

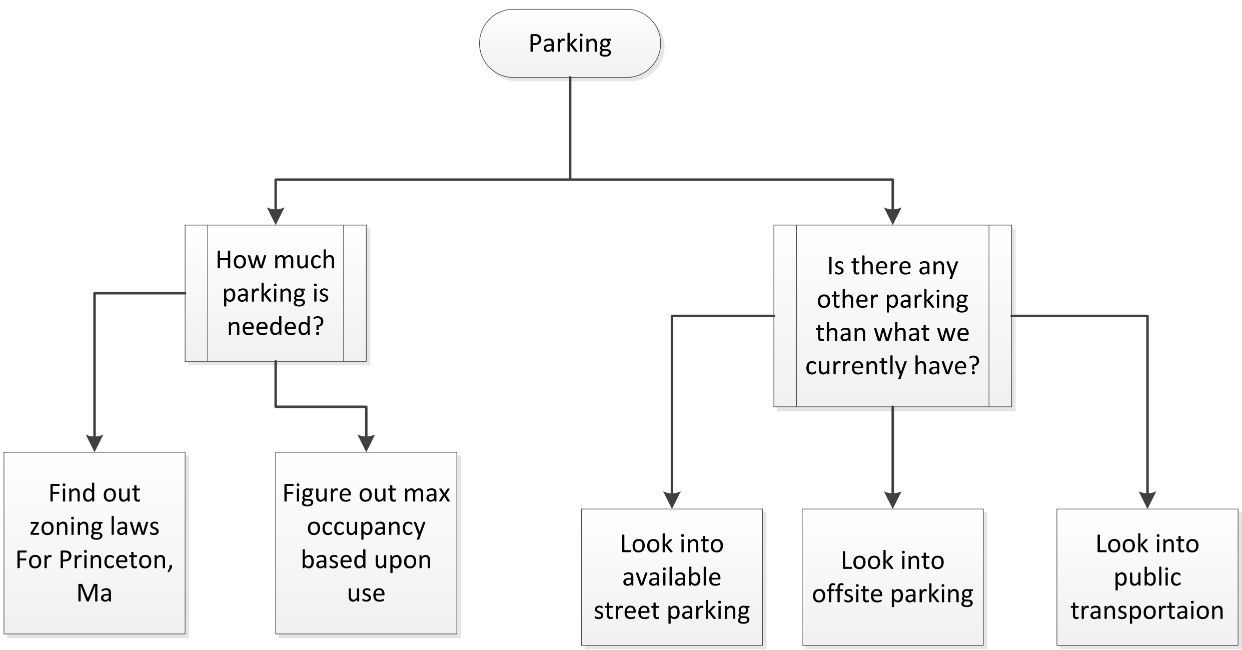

Figure 8: Parking Assessment Procedure

3.4 Code Review

In order to undertake a code review the most important element is organization. In order to create a successful review every code referenced throughout the course of the project needs to be put into the review. Because there are many pages of building codes it is necessary to document every code that was used. When the project comes to and end it is then easy to make a table and states the implications of each code. For the case of Mechanics hall this process was followed. Our framework for the code review was more of an abbreviated type of code review. A traditional code review investigates every code in a section that was looked at, and states its implication upon a project. However, for this project only the code specified within the paper was put into the code review. The point was to hopefully provide a further reference for the reader, and make the paper easy to understand.

3.5 Fire Protection

A handful of aspects make up fire protection in a building. Generally they can be summed up as Automatic Sprinkler System, Egress Requirements, Smoke and Carbon Monoxide Alarms, Water Supply, and Structural Fire Protection. This can be seen in Figure 5, which describes the procedure followed in order to investigate and solve each problem outlined below. Figure 5 includes an "other" section because originally it was not known what aspects of Fire Safety were going to be applicable to Mechanics Hall. In order to be thorough all aspects presented in the Figure 5 were taken into consideration. Because Mechanics Hall is on the historic registrar Chapter 12 of the *International Existing Building Code* (Historic Buildings) was looked into first when referencing codes. However, even if historical exceptions were met the building was then analyzed against the rest of the Existing Building Code so that its safety would be comparable to a modern building.

3.5.1 Water Supply

Water is the most important aspect when dealing with fire protection in a building. This is simply because in order to fight a fire, water is required. In order to figure out the town's water supply, Pat Schmohl, an ex-Princeton Firefighter, was contacted. He served as the Chief of the Town of Princeton's Fire department for a period of time, and is very knowledgeable on how they handle this type of situation. Through him and with the help of the Friends of Mechanics Hall, the source and quality of the water supply was determined.

3.5.2 Automatic Sprinkler System

Mechanics Hall presents an extremely difficult situation when looking at a sprinkler system. During site visits to Mechanics Hall it was easy to see that the building did not have an automatic sprinkler system. Because a sprinkler system may necessary, the background addressed the general procedure involved in designing a prescriptive sprinkler system. For Mechanics Hall a prescriptive approach was used.

Before the design could be done the building occupancy needed to be determined. The Friends of Mechanics Hall were not able to give a definitive answer in regards to the existing occupancy. Therefore, a class B business occupancy was assumed. This was assumed because class B corresponded to how the building was currently being used, and it represents a middle ground in regards to amount of protection.

One of the most difficult parts of the sprinkler section was determining whether the building actually needed sprinklers or not. By simultaneously using the IEBC, IBC, and NFPA13 a determination can be made on whether sprinklers are necessary. One important consideration was on whether the buildings use was going to change. If this happens then the building has to be able to satisfy the requirements for the new occupancy classification. Based upon this a sprinkler system was designed following the procedure outlined in the background. In general this method can be used for most sprinkler designs. One difference is the design for mechanics hall does not include a hydraulic calculation. Instead the prescriptive method per NFPA13 was used to determine pipe sizes. Although it is faster it may produce a more conservative cost estimate.

After the sprinkler design was completed a cost estimate was prepared using pricing software courtesy of Cogswell Sprinkler Co., Inc. The software gave a rough estimate of the price associated with installing the components needed for Mechanics Hall. This cost estimate will be combined with the other systems analyzed to produce a total cost for the Friends of Mechanics Hall. The full design, and cost estimate can be seen in the results section of the report.

3.5.3 Egress Requirements

Egress encompasses the necessary requirements for the occupants to safely exit the building in case of an emergency. When looking into Egress the first step is to establish a plan of the building that has the correct dimensions. Using a laser for accuracy and efficiency, floor plan measurements were taken and transferred into a drawing using design software. With respect to Mechanics Hall all the dimensions of the doors, hallways, stairways, and egress routes were compared to the requirements in the *IEBC* in order to see if there are issues with any of the elements in the building. The Historical Buildings chapter was reviewed first and then the rest of the code provisions were addressed using the same level of alteration determined in the sprinkler selection. In cases where occupancy was changed Table 1012.4 in the *IEBC* was consulted to see if Chapter 10 of the *IBC* was applicable.

3.5.4 Detectors

Using the *IEBC* it can be determined if either smoke or carbon monoxide detectors are necessary for a building depending on the historical requirements, the level of alteration the building falls under (found the same way as in egress and the sprinkler sections), and change of occupancy. The codes also refer the reader to NFPA 72, which is the current installation code for smoke detectors. If detectors are needed NFPA 72 will need to be consulted before installation.

3.5.5 Structural Fire Protection

In order to determine the structural fire requirements for a building some investigation was done at the site. The building material and construction were examined in order to verify the construction type. Because of the scenario presented the *IEBC* was looked into first in order to determine any historical exemptions. Using the alteration level described the *IEBC* was also used in determining which requirements were applicable for renovation work. This had to be simultaneously used with the *IBC* as described in the background. Because Mechanics Hall represents a low risk scenario there was not any need for advanced structural fire protection systems such as firewalls. Upon determining the different fire resistance ratings the sprinkler design was considered, and any exemptions were accounted for

3.6 Structural Analysis

A structural analysis was performed on the building frame and the foundation in order to determine if the structure will safely accommodate the required design loads. Figure 4 outlines the procedure used for the inspection and analysis of the structural systems. The only provisions regarding Historic Structures included in the *IBC for Existing Buildings* were found in Section 1206.2, which specifies that any scenarios considered "dangerous" must be fixed. Therefore, IBC 1607.1 was used to determine the required design live loads based on the occupancy of the structure. The structural analysis and design process for Mechanics Hall was completed using the Allowable Stress Design (ASD) method. In accordance with IBC 2306.1, all analysis and design work was completed using the *National Design Specifications* (NDS), produced by the American Forest and Paper Association and the American Wood Council.

To perform the structural analysis, Mechanics Hall was divided into three sections. The first section was the floor system of the main structure. The second section was the floor system for the rear addition, as the frame is very different from the original building. The last section consists of the foundation. The specie of wood used in Mechanics Hall was assumed to be Northern Red Oak based on discussions with the Friends of Mechanics Hall (A. Fiandaca, personal communication, 18 December 2013). The dimensions and layout of the structure were observed through the basement as specified in Section 3.2. During the initial analysis, all structural members were assumed to be in good condition. By not considering the condition of the structural members until after the analysis, it can be determined if the inadequate members should be replaced, or if a new frame design is required. Based on the floor layout, it was assumed that none of the interior walls are load bearing walls; all of the gravity loads are distributed to the exterior walls or the interior columns.

3.6.1 Loading

The loading on an individual structural member was approximated based on the results of the measurements taken during the building inspections and engineering judgment. The area of the floor that each structural member is responsible for is called the Tributary Area. For each beam, the Tributary Area is defined as the length of the beam multiplied by half the distance to each adjacent beam on either side, or the Tributary Width. All floor members were considered to support only gravity loads. An example of the process of determining the loading applied to a carrier beam can be seen in Appendix C. An extra dead load of 30 pounds per square foot (psf) was applied to account for interior walls, finished floor surface, and other non-structural yet permanent aspects of a building. For this analysis, a dead load is considered to be a permanent load created the self-weight of the building, including structural elements, walls, and utilities. A live load is any load that results from the occupancy or use of the building, such as furniture and people.

The self-weight of a structural member is determined using the unit weight of the material multiplied by the area of the member's cross section, as shown in Equation 2.

Equation 2

$$SW = A\mu$$

Where:

SW = Self Weight of the Structural Member (lbs/ft)

A = Area of the Cross Section of the given beam (ft^2)

μ = Unit Weight of Material of the given beam (lb/ft^3)

The superimposed dead load of the floor acting upon a particular structural member was determined using the tributary area (A_t) of the given structural member, and was calculated using Equation 3. When Equation 3 was applied to a joist or beam, the resulting superimposed dead load was divided by the span of the member, returning a load in pounds per linear foot. When Equation 3 was applied to a column, the resulting load was left as is in units of pounds.

Equation 3

$$DL_f = t_f \mu A_t$$

Where:

DL_f = Superimposed Dead Load of the Floor (lbs)

T_f = Floor Thickness (ft)

μ = Unit Weight of the Floor (lb/ft^3)

A_t = Tributary Area of the Given Beam or Column (ft^2)

The dead load exerted by a joist on a beam was determined by the Tributary Width (w_t) of the beam. Equation 4 was used to determine the superimposed dead load effect for a beam that supports one or more joists.

Equation 4

$$DL = \frac{nA\mu w_t}{l}$$

Where:

DL = Superimposed Dead Load of Structural Members (lb/ft)

n = Number of Beams Supported by the Given Beam

w_t = Tributary Width (ft)

A = Area of the Cross Section of the supported member (ft^2)

μ = Unit Weight of Material of the supported member (lb/ft^3)

l = Span of Given Beam (ft)

Equation 5 was used to determine the dead load effect of joists and beams acting on the columns.

Equation 5

$$DL = nA\mu w_t$$

For Equation 5, DL is the Dead Load of Structural Members, measured in pounds. In the case of Mechanics Hall, the columns support different beams that span in perpendicular directions. In each case, the appropriate Tributary Width was measured in the same direction as the beam. This ensured that the only structural members that are included are located within the Tributary Area of the column. This equation was applied twice on each column, once to account for the weight of the joists, and once to account for the weight of the beams. In addition, the floor load from Equation 3 was applied.

As noted earlier, an additional dead load of 30psf was added to every structural member. The effect of this superimposed dead load on a beam is equal to the load multiplied by the Tributary Width. The effect of this load on a column is equal to the load multiplied by the Tributary Area. The live load was determined in the same manner as this additional dead load.

3.6.2 Allowable Strengths and Values

The first step in the structural analysis was to determine the allowable capacity of each member using Allowable Strength Design. This was done by consulting the *NDS*. The allowable stress for the floor deck and joists were determined using Table 4A in the *NDS Supplement*, while the capacities for the carrier beams and the supporting columns were found in Table 4D in the *NDS Supplement*. The values found in these tables were then modified using the correction factors found in *NDS* Table 4.3 to reflect the environment in which the wood is located. Table 1 organizes the various adjustment factors to display which factors are applicable to each type of potential stress and the modulus of elasticity, and can be found in the *NDS* as Table 4.3.1. The final result is the factored allowable stress. Adjustment factors are also required to determine the appropriate Elastic Modulus.

Table 1: Adjustment Factors for wood design (Taken from *NDS* Table 4.3)

		ASD only	ASD and LRFD										LRFD only		
		Load duration factor	Wet service factor	Temperature factor	Beam stability factor	Size factor	Flat use factor	Incising factor	Repetitive member factor	Column stability factor	Buckling stiffness factor	Bearing area factor	Format conversion factor	Resistance factor	Time effect factor
$F_b' = F_b$	x	C_D	C_M	C_t	C_L	C_F	C_{fu}	C_i	C_r	–	–	–	K_F	ϕ_b	λ
$F_t' = F_t$	x	C_D	C_M	C_t	–	C_F	–	C_i	–	–	–	–	K_F	ϕ_t	λ
$F_v' = F_v$	x	C_D	C_M	C_t	–	–	–	C_i	–	–	–	–	K_F	ϕ_v	λ
$F_{c\perp}' = F_{c\perp}$	x	–	C_M	C_t	–	–	–	C_i	–	–	–	C_b	K_F	ϕ_c	λ
$F_c' = F_c$	x	C_D	C_M	C_t	–	C_F	–	C_i	–	C_P	–	–	K_F	ϕ_c	λ
$E' = E$	x	–	C_M	C_t	–	–	–	C_i	–	–	–	–	–	–	–
$E_{min}' = E_{min}$	x	–	C_M	C_t	–	–	–	C_i	–	–	C_T	–	K_F	ϕ_s	–

All information regarding adjustment factors in this paragraph was found in the *NDS* unless specified. The load duration factor was taken in accordance with Section 2.3.2.2 and Table 2.3.2. The temperature factor was determined by Table 2.3.3. The wet service factor was determined using

Tables 4A, 4B, 4C, 4D, 4E, and 4F of the *NDS Supplement* as described in section 3.3.3 of this report. The beam stability factor was determined using the criteria identified in Section 3.3.3 of the *NDS*. The size factor was determined in accordance with Table 4A of the *NDS Supplement*. The flat use factor was determined using Tables 4A, 4B, 4C, and 4F of the *NDS Supplement*. The incising factor was selected in accordance with Section 4.3.8, and the repetitive member factor was determined using Section 4.3.9. The buckling stiffness factor was selected in accordance with Section 4.4.2. And the column stability factor was determined using Section 3.7 and Equation 6.

Equation 6

$$C_p = \frac{\frac{1+F_{CE}}{F'_c}}{2c} - \sqrt{\left(\frac{\frac{1+F_{CE}}{F'_c}}{2c}\right)^2 - \frac{\frac{F_{CE}}{F'_c}}{c}}$$

Where:

C_p = Column Stability Factor

F_{cE} = $(0.822 \cdot E'_{min})/(l_e/d)^2$

F'_c = Adjusted Reference Design Value F_c multiplied by all applicable factors except C_p

C = 0.8 for sawn lumber

The allowed deflection limits were determined differently than the strengths of the material. As deflection limits are based on the use of the structural member and not solely on the material used, the limits can be found in IBC Table 1604.3. The appropriate deflection limit in a floor system is equal to l/360, where l is the length of the beam's span in inches.

3.6.3 Original Building Floor System

The floor system for the original building consists of floor planks, or decking, lying on top of crossbeams, or joists. These joists then connect to larger carrier beams, which are supported by the foundation walls and four columns. This plan is illustrated in Figure 9. The horizontal members were analyzed for the three failure modes of bending, shear, and deflection, while the columns were checked for crushing due to axial compression. The columns are labeled numbers 1 through 4, starting at the back left and working towards the front right.

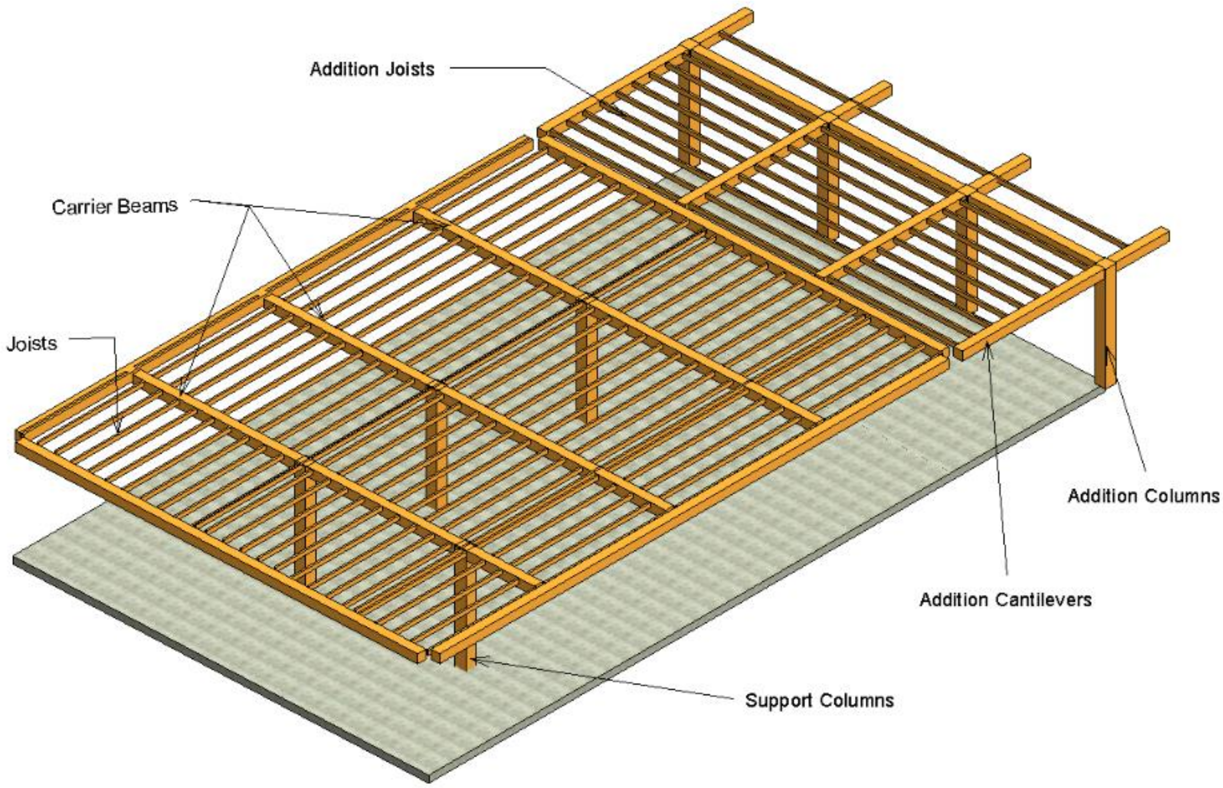

Figure 9: Mechanics Hall Framing Plan

3.6.3.1 Analysis of the Original Building Floor System

The goal of the analysis was to determine whether the existing frame is capable of holding the required live load as specified in IBC Table 1697.1. To do this, all dead loads were calculated for each structural member using the method outlined in Section 3.4.1. The floor deck, joists, and carrier beams were considered simply supported. Equation 7 was used to determine the bending moment created by a uniform load on a simply supported beam.

Equation 7

$$M = \frac{wl^2}{8}$$

Where:

w = dead load (lbs/ft)

l = span length (ft)

M = bending moment due to the dead load (ft*lbs)

Once the bending moment caused by the dead loads was determined for each horizontal member, the corresponding bending stress was calculated using Equation 8:

Equation 8

$$f_b = \frac{Mc}{I}$$

Where:

f_b = Bending Stress Due to Dead Loads (psi)

M = Bending Moment Due to Dead Loads (in*lbs)

c = Distance between the Neutral Axis and the Extreme Fiber (in)

I = Moment of Inertia (in^4)

Upon finding this bending stress, it became possible to work backwards from the member's allowable bending stress to determine how much live load each member could safely hold. This was done by subtracting the bending stress caused by the dead load from the allowable stress to find the allowable stress due to live loads. By reorganizing the above equations, one can solve for the bending moment caused by live loads, and finally the live loads themselves. This process is outlined in Appendix C.

A similar process was used to check each horizontal member for shear resistance. Equation 9 was used to determine the shear force acting on a simply supported beam due to a uniform load.

Equation 9

$$V = \frac{wl}{2}$$

Where:

V = Shear Force due to Dead Load (lbs)

After the shear force was determined, the corresponding shear stress was calculated using Equation 10. Because all of the beams in the structural frame were symmetrical, the value of Q for the cross section is the same above and below the neutral axis.

Equation 10

$$f_v = \frac{VQ}{Ib}$$

Where:

f_v = Shear Stress due to Dead Load (psi)

Q = The First Moment of Area (in^3)

b = Beam Thickness (in)

Once the shear stress due to the dead load was defined, the allowable live load was determined using a similar method described for bending moment, which is outlined in Appendix C. The next check was the check for beam deflection. According to IBC Table 1604.3, the allowed deflection limit for floor members is equal to l/240. The equation used to determine the amount of deflection a load causes on a simply supported uniformly loaded beam is Equation 11.

Equation 11

$$\Delta = \frac{5wl^4}{384EI}$$

Where:

Δ = Total Deflection (in)

E = Modulus of Elasticity (psi)

After the deflection caused by the dead loads was determined, the permissible live load was calculated using an approach similar to that described above for bending stress, as seen in Appendix C.

The four columns located in the basement were not analyzed in bending, shear, or deflection, as these are not failure criteria for columns. The columns were analyzed only in axial compression; it was determined that no bending stresses were acting on the columns based on the field inspections of the connections between the carrier beams and the columns. The strength of the columns was defined as the compression strength parallel to the grain modified by all applicable adjustment factors. Equation 12 was used to determine the axial compressive stress acting on the columns.

Equation 12

$$f_c = \frac{P}{A}$$

Where:

f_c = Compressive Stress Parallel to the Grain (psi)

P = Load (lbs)

A = Cross Sectional Area (in^2)

Once the compressive stress was found acting upon the column, the same method of working backwards discussed in detail regarding the bending stress was used to find the allowable live load.

3.6.3.2 Design Solutions for the Original Building Floor System

The results of the analysis highlighted under-capacity structural members in the frame of the initial building. Once these members were identified, solutions were developed to increase their capacity and return the structure to compliance with the requirements specified in *IBC*.

The carrier beams in the original building were determined to have insufficient strength. Due to the method of building construction, replacing the existing beams would involve taking apart every joist connection to the carrier beams. To save on labor, the proposed solution was to reinforce the beams using steel plates, which were designed using the Method of Composite Sections. This method considers a new cross section of the beam based on the Elastic Modulus of each material. First, the Modular Ratio of the two materials was calculated using Equation 13.

Equation 13

$$n = \frac{E_1}{E_2}$$

Where:

n = Modular Ratio

E_1 = Elastic Modulus of Steel (psi)

E_2 = Elastic Modulus of Red Oak (psi)

Using the Modular Ratio, the dimensions of the steel plate were adjusted to account for the difference in Elastic Modulus. The depth of the steel plate remained the same, while the width of the steel was adjusted by multiplying the original width by the Modular Ratio. This converted the steel plate into an equivalent section of Red Oak. The neutral axis of the new cross section was found using Equation 14. To use Equation 14, the new cross section of the beam was divided into 2 rectangles, one horizontal and one vertical.

Equation 14

$$\bar{y} = \frac{\sum \bar{y}_i A_i}{\sum A}$$

Where:

$\bar{y}$ = Distance (from the bottom of the cross section) to the new neutral axis (in)

$\bar{y}_i$ = Distance (from the bottom of the section) to the neutral axis of Section i. (in)

A_i + Area of Section i. (in^2)

The adjusted moment of inertia for the new cross section was determined using the Parallel Axis Theorem, as displayed in Equation 15.

Equation 15

$$I_x = \sum I_i + A_i d_i^2$$

Where:

I_x = Moment of Inertia of the cross section (in^4)

I_i = Moment of Inertia of Section I (in^4)

A_i = Area of Section I (in^2)

d_i = Distance from the neutral axis of Section i to the neutral axis of the cross section (in)

Using the new section properties, Equations 6, 8, and 9 were used to determine the stresses acting on each side of the neutral axis. The cross sectional area used to determine the stresses acting on the steel plate was the same as the equivalent section of Red Oak.

The steel plate was secured to the wooden beam by lag bolts spaced at regular intervals. The spacing of the lag bolts was determined by considering both the shear flow through the composite beam as well as the strength of the bolts. The shear flow was determined using Equation 16.

Equation 16

$$q = \frac{VQ}{I}$$

Where:

q = Shear Flow (lb/in)

Q = First Moment of Area (in^3)

The required spacing of the lag bolts was determined by relating the shear flow from Equation 14 with the capacity of the bolt. This relationship is expressed in Equation 17.

Equation 17

$$S = \frac{Capacity}{q}$$

Where:

S = Bolt Spacing (in)

Capacity = Capacity of the Lag Bolt (lbs)

The other structural members that had insufficient strength to carry the required loads were the columns. The existing columns were to be replaced with square timber columns with a large enough cross section to support the required compressive stress caused by the existing dead load of the structure and the required superimposed live load. The strength of the material was determined as outlined in Section 3.4.2. The applied live load was reduced in accordance with IBC 1607.10.2. The column was assumed to experience only axial compression stresses, and were therefore analyzed using Equation 10.

New footings were designed to support the new columns. The new footings were to be made of reinforced concrete. The footings were designed to comply with Section 1808 and 1809 of the *IBC*. To do this, the allowed bearing pressure of the soil was estimated using Table 1806.2 of the *IBC* and using soil information obtained from the United States Department of Agriculture. The applied loading was determined in accordance with Section 3.4.1, plus an additional 8% to account for the self-weight of the footing. This load was then divided by the bearing strength of the soil to obtain a required bearing area. Dimensions of the footing were chosen to satisfy this required bearing area.

Using the chosen dimensions to obtain an actual bearing area, the actual bearing pressure exerted by the soil on the footing was determined by dividing the load by the actual bearing area. The strength of the concrete was chosen based on IBC Table 1808.8.1, and a temporary depth of the footing was chosen arbitrarily.

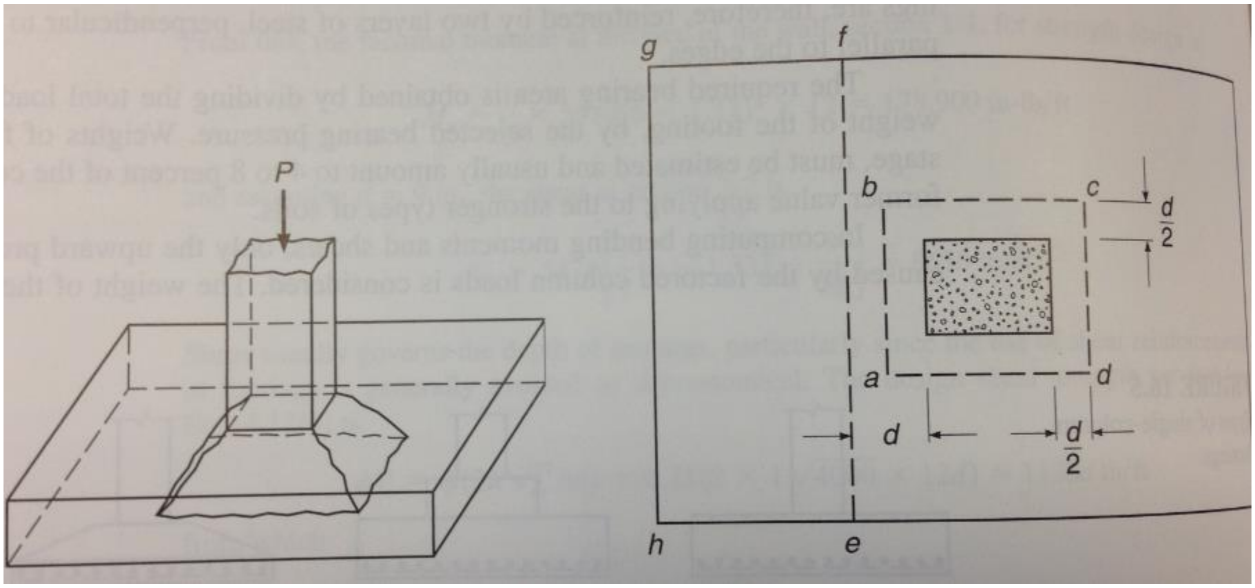

Figure 10: Illustration of Critical Shear in the Column Footing (Design of Concrete Structures)

The critical shear force acting on the footing was determined using Equation 18. The critical shear is located at a position equal to d/2 from each side of the column, where d is the depth of the footing.

Equation 18

$$V_c = P - q_a(b_c + d)$$

Where:

V_c = Critical Shear (lbs)

P = Applied Load (lbs)

q_a = Bearing Pressure of the Soil (psf)

b_c = Width of Column (ft)

d = Dept of Footing (ft)

Once the critical shear was determined, Equation 19 was used to more accurately determine the required depth of the footing to resist shear.

