Project #: LDA-1206

CAPTURING REUSE CONSIDERATIONS DURING INITIAL STRUCTURAL DESIGN

A Major Qualifying Project Report

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in Civil Engineering

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Abstract

The Major Qualifying Project focused on improving the lifecycle of public buildings by designing for adaptive reuse. The project team developed an alternate structural design for an elementary school that minimizes interior columns and anticipates future needs. Sustainable strategies were examined to lower maintenance cost and extend the building's lifespan. A cost estimate of the redesign was generated for comparisons. The final deliverable establishes a rating system that assesses the flexibility of the structural design for future use.

Acknowledgement

This Major Qualifying Project would not have been possible without the guidance and assistance of the faculty advisor, Professor Leonard Albano. His advice and dedication to the project was instrumental in the team's success. In addition, the team would like to acknowledge Consigli Construction Co., Inc. for providing the necessary documents to complete the project.

Capstone Design Statement

As part of the Accreditation Board for Engineering and Technology (ABET) requirement, all Civil Engineering degrees must complete a Capstone Design Experience. The capstone design will use skills “acquired in previous coursework, new learning, and appropriate engineering standards (…) [and] will also incorporate most of the eight, realistic constraints: economic, environmental, sustainability, constructability or manufacturability, ethical, health and safety, social and political.” This project incorporated five of these constraints: economic, sustainability, constructability, health and safety, and social.

Economic

One of the main constraints with any civil engineering project is its economic feasibility. To address this constraint, cost estimates of the determined structural schemes were performed. The estimations were benchmarked to those of Consigli, who is the primary construction manager on the project, to compare any differences and to assist in choosing the best alternative design. The project also investigated the use of geothermal energy as heating and cooling systems, which can potentially save the building owner money.

Sustainability

Sustainability was a major aspect of this project. The project’s redesign of an elementary school building took a proactive approach to the reuse of the building and prevented the abandoning of the building when the school is no longer required. The redesign introduced alternatives to the building’s structural aspects as well as its MEP systems, such as the HVAC, in order to provide a more energy efficient design.

Constructability

The constructability constraint for this project dealt with the feasibility of constructing a structural framework that allowed for a change in occupancy which included more open space and increased design floor loads, as well as ease of implementing other sustainable strategies. It also addressed established practices for construction of different framing schemes as well as the use of standard beam sections. The design of the columns considered constructability by limiting the number of column sizes used throughout the building.

Health and Safety

With any building the health and safety of its occupants is of extreme importance. Therefore, the project addressed this constraint by determining and complying with the 6th Edition of the *Massachusetts State Building Code* provisions for structural design. These provisions included the actual design requirements for load capacity as well as fire safety ratings. Also, steel and concrete were compared in terms of their fire safety ratings.

Social

The social constraint was addressed by thinking about the possible future needs of the community in the Town of Dedham. By enabling a school's design to be flexible for repurposing in the future, it allows other members of the community to be able to use the building.

Authorship

Everybody contributed equally to the outcome of this project. The following table summarizes the principal responsibilities of group members for the work of this project. The person who is accredited for the primary area performed the design calculations and was responsible for the corresponding writing and Appendix sections.

Primary Area	**Responsible Group Member(s)**
Capstone Design	Elena Fajardo
Introduction	Olivia Richards
Current Situation	Olivia Richards
Background on Framing Systems	Olivia Richards, Elena Fajardo
Revit Models	Elena Fajardo
Beam Analysis & Design	Olivia Richards, James Seed
Girder Analysis & Design	Olivia Richards, James Seed
Column Analysis & Design	Alison Corriveau
Connection Analysis & Design	James Seed
Base Plate Analysis & Design	Alison Corriveau
Footing Analysis & Design	Alison Corriveau
Lateral Force Resisting System Design	Elena Fajardo
Cost Estimate	Elena Fajardo
Demolition vs. Repurposing	James Seed
Concrete vs. Steel	Alison Corriveau
Alternate Structural Designs	Olivia Richards
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1.0 Introduction

Abandoned public buildings create problems for communities. They can create costly problems that are a drain on the town's budget and impede the development of neighborhoods (Palmer, 2008). According to a survey conducted in 2004, most U.S. public school buildings are abandoned after 60 years of use because they can no longer meet the needs of the occupants (O'Connor, 2004). This was usually due to the change in student population since the school was built - a result of "the baby-boom echo, immigration, and migration" (Lewis, 2000). In 2005, a study found that 22% of the public schools were within 5% of their capacity and another 10% exceeded the capacity of the building. Furthermore, it was discovered that in order for these towns to alleviate overcrowding, 78% of the schools have used portable classrooms, 53% have turned non-classroom space into classrooms, and 35% have built new permanent buildings or additions (Chaney, 2007). Therefore, the school buildings require costly funds in order to adapt to the changing population and to prevent an accumulation of abandoned buildings.

Many towns own school buildings that are in need of renovations or will face abandonment. A study conducted by the U.S. Department of Education in 1999 found that 1 in 4 schools possessed an onsite building that was "in less than adequate condition" (Lewis, 2000). 4 to 6% of schools reported that they had buildings that were in poor condition and 1to 2% reported that their buildings needed to be replaced because of non-operational conditions or substantial substandard performance (Lewis, 2000).

Due to limited funds and short lifespan of buildings, local governments have few options to provide their community with adequate public school facilities. Therefore, there is a critical need to sustainably extend the life cycle of a building to efficiently serve the public.

1.1 Problem Statement

Towns frequently need to provide funding for costly building renovations and modifications to their current public school buildings. Sometimes the town cannot fund the renovations needed to maintain their current school building and resort to spending taxes on a new construction as an alternate solution. However, the previous school building is then left abandoned with few options of reuse due to its limited functionality. This is a recurring cycle for most towns as they constantly face the need to abandon and tear down a school building and incur the cost of a new construction.