Equation 19

$$d = \frac{V_c}{4\lambda b_o \sqrt{f'_c}}$$

Where:

d = Depth of Footing (in)

λ= 1 for Normal Weight Concrete

b_o = Perimeter of the Critical Section for Shear (in)

f'_c = Compressive Strength of Concrete (psi)

The required depth of the footing was compared to the requirements specified in IBC 1809.4, and the greater minimum depth was chosen. The next step was to determine the applied bending moment within the cross section of the footing using Equation 20.

Equation 20

$$M = Pa_1 b$$

Where:

M = Applied Bending Moment (in*lbs)

a_1 = Distance from the Edge of the Footing to the Face of the Column (in)

b = Width of the Footing (in)

The bending moment was used to determine the necessary area of reinforcing steel to resist the tensile stresses created in the footing. The required area of reinforcing steel was found using Equation 21.

Equation 21

$$A_{s_{rec}} = \frac{M}{f_y(d-c)}$$

Where:

A_{sreq} = Required Area of Steel Reinforcement (in^2)

f_y = Yield Strength of Steel Reinforcement (psi)

d = Depth of the Footing (in)

c = Required Concrete Cover (in)

The value returned from Equation 17 was compared with the minimum allowed area of steel reinforcement based on the dimensions of the footing as determined by Equation 22.

Equation 22

$$A_{s_{\min}} = \frac{3\sqrt{f'_c}b_c d_c}{f_y} \geq \frac{200 b_c d_c}{f_y}$$

Where:

b_c = Width of Column (in)

d_c = Length of Column (in)

The largest value of A_{sreq} and A_{smin} was used to determine the reinforcement required in the footing. Table 2 was used to select the proper arrangement of rebar that would satisfy the required area. The rebar was placed in the footing in accordance with IBC 1808.8.2.

Table 2: Area of Bars in Slabs (Design of Concrete Structures)

TABLE A.3
Areas of bars in slabs, in²/ft

Spacing, in.	Bar No. Inch-Pound: 3 / SI: 10	4 / 13	5 / 16	6 / 19	7 / 22	8 / 25	9 / 29	10 / 32	11 / 36
3	0.44	0.78	1.23	1.77	2.40	3.14	4.00	5.06	6.25
3½	0.38	0.67	1.05	1.51	2.06	2.69	3.43	4.34	5.36
4	0.33	0.59	0.92	1.32	1.80	2.36	3.00	3.80	4.68
4½	0.29	0.52	0.82	1.18	1.60	2.09	2.67	3.37	4.17
5	0.26	0.47	0.74	1.06	1.44	1.88	2.40	3.04	3.75
5½	0.24	0.43	0.67	0.96	1.31	1.71	2.18	2.76	3.41
6	0.22	0.39	0.61	0.88	1.20	1.57	2.00	2.53	3.12
6½	0.20	0.36	0.57	0.82	1.11	1.45	1.85	2.34	2.89
7	0.19	0.34	0.53	0.76	1.03	1.35	1.71	2.17	2.68
7½	0.18	0.31	0.49	0.71	0.96	1.26	1.60	2.02	2.50
8	0.17	0.29	0.46	0.66	0.90	1.18	1.50	1.89	2.34
9	0.15	0.26	0.41	0.59	0.80	1.05	1.33	1.69	2.08
10	0.13	0.24	0.37	0.53	0.72	0.94	1.20	1.52	1.87
12	0.11	0.20	0.31	0.44	0.60	0.78	1.00	1.27	1.56

3.6.4 Addition Floor System

The frame supporting the addition varies considerably from the framing system that supports the main building. In the addition, the directions that the floor deck, joists, and carrier beams span are perpendicular to those in the main building. The floor planks and joists were found to be simply supported, and were analyzed using the methods discussed in Section 3.4.3.

The carrier beams were found to span continuously with a cantilever at the north end. Therefore there are two spans per carrier beam, one being an internal span and one being a cantilever. These beams were analyzed using a procedure similar to that outlined in Section 3.4.3

but using modified equations considering both positive and negative moment. The framing plan for the addition can be seen in Figure 9.

The loading applied to the carrier beams are unique in that the beams experience a uniformly distributed load as well as a point load. The first floor creates a uniformly distributed dead load that is applied to the entire length of the continuous beam. The dead load effect caused by the second story is transferred through the exterior walls of Mechanics Hall and applied as a point load to the end of the cantilevered span. The magnitude of this load was determined using the tributary area of the exterior walls. The bending moment effect applied to a typical carrier beam was determined by summing the moment caused by the first floor and the second floor. Equation 23 was used to determine the superimposed dead load applied to an internal span.

Equation 23

$$M = \frac{w}{8l^2}(l+a)^2(l-a)^2 \frac{Pax}{l}$$

Where:

M = Positive Bending Moment (ft*lb)

w = Uniform Load (lb/ft)

l = Interior Span Length (ft)

a = Cantilever Span Length (ft)

P = End Load caused by the Second Floor (lbs)

x = Location of Maximum Moment ($x = 0.5L[1-a^2/L^2]$)

Because these spans are continuous, there is an area above the supports where the beams experience a negative moment. This is created by the beam bending up and over the support column. Equation 24 was used to determine the negative moment created by both the uniformly distributed dead load and the point load at the end of the cantilever.

Equation 24

$$M = \frac{wa^2}{2} + Pa$$

The equation used to determine the shear force on an internal span has been modified to Equation 25.

Equation 25

$$V = wa + P$$

Because the beam is continuous, the upward shear forces experienced in the vicinity of the supports may be more extreme than the traditional downward shear forces. Equation 26 was used to determine the negative shear acting on the beam.

Equation 26

$$V = \frac{w}{2l}(l^2 - a^2) + \frac{Pa}{l}$$

The equation used to determine the deflection has also changed considerably from Equation 11. Equation 27 is the new equation used to find the deflection on the internal span.

Equation 27

$$\Delta = \frac{wx}{24EI}(l^4 - 2l^2x^2 + lx^3 - 2a^2l^2 + 2a^2x^2) + \frac{0.06415Pal^2}{EI}$$

The equations used to assess the deflection of the cantilevered spans differ from those used to analyze the internal span. Equation 28 was used to approximate the deflection of the cantilever. In the case of this equation, the distance x was set equal to the cantilever length, a.

Equation 28

$$\Delta = \frac{Pa^2(l+a)}{3EI} + \frac{wx}{24EI}(4la^2 - l^3 + 6xa^2 - 4ax^2 + x^3)$$

3.6.4.1 Design Solutions for the Addition Floor System

As a result of the analysis described in Section 3.6.4, certain structural members in the addition were required to be replaced. The joists and the carrier beams were found inadequate to resist the required loads. Therefore, new beams were designed to support the addition.

The design process of the joists began by determining the loading applied to the member (dead loads and live loads), as described in Section 3.6.1. As the span and the loads were known, the applied moment was found using Equation 7. Based off the applied moment, a cross section was chosen that would support the moment. This was checked using Equation 7. The proposed joist section was then checked in shear using Equations 8 and 9, as well as checked for deflection using Equation 10.

The cantilevered carrier beams were designed using the same process, except the applied moment was determined using Equations 23 and 24, the applied shear force using Equations 25 and 26, and the deflection using Equations 27 and 28.

Room for the larger beams designed using the above process will be made by replacing the existing sideboards. As the sideboards are decaying due to water damage, they will have to be replaced. When the sideboards are replaced, a new hole will be cut to accommodate the new position of the larger carrier beam.

3.6.5 Foundation

The foundation of Mechanics Hall is made of fieldstone. According to IBC 1807.1.3, a rubble stone foundation wall is not allowed to be used for a site designated Seismic Design Category C, D, E, or F. The Seismic Design Category is based off of the type of soil on site and the Occupancy Category of the structure. The Java Ground Motion Parameter tool from the United States Geological Survey (USGS) was used to determine the appropriate Seismic Design Category. This tool provided the maximum considered spectral response acceleration at 0.2 second intervals (Ss) and 1 second intervals (S1) for Princeton, MA. The Occupancy Category was decided in accordance with IBC 1604.5, and the Site Class of the soil was assumed to be Site Class D (Default), as the details of the soil beneath Mechanics Hall was unknown. With these values, the Seismic Design Category was determined to be Seismic Design Category B. Therefore, the current fieldstone foundation is in compliance with that requirement of the *IBC*.

The second requirement of IBC 1807.1.3 is that the foundation wall must be at least 16 inches thick. A measurement of the thickness of the wall taken using one of the holes in the foundation wall revealed the wall thickness to be 9 inches. Therefore it was determined that the foundation wall for the main building was to be reinforced, rather than replaced.

All of the foundation work described in this section was done to conform to *IBC* Chapter 18. According to *IEBC* 606.2.2, repairs done to the vertical members of a lateral force-resisting system need not consider earthquake loads in the analysis if the damage was not caused by an earthquake, and if the building is Seismic Design Category A, B, or C. As the damage done to Mechanics Hall was not done due to earthquakes, and Mechanics Hall is Seismic Design Category B, earthquake loads were not considered for this section.

IBC Table 1807.1.6.2 was used to determine the necessary reinforcement for the foundation in the original building. In accordance with *IBC* 1807.1.6.2 stipulation 4, a welded wire reinforcement system was used to limit the required thickness of the concrete wall instead of standard rebar. This is displayed in Table 3. For the purposes of using this table, the height of the foundation wall was rounded up from 6.5 feet to 7 feet. The design lateral soil load was taken from *IBC* Table 1610.1. The maximum unbalanced backfill height was determined by estimating the height between the basement floor and the ground surface.

Table 3: Requirements for Concrete Foundation Walls, taken from IBC 1807.1.6.2.

MAXIMUM WALL HEIGHT (feet)	MAXIMUM UNBALANCED BACKFILL HEIGHT[e] (feet)	MINIMUM VERTICAL REINFORCEMENT-BAR SIZE AND SPACING (inches)								
		Design lateral soil load[a] (psf per foot of depth)								
		30[d]			45[d]			60		
		Minimum wall thickness (inches)								
		7.5	9.5	11.5	7.5	9.5	11.5	7.5	9.5	11.5
5	4	PC	PC	PC	PC	PC	PC	PC	PC	PC
	5	PC	PC	PC	PC	PC	PC	PC	PC	PC
6	4	PC	PC	PC	PC	PC	PC	PC	PC	PC
	5	PC	PC	PC	PC	PC	PC	PC	PC	PC
	6	PC	PC	PC	PC	PC	PC	PC	PC	PC
7	4	PC	PC	PC	PC	PC	PC	PC	PC	PC
	5	PC	PC	PC	PC	PC	PC	PC	PC	PC
	6	PC	PC	PC	PC	PC	PC	#5 at 48	PC	PC
	7	PC	PC	PC	#5 at 46	PC	PC	#6 at 48	PC	PC
8	4	PC	PC	PC	PC	PC	PC	PC	PC	PC
	5	PC	PC	PC	PC	PC	PC	PC	PC	PC
	6	PC	PC	PC	PC	PC	PC	#5 at 43	PC	PC
	7	PC	PC	PC	#5 at 41	PC	PC	#6 at 43	PC	PC
	8	#5 at 47	PC	PC	#6 at 43	PC	PC	#6 at 32	#6 at 44	PC
9	4	PC	PC	PC	PC	PC	PC	PC	PC	PC
	5	PC	PC	PC	PC	PC	PC	PC	PC	PC
	6	PC	PC	PC	PC	PC	PC	#5 at 39	PC	PC
	7	PC	PC	PC	#5 at 37	PC	PC	#6 at 38	#5 at 37	PC
	8	#5 at 41	PC	PC	#6 at 38	#5 at 37	PC	#7 at 39	#6 at 39	#4 at 48
	9[d]	#6 at 46	PC	PC	#7 at 41	#6 at 41	PC	#7 at 31	#7 at 41	#6 at 39
10	4	PC	PC	PC	PC	PC	PC	PC	PC	PC
	5	PC	PC	PC	PC	PC	PC	PC	PC	PC
	6	PC	PC	PC	PC	PC	PC	#5 at 37	PC	PC
	7	PC	PC	PC	#6 at 48	PC	PC	#6 at 35	#6 at 48	PC
	8	#5 at 38	PC	PC	#7 at 47	#6 at 47	PC	#7 at 35	#7 at 47	#6 at 45
	9[d]	#6 at 41	#4 at 48	PC	#7 at 37	#7 at 48	#4 at 48	#6 at 22	#7 at 37	#7 at 47
	10[d]	#7 at 45	#6 at 45	PC	#7 at 31	#7 at 40	#6 at 38	#6 at 22	#7 at 30	#7 at 38

The steel reinforcement was designed to resist the lateral load applied by the weight of the soil. The applied soil load was determined using IBC Table 1610.1 for active pressure, measured in psf per foot of depth. The appropriate soil type was chosen based off of information gathered using the USGS Ground Motion Parameter Tool used to determine the Seismic Design Category. A 1-foot long strip of wall was used to analyze the load effects on the wall. Because the soil pressure increases with depth, the resulting stress distribution was triangular in shape, with the highest stresses occurring at the base of the foundation wall. The soil lateral load was resolved to an equivalent loading scenario using Equation 29

Equation 29

$$V = \frac{hw_z}{2}$$

Where:

V = Resolved Shear Force (lbs)

h = Height of Soil Fill (ft)

w_z = Applied Lateral Load at the Base of the Foundation Wall (lb/ft)

Equation 30 was used to verify that the steel reinforcement could adequately resist the shear force.

Equation 30

$$V = A_s F_y$$

Where:

A_s = Required Area of Steel Reinforcement (in^2/ft)

F_y = Yield Strength of Steel (psi)

The required area of steel reinforcement was then compared to the value specified in Table 3, and the greater of the two values were used. Because the bearing strength of fieldstone is very high, the existing foundation wall was deemed adequate to resist the applied compressive load of the building. Therefore, only enough concrete necessary to fully encase and protect the steel mesh was required.

The Addition required a new foundation design. Due to the terrain, the Addition is level with the outside ground. Therefore, the driving concern for the Addition foundation was frost protection. To protect against frost, a subterranean wall was designed so a footing could be placed below the frost line.

The foundation wall was designed using IBC Section 1807.1.6, as allowed by IBC 1807.1.5. IBC Table 1807.1.6.2 was used to determine the necessary size and reinforcement of the wall, as the wall met all the necessary criteria specified in IBC 1807.1.6.2. The soil loads were chosen using IBC

Table 1610.1, and the necessary depth of the wall was used using 780 CMR 5403.1.4.1 of the *Massachusetts State Building Code*. IBC Table 1807.1.6.2 is depicted in Table 3.

The footings were designed to minimize settlement while transferring the building loads to the earth. To do this, the bearing strength of the soil was estimated using the results of the USGS Ground Motion Parameter Tool in combination with IBC Table 1806.2, as seen in Table 4.

Table 4: Presumptive Load Bearing Values of Soils, Taken From IBC 1806.2.

CLASS OF MATERIALS	VERTICAL FOUNDATION PRESSURE (psf)	LATERAL BEARING PRESSURE (psf/ft below natural grade)	LATERAL SLIDING RESISTANCE	
			Coefficient of friction[a]	Cohesion (psf)[b]
1. Crystalline bedrock	12,000	1,200	0.70	—
2. Sedimentary and foliated rock	4,000	400	0.35	—
3. Sandy gravel and/or gravel (GW and GP)	3,000	200	0.35	—
4. Sand, silty sand, clayey sand, silty gravel and clayey gravel (SW, SP, SM, SC, GM and GC)	2,000	150	0.25	—
5. Clay, sandy clay, silty clay, clayey silt, silt and sandy silt (CL, ML, MH and CH)	1,500	100	—	130

This value was then adjusted to account for the soil pressure caused by the soil above the edge of the footing, next to the wall, using Equation 31.

Equation 31

$$q_s = q_a - 60z$$

Where:

q_e = Effective Soil Bearing Pressure (psf)

q_a = Presumptive Soil Bearing Pressure (psf)

z = Depth to the Top of the Footing (ft)

60 = 60 psf/ft, Taken from IBC Table 1610.1 for the Soil Pressure at a Given Depth

The loads of the building were determined as discussed in Section 3.4.1. The loads caused by the foundation wall and the self-weight of the footing were determined using the unit weight of concrete, as seen in Equation 2.

The minimum width of the footing was determined by dividing the total load by the effective bearing strength of the soil. This value was rounded up to a round number to allow for a more constructible design.

The total load acting on the footing was then divided by the chosen footing width to find the expected soil pressure acting on the footing (q_r). To determine the appropriate depth of the footing, the footing was evaluated in shear using Equation 32.

Equation 32

$$V_c = q_r(\frac{b-a}{2} - d)$$

Where:

V_c = Critical Shear (lbs/ft)

b = Width of the Footing

a = Thickness of Foundation Wall

d = Distance from the Top of the Footing to the Reinforcing Steel

The design shear force was then determined (in terms of d) using Equation 33.

Equation 33

$$V_d = 2\lambda\sqrt{f'_c}bd$$

Where:

V_d = Design Shear Force (lbs/ft)

λ = 1 for Normal Weight Concrete

The required depth of the reinforcing steel was determined by dividing V_c by V_d. In the case of this design, the value of d was much smaller than expected based on the dimensions of the footing. When choosing steel reinforcement, the original assumed depth of 9 inches was used.

The moment was analyzed using the original assumed depth for d using Equation 34.

Equation 34

$$M = 0.125q(b-a)^2$$

Where:

M = Bending Moment (ft*lbs/ft)

Once the bending moment was determined, Equation 35 was used to begin to determine the amount of reinforcement needed.

Equation 35

$$R = \frac{M}{bd^2}$$

Where:

R = Input Value to a Design Aid

Due to the extraordinarily low value for R, and the very low minimum reinforcement depth and bending moment, it was decided that reinforcement is unnecessary for this footing. Therefore, the footings were dimensioned to meet the criteria specified in IBC 1809.8. These minimum requirements specify a minimum footing thickness of 8 inches.

3.7 Accessibility

When looking into accessibility Mechanics Hall needed to first be investigated in order to determine what accessibility features were currently present. Using Figure 7 the elements within the flow chart were analyzed. Mechanize devices were not considered because of the lack of electricity, and arrangement of the current staircases. It would be a tremendous amount of work in order to install a system such as this. Through the investigation a baseline was able to be established of the amount of accessibility features within the *IEBC*. Code then had to be consulted in order to determine what the current requirements for an existing building are. The *IEBC* was mainly used because an existing building is not very easy to bring up to the level of accessibility specified in the *IBC*. Therefore, the *IEBC* specifically states that accessibility features only need to be considered if the space is renovated or changed for an existing building. Upon consulting the necessary codes specifications on how to bring the building into compliance were made.

3.8 Drainage

After performing visual inspections of the basement at Mechanics Hall, it had been determined that there is an inadequate drainage system for rain water and melting snow. It was obvious from the extent of saturated structural components and the mold growth that there are drainage problems. From the visual inspections, the first thought was that there could be a "rising damp" problem but after research it had been determined that a rising damp problem was not the cause of the poor conditions in the Mechanics Hall basement. There are a few different strategies for tackling the drainage problem, one consisting of excavating the placing an exterior drainage around the perimeter and the other by placing an interior drainage system just below the floor. They key concepts of the drainage systems would be to minimize the amount of ground water entering Mechanics Hall.

3.9 Parking

The parking requirements for Mechanics Hall were determined by applying the Town of Princeton Zoning Bylaws. Chapter 2 of the Bylaws was used to determine the type of district that Mechanics Hall is located in. The applicable parking requirements were then found in the appropriate chapter of the Zoning Bylaws and in Chapter 7 General Regulations. Parking requirements and accessibility standards were then verified using IBC Chapter 11.

3.10 Renovation Plan

In order to be able to show the Friends of Mechanics Hall an example of a finished product the first step started of any renovation begins with creating a preliminary layout of the building. From the layout a 3D CAD model was created and then applying the defined "uses" to the 3D model renderings of each use were created.

3.11 Cost Estimate

A common technique that was followed for cost estimating was to list the resources that are needed for the project and to total their costs. Typical resources include equipment, material, services and labor. In order to perform an accurate cost estimate for fire protection in Mechanics Hall, the design was broken down into the amount of pipe and sizes needed for all the necessary fitting, sprinkler heads, valves, pump and the storage tank. After gathering the necessary quantities, usually a pricing software is used to estimate the final cost including labor hours and profit. The other areas of work that were studied further to produce a cost estimate were the structural members, foundation, and also a drainage solution. These construction cost estimates were calculated by $/per linear foot, $/per square foot, $/per cubic yard. One source of estimating the cost of resources is the use of "R. S. Means Construction Cost Estimating Manual."

4. Code Review

Through the use of this chapter the reader will be able to see every code provision that was considered in this project. The implication on the work done is also outlined for each specific provision. The intent is to provide an overview of the scope of elements involved in code compliance and to provide a base for understanding the recommendations posed for opening the building to new uses.

4.1 Fire Protection

4.1.1 Sprinkler System

Per NFPA13 the definition of a sprinkler system is a network of pipes that is designed based upon engineering standards that are usually activated based upon heat from a fire, then discharges water over the fire area. The portion aboveground to which sprinklers are attached is either hydraulically or specifically designed based upon a given standard.

Table 5:Sprinkler System Code Review

Provision	Implication
MGL 146.26G	Initially indicates that sprinklers are not required in Mechanics Hall. States that a building that is greater than 7500 square feet must have a sprinkler system.
IBC Chapter 3	Provides Occupancy information to determine the correct hazard based upon building usage
IEBC 1012.2.1	States that if the occupancy changes, then the requirements per IBC for the new occupancy will have to be complied with
IEBC 504	Specifies the extent of scope of work that is defined as a level 2 alteration
IBC 903.2.1.1	For a fire area on the second floor in an Assembly occupancy a sprinkler system is required. This has applicability to Mechanics Hall.
IEBC 1012.1.1.1	If the occupancy changes without a fire barrier, then chapter 9 of the IBC must be consulted
IEBC 804.2.2	Specifies that if the occupant load is less than 30 people and there is a change of occupancy, then a sprinkler system is not required.
NFPA 13 5.2	Specifies why Mechanics Hall falls under a Light Hazard occupancy, and the implications of a Light Hazard occupancy
NFPA 13 Figure 11.2.3.1.1	Describes the necessary density/area requirement needed for Mechanics Hall
NFPA 13 Figure 11.2.3.2.3.1	Design area reduction table for quick response sprinklers. Allows Mechanics Hall to have a smaller design area based upon ceiling height.
NFPA 13 Table 11.2.3.1.2	Specifies the required hose allowance for a given occupancy classification.
NFPA 13 Table 8.6.2.2.1(a)	Gives the area and spacing requirements for a Light Hazard occupancy. This gave the specifics for the sprinkler design in Mechanics Hall.

4.1.2 Egress

The IBC defines means of egress as a continuous and unobstructed path of vertical and horizontal egress travel from any occupied portion of a building or structure to a *public way*

Table 6: Egress Code Review

Provision	Implications
IEBC Table 1012.4	States the means of egress hazard category
IBC1008.1.1	Specifies Minimum door sizes for means of egress in specific occupancies. Detailed Requirements can be seen in the egress table.
IBC1008.1.5	States that there needs to be a landing on each side of a door. Mechanics Hall does not have this so one will need to be installed.
IBC1008.1.9	Doors should be easily opened from the egress side without the use of a key or special knowledge. Mechanics Hall will need to install new door, which will make this a non-issue.
IBC1009.4	Specifies requirements for staircase width, and occupancy load.
IBC 1009.5	Specifies maximum headroom for a staircase.
IBC1009.7.2	States necessary riser height, and tread depth.
IBC Table 1016.2	Specifies egress distance for different occupancies with and without sprinklers.
IBC1011	States the requirements for exit signs and in what situations they are needed
IEBC 1203.3	Gives an exception based upon a historical building. It puts the requirements of egress up to the discretion of the code official.

4.1.3 Structural and other Fire Protection

Structural fire protection represents the required fire resistive ratings in time based upon different construction types per IBC. Code used for detectors was also specified in this table.

Table 7: Structural and other Fire Protection Code Review

Code	Implication
IBC Table 503	General Height and Area requirements based upon construction type
IBC Table 601	Fire resistive ratings for different building elements based upon construction type
IBC 704.13	Allowance to use sprayed fire resistance in order to achieve necessary fire resistance rating
IBC 907.2	Fire alarm requirements for buildings and structures.

4.2 Structural Analysis

Table 8: Structural Analysis Code Review

Code	Description
International Existing Building Code (IEBC)	
1206.2	All conditions deemed to be dangerous must be made safe, and no other work is required
606.1	Dangerous conditions must be eliminated. New structural members must conform to all standards set in IBC.
606.2.3	Significantly damaged members of the gravity load-resisting system must be repaired to conform to load requirements specified in IBC.
International Building Code (IBC)	
1604.1	Structural members must be designed using a design process approved by the appropriate material chapters in IBC.
1604.2	All structural members must be designed with adequate strength to support the required loads.

1604.3	All appropriate members must have adequate resistance to deflection. For floor systems, the limit is the length of the span/240.
1604.4	All structural members shall be analyzed for short term and long term conditions using well-established engineering principles.
1605.3.1	A list of all applicable load combinations on a structure for allowable stress design.
1607.1	A table displaying the required minimum live loads for a building based on functional use.
1607.10.2	A live load may be reduced by 20% on a member that supports multiple floors.
2306.1	All timber design using Allowable Stress Design should be done in accordance with the National Design Specifications
National Design Specifications (NDS)	
2.3.1	The applicable adjustment factors to reference design values can be found in 4.3 for sawn lumber.
2.3.2	Specifications on determining the Load Duration Factor
2.3.3	Specifications on determining the Temperature Factor
4.3	Details on the applicable reference design values for sawn lumber.
Table 4A, 4D of Supplement	Reference design values and adjustment factors for visually graded lumber and visually graded lumber 5”x5” and larger, respectively. Used to find material strength for Northern Red Oak, and Repetitive Member Factor, Wet Service Factor, Flat Use Factor, and Size Factor.
3.3.3	Method to determine the Beam Stability Factor
4.3.8	Procedure to find the Incising Factor
3.7.1	Procedure to calculate the Column Stability Factor
4.4.2	Method to calculate the Buckling Stiffness Factor
3.3.2	Flexural Design Equations
3.4.2	Shear Design Equations
4.4.3	Limitations on Notching of sawn lumber beams
3.5.1	Deflection calculations should be calculated using standard engineering mechanics.
3.7.3	Round column calculations should be based off the design calculations for a square column of similar cross-sectional area.
Appendix G	Procedure to determine the Effective Length Coefficient, K_e
Table 11.2A	Lag Screw Reference Withdrawal Design Values

4.3 Foundation

Table 9: Foundation Code Review

Code	Description
International Existing Building Code (IEBC)	
601.2	Repair work shall not result with the structure becoming less compliant with building code than it was prior to the repairs.
606.2.2	Buildings that have sustained significant damage to the vertical elements of its lateral force-resisting system shall be evaluated by a professional engineer, who shall determine if the building would comply with IBC if returned to its original condition. EXCEPTION: A building in Seismic Category A, B, or C whose damage was not caused by an earthquake need not consider earthquake loads.
606.2.3	Significantly damaged members of the gravity load-resisting system must be repaired to conform to load requirements specified in IBC.
International Building Code (IBC)	
1807.1.3	Rubble Stone foundations must be at least 16 inches thick, and cannot be used in a building of Seismic Design Category C, D, E, or F.
1604.5	A Risk Category shall be assigned to a structure in accordance with Table 1604.5 that rates the importance of the structure.
1809.5	Concrete footings must extend below the local frost line
1807.1.5	Concrete foundation walls must be designed in compliance with Chapter 19 of IBC, or with section 1807.1.6
1610.1	Tabulated Soil Loads based on soil type
1806.2	Presumptive load bearing strengths of soils
1807.1.6	Concrete foundation walls laterally supported at the top and bottom can be supported using this section.
1807.1.6.1	A foundation wall cannot be thinner than the wall it supports.
1807.1.6.2	Concrete foundation walls that comply with all requirements in this section can be dimensioned using Table 1807.1.6.2
1808.8.1	The concrete used in foundations of buildings labeled Seismic Design Category B must have an axial compression strength of at least 2500 psi.
1808.8.2	Minimum specifications of concrete cover for reinforcing bars
1809.4	Concrete footings must extend at least 12" below undisturbed soil
1809.8	Plain concrete footings must be at least 8" thick

5. Existing Conditions

This section serves as a record of the condition of Mechanics Hall as it currently stands. Included in this section are the results from analysis of fire safety, structural capacity, drainage, and parking analysis. For the proposed solutions to these issues, see Section 6. It is the goal to analyze the building in its current condition, and see if it meets code.

Based on Mechanics Halls uses presented in the background the current building will be assumed to be a Class B occupancy. This can found in Chapter 3 of the IBC. The Friends of Mechanics Hall did not specify its current occupancy, and so class B was assumed because class B presents a use that is not to stringent nor lenient. It also represents part of what the friends of Mechanics Hall want to do with the building.

5.1 Fire Safety

As this building stands the fire safety aspects are not very advanced. This is very typical in a building that was constructed such a long time ago. One reason for this is because people don't tend to worry about fire requirements until they are actually in a fire. For example, if a historical building needs a new roof most people would probably view the roof as more important than installing an Automatic Sprinkler System. Because of this, not much money gets allocated toward fire protection and historical buildings usually fall behind in this category.

5.1.1 Water Supply

Pat Schmohl, our source for determining water supply verified that Princeton does not have public water nor, as a result, does it have fire hydrants. In a situation like this town firefighters will bring the water to the building. Chief Schmohl explained that each apparatus holds a capacity of 1000 gallons, and the tanker holds 2000 gallons. This allows the fire department to have around 6000 gallons readily available to them.

The Friends of Mechanics Hall made it clear to us that the buildings water supply is a well. This well does not solely supply Mechanics Hall but it also serves an auto body shop next door. Currently there is no running water in the pipes, and there has not been for a long period of time. In order to run water through the pipes again damage control would have to take place. Because there has not been heat in the building for some time the pipes could have frozen if they were not drained sufficiently which means damage could have been sustained. It is also assumed that there is not a large enough pump at the bottom of the well to meet the flow demand for a sprinkler system. Because of the inadequate flows the water supply that currently serves Mechanics Hall cannot be trusted with respect to fire protection systems.

5.1.2 Egress

Currently the egress requirements in Mechanics Hall are not sufficiently satisfied. When looking into Egress the various aspects of the building outlined in Figure 5 were investigated based on their current conditions. When going through the different requirements those that did not apply to Mechanics Hall were not discussed (i.e. elevators, fixed seating, etc.). Table 10 below outlines the different egress requirements and whether or not they have been met. In mechanics hall the three major egress components were doors, stairways, and hallways. The egress section in the results explains solutions to issues that were not in compliance.

Table 10: Egress Requirements

Egress Requirements				
Doors	***Minimum Height***	***Minimum Width***	***Landing***	***Locking Mechanism***
Needed Measurement (Outlined Code)	80in (IBC1008.1.1)	32in (IBC1008.1.1)	Full Landing outside of door, or a slope landing (IBC 1008.1.5)	Main egress door can not have a deadbolt that will impede the exit of the building (IBC 1008.1.9)
Met/Not Met	Met	Met	Not Met	Not Met
Stairways	***Allowed Occupancy Load***	***Minimum Width***	***Riser Height***	***Tread Depth***
Needed Measurement (Outlined Code)	50 People (IBC1009.4)	36in (IBC1009.4)	4in to 7in (IBC1009.7.2)	11in (IBC1009.7.2)
Met/Not Met	Met	Not Met	Not Met	Not Met
Stairways	***Height***	***Handrails Height***		
Needed Measurement (Outlined Code)	80in (IBC 1009.5)	34in to 38in		
Met/Not Met	Not Met	Met		
Egress, and Hallways	***Distance To Exit With Sprinklers***	***Distance To Exit Without Sprinklers***	***Exit Signs***	
Needed Measurement (Outlined in Code)	250ft (IBC Table 1016.2)	200ft (IBC Table 1016.2)	Needed (IBC 1011)	
Met/Not Met	Met	Met	Not Met	

5.1.3 Detectors

Currently Mechanics Hall does not have any detectors. The only reason a manual detection system (pull station) would need to be installed for a class B occupancy is if the occupant load was greater than 100 people per IBC 907.2.2. Smoke detection/carbon monoxide detection devices are not required for this building's current usage per IBC 907

5.1.4 Structural Fire Protection

When Mechanics Hall was built the code requirements were not as stringent as currently exists. Because of this there is not much in regards to structural fire protections. This section was determined based upon the procedure outlined in the Background and the Methodology. The first step was to determine the construction type. According to *IBC* there are five different types of construction. Mechanics Hall falls under a Class V building according to *IBC* chapter 6. A Class V building is a wood framed building that uses any material that is allowable by the *IBC*. Under Class V there are two subsections. Class VA is presented as protected in the *IBC*, and Class VB is unprotected. In this situation protected means that the structural members are fire resistant.

According to Table 601 the fire resistance rating can be determined. Currently Mechanics Hall is classified as a Class VB building. The *IBC* refers to a Class VB building as being wood framed and unprotected. The reason it is called unprotected is because according to Table 601 in *IBC* there is a 0 hour fire resistant rating requirement. It does not have any structural members that have fire resistive rating. It also has some features that will likely accelerate growth of a fire. Although it is impossible to know what is inside the walls of Mechanics Hall there are numerous holes. In these holes gaps can be seen in between certain walls. Scenarios such as this allow for drafts and could potentially make for a hazardous fire scenario. Because of this, making the building into construction type VA would be beneficial.

5.2 Structural

The live load that a structure is legally required to support is dependent on the use of the building. For the purposes of this analysis, the results will be compared to the required load for an Occupancy Class B building. According to IBC Table 1697.1, the required minimum live load capacity for the proposed uses is 100 psf.

5.2.1 Analysis Results

In order to check the compliance of Mechanics Hall with the IBC, a structural analysis was performed as described in Section 3.4. The results of this analysis are displayed in Tables 11 and 12. The calculations for this analysis can be found in Appendix F.

Table 11: Safe Superimposed Live Loads - Main Building

Structural Member	Bending Capacity (psf)	Shear Capacity (psf)	Deflection Capacity (psf)	Axial Compression Capacity (psf)
Floor Decking	2493.45	3647.92	7427.43	N/A
Typical Joist	149.19	272.40	221.91	N/A
Notched Joist	N/A	UNSAFE CONDITIONS	N/A	N/A
Girder A, Span 1	UNSAFE CONDITIONS	UNSAFE CONDITIONS	UNSAFE CONDITIONS	N/A
Girder A, Span 2	133.27	171.37	129.59	N/A
Girder A, Span 3	364.90	280.92	571.01	N/A
Girder B,C, Span 1	UNSAFE CONDITIONS	UNSAFE CONDITIONS	UNSAFE CONDITIONS	N/A
Girder B,C, Span 2	UNSAFE CONDITIONS	UNSAFE CONDITIONS	UNSAFE CONDITIONS	N/A
Column 1 & 2	N/A	N/A	N/A	UNSAFE CONDITIONS
Column 3	N/A	N/A	N/A	147.81
Column 4	N/A	N/A	N/A	280.77

Table 12: Safe Superimposed Live Loads - Addition

Structural Member	Bending Capacity (psf)	Shear Capacity (psf)	Deflection (psf)	Axial Compression Capacity (psf)
Floor Decking	2493.45	3647.92	7427.43	N/A
Joists	UNSAFE CONDITIONS	184.47	104.71	N/A
Carrier Beam, Internal Span	UNSAFE CONDITIONS	UNSAFE CONDITIONS	106.95	N/A
Carrier Beam, Cantilever	UNSAFE CONDITIONS	UNSAFE CONDITIONS	355.95	N/A
Column	N/A	N/A	N/A	272.37

Tables 11 and 12 display the superimposed live load that each type of structural member can safely support. According to the *IBC*, the minimum design live load for Occupancy Class B is 100psf. Members that do not meet this standard will need to be strengthened or replaced.

Table 11 includes every beam under every condition seen in the Main Building. The primary areas for concern in the main building are the condition of the structural members, as well as the capacities of the carrier beams and the columns. The condition of the members (concerning damage done to the material) is discussed in Section 5.2.2, and was not considered for this analysis. As indicated in Table 11, the three carrier beams lack the required strength to safely operate in an Occupancy Class B structure because they failed to pass all three checks. Two of the supporting columns also failed to pass their strength check. The joists and the floor decking have sufficient strength to support the required superimposed live load.

The entire addition except for the floor decking and the columns will need strengthening to support the required loads. The joists in the addition span a longer length than the joists in the main building, which causes enough additional stress to render the joists inadequate to handle the required bending stress. The cantilevered carrier beams require reinforcement for bending and shear. The loading on the internal span of the carrier beam is so great that the cantilevered section of the beam is pushed upward rather than downward. This creates the perception that the cantilevered section can support greater loads with a lower deflection. The areas of concern for the entire Mechanics Hall frame can be seen in Figure 11.

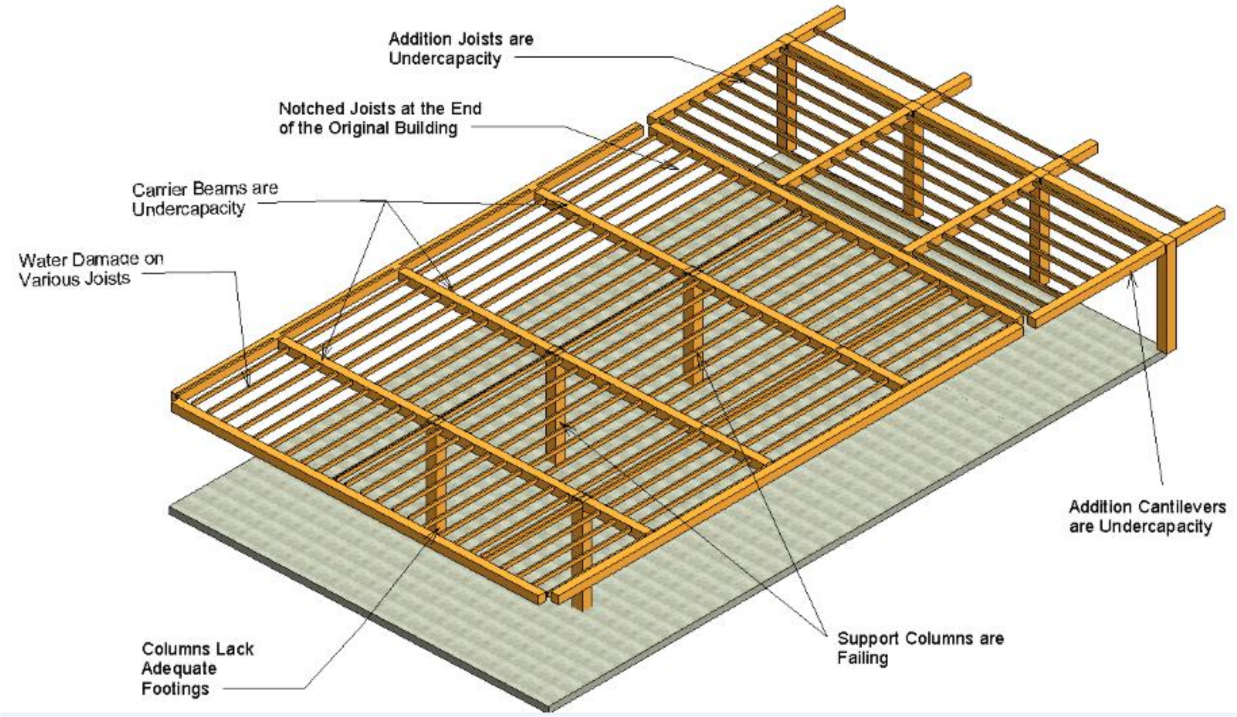

Figure 11: Mechanics Hall Frame Damages

5.2.2 Condition Assessment

The analysis that produced the results displayed in Section 5.2.1 was completed under the assumption that every structural member was in good condition with a full cross section. That is not the case with Mechanics Hall. Many of the beams in the Main Building have sustained heavy water damage. The joists in the southern bay between the South foundation wall and the South carrier beam have sustained significant water damage and have experienced large amounts of mold and decay (Figure 12). Various joists in the middle two bays are rotted and either sustained water damage or apparent insect damage (Figure 13). Select joists between the South and Middle Carrier Beam have been cut and bent to accommodate a small pipe near the western foundation wall (Figure 14 and 15).

Figure 12: Wet, moldy, rotting joists and floor decking under the South Bay.

Figure 13: Decaying joist with mold on the floor decking.

Figure 14: Warped, cut joist in between Carrier Beams A & B - Side View.

Figure 15: Warped, cut joists between Carrier Beam A and B - Front View

Every beam in the North bay between the North Carrier Beam to the addition has notches at the end of the span. As seen in Table 11, these beams fail the shear check due to the missing material. The notches in these beams also do not meet the standards specified in NDS Section 4.4.3. The standards specify that the notch may not be greater than ¼ of the depth of the beam, whereas the notch in the beam is close to half the depth, as seen in Figure 16. The Middle Column is splitting down the middle, and all columns show signs of insect damage (Figure 17). The footings for the columns consist of rocks with no concrete or grout, as the floor of the basement is dirt (Figure 18).

Figure 16: Crossbeams notched at the end of the spawn at the back end of the Main Building.

Figure 17: Basement support column splitting down the middle due to axial compression.

Figure 18: Rock footing for support column in the basement.

The structural members of the addition are generally less damaged than the members found in the original building. As seen in Figure 19, the main structural frame is free from mold and other visual signs of decay. Occasional cracks were found in structural members, as seen in Figure 19. The majority of the damage in the addition is found in the sideboards that make up the wall. These boards are exposed directly to the earth beneath the wall, resulting in rot and decay in the walls. Figure 20 depicts a hole that has rotted through on the side of Mechanics Hall. There is no foundation underneath the addition.

Figure 19: Framing as seen from the basement of the Addition

Figure 20: Hole in exterior wall as seen from the Addition Basement.

5.2.3 Foundation

Mechanics Hall has a different foundation supporting the original building than supporting the addition. The original building has a fieldstone foundation on three of the four sides. Multiple holes have formed primarily on the western foundation wall, as seen in Figure 21. These gaps right along ground level in the foundation wall allow a significant amount of water into the basement, contributing to much of the damage to the structural frame. The thickness of the wall measured through one of the larger holes is 9 inches. The southern and eastern foundation walls are in better condition and have experienced minimal damage. There is no basement floor slab.

Figure 21: Hole in Western foundation wall with visible daylight.

The addition has no foundation walls, as the land surrounding Mechanics Hall slopes downwards towards the back. The wooden exterior walls of the structure are in direct contact with the ground. This has led to significant rot and moisture damage in the exterior wall of the building, as depicted in Figure 20.

5.3 Accessibility

After analyzing Mechanics Hall for accessibility it was evident that there was not many features that made the building handicap accessible. This was not a surprise considering when the building was constructed. When beginning the analysis the first section investigated was the historic section in the *IEBC*. There are great similarities between the regular sections of the *IEBC* and the historic section in terms of accessibility. Therefore, the historic section is going to be primarily focused upon. *IEBC* 1204.1 gives suggestions for accessibility. They are suggestions and not requirements because it says that as much as possible needs to be done. Because of a historical exception accessibility does not need to be provided throughout the whole building. *IEBC* 1204.1.2 states that a building only needs to be accessible on the floor with the accessible entrance. Following Table 13 shows accessible features specified by *IEBC* 1204.1.

Table 13: Accessibility Requirements

Accessibility Requirements	
Element	***Met or Not Met***
Entrance (IEBC 1204.1.1)	Not Met
Bathrooms (IEBC 1204.1.4)	Not Met
Multilevel Buildings and Features (IEBC 1204.1.2)	Not Met

5.4 Parking

The existing parking for Mechanics Hall consists of a small dirt area in the front of the building. There is enough space for about 5 cars.

6. Proposed Uses and Recommended Rehabilitation Plans

For Mechanics Hall two future uses were established based upon the Friends of Mechanics Halls interest. Previously in the report the existing condition were established. Using this information as a benchmark, plans on how to bring the building up to code based upon the future uses were established. The components analyzed were fire protection, structural, drainage, accessibility, and parking.

6.1 Proposed Uses

As stated in the Methodology, two uses were proposed for this project. The investigation of two uses provided a context to explore specific rehabilitation schemes and their associated costs. Because this building is going to be primarily used by the contacts at Mechanics Hall, it was important that their ideas and wishes be incorporated into the uses. Their main desires were a tenable space for a business on the first floor and an assembly area on the second floor. With these guidelines in mind two uses were created that incorporated both. The first use proposes a tenable office space on the first floor that will provide income money through rent. The second floor will be isolated from the first floor and will be a function hall for any sort of assembly requirement seen fit. This solution closely follows the outlined wishes. The second proposal also has office space on the first floor but the second floor is a coffee house. The goal of providing a coffee house is to attract community involvement, and hopefully make the town aware that this building has a value and is deserving of attention.

6.1.1 Renovation Plan

The figures below show what could be expected as a result if there were a renovation of Mechanics Hall. After determining the "uses" of the building, different scenarios were put into place to show what Mechanics Hall could look like. Figure 22 shows one of the first floor rooms used as a small meeting room; displayed in Figure 23, is an office room that may be used if the building was used as a professional space. In Figure 24, the second floor auditorium is depicted as an assembly room for small town meetings. The last figure that shows a potential use for Mechanics Hall is Figure 25; in this figure the old kitchen is renovated into a modern day kitchen to support the coffee house. The building layout and section views can be found in Appendix F.

Figure 22: First Floor Meeting Room

Figure 23: First Floor Office Room

Figure 24: Second Floor Assembly Room

Figure 25: Second Floor Kitchen

6.1.1 Occupancy

In order to identify the necessary renovation work, the occupancy is required to be noted. With respect to proposal 1 there will be two different occupancies per *IBC* Chapter 3. The first floor will be a Class B occupancy because it is a business, and the upstairs will be a Class A occupancy because it is an assembly area. For proposal 2 there will only be one occupancy because the entire building will be used as a business. Therefore the second proposal refers to a class B occupancy per Chapter 3 of the *IBC*.

6.1.2 Alteration Level

When using the *IEBC* it is necessary to classify the level of alteration for the building. Chapter 5 of the *IEBC* gives definitions of each level of alteration. For the uses described, *IEBC* section 504 level 2 fits the best. It is not an overhaul to the whole building, but gives enough leeway to make ample changes. Section 504.1 of the *IEBC* describes the scope of level 2 alteration as the reconfiguration of space, the addition or elimination of any door or window, the reconfiguration or extension of any system, or the installation of any additional equipment.

6.2 Fire Protection

The development of the fire safety solution required the use of multiple codes because of how extensive the sprinkler systems are. A sprinklers system has to work in unison with every other system within the building. The three codes that were consulted for the sprinkler design were the *IEBC*, the *IBC*, and *NFPA13*. Using the *IEBC* this section also presents any historical exceptions with any repercussions from them.

6.2.1 Automatic Sprinkler System

When consulting the historical buildings section of the *IEBC* it was made clear that a historical building does not have to meet any construction requirements in the *IBC* as long as an automatic sprinkler system is installed. This is very useful for Mechanics Hall because it may be difficult to make some of the necessary structural and life safety changes. Although a sprinkler system may be costly to install, it could help with removing or reducing many other requirements, ultimately making the total cost less.

6.2.1.1 Proposal 1

For this use determining whether or not sprinklers were needed is rather challenging. Because there is a partial change of occupancy from B to A chapter 10 of the *IEBC* is applicable. Section 1012.1.1.1 of the *IEBC* states that if an occupancy changes without a fire barrier, then Chapter 9 of the *IBC* must be consulted for the most hazardous occupancy in the building. In the first proposal in the reuse of Mechanics Hall there is a group A area upstairs. Section 903.21.1 of the *IBC* states that if the fire area of a Group A-1 occupancy is located on a floor other than a level of exit discharge serving such occupancies then an automatic sprinkler system is required. Because the group A occupancy is located on the second floor sprinklers are required. The procedure outlined in the

Methodology was followed: determine fire hazard, find the necessary water discharge requirements, size a tank, and use the spacing to outline a sprinkler design.

When starting the sprinkler design it was necessary to determine the fire hazard associated with the building. NFPA13 5.2 explains that Mechanics Hall would fall under a light-hazard occupancy. Using this, NFPA13 Figure 11.2.3.1.1 states that there is an initial design area of 1500 square feet, and a flow density of 0.1 gallons per minute per square foot of the area. Because this building meets all the requirements needed to use reduction factors, a 40% ceiling reduction can be used per NFPA 13 Figure 11.2.3.2.3.1. However, because the attic has a sloped roof, a 30% sloped ceiling increase needs to be applied. The net effect is a 10% reduction in a new design area of 1350 square feet.

A water supply tank must be installed because there is not any water supplied to Mechanics Hall that can be used for an automatic sprinkler system. The size of the tank is found by calculating the amount of water needed. This is done by multiplying the flow density times the area, which results in 135 gallons per minute. The inside hose requirement needs to be satisfied, and calls for a 100 gallon per minute increase for light hazard occupancies. Therefore 235 gallons per minute for this system are required for 30 minutes per NFPA 13. If these are multiplied then a total demand of 7050 gallons per minute. Because tanks come in increments of 1000 gallons, an 8000 gallon tank is required.

After determining the sprinkler head spacing requirements the CAD files were created as seen in Figure 26-28. According to NFPA 13 Table 8.6.2.2.1(a) the maximum area between heads is 225 square feet, and the maximum spacing is 15 ft. This design could be changed depending on where the water enters the building from the water tank. It was assumed the water came in from the larger circle on the right of the building.

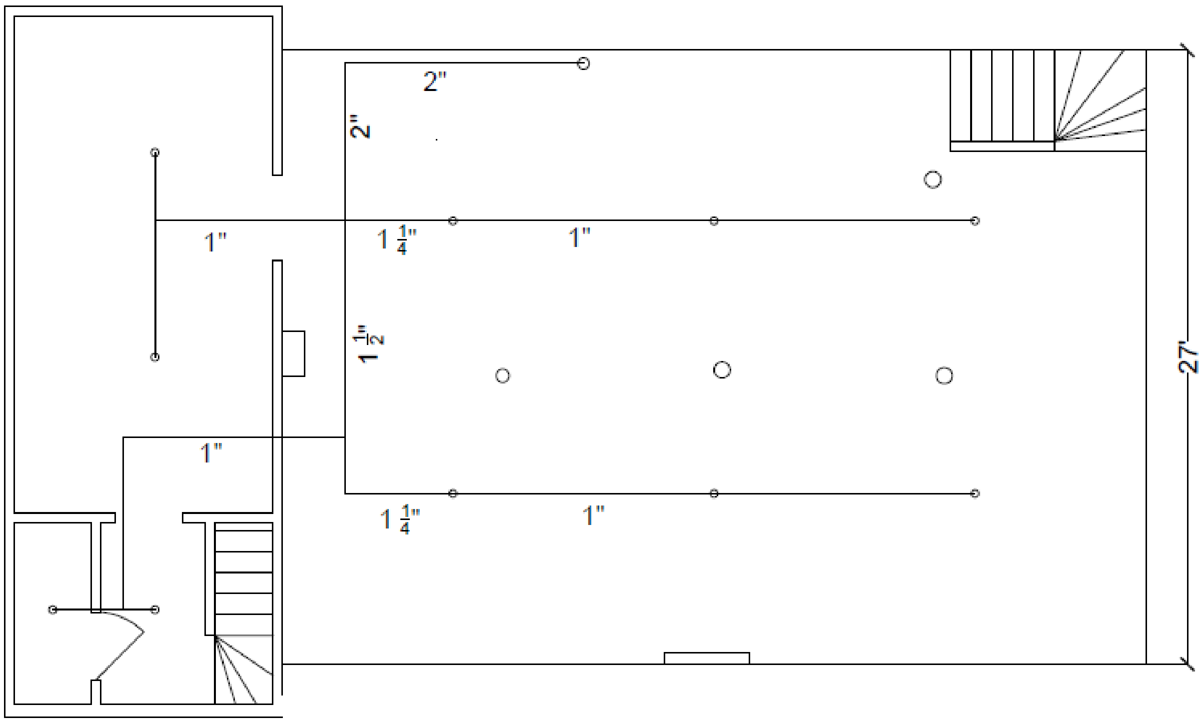

Figure 26: Sprinkler Design, Basement

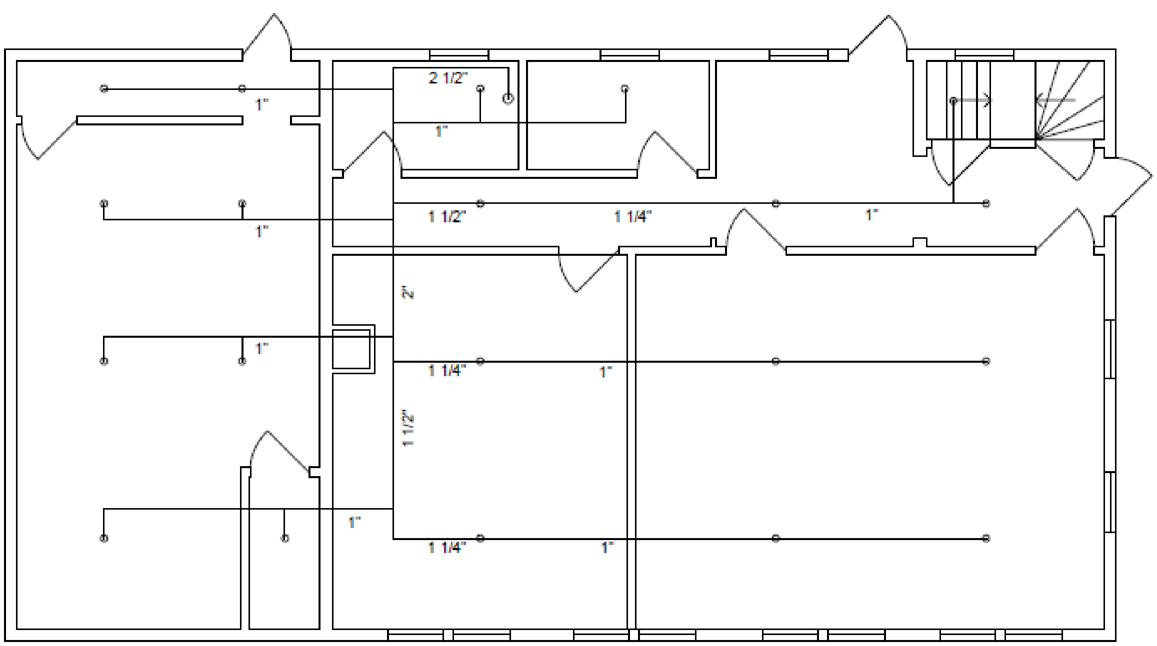

Figure 27: Sprinkler Design, First Floor

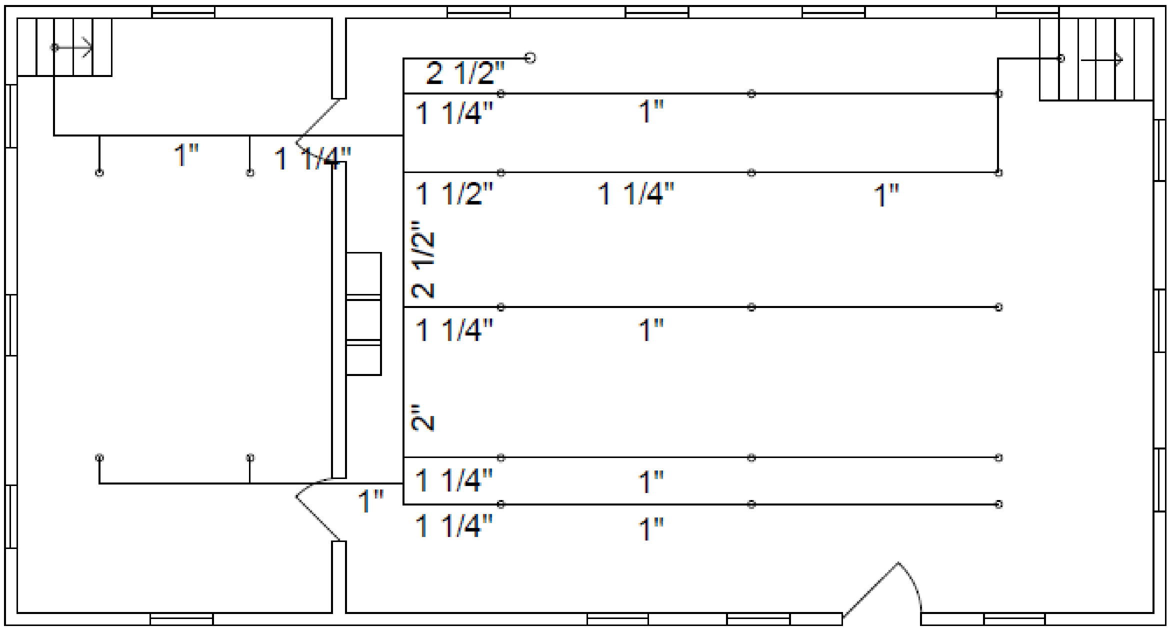

Figure 28: Sprinkler Design, Second Floor

When designing the system some further issues arose. Because there is no heat in this building the installation would have to wait until a heating system was installed. The presented drawings in Figure 26-28 do not consider the impact of the ductwork. Another issue is the observation that the attic will probably not be warm enough to permit the use of a wet pipe sprinkler system due to risk of freezing. Because of this dry uprights need to be used. These dry uprights would have to be supplied from the second floor where temperature would not be an issue. Very simply, a dry upright is a piece of pipe with a sprinkler head attached to it that does not have any water in it. When the head is activated water flows through the pipe and out of the head. NFPA 13 states that sprinkler heads must be a minimum distance of 3 feet away from a sloped roofs peak. Because of this a large enough dry upright needs to be used to get within 3 feet of the roof of the building.

Another issue that arose was in regards to aesthetic considerations. Because it is a historic building the friends of mechanics hall want it to look as original as possible. However, if a sprinkler system is installed there will be a large amount of pipe that has to be run somewhere. It is not possible to run it in the walls or the ceiling, therefore a drop ceiling will have to be installed. Because the roof is relatively tall this should not be an issue.

Lastly a pump needs to be installed. In order to determine the size of the pump a hydraulic calculation needs to be calculated. Because a sprinkler system is not the only consideration a hydraulic calculation was not done, and a pump size was estimated. A 250-gallon per minute vertical inline pump should be sufficient for what is needed. Because a hydraulic calculation was not used the schedule method of sizing pipe was used in NFPA 13.