1.2 Objective

The purpose of this project is to design a public school building that can be adapted to the community's future needs without costly renovations. This project will use the design for the Avery Elementary School in Dedham, Massachusetts as a case study. The Avery School is a new construction that is to replace the previous outdated Avery School building. Following the building layout of the new school, provided by Dore and Whittier Architects, Inc., this project will consist of a structural design that can be easily repurposed in the future. This design focuses on permitting a more versatile architectural layout. Furthermore, the proposed design will present a discussion of sustainable features, such as a geothermal system that increases the building's energy efficiency. As a result, the proposed solution is expected to increase the building's lifespan and to reduce operate maintenance costs. A cost estimation of the construction and design will be prepared and compared to the current cost of the school. After evaluating this example, the team will determine a list of recommendations for the structural design of school buildings that can be followed to achieve the construction of a building that is advantageous to the community.

2.0 Background

The team used resources from Consigli Construction Company, Inc. (Consigli), Engineers Design Group, Inc. (EDG), and Dore & Whittier, Inc. (D&W). EDG was the structural engineering firm that designed the Avery Elementary School Project. D&W then put that design on plans and published the drawings used throughout this project (D&W, 2010). Lastly, Consigli is the company responsible for the construction of the Avery Elementary School. The team used its requisition and other materials for the cost estimation for this project.

2.1 Current Situation

Every building is constructed with the intended purpose to meet the needs of the occupants. Unfortunately, the needs of the people and the integrity of the building change with time, causing the building to be considered inadequate. The building must then be evaluated for the possibility of renovations or demolition.

2.1.1 The Old Avery Elementary School

Figure 1: The Old Avery School (Friends of Avery Committee, 2011)

The town of Dedham has been utilizing the Avery Elementary School building (as displayed in Figure 1) located on High Street in East Dedham since 1921 (Friends of Avery, 2011). After 90 years of use, the town has determined that the building can no longer function as an elementary school. The conditions of the Avery School were evaluated in 2007 by teams of assessors hired by the Commonwealth of Massachusetts. The Friends of Avery, a committee formed as a part of P.R.I.D.E (Planned Reinvestment in Dedham Education, 2011), has publicized the findings of the state assessment. The assessment found that the building possesses "physical problems and structural limitations that render it obsolete" (Friends of Avery Committee, 2011). For example, the classroom sizes are below the current standard, causing the rooms to be overcrowded, as pictured in Figure 2. The building does not possess a cafeteria

Figure 2: Overcrowded Classroom (Friends of Avery Committee, 2011)

for preparation of students' meals, causing it to fail current educational standards. The exterior brick walls (pictured in Figure 3) are crumbling and portions of the building's gymnasium have collapsed and been replaced. The windows of the building are single-pane and lack insulation. Also, the doors to the exterior are warped and do not function properly (Friends of Avery, 2011).

Figure 3: Exterior Brick (Friends of Avery Committee, 2011)

The building's mechanical systems are also failing. The assessor determined that mechanical and temperature control systems are obsolete. The ventilation system is considered to be inadequate as well as the plumbing. The electrical system is "outdated and at capacity" and the lighting throughout the building is considered to be "poor" (Friends of New Avery Committee, 2001). The layout of the building is also a problem since it does not fully comply with the Americans with Disabilities Act because the restrooms are only located in the basement.

2.1.2 The New Avery Elementary School

Because of the multitude of issues with the Avery School building, the town decided that a complete new construction of the facility would be the best option. The new Avery School project has an estimated cost of $21.1 million, which is approximately $300 per square foot. The construction project was managed by Consigli Construction Co., a construction manager and general contractor headquartered in Massachusetts. The new building will have three floors instead of two, allowing the proper size for the number of students. The new building will accommodate about 310 students and have 17 general classrooms. It will allow the student population to possibly grow to 375 students in the future. The layout will consist of bathrooms on each floor off of a central corridor that will provide access to all classrooms (Friends of Avery, 2011).

Figure 4: Sketch of New Avery School Design (Friends of New Avery Committee, 2011)

The new Avery Elementary School (as displayed in Figure 4) will possess all of the needed features that the old Avery School lacked. As a 90-year-old building, the old Avery School failed to meet the needs of the occupants. The building was not built with sustainable materials to increase its lifespan. The layout of the building could not be modified to adjust to current needs, and the cost of maintenance became too high for the town to fund. The Town has created a committee to determine the fate of the old Avery School, but they have yet to decide on its new function (Dedham Transcript, 2011). Therefore, learning from the Old Avery School's current situation, it would be advantageous for the new Avery School to have a design that can change to fit the needs of the town.

2.2 Repurposing of Loughborough University Buildings

In the past few decades there have been major improvements in waste management and sustainable energy in the construction of new buildings. According to the United States Environmental Protection Agency (EPA), only 8% of the Construction and Demolition (C&D) waste is produced during construction, while 48% is generated during demolition (EPA 2011). LEED certification has driven down the waste produced during construction, but there has been significantly less research focused on the end of a building's life cycle. When a building is no longer functional or needed within the community the easy solution is demolition. Although demolition saves time and makes space for a new building, the process can be rather costly due to the heavy machinery required and the transportation of materials. Therefore, investing in the life cycle of a building and planning for reuse after the building's original occupancy is served can save time and money.

Loughborough University in Leicestershire, England, has harnessed the benefits of adaptive design for the Engineering Buildings on their campus. The design of these buildings is based on a simple grid system that "warrants continuity of future development for the University" (Fuster et al, 2006). The buildings are constructed of pre-cast concrete, comprising four 53-foot, 3-inch square units. Additionally, four corner columns support a girder system spanning the entire dimension of the square units with a floor to ceiling height of 10 feet (Fuster et al, 2006). Large structural spans has allowed for flexible functionality, permitting the University to modify the interior layout of the building over the years. This has also helped the building avoid becoming quickly outdated. The first building on campus to use this design was the civil engineering building which was constructed in 1970. With the advantages of adaptive design, the University was able to continually alter the building to accommodate new technology, increased enrollment, and a larger group of staff. Forty years after its construction, the university added an additional floor to the civil engineering building to accommodate the department's growth. Designing with the intention of repurposing has not been a popular solution for sustainability; however, this concept has been influenced by the renovations of dated buildings occurring over the past few decades.

2.3 Repurposing of Buildings

Changes in demographics, budget cutbacks, and overcrowding have encouraged communities to reuse old buildings. Initiating new construction and demolition can be costly and unpopular in a straining economy.

2.3.1 Success in Repurposing Out-Dated Buildings

A process termed "adaptive reuse" has been gaining popularity over the past decade in small rural communities and low-income, densely populated areas where they cannot afford the time, cost, and space to solve their needs for new public buildings (Spector, 2003). Critical considerations for adaptive reuse include the building's structural layout and also health, safety, and accessibility requirements. Out-of-date mills and factories are high potential candidates for adaptation due to their open space layout and resistance to large loads. Small, rural towns, such as Littleton, New Hampshire, have converted old furniture stores and factories into public school facilities due to their open space and availability (Lawrence, 2003). Lacking undeveloped or affordable land, Cartwright School District in Phoenix, Arizona transformed an abandoned mall into multiple educational facilities, including an elementary school and middle school in only ten months (Spector, 2003).

2.3.2 Issues in Repurposing

Clearly there are numerous benefits from reusing buildings; however, renovations are still costly and many issues arise. Both Miami-Dade County and Detroit Public School Districts hoped to convert dated hospitals into educational facilities, but the hospital designs proved to be "structurally inappropriate" (Spector, 2003). Hallways and stairwells were too narrow to be used for a school building, and structural columns were too close to create efficiently sized classrooms. Zoning regulations also hindered these school districts from using old office buildings. Health and safety regulations play prominent roles in adaptive reuse and often impede communities from using out-of-date buildings. For instance, in California, seismic design and construction requirements for public schools prevent or delay reuse possibilities. Therefore, these state policies generally favor new construction, despite their costs (Spector, 2003).

2.3.3 Designing for Repurposing

A promising solution to these issues is designing for reuse. Since adaptable property allows owners to alter the use of their land to respond to demographic changes without major construction and renovations, mixed developments have become increasing popular across the country (Arge, 2005). Over the past decades, numerous studies have been proving the benefits of designing for adaptive reuse. Moffat Russel provides insight about the design reusable building in the publication "Adaptability of Buildings". General recommendations for adaptive reuse design from this publication are shown in Table 1.

Table 1: Structural Strategies for Repurposing (Russell, 2001)

Foundation:	Design to allow for vertical expansion
Superstructure	The story height needs to be high enough to accommodate proposed uses and also low enough to avoid waste
	Post-tensioning floor slabs are key components for fast erection and also allow for slim slab depth
	Usable floor space per floor is also defined by service zone requirements that include HVAC, power, piping, etc.
	Rely on a central core for lateral load resistance to allow for local changes to the structure without compromising structural integrity
	Design the “lower” floors for larger loads to be capable of withstanding future additions
	Increase the height of the “lower” floors to enable a range of future uses.
	Coordinate the structural design with all planned uses
	Use a wide structural grid to provide adaptable space. Depending on the structural system and project funding, the recommended limits for span are roughly 6-12m (about 20-40ft)
Envelope	Design the building envelope so that it is independent of the structure and can be easily separated
	Design for versatility to accommodate changes in the interior layout
Interior Spaces	Opt for “Loose Fit”, Multifunctional Spaces
	Use interior partitions
	Use more than the minimal spatial areas and floor heights to ensure adaptability will comply with various building codes/regulations

Few buildings have been designed for reuse; therefore, there are a limited amount of research and resources to use as a guide (Russel, 2001). Integrating adaptability into the design of a building adds additional initial cost, causing this design to be unpopular for low-bid construction. However, according to the Norwegian Building Research Institute, not only does adaptable design promote long term investments, but it also increases cost benefits over a long period of time (Arge, 2005). Instead of having to demolish the building after its initial uses are no longer needed, the building can be adapted to adjust to a new market. Therefore, the basis of designing for reuse is looking at the life-cycle costs as opposed to just initial costs, especially in the public sector. An investigation in extending the life cycle of a building can reveal that there is true worth in fronting the high initial costs. Also, evaluating structural materials and sustainable design strategies relieves high life-cycle costs and maximizes the life-span of a building.

2.3.4 Construction Materials

In 2004, Research Scientist Jennifer O'Connor conducted a demolition survey of buildings in North America. She recorded demolitions of 227 buildings, 105 of which were non-residential. Her study included building age, building type, structural material and reason for demolition. The survey results are depicted in the pie chart in Figure 5. (O'Connor, 2004)

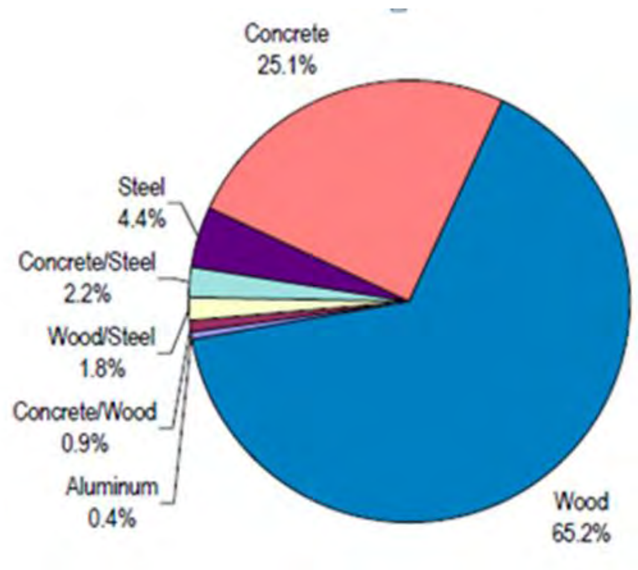


Figure 5: Proportion of all 227 demolished buildings by primary structural material (O'Connor 2004)

The figure shows that wood structures and concrete structures make up the majority of demolished buildings. Less than 9% of the buildings demolished had steel in the framework, either by itself or used with wood or concrete. Figure 6 shows the ages of buildings that were demolished, and their structural material. Out of 94 non-residential buildings, there weren't many that were over 100 years old, and the majority was constructed from steel. (O'Connor, 2004) These studies show that when comparing building materials, buildings constructed of steel are less likely to be demolished.