6.2.1.2 Proposal 2

The second proposal does not require a change from group B occupancy the IEBC can be used. In Chapter 8 (level 2 alterations) sprinklers are not required for the proposed use. Initially per section 804.2.2 it seems that if there is an occupant load greater 30 people then the IBC will have to be consulted. However, there is an exception to this that can be met in the case of Mechanics Hall. It states that if a fire pump has to be installed because there is not sufficient water to supply a sprinkler system, then the work area only need to be protected by an automatic smoke detection system in accordance with Chapter 9 of the IBC. If the occupant load is less than 30 people, then a sprinkler system is not required.

6.2.2 Egress

In a building as old as Mechanics Hall egress requirements were not necessarily considered when it was built, which can be seen by Table 10. Because of this it had to be determined if the building needed to be configured in order to meet egress standards established in Table ***. On top of this if these egress requirements were necessary a determination had to be made on a plan in which they could be achieved. In general egress standards are classified based upon the occupancy classification, but because of the specific scenario presented both proposed uses had to conform to the requirements presented in Table 10. This was because for this specific group A classification an occupancy load of less than 50 people was considered. Given the town size, and the nature of what wanted to be done with the building a situation in which there would be more than 50 people did not seem likely. If there were a greater amount of people anticipated the egress requirements may change for certain situation.

Currently the IEBC views Mechanics Hall as a building that is up to code. This is because an existing building does not have to live up to any particular code until repairs, alterations, relocations, or change of occupancies are done. Since alterations are planned for Mechanics Hall in order to update it, Table 10 presents the areas that may need updating. Similarly to other sections first historical exceptions were analyzed. Section1203.3 of the IEBC states that if the code official thinks that the building has sufficient egress dimensions for the total occupant load then the building is up to code. This means that currently as the building stands it may be sufficient to leave it the way it is, but this is assuming that the code official deems it acceptable. Looking into solutions for the buildings egress deficiencies will provide an insurance against a code official having a negative view.

The first section that needs to be addressed is the doors. Most of the issues with the doors are not very intensive, and will not take very long to remedy. Currently not having a landing outside the front door presents a problem. One can easily be made with concrete, and is allowed to be either level or sloped per IBC 1008.1.5. The doors also need an acceptable locking mechanism per IBC 1008.1.9. A lock can easily be cut into the existing door, but it may make sense to install a new door. This would enhance security, safety, and serviceability.

The main area of concern is the stairways. Table 10 shows that currently there are not many aspects of Mechanics Halls stairs that meet code. When in the building it is evident the staircases are not very sturdy, and the level of safety is not sufficient. According to the slope and width requirements it is not possible to quickly adjust the staircases in order to bring them into code compliance. Therefore, in order to meet requirements new staircases will have to be built. This should make the building much safer to move around in. Along with new staircases exit signs need to be installed along the path of egress according to Table 10.

Once these changes are implemented the building will be fully code compliant. One strategy could be doing the changes that are not extremely expensive (everything except the stairs) and then having a code official come to try and meet a historical exception. This way he/she would see that there was an attempt to make the building safer. This along with a sprinkler system should be plenty to convince an authority having jurisdiction that Mechanics Hall is safe enough for a historic building.

6.2.4 Structural Fire Protection

For each proposed use the structural fire protection requirements will be slightly different. The main reason for this is because there is an assembly area on the second floor in one of the uses. Like with the sprinkler design this poses a challenge for code compliance.

6.2.4.1 Proposal 1

From the existing conditions it is known that Mechanics Hall falls under Type V wood framed construction. However, what is not known is whether or not the building is Type VA or VB. As specified in the existing structural fire protection section A refers to being protected and B is unprotected. Per Table 601 Type VA construction has a 1-hour fire resistance rating on certain members. In order to determine if the building needs to be protected or unprotected, Table 503 in the *IBC* needs to be consulted. Table 503 gives maximum allowable areas and heights depending on certain building usages. For an assembly occupancy Type VB construction is only allowed if there is one floor. Because the assembly occupancy is on the second floor Type VA is required. As stated in existing condition Type VA required a 1-hour fire resistance rating for all structural members per Table 601 in the *IBC*. The structural members consist of bearing walls, the roof, the floor, and the structural frame. Because it would not be realistic to replace all of these members in Mechanics Hall with fire resistant members, fire resistant paint/spray is allowed to bring the elements up to code per IBC 704.13. One option is installing sprinklers. According to Table 601, if sprinklers are installed all of the rating requirements except the exterior walls drop to 0 hours. It would be a lot easier to paint/spray the exterior walls, opposed to every surface in the building.

6.2.4.2 Proposal 2

Because an assembly occupancy is not being considered for the second proposal Table 503 was revisited. Mechanics Hall is able to be either protected (VA) or unprotected (VB) for the second proposed use. Table 503 shows that for a business occupancy both Type VA and VB construction are above the height and area of Mechanics Hall. Therefore for the second proposal there is not a requirement for fire resistant construction.

6.3 Structural

Unlike the requirements for fire protection, the two proposed future uses do not have significant impact on the structural requirements for Mechanics Hall. According to *IBC* 1607.1, the minimum live load capacity needed to support an Occupancy Class B building, such as an office space with computers or a dining room (coffee house) is 100 psf. Many recreational uses require either the same live load capacity or less, making 100 psf a good benchmark capacity so that Mechanics Hall can house a variety of potential uses. Therefore, the following section provides design solutions to ensure the floor system can safely support a live load of 100 psf.

6.3.1 Main Building

The issues with the main building discovered during the structural analysis described in Section 3.4 and identified in Section 4.2 are addressed in this section. The primary areas of concern are the condition of the joists, carrier beams failing all three critical checks (bending, shear, and deflection), and the columns with insufficient strength supported by inadequate footings. The solutions presented in this section are intended to immediately increase the strength and stability of the floor system. For design solutions intended to preserve the condition of the structural members, see Sections 6.3.3 and 6.6.

6.3.1.1 Joist Condition

Many joists in the original building have sustained significant damage that compromises their ability to resist load. A total of 51 joists were identified as inadequate. Fourteen joists, primarily in the south-west corner, were identified in the south bay. Ten beams in the middle-south bay and six beams in the middle-north bay also need replacing. Every beam in the north bay needs to be replaced on account of the notches at the ends of the spans, as discussed in Section 4.2.2 and seen in Figure 13. These beams should be replaced by Northern Red Oak joists of the same size and cross section.

6.3.1.2 Carrier Beams

The carrier beams were determined to fail primarily in bending and deflection. Because the beams are attached to the floor deck above, and because the joists are flush with the face of the beams, the decision was made to reinforce the existing beams rather than replace them. This will allow the current joists that do not need replacing to remain in place.

To adequately reinforce the beam, a steel plate should be bolted to the bottom of the beams. The cross section should be 8 inches wide by one half inch in depth, and the material should be grade A36 Steel, as seen in Figure 29. The plate will be secured to the beam by 7/8" diameter lag bolts spaced every 12.5" on center, as depicted in Figure 30. The steel sheets will require 15/16 inch diameter holes to allow for adequate space for the lag bolts.

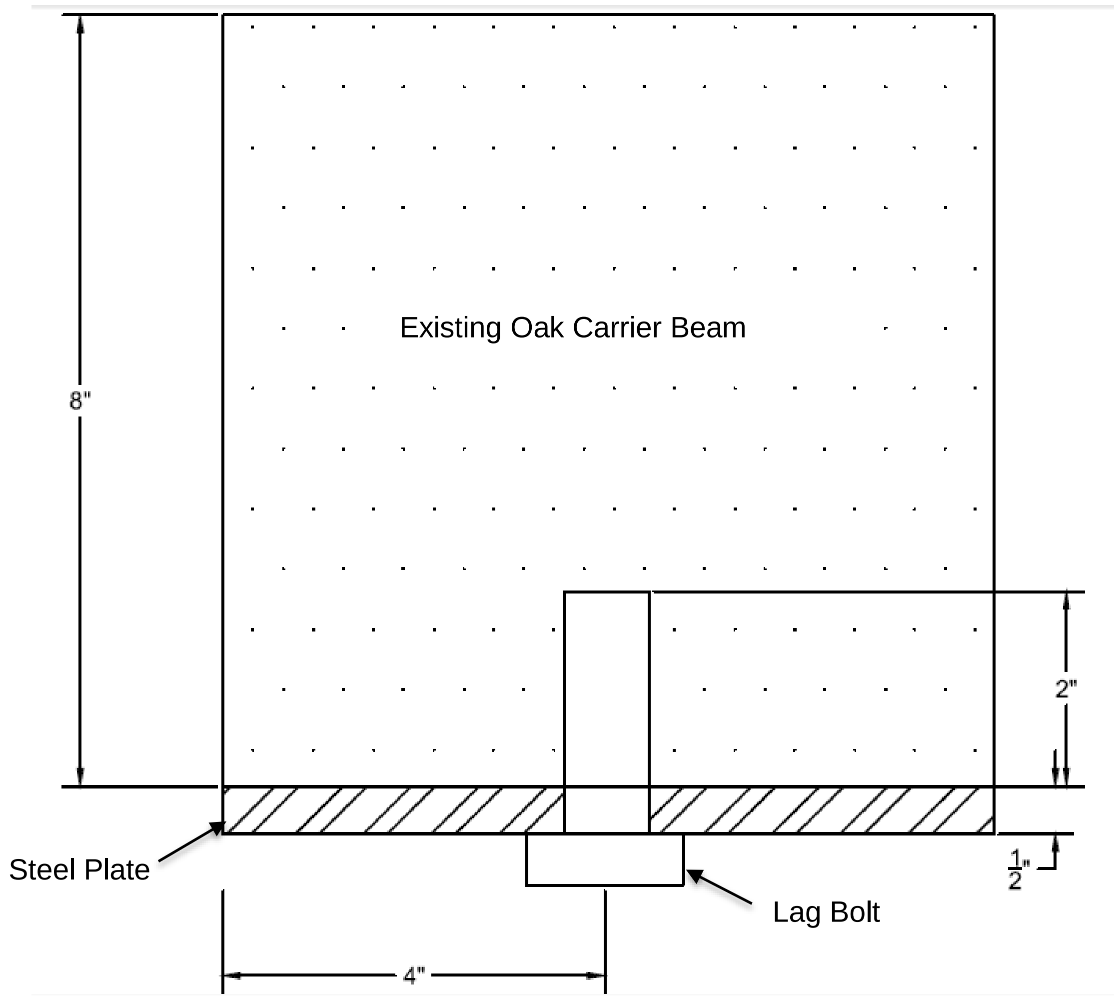

Figure 29: Composite Beam Section View

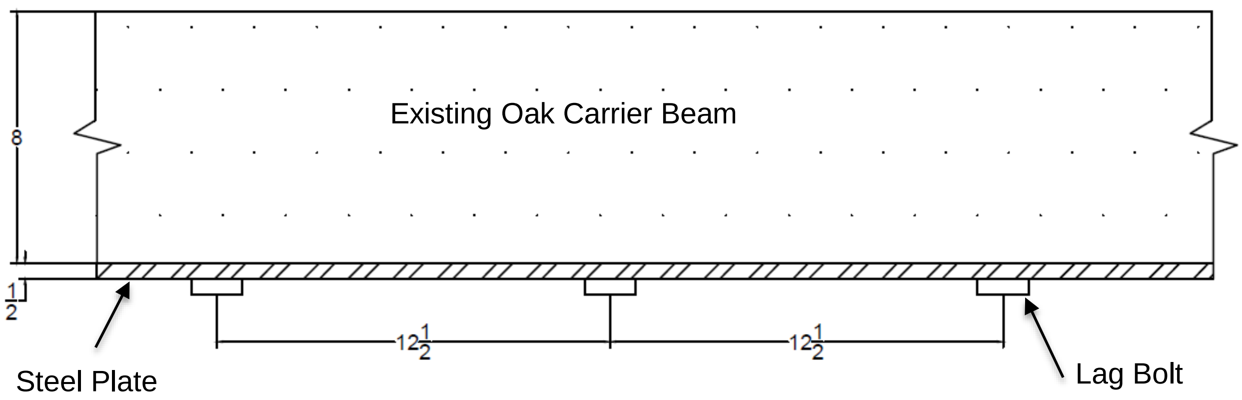

Figure 30: Composite Beam Profile View

The cross section for this beam was designed using the method of composite sections, as described in Section 2.2.2.2. The calculations for this design can be found in Appendix F.

6.3.1.3 Columns

The four wooden support columns have sustained damage as identified in Section 4.2.3 and can no longer carry the required load. The footings for these columns are also inadequate. To solve these issues, the old columns should be removed and replaced, and new concrete footings for the columns should be poured.

The replacement columns should be square Northern Red Oak beams with a 10” x 10” cross section. These columns will have a larger cross sectional area than the existing columns, which will allow them to support the required loads. At the same time, the dimensions are similar to the existing structural members, and will allow for easy installation. It is important to note that these given dimensions are exact measurements, not nominal. When ordering a 10x10 timber beam, the actual dimensions will be less than 10” x 10”. Installing a nominal 11x11 may be necessary to provide the required cross sectional area. Figure 31 depicts the base of the new column as it meets the new footing.

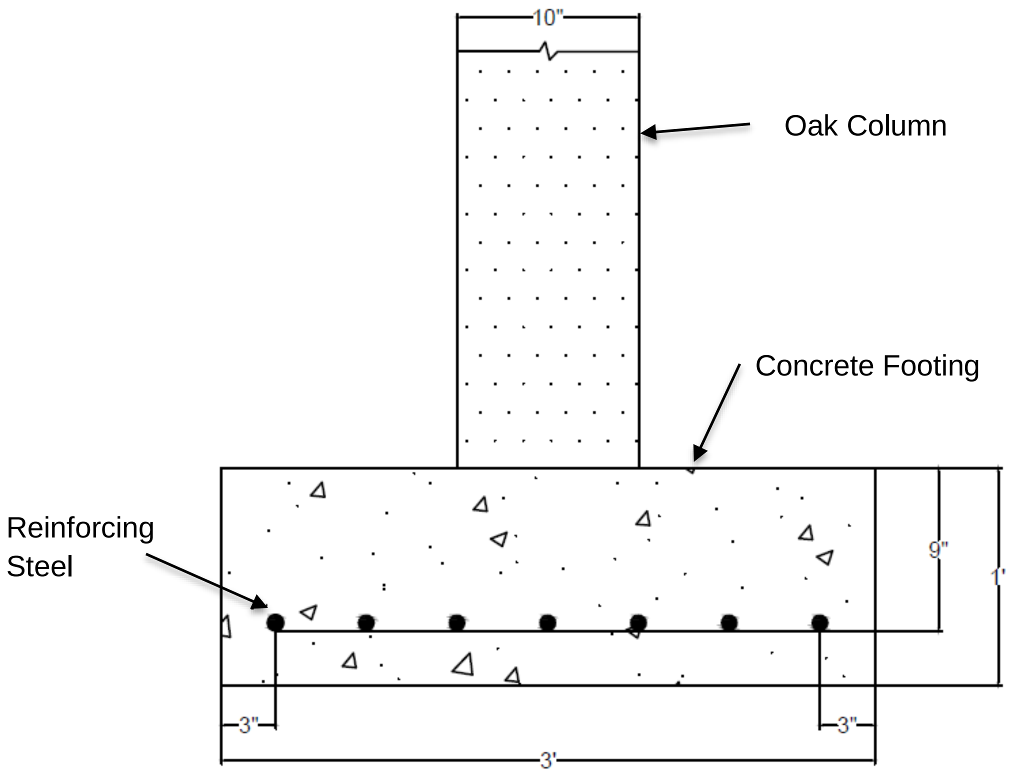

Figure 31: Column and Footing Elevation View

The new columns will require new footings to adequately disperse the load to the earth. The existing footings are insufficient for this purpose. The new columns should be made of reinforced concrete. In accordance with IBC 1809.4, the base of the footing must sit a minimum of 12 inches below the existing grade. For Mechanics Hall, this minimum will suffice. The footing should be a square 36" x 36" x 12" deep centered beneath the column. Due to the minimum required reinforcement needed, No. 7 rebar must be placed at a depth of 9" running in both directions to form a grid, as depicted above in Figure 31. The rebar should be spaced every 5" on center, with no rebar placed within 3" of any edge of the footing. This is shown below in Figure 32, with the red dotted lines indicated the placement of the reinforcing rebar. The supporting calculations for the column and the footings can be found in Appendix D and E respectively.

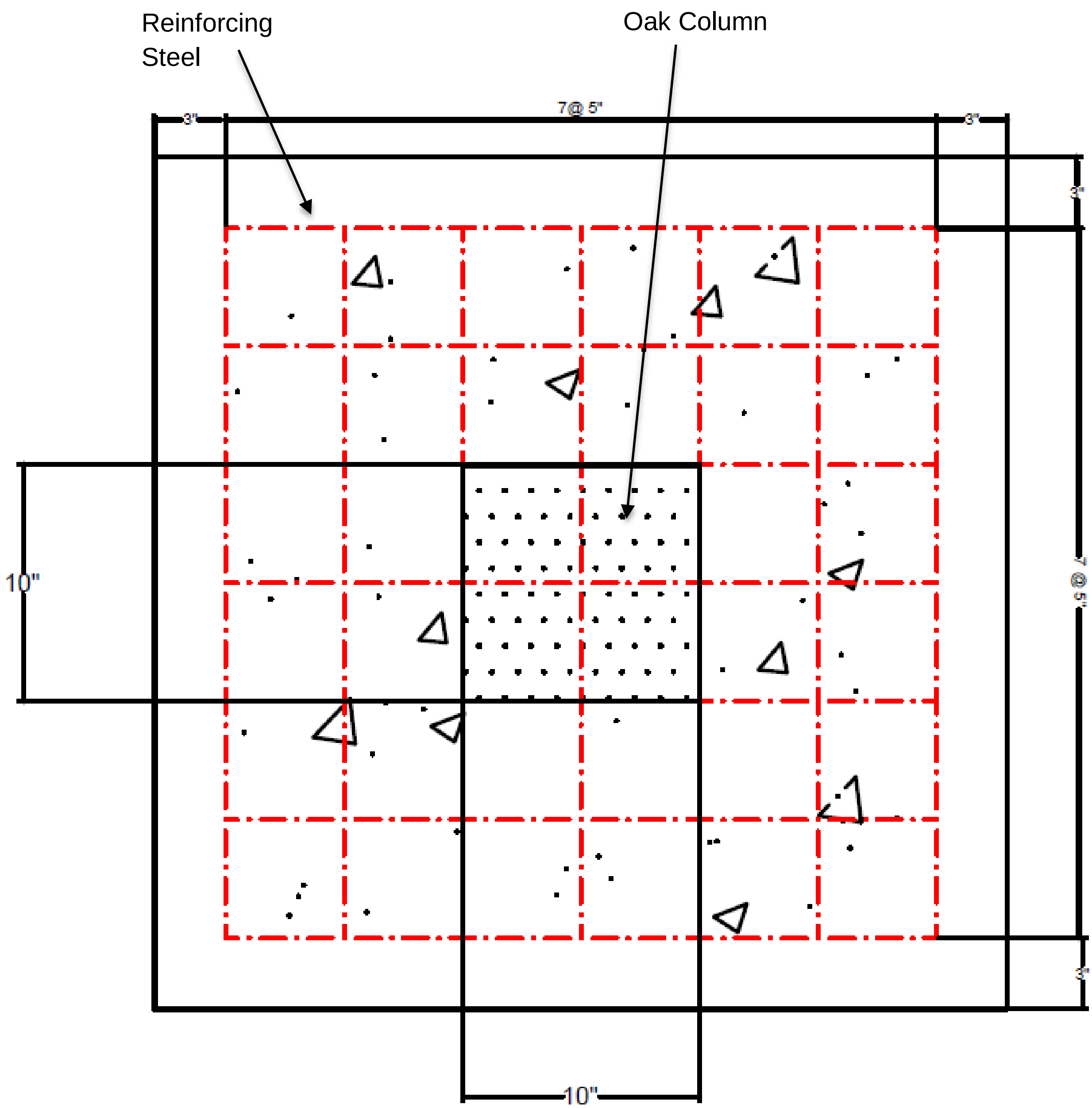

Figure 32: Reinforcement Distribution and Spacing for the Column Footings

6.3.2 Addition

The Addition has a significant problem with the strength of the structural members. The joists and the carrier beams are insufficient to carry the loads specified by the IBC. As such, the existing members must be replaced or reinforced. The structural frame of the Addition is also exposed to the outside through holes in the sideboards caused by rotting.

6.3.2.1 Joists

The joists in the Addition lack the required strength to resist moment. This is a result of the depth of the beam being too shallow. The existing joists should be replaced with Northern Red Oak beams 2" wide and 10" deep. This will provide the extra strength needed to allow the joists to adequately resist bending stresses. The design calculations for the joists can be found in Appendix D.

6.3.2.2 Carrier Beams

The carrier beams require strengthening to account for the required bending and shear stresses. To comply with the IBC, a Northern Red Oak beam with an 8”x10” cross section should replace the existing beam. These dimensions, like the columns discussed in Section 6.3.1.3, are exact dimensions, and a beam with larger nominal dimensions may be required. The original span and cantilever length will remain the same. Due to the increase in depth of these beams as well as the joists, the columns supporting the frame must be cut to be 4 inches shorter than the existing column. The design calculations and the strength checks for the columns due to the additional loading can be found in Appendix F.

6.3.2.4 Exterior Sideboards

The exterior sideboards have rotted through due to exposure to the damp earth. This has caused holes to form in the exterior, which allows water into the basement, causing damage. The sideboards should be replaced. It is important to note that due to the changes of the structural frame, the carrier beams extending from the interior of the cantilever will sit 4” lower than the existing beams; the holes in the new siding must accommodate these alterations.

6.3.3 Foundation

The substructure for the entire building must be improved for Mechanics Hall to comply with the requirements specified in IBC Chapter 18. As the original building was built with a different foundation than the addition, different approaches were taken to improve the existing foundation.

6.3.3.1 Original Building

As discussed in Section 3.6.5, Mechanics Hall meets the necessary criteria to retain the existing field stone foundation. However, due to the condition and slenderness of the foundation wall, repairs and reinforcement must happen for the foundation to comply with *IBC*.

Every hole in the foundation must be filled with 3000 psi concrete. A reinforced concrete wall should be cast on the inside of the field stone. The wall should be 3” thick, and should be reinforced with a 6 X 6-W4.0 X 4.0 welded wire reinforcement placed no closer than ¾” from the interior face of the wall, and no closer than 1.5” from the field stone. The concrete should fill every gap in the existing foundation wall so adequate transfer of stresses occur. The concrete should be cast monolithically. The 3 inches of concrete will provide enough cover to prevent the reinforcement from corroding. This will provide a full depth foundation wall thickness of 12”. The reinforcement satisfies the requirements specified by IBC 1807.1.6.2. Supporting reasoning and calculations can be found in Appendix F. Figure 33 illustrates a section view of the reinforcing wall acting with the existing foundation. The dotted red line indicates the steel wire mesh placed greater than ¾" from the basement interior and greater than 1.5" from the existing fieldstone foundation wall.

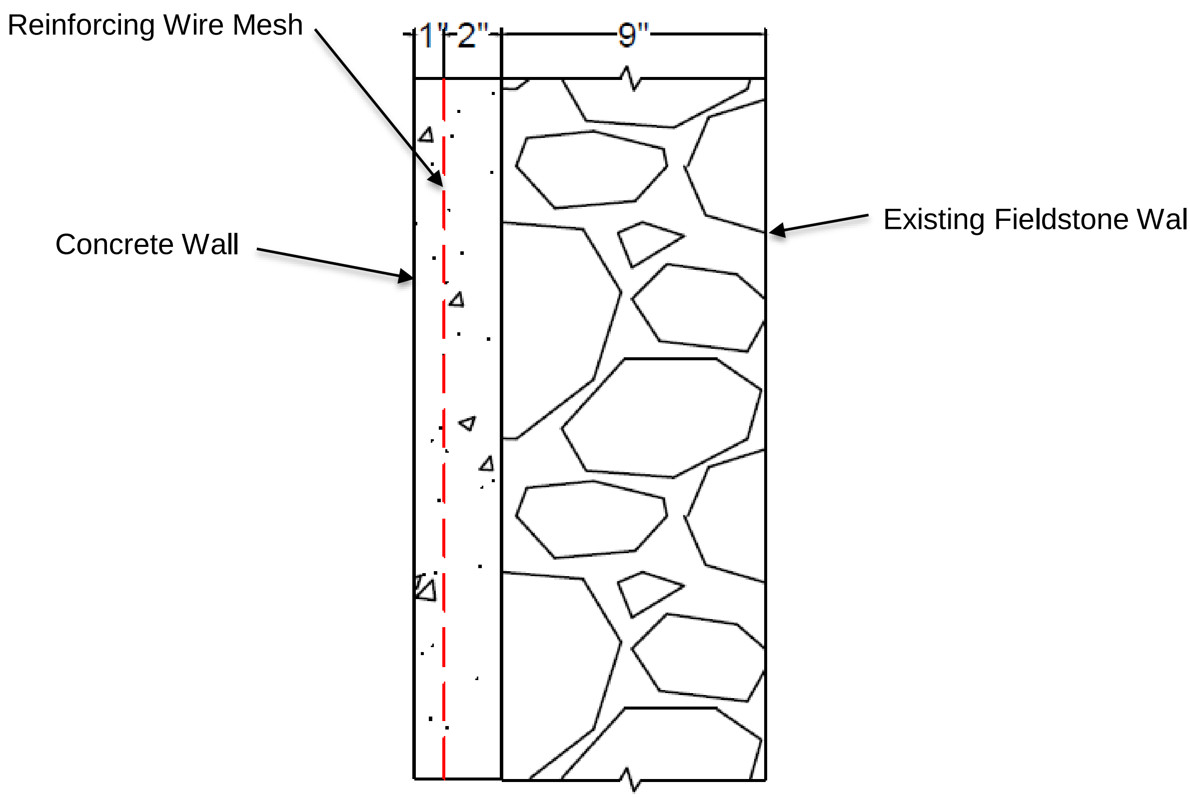

Figure 33: Section View of the Reinforcing Foundation Wall

To reduce moisture in the basement, it is recommended to place a slab-on-grade in the basement. This will serve as a moisture barrier and a smooth floor. A 2" thick slab of 2500 psi concrete should minimize the drainage issue in the basement and preserve the structural integrity of the building frame.

6.3.3.2. Addition

A new foundation for the addition was designed to support the required loads from the first and second floor of the building and to protect the columns and sideboards from moisture and rot. To comply with IBC 1809.5, a concrete wall is required to ensure that the footings for the building are placed below the frost line to prevent frost-heaving. According to *Massachusetts State Building Code* (780 CMR 5403.1.4.1), all footings must extend at least four feet below the existing grade to protect against frost. To do this, the area underneath the addition walls must be excavated to a depth of 4 feet. Starting at 4 feet below the existing grade, a concrete footing measuring 16” wide by 1 foot thick should be placed. On top of the footing, an 11.5” thick by 3 feet tall concrete wall should be cast. All concrete should be rated at a compressive strength of 3000 psi. No steel reinforcement is required for this foundation. Design calculations can be found in Appendix F. A section view of the proposed wall and footing can be seen in Figure 34.

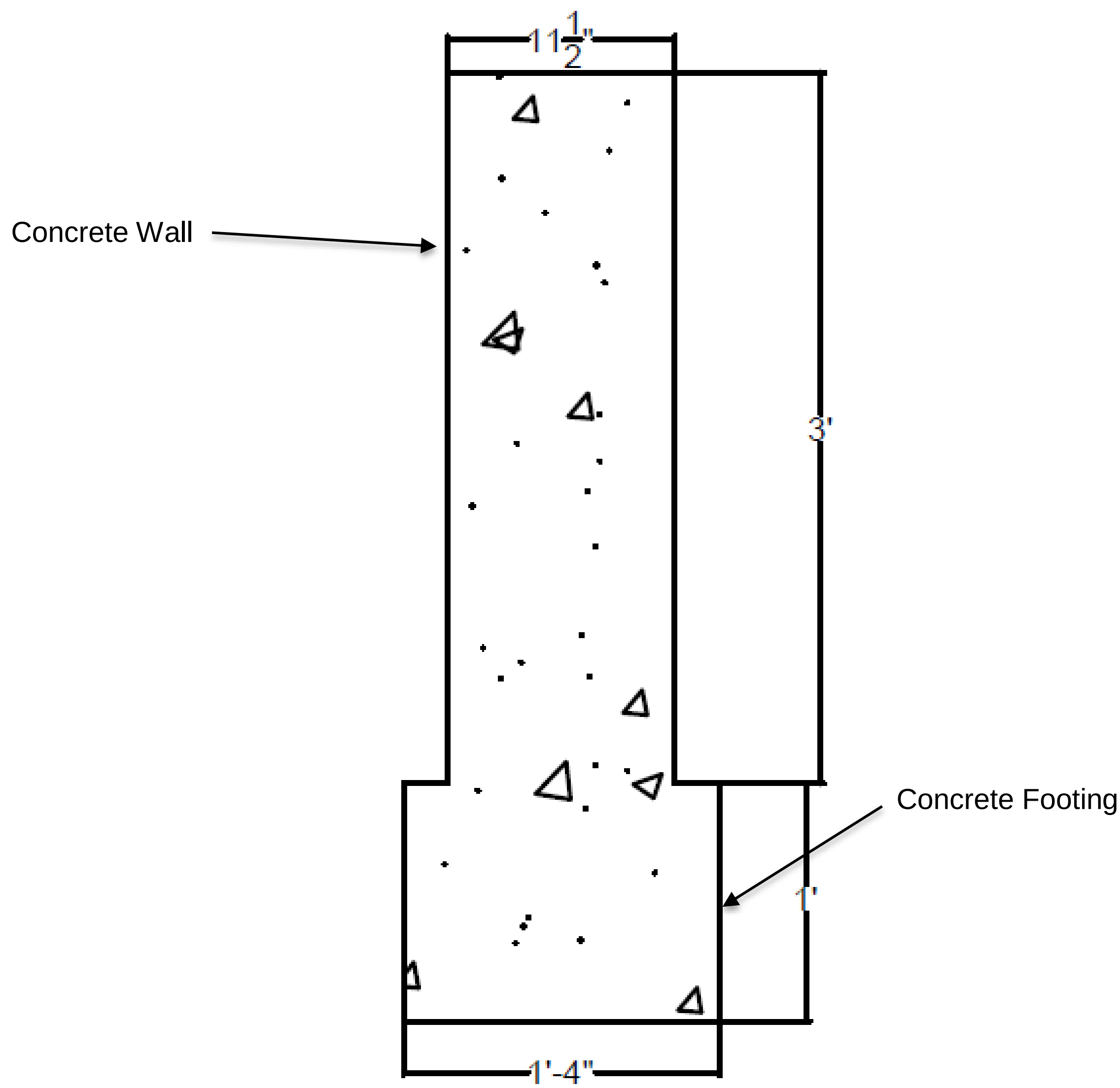

Figure 34: Section View of the Proposed Strip Footing Supporting the Addition.