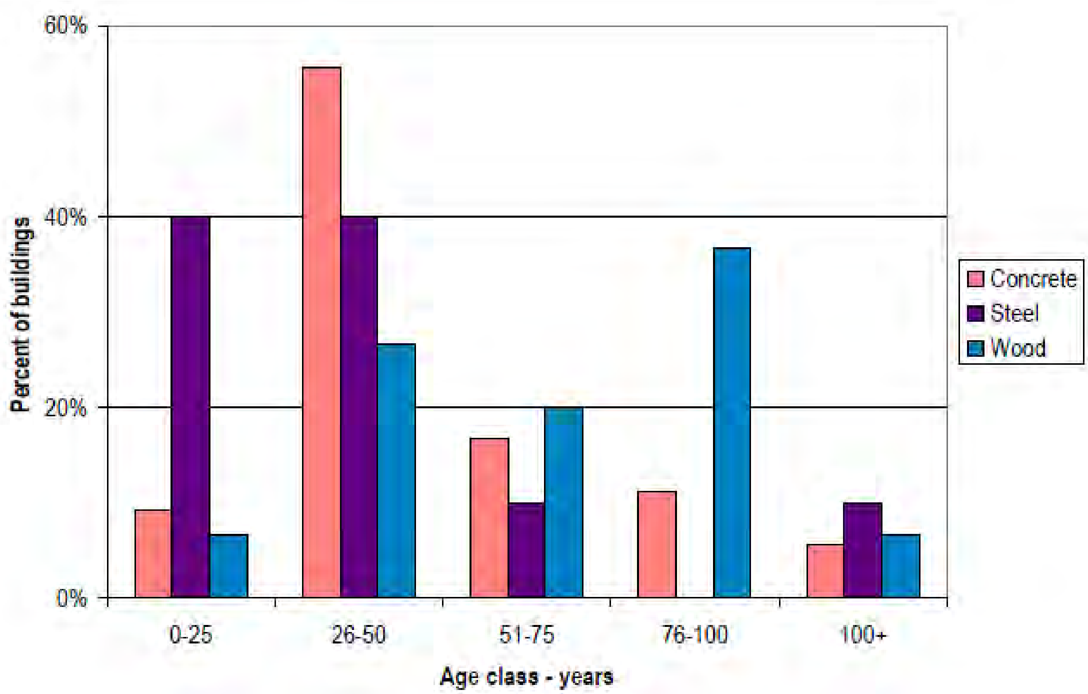


Figure 6: Distribution of 94 non-residential buildings by age class and by structural material, (O'Connor 2004)

When it comes to new construction, sustainability has had a large impact on the construction-sector. Concrete and steel both have sustainable aspects to their use. Concrete is usually made up of materials from local suppliers, which reduces the carbon dioxide emissions from transport of the aggregate and cement. Steel, however, is easily recyclable. 28% of the steel going into buildings today is recycled steel. Also, when a steel building is taken down, 66% of that steel is recycled (Emerson, 2005).

In the Northeast, steel is a much more commonly used construction material than steel. Concrete and steel both have their advantages and disadvantages. Steel does have greater spanning capabilities; however a precast concrete system can also span great lengths. A life-cycle cost analysis can determine which system is more beneficial.

2.4 Sustainable Design Strategies

A building design is truly sustainable when it meets the criteria of economic viability, social awareness and environmental sensitivity (Cunningham Group, 2011). Therefore, a building must be cost effective, take into account the needs of the owner and community, and be environmentally friendly. This can be achieved by having a building that has low maintenance costs, adaptable to the needs of the community, has a long life-span (i.e. durable) and minimizes carbon-emission. According to a Commercial Building Energy Survey made in 2008, heating, ventilation and air-condition systems (HVAC) and lighting accounted to more than 50% of energy use for commercial buildings (CBECS, 2008). Energy consumption can be reduced by implementing passive design strategies, improving building envelope, and using more efficient system and/or using renewable energy technologies such as geothermal energy (Architecture 2030, 2011).

2.4.1 Passive Design Strategies

A reduction of energy consumption in a building due to maintaining internal climate and lighting can be achieved by implementing passive solar designs, having natural ventilation and utilizing natural lighting. Passive solar designs, which refer to the use of the sun's energy to heat and cool living spaces, does not use any mechanical systems and are used because it takes advantage of natural energy. It also lowers the cost of building operations. For new buildings in cold climate regions, it can have passive solar heating in the winter, and passive solar cooling in the summer. Examples of passive solar designs are operable windows and thermal chimneys which permit flow of outside air into the building and vice-versa. When combined properly, they can contribute to the heating, cooling, and day lighting of nearly any building. Energy consumption and costs due to lighting can be reduced by having more natural lighting. This can be achieved by having more windows in a building. Through these passive design strategies, the energy loads/demands can be decreased, benefitting the owner by lowering maintenance costs and benefitting the environment as well. (Passive Solar Design, 2012)

2.4.2 Geothermal Energy

To heat and cool buildings, geothermal energy, which is simply heat generated from the Earth, can be used. It can be obtained from one of two sources, ground source geothermal, which refers to the shallow ground and hot water, or deep well geothermal, which consists of drilling wells a few miles beneath the Earth's surface or even deeper to extremely high temperature of molten rock (Renewable Energy World, 2011). For institutional buildings, geothermal energy is generally obtained through the use of geothermal heat pumps (GHP), also referred to as ground-source heat pumps (GSHP) or geo-exchange (Whole Building Design Guide, 2011). For most locations, the shallow ground of the Earth, which is considered the upper 10 feet, maintains a nearly constant temperature of 50° to 60°F. Therefore, for regions such as Northeast, the earth can be used as a heat source during the winter and a heat sink during the summer through the use of GSHP. As a result, the internal climate of a building can be easily maintained. The heat obtained from the ground can also be used to heat water.