6.4 Accessibility

After analyzing the historic requirements section in the *IEBC* Mechanics Hall does not pose a requirement for further accessibility features. Because of the layout of the building a handicapped bathroom may not be possible without extensive work to the building. Therefore, one is not going to be specified for either use. However, because a landing is going to be installed for egress requirements it would make sense to install a handicapped ramp attached to the landing. If this were done the only missing component would be the bathroom. Because the handicapped ramp was installed it would show that some effort was put into handicap accessibility. If effort is put into areas such as this it will help with the approval by the code official. For example, if a code official sees something that he is weary about, he/she may take the installation of the handicapped ramp into consideration. It is important to remember in the end the goal is to get the code officials approval for the use that is desired.

6.5 Parking

According to the Town of Princeton Zoning Bylaws Chapter 2, Mechanics Hall is located in a Business District. Therefore, any special parking requirements would be found in Chapter 4 Business District. Chapter 4 does not have any specific parking requirements, so the requirements from Chapter 7 General Regulations should be used. Chapter 7 Section 3 states that the parking for any building must be adequate for the customers' needs. However, according to Chapter 4 Section 3, any building in a business district that is externally enlarged must have a site plan approved by the town Planning Board. Therefore, while there is some room to expand parking, particularly on the east side of the building, parking expansions will have to be approved by the Princeton Town Planning Board.

6.6 Drainage

To solve the drainage issue there are a few ways to approach the problem. The most conventional way to deal with leaking foundations is to excavate from the outside to install a perimeter drain, and install a drainage layer over waterproofing on the outside. This method tends to be expensive because of the amount of excavation that would need to be performed. Another solution that would be less expensive is to install an interior French drain. In Figure 35, shows a section view of an interior French drain system and its components.

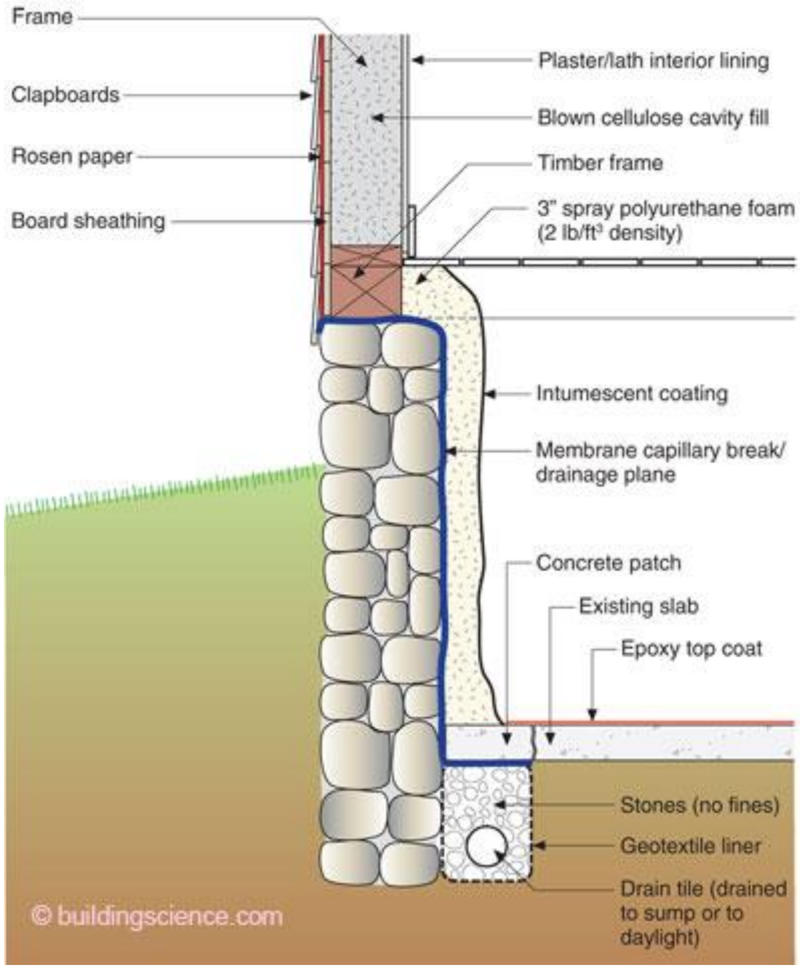

Figure 35: Interior French Drain (Lstiburek, Interior Perimeter Drain)

This trench is lined with a geotextile filter fabric and contains a perforated drain and gravel. The drain pipe is either connected to a sump pump that pumps the water out or the pipe is directly extends out of the basement to daylight. Also, installing a sheet of polyethylene butyl composite liner to the interior perimeter of the foundation will create a barrier that does not let moisture into the basement. Additionally a layer of polyurethane foam (spray insulation) should be applied to the foundation walls. This foam insulation acts as a water barrier and it helps insulate the basement and also, and intumescent coasting could be applied to the foam as a fire protection barrier. To maximize the amount of water kept out of the basement a slab should be placed over the existing floor. Either a sheet of polyethylene butyl composite liner should be placed under the slab or an epoxy top coating of paint applied to the surface of the slab will prevent vapor transmission into the basement.

6.7 Cost Estimate

6.7.1 Fire Protection

The fire protection cost estimate was broken into two major sections, which included egress and automatic sprinklers. Towards the end of this section there will be an additional miscellaneous fire protection cost that takes into consideration the other aspects of fire protection. A sprinkler system will be supplied because it will make the egress, and structural fire protection costs less expensive.

6.7.1.1 Automatic Sprinkler System

The sprinkler system cost will only be considered for proposal 1. Proposal 2 does not require a sprinkler system and therefore will not have any of these added costs. In order to determine the cost of the fire protection system organization was crucial. Using the sprinkler layout an inventory of all necessary elements was prepared and can be seen in Table 14. Table 14 includes all anticipated items, and was only intended to give a general estimate. Using a sprinkler pricing software provided by Cogswell Sprinkler Co.,Inc. an estimate was prepared. One benefit of using a contractors pricing software, is it depicts a more accurate cost than library references. The software used by the contractor automatically includes a price for labor, and an added percentage for profit.

Table 14: Sprinkler System Inventory

Element	Number	Size	Length
Wet Upright Sprinkler	39	N/A	N/A
Dry Uprights	7	N/A	4' Barrels
Horizontal Main	N/A	2.5"	91.5'
Vertical Main	N/A	3"	25.3'
Line Pipe	N/A	1.5"	10'
Line Pipe	N/A	1.25"	74.5'
Line Pipe	N/A	1"	281'
Elbow	38	1"	N/A
Grooved Tee	3	3"x3"x2.5"	N/A
Grooved Elbow	4	2.5"	N/A
Tee	18	1"	N/A
Tee	11	1.25"x1"x1"	N/A
Tee	2	1.5"x1.25"x1"	N/A
Valve and Trim	Normal Wet Valve and Trim		
Pump with Valves	1	250gpm Vertical Inline	N/A
Fiberglass Tank	1	8000 gallons	N/A

The final outcome of the pricing software came out to $94,000. This total does not include the costs for excavation. Extensive excavation will be required to install a tank, and any piping needed from the tank to the pump. According to a cost evaluation software for excavation called *homewyse* it will cost around $400 per cubic yard of excavation. Another added cost not included is the pump house. Taking these two factors into consideration a final price of around $100,000 would seem reasonable. This price may change depending on if any unforeseen factors arise.

6.7.1.2 Egress Requirements

Both proposed uses will incorporate the suggested changes to bring the building up to code in terms of egress as stated in the proposed egress section. Because a historical exception is met the egress changes are not completely necessary, however for the reasons stated in previous sections the cost of the changes was furnished. The main cost for this section is the addition of new staircase within the building. Mechanics Hall has four staircases that need to be replaced and brought up to code. A cost estimating service called FIXR was used in order to determine the different steps involved in installing a new staircase, determining prices, and estimating time for each step. One of the first necessary tasks is to demolish the existing staircases. This can be done without professional help, but great care needs to be taken in order to ensure that important structural elements are not being destroyed. However, if professional help is desired then it can be assumed one staircase will take around eight hours. According to FIXR the average carpenter rate is $45 per hour. Therefore four staircases will cost around $1400. For Mechanics Hall a professional may be the desired approach because the stairs need to be widened, and walls will have to come down.

Depending on the type of staircases different costs will be associated. The average price of a premade staircase is around $800. Therefore four staircases will cost around $3200. Based upon the desired outcome new walls may have to be constructed, and the staircase will have to be installed. Therefore another price of $1000 will be added to the cost. The rest of the scope of work such as finishing, and painting can easily be done by the owner and will help save cost. Another Egress Cost will be a new door. Currently the major home improvement stores have exterior doors listed around $200. Using these prices the final cost for egress requirements is $5800. In order to be conservative, and account for unforeseen costs $6000 is reasonable.

6.7.1.3 Miscellaneous Costs

In order to bring the building into code compliance for structural fire protection it makes the most sense to use the fire resistive paint specified in the proposed section. This cost will only apply to the first proposed use, but because a sprinkler system is specified the interior walls do not have to be considered. Therefore, all that is needed is enough paint in order to cover the exterior walls. The exterior walls have a surface area of 4472 square feet. Therefore the coverage per pail needs to be divided from the total surface area to get the needed amount of pails. There are multiple different companies who supply paint such as this. The manufacturer Grainger states that a 5 gallon pale costs around $400. Grainger states that a 5-gallon pale will cover 1500 square feet of wall. Using these numbers 3 pales or 15 gallons is required. This comes to a cost of $1200 excluding professional help. Another cost will be smoke detectors and fire alarms. These do not cost much, and will be covered by the conservative prices already established. In the end

6.7.1.4 Final Fire Protection Cost

When all of the above costs are brought together a final cost estimate can be seen in Table 15. A star next to the cost denotes that element is not completely necessary.

Table 15: Fire Protection Cost

Element	Proposal 1	Proposal 2
Automatic Sprinklers	$100,000	$0
Egress Requirements	$6,000	$6000
Structural Fire Protection	$1200	$0
Total	$107,200	$6000

6.7.2 Structural and Foundation Cost Estimate

This section contains a cost estimate of the materials required to perform the work specified in Section 6.3. The estimated costs are conservative, but do not include the cost of transportation or labor.

Structural Members

The costs associated with the structural members are presented in Table 16. The figures for the structural members are intended to represent the cost of the material only. Due to the nature of the repairs, significant labor costs will be associated with the installation of some members due to the need for temporary bracing and jacking of the building.

Table 16: Structural Members Cost

Structural Member	Quantity	Cost ($)
2”x8”x9’ Joist	459 LF	950.00
10”x10” Column	26 LF	335.00
8” x ½” x 13’ Steel Plate	6	1,320.00
¾” Lag Bolts	78	235.00
2”x10”x10.33’ Joists	495.84 LF	1,500.00
8”x10”x15’ Carrier Beam	60 LF	735.00
Total Structural Cost	--	5,075.00

Foundation and Footings

The costs of the foundation and footings are the costs of the materials and excavation only. There will be other labor costs associated with the renovation of the foundation, such as the transportation of the concrete from the mixing plant to Mechanics Hall in a mixing truck.

Table 17: Foundation and Footings Cost

Material	Quantity	Estimated Cost ($)
Concrete	4.84 CY	1,300.00
Formwork	704.4 SF	710.00
Welded Wire Web	247 SF	30.00
Rebar	140 LF	280.00
Excavation	14.25 CY	5,700.00
Total Foundation Cost	--	8,020.00

6.7.3 Drainage Cost Estimate

The cost of a French drain can vary widely depending upon the soil conditions. The length of the drain, the depth and the width of the trench are also factors. Usual drain systems are charged by $/per linear foot. Typical costs for an interior French drain system range between $2,000 and $15,000 dollars depending on the amount of work being performed (CostHelper). With the amount of work that needs to be completed in Mechanics Hall, it is assumed that it could cost up to $10,000 dollars to have a complete waterproofing system.

6.7.4 Accessibility Cost

As specified in the proposed section a handicapped ramp would be a good feature to add. Because the landing cost was specified in the egress cost with a conservative value cost of the handicapped ramp is considered to be included in that price. Therefore, there will not be any cost specified for accessibility.

6.7.5 Final Cost Estimate

Table 18: Final Cost Estimate

Element	Proposal 1	Proposal 2
Fire Protection	$107,200.00	$6000.00
Structural	$5,075.00	$5,075.00
Foundation	$8,020.00	$8,020.00
Drainage	$10,000.00	$10,000.00
Total	$130,295.00	$29,095.00

Conclusions and Recommendations

The goal of this Major Qualifying Project was to determine the necessary steps to bring Mechanics Hall into compliance with the International Building Code so it could safely open to the public while remaining on the National Register of Historic Buildings. The results of this study were presented to the Friends of Mechanics Hall. To do this, two separate uses for the building were proposed and examined. Because building code requirements are often based on the use of the building, comparing two different uses meant different requirements had to be met. This allowed multiple available renovation options for the Friends of Mechanics Hall. The scope of work on this project was limited to fire safety, structural, drainage, accessibility, and parking concerns.

The initial work performed was an inspection of Mechanics Hall. This inspection resulted in the creation of plan drawings and a 3D model of Mechanics Hall. Extensive data was gathered to allow for proper analysis of fire safety and structural concerns. Examples of collected data include doorway width and height, stairway steepness, railing heights, and the dimensions of structural members.

The gathered information was used to perform an analysis on the building. The data was used to summarize the existing condition of Mechanics Hall with respect to *the International Existing Building Code* and *International Building Code*. Using the requirements specified in this code, fire safety and structural issues were identified. A code review was compiled to organize every section of code used to conduct the analysis. Other issues of concern were identified during the building inspections, and include drainage, accessibility, and parking issues.

Once the issues preventing code compliance were identified, design solutions were proposed to return Mechanics Hall to a safe condition. These solutions vary depending on the proposed use of the building. The largest difference involves fire safety. In the first proposed building use, a sprinkler system would be necessary due to occupancy concerns on the second floor stage area. However for the second proposed use, merely limiting the maximum occupancy of the second floor would eliminate the need for a sprinkler system. Other design solutions include the design of new foundation walls and structural members to allow the building to adequately resist the required live loads. Proposed solutions included improved drainage systems to prevent moisture from entering the basement and damaging the structural members. Solutions regarding accessibility were examined to make Mechanics Hall ADA compliant.

The results of this study were presented to the Friends of Mechanics Hall, a non-profit group who is looking to rehabilitate Mechanics Hall. This study is intended to portray the amount of work that would be necessary for the Friends to achieve their goals.

There are limitations in this study that prevent the results from being a comprehensive list of repairs needed to renovate Mechanics Hall. The scope of work was limited by the expertise of the project group. As such, fields critical to the function of a building, such as HVAC, plumbing, and electricity, were not considered. Another significant limitation was the limited equipment available for use during the building inspections. The lack of equipment limited the availability of soil data. The characteristics and strengths of the soil were approximated using tools provided by the USGS. This means that the proposed design solutions for the foundation walls and footings may be very conservative.

There are areas available for future study in Mechanics Hall. As mentioned in the previous paragraphs, heating and ventilation systems were not considered due to lack of knowledge in those fields. An area of study for an architectural engineer could include methods of insulating and heating Mechanics Hall. The entire building envelope was not examined as a part of this study. Another area of study could be upgrading or replacing the existing plumbing and electrical circuits in Mechanics Hall. And another opportunity for study would involve the construction processes of implementing the repairs discussed in this report. Construction management is critical to turning a design into a reality, and Mechanics Hall offers some unique management challenges.

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Appendices:

Appendix A: MQP Proposal/ Deliverables

Rehabilitation of Mechanics Hall

Project # LDA1402-A13-C14

A Major Qualifying Project Proposal By

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Abstract

Mechanics Hall is an important historical building to the town of Princeton, Massachusetts. The goal of this project is to help the people who hope to see a building that has been part of their town for over a century put to good use. Unfortunately in our current economic state, price is extremely important. Because of this it is vital to put forward ideas that are both economical and useful for the community. We will provide a restoration plan for two suggested uses to aid the Friends of Mechanics Hall realize their goal of renovating Mechanics Hall.

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Section 1: Introduction

Mechanics Hall is situated in eastern Princeton, Massachusetts, a short distance from the WPI campus (Figure 1). It has a wealth of history, having been used for many different purposes over its 161-year lifetime. When Mechanics Hall was being constructed in 1852, wood was salvaged from a collapsed building in Princeton and used for construction. This salvaged wood is dated to 1843, and is the reason for this date appearing on the front of the building. Originally it was used as a school and served that purpose until 1945. During this span it was also used for community gatherings, lectures, and banquets. Since then it has been used as everything from an extra library to an American Legion post. Because of the building's history, there are many groups that are extremely interested in its preservation and continued use. These include The Princeton Farmer and Mechanics Association, The East Princeton Village Improvement Society and The Friends of Mechanics Hall. All of these organizations have helped with the usage and upkeep of the building. Currently Mechanics Hall is listed on the National Register of Historic places adding to its importance throughout the history of Princeton.

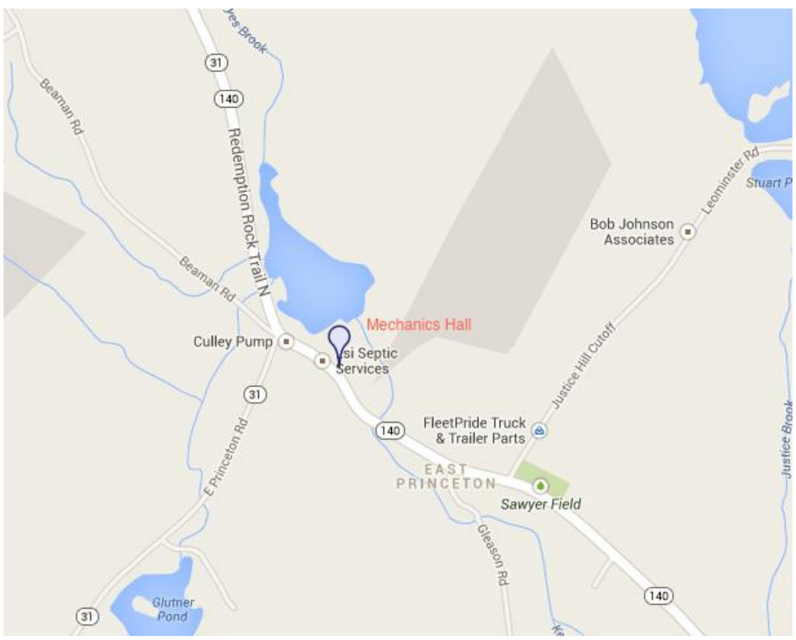

Figure 1: Mechanics Hall Locus Map

Mechanics Hall can be classified as a Greek-Revival building. The pillars and pediment out front can attest to this. It also has the classic look of a front gabled building. It rests on a 0.3-acre rectangular plot of land and has an approximate floor area of 3200 square feet (Figure 2). Sometime after the building was built, there was an addition added onto the back that houses a kitchen, and an extra back room. It is a two-story building with a banquet hall on the second floor. The second floor is a great area for function because of the kitchens location. The first floor is made up of three rooms including the back addition. There are also two bathrooms within the building. Because of its age many of the mechanical systems within the building need a lot of updating. Plumbing and heating are currently nonexistent. The electrical configuration in the building works, but is extremely outdated. Because of the buildings age the structural integrity and code compliance need to be further investigated.

Figure 2: Mechanics Hall Front

Preservation Massachusetts listed Mechanics Hall as one of Massachusetts Most Endangered Historic Resources in 2012. This is exactly where our project team comes into play. We plan on looking into the building, and assessing some of the above problems associated with it as well as any additional found along the way. This is further explained in the scope of work. Along with assessing the issues associated with the building we want to propose some viable uses for Mechanics Hall, and the design descriptions cost estimates for each proposed use will also be prepared. A report on how much this will roughly cost will also be provided. This information will be presented to the friends of Mechanics Hall, and hopefully provide background to raise the needed funds.

Section 2: Background

Before beginning work on renovating Mechanics Hall, it is important to research the issue at hand. An understanding of the historical aspect of the building as well as engineering is necessary to develop an appropriate restoration plan.

Section 2.1: Historical Value

*Note:(At the time of writing this section limited information was available due to the Government shutdown, information is subject to change once the information is able to be accessed.)

The current state of Mechanics Hall features many problems that threaten the future of the building, such as structural issues, faulty wiring, water infiltration and vandalism. The Friends of Mechanics Hall was established and set out a list of short and long-term goals for the preservation of Mechanics Hall. They hope that endangered listing will help to educate the public about the importance of Mechanics Hall, what it means to their town, and their connections as a community.

In 2004, a committee surveying Princeton found 97% of residents were interested in seeing the building put to good use (UTAC, 2012). East Princeton, including Mechanics Hall, was accepted by the National Parks Service for inclusion on the National Register of Historic Places in September 2004. The National Register of Historic Places is the list of individual buildings and districts, which are important in American history. It is a federal designation and is administered by the Secretary of the Interior through the Massachusetts Historical Commission as the State Historic Preservation Office (UTAC, 2012).

As a municipal building Mechanics Hall qualifies for a 50/50 matching grant from the Massachusetts Historical Commission for renovations. The National Register does not restrict a property owner's private property rights. The owners of National Register properties are able to remodel, renovate, sell, or even demolish the property with no restrictions. However, significant modifications could result in removal from the National Register. Furthermore, the federal government does regulate alterations to historic properties where federal funds have been invested. If the owner has not received federal grant funds or federally sponsored tax benefits then there is no federal restriction on the property owner. The same applies at the state and local level. Recipients of state or local funds or tax benefits to preserve their historic property may be subject to design review for any alterations (Galvin, 2012).

Section 2.2: Building Analysis

Section 2.2.1: Field Inspections

An integral part of the work that will be performed on Mechanics Hall will be on-site inspections. The purpose of the inspections is to gather data regarding the dimensions and conditions of the structural members that support the building, and to identify issues that must be fixed before the building can open to the public. The ASCE (American Society of Civil Engineers) Guideline for Structural Condition Assessment will be consulted as an aid to the inspection process, and the building will be analyzed according to the Massachusetts Building Code.

Section 2.2.2: Structural Systems

The ASCE Guideline for Structural Condition Assessment highlights potential structural issues that should be noted during a typical building inspection. The critical issues expected to be found in Mechanics Hall are damages due to moisture, mold, and load duration (ASCE 2000). As the moisture content of the wooden members change, the physical properties and dimensions of the wood will change. If the wood becomes too dry after installation, the beams will shrink and crack, resulting in a reduction of capacity. If the wood becomes too wet after installation, the wood may become soft and malleable. Water damage will be inspected using a combination of visual inspection and physical testing of a member to identify soft areas.

Certain types of molds and mildew will also be a concern, as these organisms will grow on wood structural members. These organisms feed on the wood, which deteriorates the structure and reduces the capacity of each member.

Load duration is a concern as the wood members were installed in the mid-1800s, and are likely not strong enough to satisfy modern building code requirements. A wooden beams capacity decreases over time when subjected to consistent loading. Figure 3 displays how the load duration factor changes with regard to the duration of the load. These values are applied to the loads when analyzing a structural member. As these beams have been subjected to consistent load since 1852, this issue could be magnified by other issues such as moisture.

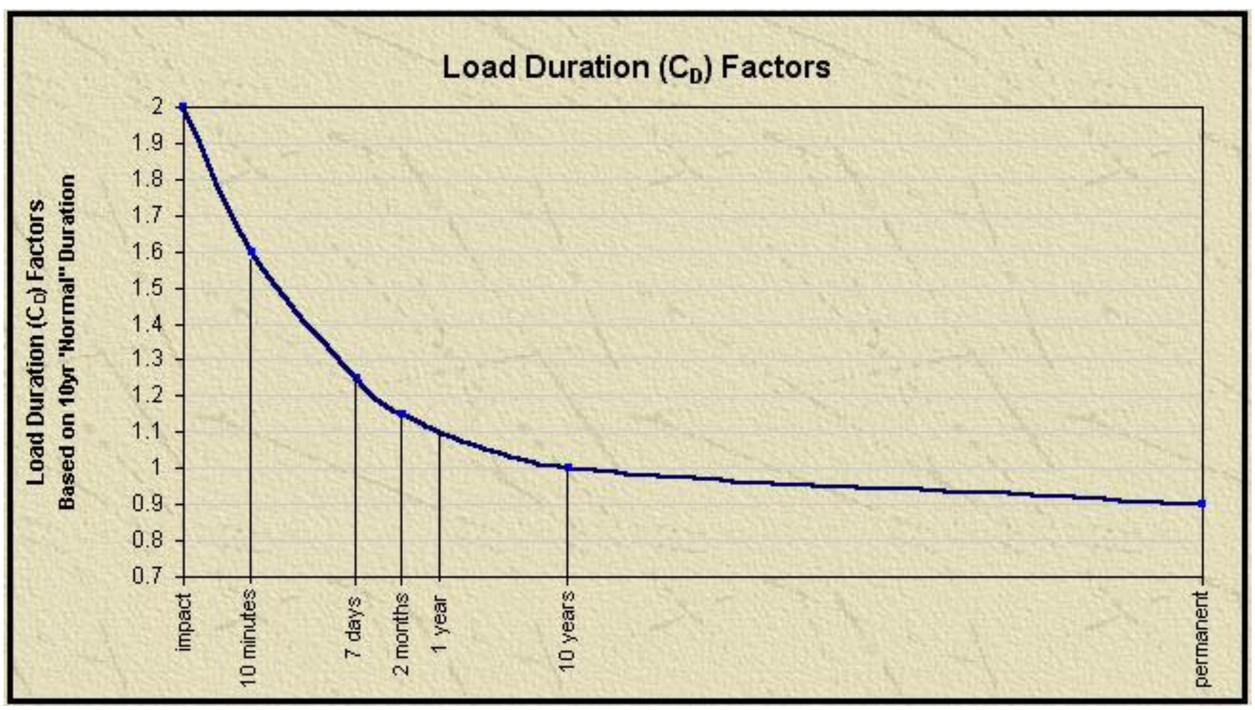

Figure 3: Load Duration Factor with respect to Duration of Applied Load (WSU, 2000)

It is important to consider Massachusetts State Building Code when renovating an existing structure. The three chapters of the building code most pertinent to structural stability are Chapters 16 (Structural Design) and 32 (Existing Structures). Chapter 16 details the different allowed loads for buildings based off of functional use. These values are tabulated in the building code in table 1606.1 (Mass, 1997). It is important to establish uses for Mechanics Hall before developing a renovation plan as the allowed stresses vary depending on building use.

Chapter 32 classifies the level of work done to existing structures into 5 separate levels, each with their own specifications (Mass, 2008). From the descriptions in Chapter 32, the work done for Mechanics Hall will most likely be Level 2 or Level 3 work. These classifications are used to ensure the safety of the structure.

Section 2.2.3: Fire Safety

A major consideration when opening a building for public use is fire safety and egress. As specified in Chapter 32 of the Massachusetts Building Code, existing buildings designed to hold 50 or more people must conform to most current fire safety and egress standards except for those which are impractical and those which would remove the building from the National Registry of Historic Places (Mass, 2005b). Important criteria to look for when considering egress are hallway dimensions, door dimensions, and door locations. The floor plan is also important to consider, as are aspects such as stair steepness and railing height.

Section 2.2.4: Other Concerns

There are major concerns that must be investigated when inspecting older buildings. These issues include weatherproofing, and insulation. For a building to become livable, it must be sealed from nature. For Mechanics Hall to open to the public, it must be properly insulated and heated to protect from the cold. Waterproofing to avoid the above mentioned moisture issues is also important. Other elements that must be considered are electrical and plumbing issues. While the installation of these facilities are outside the scope of this project, it is important to be aware of these concerns when developing a designs for the renovated building.

Section 2.2.5: Cost Estimate

Part of preparing a design solution to a problem is acknowledging the cost of the project. Until the project is complete, it is impossible to know the actual price. The costs can be estimated once the design is completed based on the amount of materials required and the type of work being performed. As part of the design, an itemized list is prepared with the every material required for the job, the amount required, and the unit price. This allows for a relatively accurate cost estimate.

Section 3: Scope

The purpose of this project is to propose a plan to renovate Mechanics Hall to meet modern building code while maintaining the historical significance of the structure. For this to be practical, the building must have a purpose. We will develop two different potential uses for the structure based on the needs of the community and cost. We will compare the benefits and drawbacks of each purpose.

Once two purposes for Mechanics Hall have been identified, the building will be analyzed in its current condition. We will focus on concerns regarding structural integrity, egress, fire safety, drainage, handicapped access, and parking. We will identify the causes of the current issues with the building and consider preventative measures for the future structure. Plumbing and electrical work will be considered, but is not the focus of our investigation.

We will use the results obtained during our analysis of the structure to identify the major issues obstructing current use. Using the current issues and potential issues that we see, we will develop a set of solutions to these problems. These solutions will be intended to resolve deficiencies in the areas mentioned in the above paragraph, as well as attempt to address any concerns that may result in future damage to the building. These ideas, as well as computer models of the entire building, will be combined into a Restoration Plan. This Restoration Plan will include the results of our analysis, calculations, computer models, a cost estimate, and an estimate of potential revenue for the two purposes developed for the building.

Section 4: Capstone Design

Mechanics Hall is a building that was constructed in 1852 in Princeton, Massachusetts, and has been abandoned since the 1980s. The structure consists of a ground floor, a second floor, and a basement, as well as a two-story addition that is cantilevered at the rear of the building. In order to re-open this building to public use, repairs and renovations must be performed to ensure that the building meets modern building codes.

In order to complete this project, intensive field investigations will be performed on site. Using data collected at these inspections, computer models will be generated and building code issues will be identified. Structural issues will be identified and solutions will be proposed along with measures to limit future damage to the structure. Building code issues will be investigated in terms of egress and fire safety, as the building must be safe for public use. Using the computer models, updated versions of the building will be compiled and presented with a new intended use for the building.

This project addresses all eight realistic constraints specified by the ASCE. This project, like most engineering projects, deals heavily with economics. The sponsor group that will be funding the project is a non-profit organization that obtains its funds through fundraising. As such, the renovations proposed for this building will be very sensitive to cost. Our design will have to be the most economical design possible while still fulfilling the design requirements.

This projects addresses health and safety issues through the consideration of building code concerns. Different aspects of this project address structural issues and fire safety issues identified during the building inspections. Other issues include, insulation, heating and ventilation, drainage, and electrical and plumbing concerns. All of these issues affect the health and safety of those who will utilize this building.

This projects addresses constructability through the proposed restoration plan. If there are structural issues found with the framing or with the foundation, these issues will have to be addressed. Constructability will be addressed when developing the design solutions for these issues to allow for ease of construction.

This project addresses environmental concerns through the consideration of drainage issues. Due to potential water damage to the structure, preventative measures may have to be installed to redirect water runoff. This raises concerns of potential local wetland issues.

This project addresses sustainability in the form of historic context and environmental concerns. The goal of this project is to renovate Mechanics Hall to modern building standards while maintaining the historical feel of the building. As much of the original building will be preserved, minimizing the impact of producing new materials.

This project is subjected to ethical concerns due to the heavy burden of keeping the cost low. It can be difficult to properly design a safe restoration plan for buildings as performing the necessary work can be costly. While it is important to keep the design economical, the safety and integrity of the building design or construction cannot be compromised. At the same time, it is important to control the scope of the project to areas within our area of expertise. It is important to keep within our background and to keep from advising in areas where we have no background.

This project faces social issues due to the need to develop a use for the building. An appropriate use for the building is determined by recognizing what the needs are of the residents of Princeton and what the building is capable of providing.

This project faces political issues due to the challenge of obtaining funding from the Town of Princeton. An aspect of this project proposal is developing a use for the building that will encourage the to Town to provide funding turn the restoration plan into a reality.

Section 5: Methodology

The methodology for the project is one of the most important aspects of the proposal. It is what our group will follow to guide us in the right direction, and it dictates how we are going to accomplish different tasks that we want to achieve. From the beginning our group felt that it was important to establish a framework to follow. We came up with five major sections to our methodology.

Section 5.1: Understanding Existing Conditions

In order to understand the current state of Mechanics Hall, it is necessary to schedule an initial meet and greet with the Friend of Mechanics Hall. Before looking into the building it is in our interest to talk to them, and get a feel for what they expect the outcome to be. One major reason for doing this is to get an initial tour of the building. As we will be going into this project blind, it is necessary to listen intently during the tour to any history explained or problems with the building. Another major reason for this meeting is to get an initial idea for the eventual use of the building. We will develop the restoration plan based on the needs of the Friends of Mechanics Hall, as we want the final product to be something that they will enjoy and approve of.

Our Second site visit at Mechanics Hall will be designated to measuring the building. We were given a document that had part of the background description of the building, but it is necessary to go much deeper into the building. It is in our interest to obtain measurements of the electrical equipment, structural members, room and hallway dimension, windows, heating equipment, and plumbing equipment. We will obtain these measurements for every room in the building and the exterior. Using a laser, these measurements will be much easier to come by. Doing this will help us paint a much better picture of Mechanics Hall.

From the dimension obtained both a CAD and Revit model will be created. These models will serve as our plans throughout the project. Having these models is an important step because it will be much easier to look into intended uses, and changes within the building. It also is necessary because Mechanics Hall does not have any plans associated with it.

Section 5.2: Uses and Layout

Per the scope of work our overall goal is to come up with two potential uses for Mechanics Hall. One major reason for coming up with two uses is to be able to give the Friends of Mechanics Hall multiple options. By showing both pros and cons of different uses along with price it will be easier for them to decide which they think is more appealing. The most important goal of each use is to get people involved with the building. The more people involved with the building the more likely added money will be granted for the restoration of the building. This goal will be a major consideration when deciding on potential uses.

The models made will play a big role in deciding what the building will be used for. They will help us understand what is realistic and what is not. The layout can also be done using the created models. Because a Revit Model has already been created it will be easy to see a 3-D rendering of a potential use. This will greatly help in seeing how effective a certain layout is.

Section 5.3: Asses Building Systems

We plan on assessing the building systems for each proposed use and defining the restoration work that will need to take place. The major areas of the building that will be assed as established in the scope will be structural integrity, egress, fire safety, drainage, handicapped access, and parking. The following flow charts outline our assessment procedure (Figures 4-8). Research will be done in all of the fields presented, and will be added to as the project moves forward. All of the following will have a focus on the particular uses decided upon.

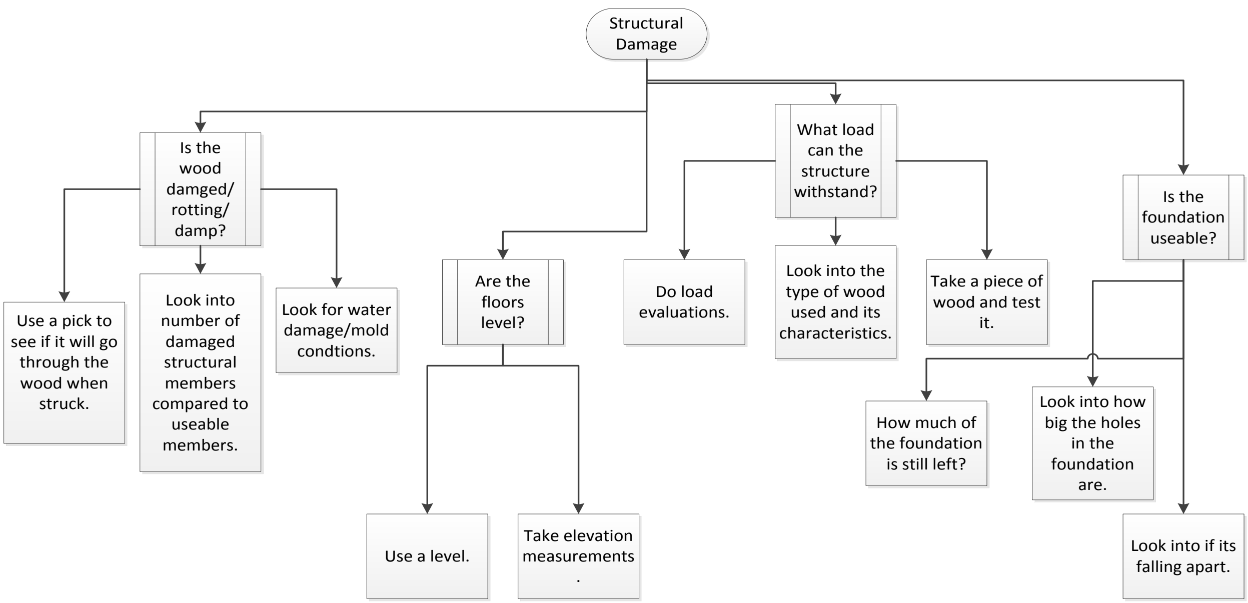

Figure 4: Structural Damage Assessment Procedure

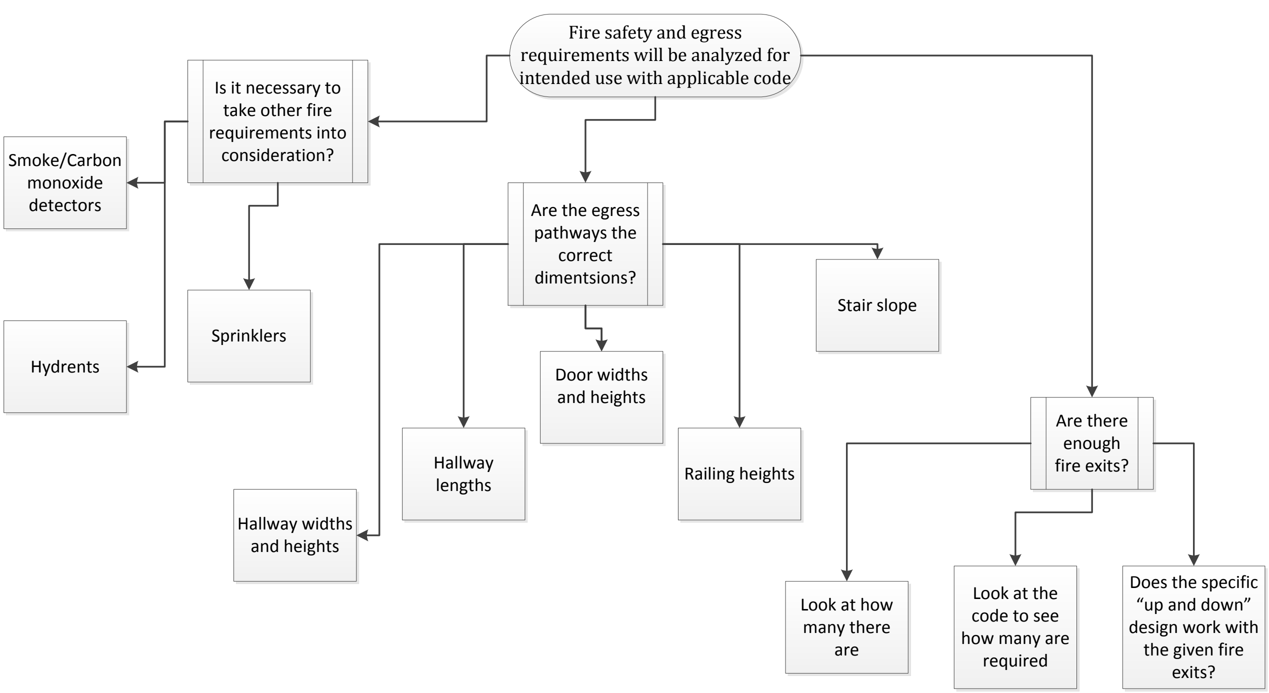

Figure 5: Fire Safety and Egress Assessment Procedure

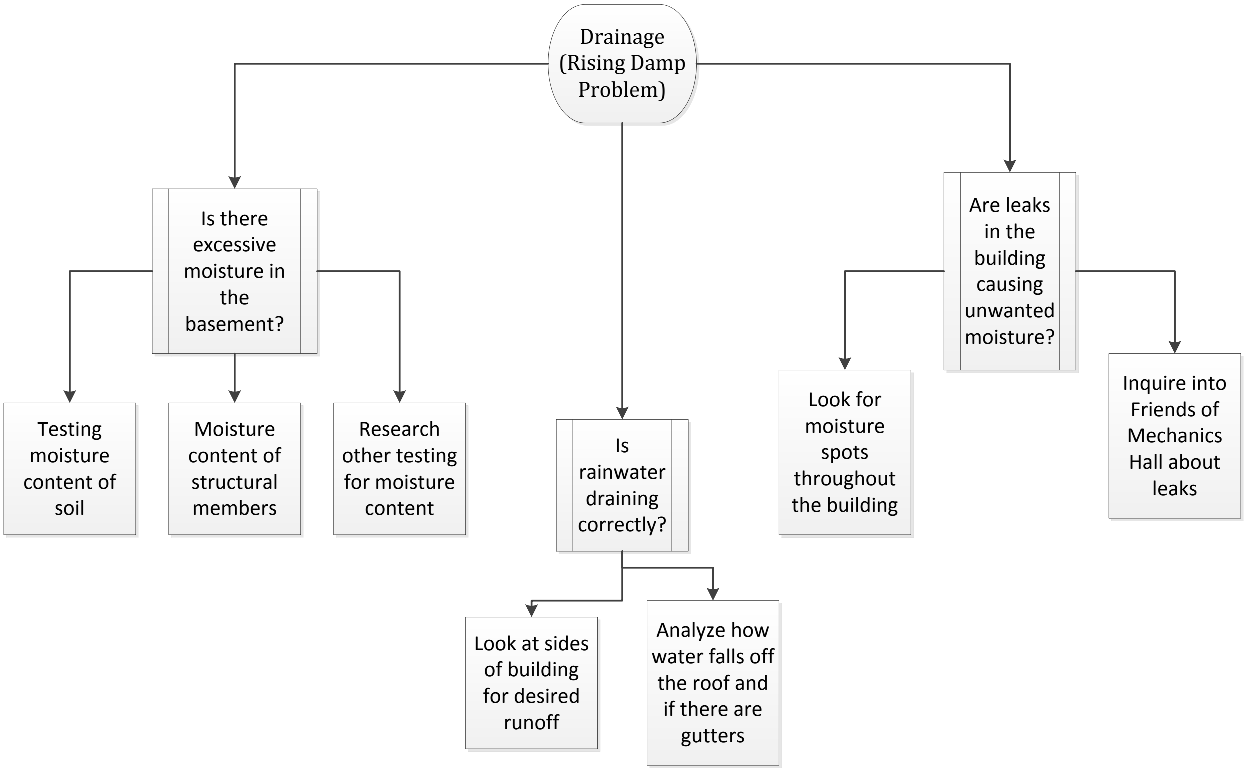

Figure 6: Drainage Assessment Procedure

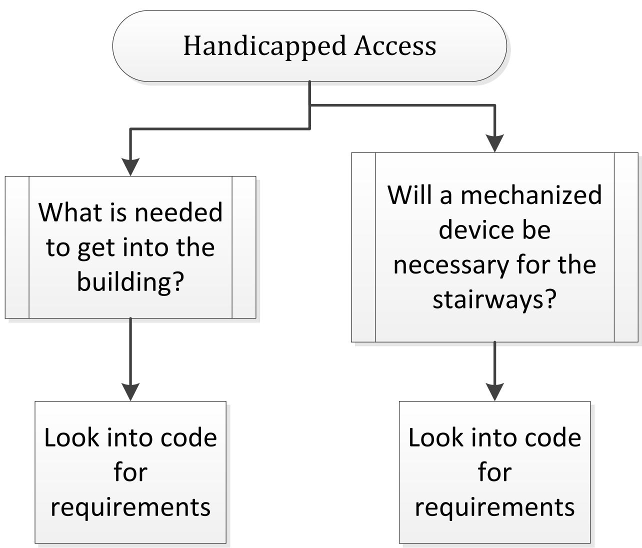

Figure 7: Handicapped Access Assessment Procedure

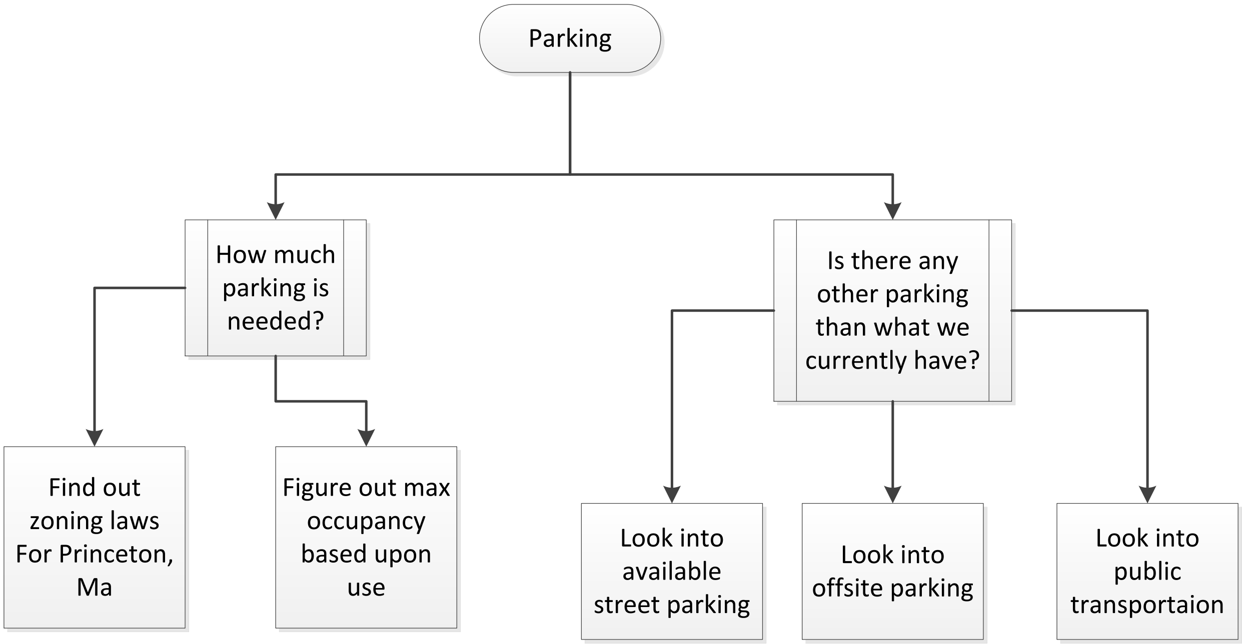

Figure 8: Parking Assessment Procedure

After looking into all of these systems it will be in our best interest to see if the building meets any historical exceptions. These exceptions may be beneficial for cost savings and historical integrity. Either way both the problem and potential exceptions to it will be investigated.

Section 5.4: Estimate Cost

We plan to look into the cost required for each proposed use. By displaying a rough cost to the Friends of Mechanics Hall they will get a better understanding of which use they think fits the best. With the help of our models we will be able to look into what materials are needed. A list will then be furnished of materials and their prices in order to come up with a rough estimate for cost of materials. Another major expense is labor. Through research and current wage information we will be able to come up with the labor costs. Estimation on the time spent on certain areas will have to take place. Lastly, demo costs will again be determined through wage data, and estimated time spent per area. Using the three areas above a rough picture of the cost of each use will be able to be portrayed.

Section 5.5: Compile Restoration Plan

Construction projects such as these require a plan in writing to accurately portray the project. We will use this to convey our project to the Friends of Mechanics Hall, and hopefully use it to achieve the necessary funds for renovation. A renovation plan consists of a problem statement, results of an investigation, a design, and a cost estimate. The renovation plan for Mechanics Hall will include the results from field inspections and analysis for the aforementioned deficiencies. After identifying the potential uses for the building, these deficiencies will be addressed in a design with the purpose of bringing Mechanics Hall into compliance with Massachusetts Building Code. Based on the design, computer models of the structure will be presented using

AutoCAD and Revit software, and a cost estimate will be proposed. A completed renovation plan includes these items as well as a set of plans and specifications, and an itemized list of quantities required for the project.

Section 6: Schedule

Our schedule is made up of both project milestones and deliverables. It is presented in the table listed below (Table 1).

Table 1: Schedule

Term	A-Term Milestones	B-Term Milestones	C-Term Milestones
	Meet and Greet with Friends of Mechanics Hall	Continue Background Research	Develop Cost Estimates Based on Prelim Design
	Compile Building Dimensions	Building Layout for Each Intended Use	Presentations to Friends of Mechanics Hall
	Compile Preliminary Cad Drawing	Structural Analysis	Refine Design
	Define Project Scope	Fire Safety and Egress Requirements	Finalize Cost for Each Solution
	Develop Anticipated Building Uses	Drainage Requirements	Prepare Draft of MQP Report (Advising Day)
	Begin Background Research	Handicapped Access	Develop Necessary Models
		Parking Requirements	Compile Restoration Plan
		Prelim Design Solutions	Finalize MQP Report
	Deliverables	Deliverables	Deliverables
	Proposal	Progress on MQP Report	MQP Report
	CAD/Revit Drawings	Layout Models	Restoration Plan
	A-Term Accomplishments and Beginning of B-Term Expectations	B-Term Accomplishments and Beginning of C-Term Expectations	MQP Poster

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Appendix B: Structural Analysis

Floors:

Floor Decking - Original Building						
Dimensions						
Span	1.08	ft				
Span	13.00	in				
Depth	1.50	in				
Width	8.00	in				
Tributary Width	8.00	in				
Tributary Area	0.72	sf				
Moment of Inertia	2.25	in^4				
X-Sectional Area	12.00	si				
Plank Thickness	1.50	in				
Floor Finish Thickness	1.00	in				
Section Modulus	3.00	ci				
Q	2.25	ci				
Material Properties	Table 4A					
Species	Northern Red					
Unit Weight	42.00	pcf				
E	1400000.00	psi				
Bending Strength	1000.00	psi				
Shear Strength	170.00	psi				
Tensile Strength	575.00	psi				
Compression ll to Grain	925.00	psi				
Compression Perp. to Grain	885.00	psi				
Loading						
Self Weight	3.50	plf				
Dead Load - Structural	0.00	plf				
Dead Load - Other	27.00	psf				
Service Live Load	100.00	psf				
C Factors	fb	fv	fc	fcperp	Emin	
Load Duration Factor	1.00	1.00	1.00	1.00	1.00	Occupancy LL
Temperature Factor	1.00	1.00	1.00	1.00	1.00	
Wet Service Factor	0.85	0.97	0.80	0.67	0.90	
Beam Stability Factor	1.00	-				
Size Factor	1.00	-	1.05			
Flat Use Factor	1.00	-	-	-	-	
Incising Factor	1.00	1.00	1.00	1.00	1.00	
Repetative Member Factor	1.15	-	-	-	-	
Column Stability Factor						
Buckling Stiffness Factor	-	-	-	-	1.00	
Bearing Area Factor	-	-	-	1.00	-	

Floor Decking (Cont)		
Typical Interior Beam		
Applied Dead Load	3.50	plf
Bending Moment		
Allowed Bending Stress	977.50	psi
Bending Moment due to Dead Load	0.51	lb*ft
Dead Load Bending Stress	2.05	psi
Maximum Allowed Live Load Stress	975.45	psi
Maximum Allowed Live Load Moment	243.86	lb*ft
Maximum Allowed Live Load	1662.30	lb/ft
Maximum Allowed Live Load	2493.45	psf
Shear		
Allowable Shear Stress	164.90	psi
Shear Force - Dead Load	1.90	lbs
Shear Stress due to Dead Load	0.24	lb
Maximum Allowed Live Load Shear Stress	164.66	lb/ft
Maximum Allowed Live Load Shear Force	1317.30	lb
Maximum Allowed Live Load	2431.95	plf
Maximum Allowed Live Load	3647.92	psf
Deflection		
Allowable Deflection	0.05	in
Deflection due to Dead Load	0.00	in
Maximum Allowed Live Load Deflection	0.05	in
Maximum Allowed Live Load	4951.62	plf
Maximum Allowed Live Load	7427.43	psf

Joists:

Joists - Original Building						
Dimensions						
Span	9.00	ft				
Span	108.00	in				
Depth	8.00	in				
Width	2.00	in				
Tributary Width	15.00	in				
Tributary Area	11.25	sf				
Moment of Inertia	85.33	in^4				
X-Sectional Area	16.00	si				
Plank Thickness	1.50	in				
Floor Finish Thickness	1.00	in				
Section Modulus	21.33	ci				
Q	16.00	ci				
Material Properties	Table 4A					
Species	Northern Red Oak					
Unit Weight	42.00	pcf				
E	1400000.00	psi				
Bending Strength	1000.00	psi				
Shear Strength	170.00	psi				
Tensile Strength	575.00	psi				
Compression ll to Grain	925.00	psi				
Compression Perp. to Grain	885.00	psi				
Loading						
Self Weight	4.67	plf				
Dead Load - Structural	6.56	psf				
Dead Load - Other	30.00	psf				
Service Live Load	100.00	psf				
C Factors	fb	fv	fc	fcperp	Emin	
Load Duration Factor	1.00	1.00	1.00	1.00	1.00	Occupancy LL
Temperature Factor	1.00	1.00	1.00	1.00	1.00	
Wet Service Factor	0.85	0.97	0.80	0.67	0.90	
Beam Stability Factor	1.00	-				
Size Factor	1.20	-	1.05			
Flat Use Factor	1.15	-	-	-	-	
Incising Factor	1.00	1.00	1.00	1.00	1.00	
Repetative Member Factor	1.15	-	-	-	-	
Column Stability Factor						
Buckling Stiffness Factor	-	-	-	-	1.00	
Bearing Area Factor	-	-	-	1.00	-	

Joists - Original Building (Cont)		
Typical Interior Beam-Uniform Load		
Applied Dead Load	50.37	plf
Applied Live Load	125.00	plf
Bending Moment		
Allowed Bending Stress	1348.95	psi
Bending Moment due to Dead Load	509.99	lb*ft
Dead Load Bending Stress	286.87	psi
Maximum Allowed Live Load Stress	1062.08	psi
Maximum Allowed Live Load Moment	1888.14	lb*ft
Maximum Allowed Live Load	186.48	lb/ft
Maximum Allowed Live Load	149.19	psf
Shear		
Allowable Shear Stress	164.90	psi
Shear Force - Dead Load	226.66	lbs
Shear Stress due to Dead Load	21.25	psi
Maximum Allowed Live Load Shear Stress	143.65	psi
Maximum Allowed Live Load Shear Force	1532.27	lb
Maximum Allowed Live Load	340.50	plf
Maximum Allowed Live Load	272.40	psf
Deflection		
Allowable Deflection	0.45	in
Deflection due to Dead Load	0.07	in
Deflection due to Live Load	0.15	in
Maximum Allowed Live Load Deflection	0.38	in
Maximum Allowed Live Load	277.39	plf
Maximum Allowed Live Load	221.91	psf

Notched Beams					
Dimensions					
Span	9.00	ft			
Span	108.00	in			
Depth	4.50	in			
Width	2.00	in			
Tributary Width	15.00	in			
Tributary Area	11.25	sf			
Moment of Inertia	15.19	in^4			
X-Sectional Area	9.00	si			
Plank Thickness	1.50	in			
Floor Finish Thickness	1.00	in			
Section Modulus	6.75	ci			
Q	5.06	ci			
Material Properties	Table 4A				
Species	Northern Red Oak				
Unit Weight	42.00	pcf			
E	1400000.00	psi			
Bending Strength	1000.00	psi			
Shear Strength	170.00	psi			
Tensile Strength	575.00	psi			
Compression ll to Grain	925.00	psi			
Compression Perp. to Grain	885.00	psi			
Loading					
Self Weight	2.63	plf			
Dead Load - Structural	47.25	psf			
Dead Load - Other	30.00	psf			
Service Live Load	100.00	psf			
C Factors	fb	fv	fc	fcperp	Emin
Load Duration Factor	1.00	1.00	1.00	1.00	1.00
Temperature Factor	1.00	1.00	1.00	1.00	1.00
Wet Service Factor	0.85	0.97	0.80	0.67	0.90
Beam Stability Factor	1.00	-			
Size Factor	1.20	-	1.05		
Flat Use Factor	1.15	-	-	-	-
Incising Factor	1.00	1.00	1.00	1.00	1.00
Repetative Member Factor	1.15	-	-	-	-
Column Stability Factor					
Buckling Stiffness Factor	-	-	-	-	1.00
Bearing Area Factor	-	-	-	1.00	-

Notched Beams (Cont)

Typical Interior Beam-Uniform Load				
Applied Dead Load	99.19	plf		
Applied Live Load	125.00	plf		
Bending Moment				
Allowed Bending Stress	1348.95	psi		
Bending Moment due to Dead Load	1004.27	lb*ft		
Dead Load Bending Stress	1785.38	psi		
Maximum Allowed Live Load Stress	-436.43	psi		
Maximum Allowed Live Load Moment	-245.49	lb*ft		
Maximum Allowed Live Load	-24.25	lb/ft		
Maximum Allowed Live Load	-19.40	psf		
Shear				
Allowable Shear Stress	164.90	psi		
Shear Force - Dead Load	446.34	lbs		
Shear Stress due to Dead Load	74.39	psi		
Maximum Allowed Live Load Shear Stress	90.51	psi		
Maximum Allowed Live Load Shear Force	543.06	lb		
Maximum Allowed Live Load	120.68	plf		
Maximum Allowed Live Load	96.54	psf		
Deflection				
Allowable Deflection	0.30	in		
Deflection due to Dead Load	0.14	in		
Deflection due to Live Load	0.15	in		
Maximum Allowed Live Load Deflection	0.16	in		
Maximum Allowed Live Load	23.60	plf		
Maximum Allowed Live Load	18.88	psf		