A GSHP system consists of basically three principle components: 1) earth connection subsystem 2) heat pump subsystem and 3) heat distribution system, which are shown in Figure 7 (Geothermal Heat Pumps, 2012). In the earth connection subsystem, there are a series of flexible pipe "loops" that contain water and run through the shallow ground. The water circulating in the pipes absorbs/relinquishes heat within the ground, when the soil is warmer/cooler than the ambient air. For heating, the heat pump subsystem then removes the heat from the pipes, and transfers it to the building. This process is reversed for cooling. A schematic view of the heating and cooling system is shown in Figure 7. Through the heat distribution system consisting of ductwork, the heated and cooled air from the geothermal pump is transferred throughout the building. There are four basic types of GSHP systems which are horizontal closed-loop, vertical closed-loop, pond/lake closed loop and open-loop system. Closed loop systems uses heat absorbed by the circulated fluid in the pipes while open loop systems use groundwater directly as the heat transfer fluid.

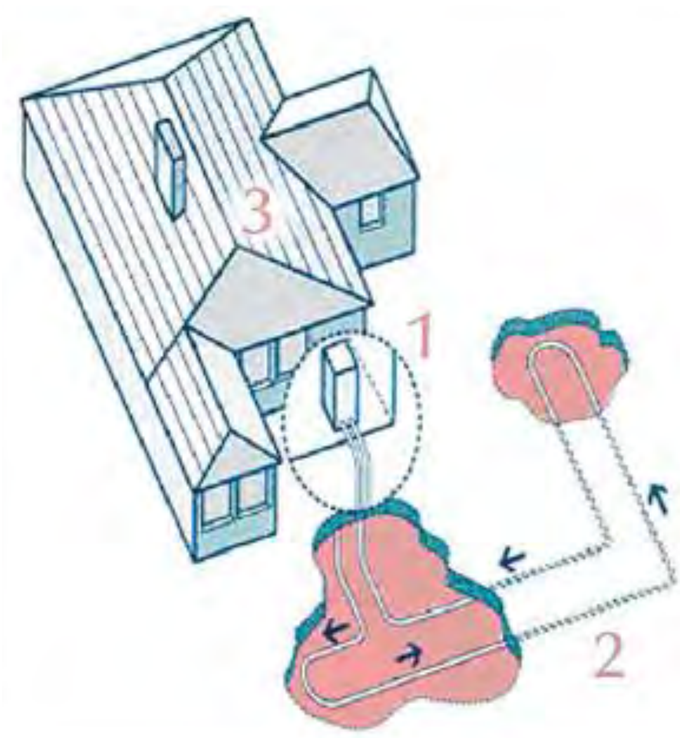


Figure 7: The Three GSHP System Major Components: (1) Heat Pump (2) Earth Connection, and (3) Heating/Cooling Distribution System from RETScreen

GSHP systems are a clean alternative for heating and cooling buildings and are very energy efficient. According to RETScreen International, GSHP systems are one of the fastest growing applications of renewable energy in the world (RETScreen, 2005). They are more advantageous than traditional HVAC system which use an air or water source because they

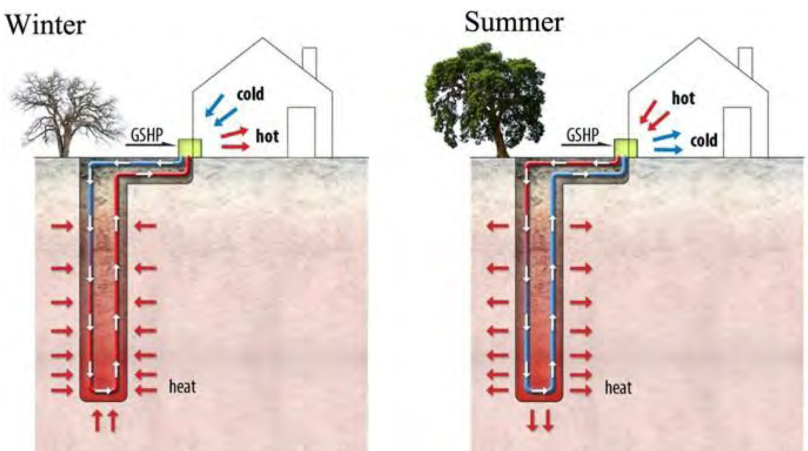


Figure 8: Schematic View of a Ground Source Heat Pump (GSHP) Heating and Cooling System from Emerging Geothermal Energy Technologies

transfer natural existing heat, rather than producing heat from electricity, fossil fuels, or biomass. GSHP also help provide better comfort levels in non-residential buildings than traditional systems because it maintains an appropriate zone-level temperature. The installation costs of GSHP system is typically twice as much as conventional air source systems (Geothermal Heat Pump Resource, 2011). Nonetheless, GHPs typically reduce energy consumption by 30% to 70% in the heating mode and 20% to 50% in the cooling mode (RETScreen International, 2005). With the benefits of GSHP systems, it would be ideal to determine the feasibility of having these systems for school buildings in order to save on costs and consumption.

2.5 Structural Framing Systems

The structural scheme of a building plays a significant role on the ability of a building to be repurposed. The location of load-bearing columns or walls limits the amount of open space to move and add partition walls. Therefore, in order to be flexible with altering architectural layouts, it is ideal to minimize or even eliminate the amount of load-bearing columns or walls. However, by doing so, there is increased necessity for floor systems to be able to resist a greater amount of load over a larger span, which causes a need for deeper beams, or a different framing system. Some of these framing systems include trusses, truss girders and open-web steel joists.

2.5.1 Truss Design

A truss consists of a combination of triangles and tetrahedrons, and it can resist a great amount of load depending on the spacing and height of the truss. It can span an entire width of a building, and the depth of the truss can cover the height of floor. As a result, trusses allow more flexibility than girders in altering the interior layout of a building. Interior walls may be moved/added with the redesign. However, if the truss systems require a certain height, this must be taken into consideration to the overall height of Avery School. Therefore, the appropriateness of a truss system for a floor must be determined.

2.5.2 Truss Girders

A truss consists of a combination of I-sections and triangles to support a member system. The I-sections are oriented in a way so that the truss members are only subject to compression and tension forces as opposed to a girder which is subject to bending. This type of system will allow for forces due to concentrated loadings to be dispersed over the system. An example of a truss girder is shown in Figure 9. This type of structural framework is usually used in bridges, but can also be used to support floors, ceilings and roofs.

Figure 9: Example of Truss Girders (Osborn's Models, 2011)

2.5.3 Joist Design

According to the Consulting Engineers, Corp., "joists are closely spaced beams, which are used to support floor sheathing" (Structural Engineering Framing Design, 2011). They can be made from wood, concrete or steel and resist bending, bearing and deflection. Joists can be defined as small parallel chord trusses that usually consist of members made from bars, small angles or other rolled shapes (McCormac, 2008). An example of joist is shown in Figure 10.

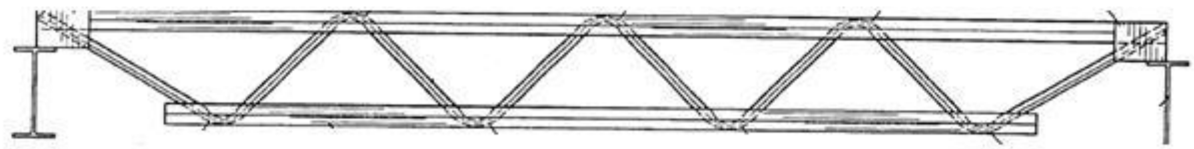


Figure 10: Steel Joist with Bent Rolled Bars (Free Patents Online, 2011)

This type of structural design is suitable for use in low-rise structures such as schools and houses and for floor systems that do not experience a lot of concentrated loads, but mainly uniform loads (McCormac, 2008). For all joists, their key benefits are that different MEP systems can be easily placed between the bracing, and a ceiling can be suspended on the bottom of the joists or a floor added on top. With the K-series joists, they are also relatively light in weight when compared to I-beams. They contribute to lowering building costs because they are generally a lower cost per foot because they require less volume of steel. The longspan joists are generally used for directly supporting floor or roof slabs or decks between walls, beams and main structural members.

Longspan steel joists provide more open space because they are able to span long lengths. Therefore, they are an important tool in anticipating the structural design of a building for reusability. Longspan Steel Joists are used for direct support of floors and roof decks for buildings. These members are able to hold loads while spanning long distances. They are high strength and their design is economical. When the longspan steel joists are used for roofs they can span up to 144 feet. The steel joists can be used for floor spans that span up to 120 feet (Vulcraft, 2007). Figure 11 below shows the details of the structure of a longspan steel joist member.

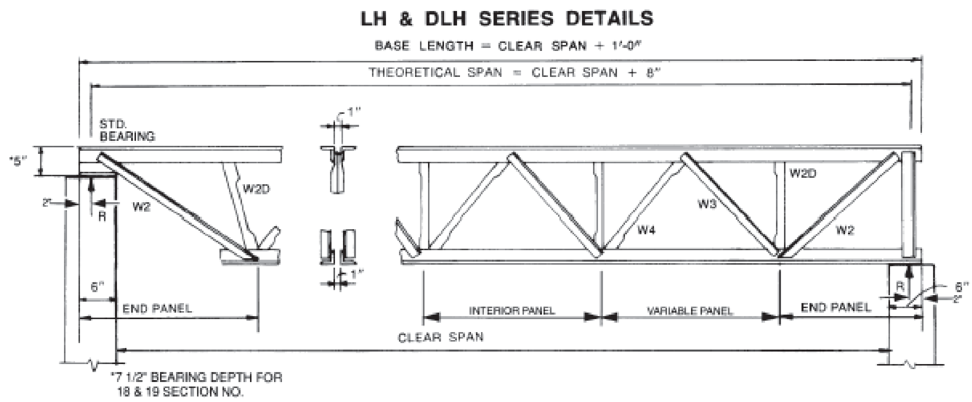


Figure 11: Section of Longspan Steel Joist (Vulcraft Steel Joists and Joist Girders Catalog, 2007)

Vulcraft is a manufacturer of longspan steel joists and joist girders. They have provided steel joists for buildings such as the Prairie School in Racine, Wisconsin. The Prairie School is a 68,000 sq. ft. facility that utilized Vulcraft compound-curved steel frame supporting long span barrel vaulted steel joists. Vulcraft is the largest manufacturer of steel joists in the United States and a division of Nucor Corporation (Vulcraft, 2007).

2.5.4 Staggered Truss System

A staggered truss system allows for the elimination of interior columns and provides a structural design with open floor space. A staggered truss system is useful when designing a building that can be easily repurposed.

The staggered truss system was developed in the 1960s. It is used in the design of hotels, apartment buildings, dormitories, office buildings, and hospitals. (Ochshorn, 2003) It is an innovative system that allows for flexibility in the architectural layout of the building. A staggered truss system is ideal mostly for tall rectangular buildings because of the way it supports the load. Figure 12 shows an example of a staggered truss system. The system consists of story-high steel trusses that alternate sides of the frame on each floor. With this system there is always a truss in between two trusses on

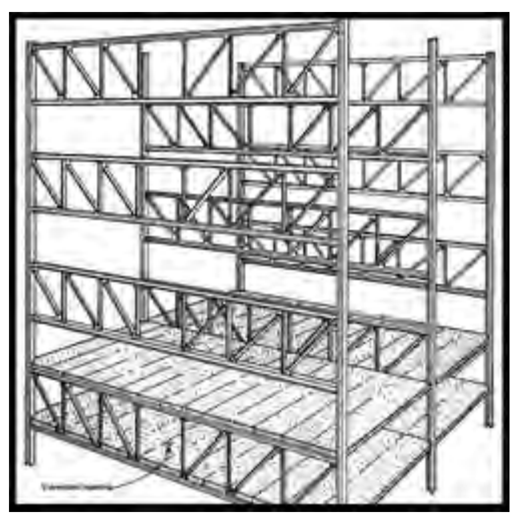


Figure 12: Staggered Truss System Design (Ochshorn, 2003)

the floor below. This system allows for a design with a lower floor-to-floor height and large column-free areas that promote design flexibility.