Front Girder:

Southern Carrier Beam						
Dimensions				**Material Properties**	**Table 4A**	
Span	13.00	ft		Species	Northern Red Oak	
Span	156.00	in		Unit Weight	42.00	pcf
Depth	8.00	in		E	1300000.00	psi
Width	8.00	in		Bending Strength	1350.00	psi
Tributary Width	120.00	in		Shear Strength	205.00	psi
Tributary Area	16.25	sf		Tensile Strength	675.00	psi
Moment of Inertia	341.33	in^4		Compression ll to Grain	800.00	psi
X-Sectional Area	64.00	si		Compression Perp. to Grain	885.00	psi
Plank Thickness	1.50	in				
Floor Finish Thickness	1.00	in				
Section Modulus	85.33	ci				
Q	64.00	ci				
Loading						
Self Weight	18.67	plf				
Dead Load - Structural Span 1	42.10	plf				
Dead Load - Other	30.00	psf				
Service Live Load	100.00	psf				
Dead Load - Structural Span 2	67.74	plf				
Dead Load - Structural Span 3	104.26	plf				
C Factors	fb	fv	fc	fcperp	Emin	
Load Duration Factor	1.00	1.00	1.00	1.00	1.00	Occupancy LL
Temperature Factor	1.00	1.00	1.00	1.00	1.00	
Wet Service Factor	0.85	0.97	0.80	0.67	0.90	
Beam Stability Factor	1.00	-				
Size Factor	1.30	-	1.05			
Flat Use Factor	1.15	-	-	-	-	
Incising Factor	1.00	1.00	1.00	1.00	1.00	
Repetative Member Factor	1.15	-	-	-	-	
Column Stability Factor						
Buckling Stiffness Factor	-	-	-	-	1.00	
Bearing Area Factor	-	-	-	1.00	-	

Southern Carrier Beam (Cont)		
West Span		
Applied Dead Load	360.77	plf
Applied Live Load	1000.00	plf
Span 1	13.00	ft
Bending Moment		
Allowed Bending Stress	1972.84	psi
Bending Moment due to Dead Load	7621.27	lb*ft
Dead Load Bending Stress	1071.74	psi
Maximum Allowed Live Load Stress	901.10	psi
Maximum Allowed Live Load Moment	6407.81	lb*ft
Maximum Allowed Live Load	303.33	plf
Maximum Allowed Live Load	30.33	psf
Shear		
Allowable Shear Stress	198.85	psi
Shear Force - Dead Load	2345.01	lbs
Shear Stress due to Dead Load	54.96	psi
Maximum Allowed Live Load Shear Stress	143.89	psi
Maximum Allowed Live Load Shear Force	6139.26	lb
Maximum Allowed Live Load	944.50	plf
Maximum Allowed Live Load	94.45	psf
Deflection		
Allowable Deflection	0.65	in
Deflection due to Dead Load	0.58	in
Maximum Allowed Live Load Deflection	0.07	in
Maximum Allowed Live Load	43.17	plf
Maximum Allowed Live Load	4.32	psf

Middle Span		
Applied Dead Load	386.41	plf
Applied Live Load	1000.00	plf
Span 2	8.08	ft
Bending Moment		
Allowed Bending Stress	1972.84	psi
Bending Moment due to Dead Load	3153.39	lb*f
Dead Load Bending Stress	443.45	psi
Maximum Allowed Live Load Stress	1529.39	psi
Maximum Allowed Live Load Moment	10875.69	lb*f
Maximum Allowed Live Load	1332.67	lb/f
Maximum Allowed Live Load	133.27	psf
Shear		
Allowable Shear Stress	198.85	psi
Shear Force - Dead Load	1561.09	lbs
Shear Stress due to Dead Load	36.59	lb
Maximum Allowed Live Load Shear	162.26	lb/f
Maximum Allowed Live Load Shear	6923.18	lb
Maximum Allowed Live Load	1713.66	plf
Maximum Allowed Live Load	171.37	psf
Deflection		
Allowable Deflection	0.40	in
Deflection due to Dead Load	0.09	in
Maximum Allowed Live Load Deflection	0.31	in
Maximum Allowed Live Load	1295.95	plf
Maximum Allowed Live Load	129.59	psf

East Span		
Applied Dead Load	422.92	plf
Applied Live Load	1000.00	plf
Span 3	5.25	ft
Bending Moment		
Allowed Bending Stress	1972.84	psi
Bending Moment due to Dead Load	1457.10	lb*ft
Dead Load Bending Stress	204.90	psi
Maximum Allowed Live Load Stress	1767.93	psi
Maximum Allowed Live Load Moment	12571.98	lb*f
Maximum Allowed Live Load	3649.01	lb/f
Maximum Allowed Live Load	364.90	psf
Shear		
Allowable Shear Stress	198.85	psi
Shear Force - Dead Load	1110.17	lbs
Shear Stress due to Dead Load	26.02	lb
Maximum Allowed Live Load Shear	172.83	plf
Maximum Allowed Live Load Shear	7374.09	lb
Maximum Allowed Live Load	2809.18	plf
Maximum Allowed Live Load	280.92	psf
Deflection		
Allowable Deflection	0.26	in
Deflection due to Dead Load	0.02	in
Maximum Allowed Live Load Deflection	0.24	in
Maximum Allowed Live Load	5710.09	plf
Maximum Allowed Live Load	571.01	psf

Mid Girder:

Middle Carrier Beam						
Dimensions						
Span	13.00	ft				
Span	156.00	in				
Depth	8.00	in				
Width	8.00	in				
Tributary Width	124.92	in				
Tributary Area	16.25	sf				
Moment of Inertia	341.33	in^4				
X-Sectional Area	64.00	si				
Plank Thickness		in				
Floor Finish Thickness		in				
Section Modulus	85.33	ci				
Q	64.00	ci				
Material Properties	Table 4A					
Species	Northern Red Oak					
Unit Weight	42.00	pcf				
E	1300000.00	psi				
Bending Strength	1350.00	psi				
Shear Strength	205.00	psi				
Tensile Strength	675.00	psi				
Compression ll to Grain	800.00	psi				
Compression Perp. to Grain	885.00	psi				
Loading						
Self Weight	18.67	plf				
Dead Load - Structural	35.54	plf				
Dead Load - Other	30.00	psf				
Service Live Load	100.00	psf				
Dead Load - Structural Span 2	34.66	plf				
C Factors	fb	fv	fc	fcperp	Emin	
Load Duration Factor	1.00	1.00	1.00	1.00	1.00	Occupancy LL
Temperature Factor	1.00	1.00	1.00	1.00	1.00	
Wet Service Factor	0.85	0.97	0.80	0.67	0.90	
Beam Stability Factor	1.00	-				
Size Factor	1.30	-	1.05			
Flat Use Factor	1.15	-	-	-	-	
Incising Factor	1.00	1.00	1.00	1.00	1.00	
Repetative Member Factor	1.15	-	-	-	-	
Column Stability Factor						
Buckling Stiffness Factor	-	-	-	-	1.00	
Bearing Area Factor	-	-	-	1.00	-	

West Span		
Applied Dead Load	366.51	plf
Applied Live Load	1041.00	plf
Span 1	13.00	ft
Bending Moment		
Allowed Bending Stress	1972.84	psi
Bending Moment due to Dead Load	7742.47	lb*ft
Dead Load Bending Stress	1088.79	psi
Maximum Allowed Live Load Stress	884.05	psi
Maximum Allowed Live Load Moment	6286.61	lb*ft
Maximum Allowed Live Load	297.59	plf
Maximum Allowed Live Load	28.59	psf
Shear		
Allowable Shear Stress	198.85	psi
Shear Force - Dead Load	2382.30	lbs
Shear Stress due to Dead Load	55.84	psi
Maximum Allowed Live Load Shear Stress	143.01	psi
Maximum Allowed Live Load Shear Force	6101.97	lb
Maximum Allowed Live Load	938.76	plf
Maximum Allowed Live Load	90.18	psf
Deflection		
Allowable Deflection	0.65	in
Deflection due to Dead Load	0.59	in
Maximum Allowed Live Load Deflection	0.06	in
Maximum Allowed Live Load	37.44	plf
Maximum Allowed Live Load	3.60	psf

East Span		
Applied Dead Load	365.63	plf
Applied Live Load	1041.00	plf
Span 2	13.33	ft
Bending Moment		
Allowed Bending Stress	1972.84	psi
Bending Moment due to Dead Load	8121.00	lb*f
Dead Load Bending Stress	1142.02	psi
Maximum Allowed Live Load Stress	830.82	psi
Maximum Allowed Live Load Moment	5908.08	lb*f
Maximum Allowed Live Load	266.00	lb/f
Maximum Allowed Live Load	25.55	psf
Shear		
Allowable Shear Stress	198.85	psi
Shear Force - Dead Load	2436.91	lbs
Shear Stress due to Dead Load	57.12	lb
Maximum Allowed Live Load Shear	141.73	lb/f
Maximum Allowed Live Load Shear	6047.36	lb
Maximum Allowed Live Load	907.33	plf
Maximum Allowed Live Load	87.16	psf
Deflection		
Allowable Deflection	0.44	in
Deflection due to Dead Load	0.65	in
Maximum Allowed Live Load Deflection	-0.21	in
Maximum Allowed Live Load	-115.84	plf
Maximum Allowed Live Load	-11.13	psf

Back Girder:

Dimensions							
Span	13.00	ft					
Span	156.00	in					
Depth	8.00	in					
Width	8.00	in					
Tributary Width	120.00	in					
Tributary Area	16.25	sf					
Moment of Inertia	341.33	in^4					
X-Sectional Area	64.00	si					
Plank Thickness		in					
Floor Finish Thickness		in					
Section Modulus	85.33	ci					
Q	64.00	ci					
Material Properties	Table 4A						
Species	Northern Red Oak						
Unit Weight	42.00	pcf					
E	1300000.00	psi					
Bending Strength	1350.00	psi					
Shear Strength	205.00	psi					
Tensile Strength	675.00	psi					
Compression ll to Grain	800.00	psi					
Compression Perp. to Grain	885.00	psi					
Loading							
Self Weight	18.67	plf					
Dead Load - Structural	35.54	plf					
Dead Load - Other	30.00	psf					
Service Live Load	100.00	psf					
Dead Load - Structural Span 2	34.66	plf					
C Factors	fb	fv	fc	fcperp	Emin		
Load Duration Factor	1.00	1.00	1.00	1.00	1.00		
Temperature Factor	1.00	1.00	1.00	1.00	1.00		
Wet Service Factor	0.85	0.97	0.80	0.67	0.90		
Beam Stability Factor	1.00	-					
Size Factor	1.30	-	1.05				
Flat Use Factor	1.15	-	-	-	-		
Incising Factor	1.00	1.00	1.00	1.00	1.00		
Repetative Member Factor	1.15	-	-	-	-		
Column Stability Factor							
Buckling Stiffness Factor	-	-	-	-	1.00		
Bearing Area Factor	-	-	-	1.00	-		

Northern Carrier Beam (Cont)		
West Span		
Applied Dead Load	354.21	plf
Applied Live Load	1000.00	plf
Span 1	13.00	ft
Bending Moment		
Allowed Bending Stress	1972.84	psi
Bending Moment due to Dead Load	7482.64	lb*ft
Dead Load Bending Stress	1052.25	psi
Maximum Allowed Live Load Stress	920.59	psi
Maximum Allowed Live Load Moment	6546.44	lb*ft
Maximum Allowed Live Load	309.89	plf
Maximum Allowed Live Load	30.99	psf
Shear		
Allowable Shear Stress	198.85	psi
Shear Force - Dead Load	2302.35	lbs
Shear Stress due to Dead Load	53.96	psi
Maximum Allowed Live Load Shear Stress	144.89	psi
Maximum Allowed Live Load Shear Force	6181.92	lb
Maximum Allowed Live Load	951.06	plf
Maximum Allowed Live Load	95.11	psf
Deflection		
Allowable Deflection	0.65	in
Deflection due to Dead Load	0.57	in
Maximum Allowed Live Load Deflection	0.08	in
Maximum Allowed Live Load	49.74	plf
Maximum Allowed Live Load	4.97	psf

Northern Carrier Beam (Cont)		
East Span		
Applied Dead Load	353.33	plf
Applied Live Load	1000.00	plf
Span 2	13.33	ft
Bending Moment		
Allowed Bending Stress	1972.84	psi
Bending Moment due to Dead Load	7847.80	lb*f
Dead Load Bending Stress	1103.60	psi
Maximum Allowed Live Load Stress	869.24	psi
Maximum Allowed Live Load Moment	6181.28	lb*f
Maximum Allowed Live Load	278.30	lb/f
Maximum Allowed Live Load	27.83	psf
Shear		
Allowable Shear Stress	198.85	psi
Shear Force - Dead Load	2354.93	lbs
Shear Stress due to Dead Load	55.19	lb
Maximum Allowed Live Load Shear	143.66	lb/f
Maximum Allowed Live Load Shear	6129.34	lb
Maximum Allowed Live Load	919.63	plf
Maximum Allowed Live Load	91.96	psf
Deflection		
Allowable Deflection	0.44	in
Deflection due to Dead Load	0.63	in
Maximum Allowed Live Load Deflection	-0.18	in
Maximum Allowed Live Load	-103.54	plf
Maximum Allowed Live Load	-10.35	psf

Column 1&2 (towards back)

Northern and Middle	Columns					
Dimensions				**Loads**		
Column Thickness	8.50	in		Self Weight - Floor	5.25	psf
Column Radius	4.25	in		Self Weight - Joists	4.67	plf
Length	6.50	ft		Self Weight - Girder	18.67	plf
Ke	2.40	**NDS App. G**		Dead Load - Floor	681.37	lb
Effective Length	15.60	ft		Dead Load - Joists	102.67	lb
Tributary Length	9.36	ft		Dead Load - Girder	258.97	lb
Tributary Width	13.87	ft		Dead Load - Column	107.58	lb
Tributary Area	129.79	sf		DL - Structural	1150.586	lb
Cross Sec Area	56.74	si		DL - Other	12978.5	lb
D1	7.53	in		Total Dead Load	14129.09	lb
D2	7.53	in				
Slenderness Ratio	9.18					
Material Properties	Table 4D			**C Factors**		
Species	Northern Red			Load Duration Factor	1.00	
E	1300000.00	psi		Temperature Factor	1.00	
Bending Strength	1350.00	psi		Wet Service Factor	0.80	
Shear Strength	205.00	psi		Beam Stability Factor	-	
Tensile Strength	675.00	psi		Size Factor	1.00	
Compression ll to Grain	800.00	psi		Flat Use Factor		
Compression Perp. to Grain	885.00	psi		Incising Factor	1.00	
Emin	470000.00	psi		Repetative Member Factor		
Unit Weight Wood	42.00	pcf		Column Stability Factor	0.75	
				Buckling Stiffness Factor		
				Bearing Area Factor	1.00	
Capacity						
Comp. Strength	477.60	psi				
Load	14129.09	lb				
Comp. Stress	248.99	psi				
Available Comp. Stress	228.61	psi				
Allowable Live Load	12972.38	lb				
Allowable Live Load	99.95	psf				

Column 3 (Front-Main Room)

South-West Column					
Dimensions			**Loads**		
Column Thickness	8.50	in	Self Weight - Floor	5.25	psf
Column Radius	4.25	in	Self Weight - Short Beams	4.67	plf
Length	6.50	ft	Self Weight - Girder	18.67	plf
Ke	2.40		Dead Load - Floor	552.52	lb
Effective Length	15.60	ft	Dead Load - Short Beam	102.67	lb
Tributary Length	9.36	ft	Dead Load - Girder	258.97	lb
Tributary Width	11.25	ft	Dead Load - Column	107.58	lb
Tributary Area	105.24	sf	DL - Structural	1021.74	lb
Cross Sec Area	56.74	si	DL - Other & Roof	10524.22	lb
D1	7.53	in	Total Dead Load	11545.96	lb
D2	7.53	in			
Slenderness Ratio	9.18				
Material Properties	Table 4D		**C Factors**		
Species	Northern Red Oak		Load Duration Factor	1.00	
E	1300000.00	psi	Temperature Factor	1.00	
Bending Strength	1350.00	psi	Wet Service Factor	0.80	
Shear Strength	205.00	psi	Beam Stability Factor	-	
Tensile Strength	675.00	psi	Size Factor	1.00	
Compression ll to Grain	800.00	psi	Flat Use Factor		
Compression Perp. to	885.00	psi	Incising Factor	1.00	
Emin	470000.00	psi	Repetative Member Factor		
	42.00		Column Stability Factor	0.75	
			Buckling Stiffness Factor		
			Bearing Area Factor	1.00	
Capacity					
Comp. Strength	477.60	psi			
Load	11545.96	lb			
Comp. Stress	203.47	psi			
Available Comp. Stress	274.13	psi			
Allowable Live Load	15555.51	lb			
Allowable Live Load	147.81	psf			

Column 4(Front Stairs)

South-East Column					
Dimensions			**Loads**		
Column Thickness	8.50	in	Self Weight - Floor	5.25	psf
Column Radius	4.25	in	Self Weight - Short	4.67	plf
Length	6.50	ft	Self Weight - Girder	18.67	plf
Ke	2.40	NDS App. G	Dead Load - Floor	362.21	lb
Effective Length	15.60	ft	Dead Load - Short	102.67	lb
Tributary Length	9.36	ft	Dead Load - Girder	258.97	lb
Tributary Width	7.37	ft	Dead Load -	107.58	lb
Tributary Area	68.99	sf	DL - Structural	831.42	lb
Cross Sec Area	56.74	si	DL - Other	6899.16	lb
D1	7.53	in	Total Dead Load	7730.58	lb
D2	7.53	in			
Slenderness Ratio	9.18				
Material Properties	Table 4D		**C Factors**		
Species	Northern Red Oak		Load Duration	1.00	
E	1300000.00	psi	Temperature Factor	1.00	
Bending Strength	1350.00	psi	Wet Service Factor	0.80	
Shear Strength	205.00	psi	Beam Stability	-	
Tensile Strength	675.00	psi	Size Factor	1.00	
Compression ll to Grain	800.00	psi	Flat Use Factor		
Compression Perp. to Grain	885.00	psi	Incising Factor	1.00	
Emin	470000.00	psi	Repetative Member		
	42.00		Column Stability	0.75	
			Buckling Stiffness		
			Bearing Area	1.00	
Capacity					
Comp. Strength	477.60	psi			
Load	7730.58	lb			
Comp. Stress	136.23	psi			
Available Comp. Stress	341.37	psi			
Allowable Live Load	19370.89	lb			
Allowable Live Load	280.77	psf			

Addition Floor

Dimensions						
Span	1.08	ft				
Span	13.00	in				
Depth	1.50	in				
Width	8.00	in				
Tributary Width	8.00	in				
Tributary Area	0.72	sf				
Moment of Inertia	2.25	in^4				
X-Sectional Area	12.00	si				
Plank Thickness	1.50	in				
Floor Finish Thickness	1.00	in				
Section Modulus	3.00	ci				
Q	2.25	ci				
Material Properties	Table 4A					
Species	Northern Red Oak					
Unit Weight	42.00	pcf				
E	1400000.00	psi				
Bending Strength	1000.00	psi				
Shear Strength	170.00	psi				
Tensile Strength	575.00	psi				
Compression ll to Grain	925.00	psi				
Compression Perp. to Grain	885.00	psi				
Loading						
Self Weight	3.50	plf				
Dead Load - Structural	0.00	plf				
Dead Load - Other	27.00	psf				
Service Live Load	100.00	psf				
C Factors	fb	fv	fc	fcperp	Emin	
Load Duration Factor	1.00	1.00	1.00	1.00	1.00	Occupancy LL
Temperature Factor	1.00	1.00	1.00	1.00	1.00	
Wet Service Factor	0.85	0.97	0.80	0.67	0.90	
Beam Stability Factor	1.00	-				
Size Factor	1.00	-	1.05			
Flat Use Factor	1.00	-	-	-	-	
Incising Factor	1.00	1.00	1.00	1.00	1.00	
Repetative Member Factor	1.15	-	-	-	-	
Column Stability Factor						
Buckling Stiffness Factor	-	-	-	-	1.00	
Bearing Area Factor	-	-	-	1.00	-	

Typical Interior Beam		
Applied Dead Load	3.50	plf
Bending Moment		
Allowed Bending Stress	977.50	psi
Bending Moment due to Dead Load	0.51	lb*ft
Dead Load Bending Stress	2.05	psi
Maximum Allowed Live Load Stress	975.45	psi
Maximum Allowed Live Load Moment	243.86	lb*ft
Max Allowed Live Load	1662.30	lb/ft
Max Allowed Live Load	2493.45	psf
Shear		
Allowable Shear Stress	164.90	psi
Shear Force - Dead Load	1.90	lbs
Shear Stress due to Dead Load	0.24	lb
Maximum Allowed Live Load Shear Stress	164.66	lb/ft
Maximum Allowed Live Load Shear Force	1317.30	lb
Max Allowed Live Load	2431.95	lb/ft
Max Allowed Live Load	3647.92	psf
Deflection		
Allowable Deflection	0.05	in
Deflection due to Dead Load	0.00	in
Maximum Allowed Live Load Deflection	0.05	in
Maximum Allowed Live Load	4951.62	plf
Maximum Allowed Live Load	7427.43	psf

Addition Joists

Dimensions						
Span	10.33	ft				
Span	123.96	in				
Depth	8.00	in				
Width	2.00	in				
Tributary Width	15.00	in				
Tributary Area	12.91	sf				
Moment of Inertia	85.33	in^4				
X-Sectional Area	16.00	si				
Plank Thickness	1.50	in				
Floor Finish Thickness	1.00	in				
Section Modulus	21.33	ci				
Q	16.00	ci				
Material Properties	Table 4A					
Species	Northern Red Oak					
Unit Weight	42.00	pcf				
E	1400000.00	psi				
Bending Strength	1000.00	psi				
Shear Strength	170.00	psi				
Tensile Strength	575.00	psi				
Compression ll to Grain	925.00	psi				
Compression Perp. to Grain	885.00	psi				
Loading						
Self Weight	4.67	plf				
Dead Load - Structural	54.23	psf				
Dead Load - Other	30.00	psf				
Service Live Load	100.00	psf				
C Factors	fb	fv	fc	fcperp	Emin	
Load Duration Factor	1.00	1.00	1.00	1.00	1.00	Occupancy LL
Temperature Factor	1.00	1.00	1.00	1.00	1.00	
Wet Service Factor	0.85	0.97	0.80	0.67	0.90	
Beam Stability Factor	1.00	-				
Size Factor	1.20	-	1.05			
Flat Use Factor	1.15	-	-	-	-	
Incising Factor	1.00	1.00	1.00	1.00	1.00	
Repetative Member Factor	1.15	-	-	-	-	
Column Stability Factor						
Buckling Stiffness Factor	-	-	-	-	1.00	
Bearing Area Factor	-	-	-	1.00	-	

Typical Interior Beam-Uniform Load		
Applied Dead Load	109.96	plf
Applied Live Load	125.00	plf
Bending Moment		
Allowed Bending Stress	1348.95	psi
Bending Moment due to Dead Load	1466.68	lb*ft
Dead Load Bending Stress	825.01	psi
Maximum Allowed Live Load Stress	523.94	psi
Maximum Allowed Live Load Moment	931.46	lb*ft
Maximum Allowed Live Load	69.83	lb/ft
Maximum Allowed Live Load	55.87	psf
Shear		
Allowable Shear Stress	164.90	psi
Shear Force - Dead Load	567.93	lbs
Shear Stress due to Dead Load	53.24	psi
Maximum Allowed Live Load Shear Stress	111.66	psi
Maximum Allowed Live Load Shear Force	1191.00	lb
Maximum Allowed Live Load	230.59	plf
Maximum Allowed Live Load	184.47	psf
Deflection		
Allowable Deflection	0.52	in
Deflection due to Dead Load	0.24	in
Deflection due to Live Load	0.27	in
Maximum Allowed Live Load Deflection	0.28	in
Maximum Allowed Live Load	130.89	plf
Maximum Allowed Live Load	104.71	psf

Addition Cantilevers

Dimensions						
Span	11.00	ft				
Span	132.00	in				
Depth	8.00	in				
Width	8.00	in				
Tributary Width	123.96	in				
Tributary Area	113.63	sf				
Moment of Inertia	341.33	in^4				
X-Sectional Area	64.00	si				
Plank Thickness	1.50	in				
Floor Finish Thickness		in				
Section Modulus	85.33	ci				
Q	64.00	ci				
Cantilever Length	4.42	ft				
Material Properties	**Table 4A**					
Species	Northern Red Oak					
Unit Weight	42.00	pcf				
E	1300000.00	psi				
Bending Strength	1350.00	psi				
Shear Strength	205.00	psi				
Tensile Strength	675.00	psi				
Compression ll to Grain	800.00	psi				
Compression Perp. to Grain	885.00	psi				
Loading						
Self Weight	18.67	plf				
Dead Load - Structural Span 1	98.06	plf				
Dead Load - Other	30.00	psf				
Service Live Load	100.00	psf				
C Factors	fb	fv	fc	fcperp	Emin	
Load Duration Factor	1.00	1.00	1.00	1.00	1.00	Occupancy LL
Temperature Factor	1.00	1.00	1.00	1.00	1.00	
Wet Service Factor	0.85	0.97	0.80	0.67	0.90	
Beam Stability Factor	1.00	-				
Size Factor	1.30	-	1.05			
Flat Use Factor	1.15	-	-	-	-	
Incising Factor	1.00	1.00	1.00	1.00	1.00	
Repetative Member Factor	1.15	-	-	-	-	
Column Stability Factor						
Buckling Stiffness Factor	-	-	-	-	1.00	
	-	-	-	1.00	-	

Applied Uniform Dead Load	426.63	plf
Applied Point Dead Load (2nd Floor)	452.71	lb
Span 1	11.00	ft
x	4.58	
x	54.96	
x1	53.04	
Positive Bending Moment		
Allowed Bending Stress	1972.84	psi
Bending Moment due to Dead Load	5370.39	lb*ft
Dead Load Bending Stress	755.21	psi
Maximum Allowed Live Load Stress	1217.63	psi
Maximum Allowed Live Load Moment	8658.69	lb*ft
Maximum Allowed Live Load	814.15	plf
Maximum Allowed Live Load	78.81	psf
Negative Bending Moment		
Allowed Bending Stress	1972.84	psi
Bending Moment due to Dead Load	6168.37	lb*ft
Dead Load Bending Stress	1489.30	psi
Maximum Allowed Live Load Stress	483.54	psi
Maximum Allowed Live Load Moment	3438.52	lb*ft
Maximum Allowed Live Load	352.01	plf
Maximum Allowed Live Load	34.08	psf
Allowed Bending Stress	1972.84	psi
Bending Moment due to Dead Load	4422.19	lb*ft
Dead Load Bending Stress	621.87	psi
Maximum Allowed Live Load Stress	1350.97	psi
Maximum Allowed Live Load Moment	9606.89	lb*ft
Maximum Allowed Live Load	4347.01	plf
Maximum Allowed Live Load	420.81	psf
Negative Shear		
Allowable Shear Stress	198.85	psi
Shear Force - Dead Load	2907.21	lbs
Shear Stress due to Dead Load	68.14	psi
Maximum Allowed Live Load Shear Stress	130.71	psi
Maximum Allowed Live Load Shear Force	5577.06	lb
Maximum Allowed Live Load	873.05	plf
Maximum Allowed Live Load	84.52	psf

Positive Shear			
Allowable Shear Stress	198.85	psi	
Shear Force - Dead Load	2338.40	lbs	
Shear Stress due to Dead Load	54.81	psi	
Maximum Allowed Live Load Shear Stress	144.04	psi	
Maximum Allowed Live Load Shear Force	6145.86	lb	
Maximum Allowed Live Load	1390.47	plf	
Maximum Allowed Live Load	134.60	psf	
Deflection - Between Supports			
Allowable Deflection	0.55	in	**IBC Table 1604.3**
Deflection due to Dead Load	0.09	in	
Maximum Allowed Live Load Deflection	0.46	in	
Maximum Allowed Live Load	1104.84	plf	
Maximum Allowed Live Load	106.95	psf	
Deflection - Cantilever			
Allowable Deflection	0.55	in	**IBC Table 1604.3**
Deflection due to Dead Load	-0.07	in	
Maximum Allowed Live Load Deflection	0.62	in	
Maximum Allowed Live Load	3676.97	plf	
Maximum Allowed Live Load	355.95	psf	

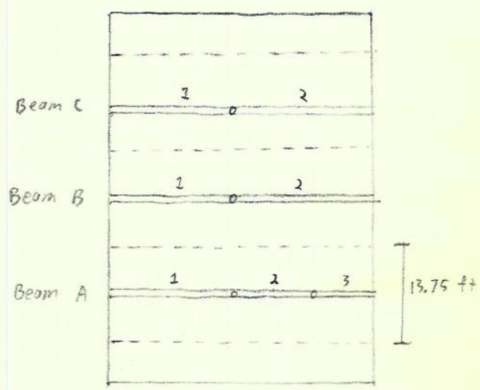
Addition Columns

Dimensions			**Loads**		
Column Thickness	8.00	in	Floor	542.33	lb
Column Radius	--		Joists	482.07	lb
Height	10.00	ft	Girders	186.67	lb
Tributary Length	10.33	ft	Second Floor	1033.00	lb
Tributary Width	10.00	ft	Self Weight	186.67	lb
Tributary Area	103.30	sf	MEP	0.29	lb
Cross Sec Area	64.00	si	Total Dead Load	2431.02	lb
Plank Thickness	1.50	in			
Tributary Length	120.00	in			
Tributary WIdth	1239.60	in			
Tributary Area	148752.00	si			
Material Properties	Table 4D		**C Factors**		
Species	Northern Red Oak		Load Duration Factor	1.00	
E	1300000.00	psi	Temperature Factor	1.00	
Bending Strength	1350.00	psi	Wet Service Factor	0.80	
Shear Strength	205.00	psi	Beam Stability Factor	-	
Tensile Strength	675.00	psi	Size Factor	1.00	
Compression II to Grain	800.00	psi	Flat Use Factor		
Compression Perp. to Grain	885.00	psi	Incising Factor	1.00	
Emin	470000.00	psi	Repetative Member Factor		
Unit Weight Wood	42.00	pcf	Column Stability Factor	0.75	
			Buckling Stiffness Factor		
			Bearing Area Factor	1.00	
Capacity					
Applied Dead Load	2431.02	lbs			
Comp. Strength	477.60	psi			
Load	2431.02	lb			
Comp. Stress	37.98	psi			
Available Comp. Stress	439.62	psi			
Allowable Live Load	28135.47	lb			
Allowable Live Load	272.37	psf			

Appendix C: Structural Analysis (Hand Calculations)

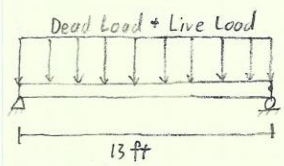
APPENDIX C

Example Beam Analysis

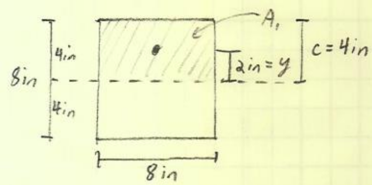


Beam A, Span 1

Profile



Cross Section



Dimensions

Span = ℓ = 13 ft

Beam Depth = h = 8in

Beam Width = b = 8in

Area of Cross Section = $A = 64 \, in^2$

Moment of Inertia = $I = \frac{1}{12} bh^3 = 341.33 \, in^4$

Static Moment of Area = $Q = A_1 \cdot y = 64 \, in^3$

$C = 4 \, in$

Tributary Width = $w_T = 13.75$ ft

APPENDIX C (cont.)