2.5.5 Beam Grid System

A single layer grid system of beams is another structural framing system that allows for large areas of open space in the floor plan. The grid system consists of beams that are arranged into a grid and rigidly connected to each other. There are various grid patterns that are used in the design of a beam grid system. The different patterns are displayed in Figure 13. The basic two-way grid is the most common design (University of Surrey, 2011). In the grid system, the force is applied perpendicularly to the plane as it would in a regular beam and girder system. However, a column is not needed at each rigid connection between beams. The force is distributed throughout the interconnected beams through multiple force paths. Since the force is distributed in four directions at each point, a single member does not have to account for all of the force.

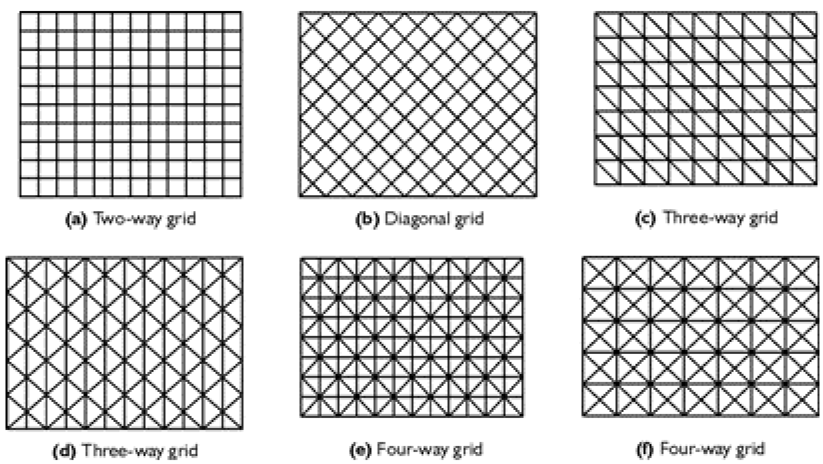


Figure 13: Example Grid Designs (Nichols, 2012)

2.6 Rigid and Braced Frames

In order to resist lateral forces on the frame of the building, two different framing systems maybe be used. One method includes the use of rigid frames in the building and the other method includes the use of braced frames.

Rigid frames do not use pinned joints in the frame. They resist rotation of the frame and are supported by fixed supports or pins as displayed in Figure 14. Connections of members in a rigid frame consist of welding or bolted plates because the flanges of a member need to be fully attached to the flange of the other member. The members in the rigid frame resist forces such as shear, bending, and axial forces. In a rigid frame the whole joint is inclined to rotate from the lateral forces acting on it. The amount of rotation that the joint will experience depends on how stiff the members of the frame are. The stiffness of the member can be measured by EI/L. In the case of rigid frames, the effective length (L) of the columns is reduced because of the end restraints. Therefore, columns can be slender and the deflection and moment of each beam are reduced. A drawback in the use of rigid frames is that settlement can induce strains and change the stress distribution in the frame, changing the behavior of the rigid frame (Nichols, 2012).

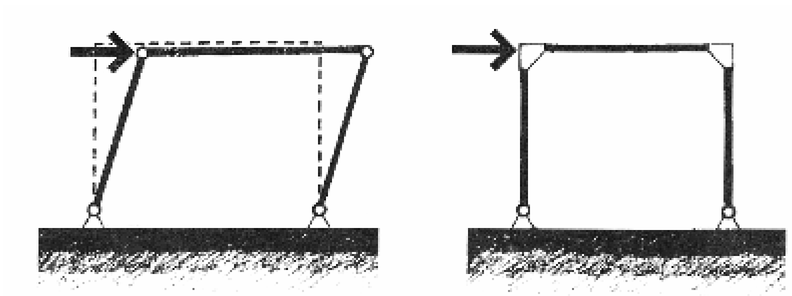


Figure 14: Rigid Frames (Nichols, 2012)

Braced frames use pins to connect the columns and members in the frame in order to resist the lateral forces. Some types of bracing are displayed in Figure 14: Rigid Frames (Nichols, 2012) and include knee-bracing, diagonal bracing, X bracing, K bracing, and shear walls. Shear walls resist the load by resisting lateral forces in the plane of the wall. Braced frames are analyzed using the method of joints in order to determine the forces acting through the members. In the case of the Avery Elementary School, braced frames are used in the design of the building in order to resist lateral forces (Nichols, 2012).

2.7 *Massachusetts State Building Code*

When designing any building, architects/engineers need to ensure that the building meets local building codes in order to comply with state regulations. Massachusetts has recently released their 8th edition of their Base Code in August 2011, which is comprised of the *International Building Code (IBC)* and several companion I-codes that are used in combination with Massachusetts amendments (International Code Council, 2011). This body of documents provided a foundation for the design of the project. "Chapter 16 – Structural Designs" is a critical chapter of the *IBC* that was referenced during the design segment of the project since this chapter encompasses all of the regulations for structural design elements including live loads, dead loads, lateral loads, snow loads, etc. Another section is “Chapter 34 – Existing Buildings and Structures,” which is also essential to repurposing a building. This chapter includes regulations on additions, alterations, change of occupancy, etc. Aside from a few exemptions listed in the *IBC*, alterations to a building’s structure must comply with the requirements for the new construction including, height and area provisions found in Chapter 5 and also follow any local zoning regulations. These factors will be critical in evaluating possible uses for the building when it is repurposed. According to the Massachusetts amendments in the *IBC*, it is mandatory to review all of the structural elements of a building before any alterations or changes in occupancy are pursued to make sure that they will meet the requirements of the new construction. Since these operations will take time and money, designing with repurposing in mind will substantially save time and cost (International Code Council, 2011).