Material Properties

Wood Type = Northern Red Oak - Structural NO. 1

Unit weight (seasoned) = $\gamma = 42\ \text{lb}/\text{ft}^3$

Elastic Modulus = $E = 1{,}300{,}000$ psi

Bending Strength = $f_b = 1350$ psi

Shear Strength = $f_v = 205$ psi

Tensile Strength = $f_t = 675$ psi

Compression Strength Parallel to Grain = $f_c = 800$ psi

Compression Strength Perpendicular to Grain = $f_{c\perp} = 885$ psi

Loading

self weight = $\gamma A = (42\ \text{lb}/\text{ft}^3)(64\ \text{in}^2)\left(\frac{1\ \text{ft}^2}{144\ \text{in}^2}\right) = 18.667\ \text{lb}/\text{ft}$

Dead Load - Short beams = $\dfrac{2n\gamma A_{sb}\cdot\frac{l_{sb}}{2}}{l} = \dfrac{(2)(11)(42\ \text{lb}/\text{ft}^3)\left(\frac{16\ \text{in}^2}{144}\right)\left(\frac{9\ \text{ft}}{2}\right)}{13\ \text{ft}} = 35.54\ \text{lb}/\text{ft}$

Dead Load - Floor = $\dfrac{2n\gamma A_p\frac{l_p}{2}}{l} = \dfrac{(2)(20)(42\ \text{lb}/\text{ft}^3)\left(\frac{12\ \text{in}^2}{144}\right)\left(\frac{1.083}{2}\right)}{13\ \text{ft}} = 5.832\ \text{lb}/\text{ft}$

Assumed Dead Load due to Walls, MEP, etc = $(30\ \text{psf})(13.75\ \text{ft}) = 412.5\ \text{lb}/\text{ft}$

Total Dead Load = $w_D = 472.539\ \text{lb}/\text{ft}$

Bending

Applicable Adjustment Factors:

Load Duration Factor = $C_D = 1$
Temperature Factor = $C_T = 1$
Wet Service Factor = $C_w = 0.85$
Beam Stability Factor = $C_L = 1$
Size Factor = $C_F = 1.3$
Flat Use Factor = $C_{FU} = 1.15$
Incising Factor = $C_i = 1$
Repetative Member Factor = $C_r = 1.15$

APPENDIX C (Cont.)

Allowable Bending strength = $f'_b = f_b \cdot C_D \cdot C_T \cdot C_w \cdot C_L \cdot C_F \cdot C_{fu} \cdot C_i \cdot C_r$

$$f'_b = (1350\ psi)(1)(1)(0.85)(1)(1.3)(1.15)(1)(1.15)$$

$$f'_b = 1972.839\ psi$$

Bending Moment due to Dead Load = $M_D = \frac{w_D \ell^2}{8}$

$$M_D = \frac{(472.539\ lb/ft)(13\ ft)^2}{8}$$

$$M_D = 9982.39\ lb \cdot ft$$

Bending Stress due to Dead Load = $F_{bd} = \frac{M_D c}{I}$

$$F_{bd} = \frac{(9982.39\ lb \cdot ft)(12\ \frac{in}{ft})(4\ in)}{341.33\ in^4}$$

$$F_{bd} = 1403.77\ psi$$

Remaining Available strength for Live Load = $F_{bL} = f'_b - F_{bd}$

$$F_{bL} = 1972.839\ psi - 1403.77\ psi$$

$$F_{bL} = 569.07\ psi$$

Maximum Allowed Live Load Moment = $M_L = \frac{F_{bL} I}{c}$

$$M_L = \frac{(569.07\ psi)(341.33\ in^4)}{4\ in}$$

$$M_L = 4046.69\ ft \cdot lb$$

Maximum Allowed Live Load = $w_L = \frac{8 M_L}{\ell^2}$

$$w_L = \frac{(8)(4046.69\ ft \cdot lb)}{(13\ ft)^2}$$

$$w_L = 191.56\ lb/ft$$

Maximum Allowed Live Load = $\frac{w_L}{w_t} = \frac{191.56\ lb/ft}{13.75\ ft} = 13.93\ psf$

Allowed Live Load = 13.93 psf < 100 psf = Required Live Load

APPENDIX C (cont.)

Shear

Applicable Adjustment Factors:

Load Duration Factor = $C_D = 1$
Temperature Factor = $C_t = 1$
Wet service Factor = $C_w = 0.97$
Incising Factor = $C_i = 1$

Allowable Shear Stress = $f'_v = f_v \cdot C_D \cdot C_t \cdot C_w \cdot C_i$

$$f'_v = (205\,psi)(1)(1)(0.97)(1)$$

$$f'_v = 198.85\,psi$$

Shear Force Due To Dead Load = $V_d = \dfrac{w_d l}{2}$

$$V_d = \frac{(472.539\,lb/ft)(13\,ft)}{2}$$

$$V_d = 3071.504\,lb$$

Shear Stress Due To Dead Load = $F_{vd} = \dfrac{V_d Q}{I\,b}$

$$F_{vd} = \frac{(3071.504\,lb)(64\,in^3)}{(341.33\,in^4)(8\,in)}$$

$$F_{vd} = 71.989\,psi$$

Remaining Available Strength for Live Load = $F_{vL} = f'_v - F_{vd}$

$$F_{vL} = 198.85\,psi - 71.989\,psi$$

$$F_{vL} = 126.86\,psi$$

Maximum Allowed Live Load Shear Force = $V_L = \dfrac{F_{vL} I b}{Q}$

$$V_L = \frac{(126.86\,psi)(341.33\,in^4)(8\,in)}{64\,in^3}$$

$$V_L = 5412.64\,lb$$

APPENDIX C (cont.)

Maximum Allowed Live Load $= w_L = \frac{2V_L}{\ell}$

$$w_L = \frac{(2)(5412.64\ lb)}{13\ ft}$$

$$w_L = 832.714\ lb/ft$$

Maximum Allowed Live Load $= \frac{w_L}{w_t} = \frac{832.714\ lb/ft}{13.75\ ft} = 60.56\ psf$

Allowed Live Load $= 60.56\ psf < 100\ psf =$ Required Live Load

Deflection

Applicable Adjustment Factors:

Wet Service Factor $= C_w = 0.9$
Temperature Factor $= C_t = 1$
Incising Factor $= C_i = 1$

Total Allowed Deflection $= \frac{\ell}{360} = \frac{13\ ft \cdot 12\ \frac{in}{ft}}{360} = 0.43\ in = \Delta_t$

Adjusted Elastic Modulus $= E' = E \cdot C_w \cdot C_t \cdot C_i$

$$E' = (1{,}300{,}000\ psi)(0.9)(1)(1)$$

$$E' = 1{,}170{,}000\ psi$$

Deflection Due To Dead Load $= \Delta_d = \frac{5 w_d \ell^4}{384 E' I}$

$$\Delta_d = \frac{(5)(472.539\ lb/ft \cdot \frac{1}{12\ in/ft})(13\ ft \cdot 12\ in/ft)^4}{(384)(1{,}170{,}000\ psi)(341.33\ in^4)}$$

$$\Delta_d = 0.76\ in$$

Remaining Allowed Live Load Deflection $= \Delta_t - \Delta_d = \Delta_L$

$$\Delta_L = 0.43in - 0.76in$$

$$\Delta_L = -0.33in$$

APPENDIX C (cont.)

Maximum Allowed Live Load = $w_L = \left[\frac{384\,\Delta_L E I}{5\ell^4}\right] 12\ \text{in}/\text{ft}$

$$w_L = \left[\frac{(384)(-0.33\text{in})(1{,}170{,}000\text{psi})(341.33\text{in}^4)}{(5)(13\text{ft}\cdot 12\ \text{in}/\text{ft})^4}\right] 12\ \text{in}/\text{ft}$$

$$w_L = -205.08\ \text{lb}/\text{ft}$$

Maximum Allowed Live Load = $\frac{w_L}{w_t} = \frac{-205.08\ \text{lb}/\text{ft}}{13.75\ \text{ft}} = -14.91\ \text{psf}$

Allowed Live Load = -14.91 psf < 100 psf = Required Live Load

Appendix D: Design Calculations

Composite Beams:

Material Properties: Red Oak			**Material Properties: A36 Steel**		
Elastic Modulus	1170000.00	psi	Elastic Modulus	29000000.00	psi
Unit Weight	42.00	pcf	Unit Weight	483.84	pcf
Bending Strength	1972.84	psi	Yield Strength	36000.00	psi
Shear Strength	198.85	psi			
Section Properties			**Equivalent Section**		
Width	8.00	in	Width	198.29	in
Steel Thickness	0.50	in	Depth	0.50	in
Wood Thickness	8.00	in	Steel Moment of Inertia	2.07	in^4
Modular Ratio	24.79		Wood Moment of Inertia	341.33	in^4
			Steel Area	99.15	si
			Wood Area	64.00	si
			Centroid Location	0.82	in
			Total Moment of Inertia	2162.59	in^4
Loading					
Dead Load	374.21	plf	dead load steel	13.44	
Live Load	1000.00	plf			
Total Load	1374.21	plf			
Beam Conditions					
Span	13.00	ft			
Qtop	234.28	in^3			
Qbottom	33.14	in^3			

Bending		
Moment	29030.19	ft*lb
Bending Stress - Top	1479.14	psi
Bending Stress - Bottom	131.71	psi
Shear Studs		
shear flow	136.90	lb/in
shear stud capacity	1810.00	lb
required spacing	13.22	in
use spacing	12.50	in
Use 3/4" Stud with 2" Thread Penetration @ 12.5"OC		
Shear		
Shear Force	8932.37	lbs
Shear Stress - Top	120.96	psi
Shear Stress - Bottom	0.69	psi
Deflection		
Limit	0.43	in
Deflection	0.35	in

Column:

Loading				**Material Properties**			
dead load	14398.01	lbs		Unit Weight	42.00	pcf	
live load	10383.20	lbs		Compresssion ll to Grain	800.00	psi	
total load	49562.41	lbs		E	1300000.00	psi	
				Adjusted Fc (no Cp)	840.00	psi	
Section				Emin'	470000.00	psi	
Width	10.00	in		Fce	914414.20	psi	
Length	10.00	in					
Area	100.00	si		F'c	524.38	psi	
Area (ft)	0.69	sf					
Ke	1.00						
le	6.50	ft					
Adjustment Factors	Fc	E					
Load Duration Factor	1.00						
Temperature Factor	1.00	1.00					
Wet Service Factor	1.00	1.00					
Beam Stability Factor	-						
Size Factor	1.05						
Flat Use Factor							
Incising Factor	1.00	1.00					
Repetative Member Factor							
Column Stability Factor	0.62						
Buckling Stiffness Factor		1.00					
Bearing Area Factor	1.00						
Axial Compression							
Allowable Strength	524.38	psi					
Compression Stress	495.62	psi					

Addition Joists:

Loading				Material Properties		
Span (in)	123.96	in		Species	Northern Red	
Span (ft)	10.33	ft		Unit Weight	42.00	pcf
Tributary Width (in)	15.00	in		E	1400000.00	psi
Tributary Area (in)	1859.40	si		Bending Strength	1000.00	psi
Tributary Width (ft)	1.25	ft		Shear Strength	170.00	psi
Tributary Area (ft)	12.91	sf		Tensile Strength	575.00	psi
Dead Load (Structural)	60.06	plf		Compression ll to Grain	925.00	psi
Live Load	125.00	plf		Compression Perp. to Grain	885.00	psi
Dead Load (Other)	37.50	plf				
Total Load	222.56	plf				
C Factors	fb	fv	fc	fcperp	Emin	
Load Duration Factor	1.00	1.00	1	1.00	1.00	
Temperature Factor	1.00	1.00	1	1.00	1.00	
Wet Service Factor	1.00	1.00	1	1.00	1.00	
Beam Stability Factor	1.00	-				
Size Factor	1.20	-	1			
Flat Use Factor	1.15	-	-	-	-	
Incising Factor	1.00	1.00	1	1.00	1.00	
Repetative Member Factor	1.15	-	-	-	-	
Column Stability Factor						
Buckling Stiffness Factor	-	-	-	-	1.00	
Bearing Area Factor	-	-	-	1.00	-	

Bending Moment		
Allowable Bending Stress	1587.00	psi
Applied Moment	2968.69	lb*ft
USE 2"x10" JOIST		
Width	2.00	in
Depth	10.00	in
c	5.00	in
Ix	166.67	in^4
Applied Bending Stress	1068.73	psi
1040.72<1587		
Shear Stress		
Allowable Shear Stress	170.00	psi
Area of Cross Section	20.00	si
Q	25.00	in^3
Applied Shear Force	1149.54	lbs
Applied Shear Stress	86.22	psi
83.96<170		
Deflection		
Allowed Deflection	0.52	in
Actual Deflection	0.24	in
0.24<0.52		

Cantilever Beam:

Dimensions				Material Properties	Table 4A	
Span	11.00	ft		Species	Northern Red Oak	
Span	132.00	in		Unit Weight	42.00	pcf
Depth	10.00	in		E	1300000.00	psi
Width	8.00	in		Bending Strength	1350.00	psi
Tributary Width	123.96	in		Shear Strength	205.00	psi
Tributary Area	113.63	sf		Tensile Strength	675.00	psi
Moment of Inertia	666.67	in^4		Compression ll to Grain	800.00	psi
X-Sectional Area	80.00	si		Compression Perp. to Grain	885.00	psi
Plank Thickness	1.50	in				
Floor Finish Thickness		in				
Section Modulus	133.33	ci				
Q	100.00	ci				
Cantilever Length	4.42	ft				
Loading						
Self Weight	23.33	plf				
Dead Load - Structural Span 1	98.06	plf				
Dead Load - Other	30.00	psf				
Service Live Load	100.00	psf				
C Factors	fb	fv	fc	fcperp	Emin	
Load Duration Factor	1.00	1.00	1	1.00	1.00	
Temperature Factor	1.00	1.00	1	1.00	1.00	
Wet Service Factor	1.00	1.00	1	1.00	1.00	
Beam Stability Factor	1.00	-				
Size Factor	1.30	-	1			
Flat Use Factor	1.15	-	-	-	-	
Incising Factor	1.00	1.00	1	1.00	1.00	
Repetative Member Factor	1.15	-	-	-	-	
Column Stability Factor						
Buckling Stiffness Factor	-	-	-	-	1.00	
Bearing Area Factor	-	-	-	1.00	-	
x	4.58					
x	54.96					
x1	53.04					

Applied Uniform Dead Load	431.29	plf	
Applied Point Dead Load (2nd Floor)	529.47	lb	
Span 1	11.00	ft	
Positive Bending Moment			
Allowed Bending Stress	2320.99	psi	
Bending Moment due to Dead Load	5561.28	lb*ft	
Dead Load Bending Stress	500.51	psi	
Maximum Allowed Live Load Stress	1820.47	psi	
Maximum Allowed Live Load Moment	20227.47	lb*ft	
Maximum Allowed Live Load	1901.94	plf	
Maximum Allowed Live Load	184.12	psf	
Negative Bending Moment			
Allowed Bending Stress	2320.99	psi	
Bending Moment due to Dead Load	6553.20	lb*ft	
Dead Load Bending Stress	589.79	psi	
Maximum Allowed Live Load Stress	1731.20	psi	
Maximum Allowed Live Load Moment	19235.55	lb*ft	
Maximum Allowed Live Load	1969.20	plf	
Maximum Allowed Live Load	190.63	psf	
Negative Shear			
Allowable Shear Stress	205.00	psi	
Shear Force - Dead Load	2967.86	lbs	

Shear Stress due to Dead Load	55.65	psi
Maximum Allowed Live Load Shear Stress	149.35	psi
Maximum Allowed Live Load Shear Force	7965.48	lb
Maximum Allowed Live Load	1246.94	plf
Maximum Allowed Live Load	120.71	psf
Positive Shear		
Allowable Shear Stress	205.00	psi
Shear Force - Dead Load	2435.78	lbs
Shear Stress due to Dead Load	45.67	psi
Maximum Allowed Live Load Shear Stress	159.33	psi
Maximum Allowed Live Load Shear Force	8497.55	lb
Maximum Allowed Live Load	1922.52	plf
Maximum Allowed Live Load	186.11	psf
Deflection - Between Supports		
Allowable Deflection	0.37	in
Deflection due to Dead Load	0.04	in
Maximum Allowed Live Load Deflection	0.32	in
Maximum Allowed Live Load	1661.32	plf
Maximum Allowed Live Load	160.82	psf
Allowable Deflection	0.37	in
Deflection due to Dead Load	-0.03	in
Maximum Allowed Live Load Deflection	0.40	in
Maximum Allowed Live Load	5133.75	plf
Maximum Allowed Live Load	496.98	psf

Addition Columns (New Loads)

Dimensions			**Loads**		
Column Thickness	8.00	in	Floor	542.33	lb
Column Radius	--		Joists	602.58	lb
Height	10.00	ft	Girders	233.33	lb
Tributary Length	10.33	ft	Second Floor	1200.17	lb
Tributary Width	10.00	ft	Self Weight	186.67	lb
Tributary Area	103.30	sf	MEP	0.29	lb
Cross Sec Area	64.00	si	Total Dead Load	2765.37	lb
Plank Thickness	1.50	in			
Tributary Length	120.00	in			
Tributary WIdth	1239.60	in			
Tributary Area	148752.00	si			
Material Properties	Table 4D				
Species	Northern Red Oak		**C Factors**		
E	1300000.00	psi	Load Duration Factor	1.00	
Bending Strength	1350.00	psi	Temperature Factor	1.00	
Shear Strength	205.00	psi	Wet Service Factor	0.80	
Tensile Strength	675.00	psi	Beam Stability Factor	-	
Compression ll to Grain	800.00	psi	Size Factor	1.00	
Compression Perp. to Grain	885.00	psi	Flat Use Factor		
Emin	470000.00	psi	Incising Factor	1.00	
Unit Weight Wood	42.00	pcf	Repetative Member Factor		
			Column Stability Factor	0.75	
			Buckling Stiffness Factor		
			Bearing Area Factor	1.00	

Applied Dead Load	2765.37	lbs
Capacity		
Comp. Strength	477.60	psi
Load	2765.37	lb
Comp. Stress	43.21	psi
Available Comp. Stress	434.39	psi
Allowable Live Load	27801.11	lb
Allowable Live Load	269.13	psf

Appendix E: Foundation

Seismic Design Category:

Site Information	
Ss	0.254
S1	0.069
Fa	1.6
Fv	2.4
Sds	0.270933
Sd1	0.1104
Seismic Design Category	B

Existing Conditions:

Compression		
Granite Compressive Strength	29000.00	psi
Concrete Comp. Strength	3000.00	psi
Steel Yield Strength	60000.00	psi
Unit Weight - Red Oak	42.00	pcf
Floor Load	2593.50	lb/ft
Joists	61.89	lb/ft
Beams	19.16	lb/ft
Superimposed Dead Load	390.00	lb/ft
Superimposed Live Load	1040.00	lb/ft
Total Load on Sill	4104.55	lb/ft
Stress on Sill	42.76	psi
Load on Granite Wall	4123.22	lb/ft
Comp. Stress on Granite Wall	24.01	psi
Wall OK In Compression		
Shear Reinforcement		
6x6 W4x4	0.08	si/ft
Minimum Soil Load	0.00	lb/ft
Maximum Soil Load	390.00	lb/ft
Resultant Soil Force	1267.50	lbs
Reaction At Base of Wall	845.00	lbs
Asmin	0.01	si
USE 6x6 W4x4 to Comply With IBC 1807.1.6.2		

Addition Foundation Wall:

Loading		
Building Load	1800.00	plf
Soil Lateral Load	60.00	psfpf
Material Properties		
Compressive Strength	3000.00	psi
Rebar Yield Strength	60000.00	psi
Concrete Unit Weight	150.00	pcf
Dimensions		
Assumed Wall Thickness	12.00	in
Wall Height	3.00	ft
Wall Length	31.25	ft
Design		
Total Load	2250.00	plf
Use Thickness	11.50	in
1.2*t*f'c	41400.00	good
No Reinforcement Necessary	IBC Table 1807.1.6.2	

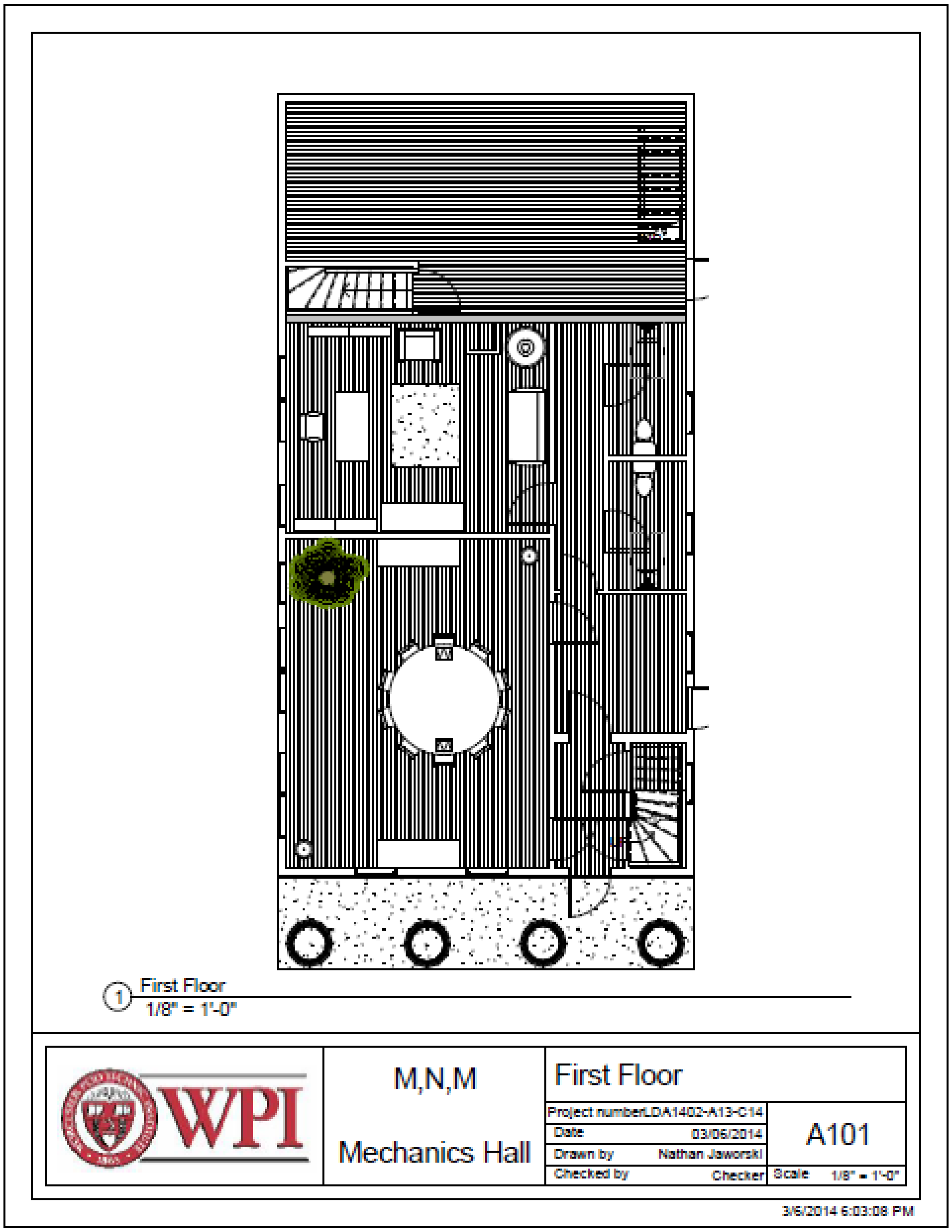
Wall Footings:

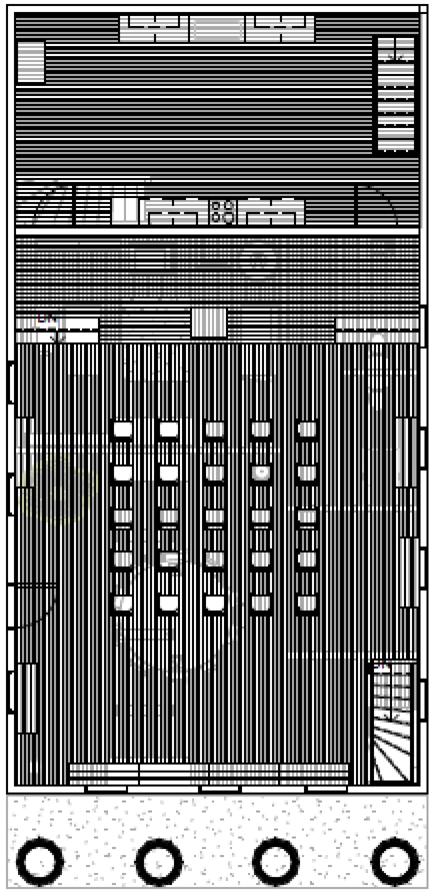
Material Properties		
Soil Bearing Pressure	2000.00	psf
depth of soil above footing	3.00	ft
Effective Bearing Pressure	1820.00	psf
Compressive Strength	3000.00	psi
Yield Strength Rebar	60000.00	psi
Unit Weight Concrete	150.00	pcf
Prelim Dimensions		
Footing Height	12.00	in
Loading		
Footing Weight	150.00	plf
Building/Wall Load	2250.00	plf
Total Load	2400.00	plf
Design		
bmin	1.32	ft
USE b =	16.00	in
Actual Bearing Pressure	1800.00	psf
Reinforcement Depth	9.00	in
Vc	-1012.50	lb/ft
Design Shear	1752.71	lb/ft
dreq	0.58	in
M	31.64	ft*lb/ft
R	0.29	
USE PLAIN CONCRETE		
Footing Height	12.00	in
Footing Width	16.00	in

Column Footings:

Loads/Resistance			
Building Load	15274.81	lbs	
Allowable Bearing Pressure	2000.00	psf	
Concrete Compressive Strength	3000.00	psi	
Rebar Yield Strength	60000.00	psi	
Column Width	10.00	in	
Minimum Cover	3.00	in	
Design			
Areq	7.64	sf	
Minimum Length	2.76	ft	
Minimum Length	33.16	in	
Actual Length	36.00	in	
Actual Area	1296.00	si	
Effective Bearing Pressure	1697.20	psf	
Assumed Depth	18.00	in	
Vc	6034.49	lbs	
Bo	112.00	in	
Required Depth	0.25	in	
USE D =	12.00	in	**IBC 1809.4**
M	83162.85	in*lb	
Asreq	0.15	si	
Asmin1	1.18	si	
Asmin2	1.44	si	
USE NO 7 @ 5"			
Depth of Rebar	9.00		

Appendix F: BIM Models





1 Second Floor
1/8" = 1'-0"

WPI	M,N,M	Second Floor	
	Mechanics Hall	Project number LDA1402-A13-C14	A102
		Date 03/06/2014	
		Drawn by Nathan Jaworski	
		Checked by Checker	Scale 1/8" = 1'-0"

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1 Section 1
1/8" = 1'-0"

WPI	M,N,M	Elevation Section View
	Mechanics Hall	

Project number	LDA1402-A13-C14	A103
Date	03/06/2014	
Drawn by	Nathan Jaworski	
Checked by	Checker	Scale 1/8" = 1'-0"

3/6/2014 6:03:09 PM