2.8 Future Changes and Uses of Avery Elementary School

Several decades from now, the town may experience a significant change in age group populations, whether it is an increase or a decrease in the number of elementary students, or an increase in the number of elderly. With this in mind, the town will either need to 1) expand the school building due to population increase, 2) change the use of the building to an elderly home or 3) determine another non-residential use for the building.

For the first change, the expansion will need to be done vertically with added floors as opposed to horizontally because of the limited area of the site. Therefore, it can be proposed that the initial structural design be able to accommodate the addition of a fourth floor. For the added floor to be possible, a few changes to the current design should be made. First of all, the columns will need to be larger in order to support the additional loading due to the extra floor. Second, the current roof will have to be a flat roof instead of a pitched roof for ease of construction. Third, the roof framing will have to be designed to account for the future floor live loads as well, as opposed to just the current dead and snow loads. However, the costs of overdesigning the building for a possible addition of a floor might be too expensive as column sizes will have to be larger. Furthermore, the addition of a new floor to an operating school building may require areas of the school to be closed, causing smaller working areas for the students. Therefore, although adding another floor sounds like a good idea, it is not the ideal design due to costs.

For the second scenario of making the school building into an elderly home and third scenario of determining another non-residential use, the floor layouts will need to be altered and possibly the MEP systems will be modified as well. Therefore, the initial design of the building will need to possess minimal or no interior columns to allow greater flexibility in the location of partition walls. Currently, each room/area in the school is designed to support the maximum design live load for the given use of area. The occupant load criteria for the main areas of the building are shown in Table 2 below, which was obtained from the architectural package created by D&W (D&W, AC1.00). This table also includes floor live load criteria for other types of building uses in order to compare the different design loadings.

Table 2: Occupant Load Criteria for Avery Elementary School and for Other Buildings

Classification	Load (lbs per square foot)
Educational	
Classrooms	50
Reading Room	60
Corridors – First Floor	100
Corridors – Upper Floor	80
Open Plan Areas	100
Stairs/Lobby	100
Storage	125
Mechanical Penthouse	150
Partition Walls	20
Office buildings	
Offices	50
Residential	
Habitable attics & Sleeping Areas	30*
All other areas except stairs	40*

*Obtained from Table 4-1 ASCE 7-05

If the school is to be remodeled for a different use in the future, then the structural floor framing needs to be able to support the maximum design live load suggested by these uses to provide the most flexibility for future use. Hence, the upper floor needs to be designed to support a uniform live load of 100 psf, which would accommodate an open plan area, instead of designing different sections of a floor differently. With this approach, many of the areas will be overly designed for the initial use as a school causing an increase in construction cost relative to the current estimate. But, this overdesign may save the town money when the building is repurposed, decades from now.

2.9 Design Resources and References

In order to perform an analysis of the current structural design, create an alternate design and determine a cost estimate for this project, the group used several references and resources. A few of these references included the architectural and structural drawings of the new Avery School that was created by EDG and D&W, and provided to the group by Consigli. These drawings show the different uses of each area of the building as well as all the sizes and locations of the different structural members. The drawings also provide the specifications used for the design of the building. Consigli also provided the group with one of their payment requisitions, which showed Consigli's cost breakdown of the different materials.

With steel design, the main reference book that was used was the *AISC Steel Construction Manual 2005*. This manual contains all the information regarding beam sizes such as their gravity loads, moment capacities and moment of Inertia. All the data in this manual is also up to code. Other resources that were helpful for structural design and cost estimate included the following computer software: *Autodesk Revit Structure 2012*, *Microsoft Excel*, *AutoCAD 2012*, *MDSolids*, *RISA 2D educational* and *Autodesk Robot Structural Analysis Professional*.

Autodesk Revit is three-dimensional software that enables designers and other viewers to easily visualize the building. A three-dimensional model of a building can be made based on two-dimensional architectural and structural layouts. Through this software, updates with a drawing can be performed quickly and measurements between elements can be determined easily. Furthermore, this software is useful because building data, such as the quantities of different materials, can be extracted and input into a cost estimate. To determine the most suitable structural members, several steps of calculations need to be performed.

Microsoft Excel is a software program that enables several equations to be developed and executed in spreadsheet format, allowing an efficient manner of performing repetitive calculations for the design of a building. This software can also be used to calculate subtotals of different elements and their total costs.

AutoCAD allows two-dimensional drawings to be made, which is useful when creating alternative layouts for the building. *MDSolids* allows a quick analysis of a beam's reaction forces due to different loadings. Thus, it is advantageous to use it when analyzing a girder that has several point loads due to supporting different beams.

RISA is a computer program that allows for the structural analysis of a two-dimensional frame due to gravity loads and lateral loads. Through the use of all these tools, projects can be performed more efficiently. *Autodesk Robot Structural Analysis Professional* (*Robot*) is a software by Autodesk that performs advanced structural analyses in 3D. Using this program, drawings can be exported from *Autodesk Revit Structural* to *Robot* to perform a full structural analysis. Results for forces, moments, and deflections can be used in the design of beams, girders, and columns. The program gives the user a chance to see the structure in 3D as well as effect the loads have on the members. *Scia Engineer* is a structural analysis and design software that was created by Nemetschek Scia. Nemetschek Scia was founded in 1974 that is headquartered in Belgium. *Scia Engineer* is a very advanced analysis software that is compatible with *Revit*. Therefore, *Scia* can be used to analyze a *Revit* structure once it is exported to *Scia*. *Scia* is similar to *Autodesk Robot* in its features, but it allows for the user to customize the analysis more than *Robot* allows. *Scia* analyzes the structure using the provided member sizes and determines the deflections, moments, and other properties of the system. *Scia* provides diagrams to display deflections, as well as a table of results.

2.10 Summary

The Town of Dedham's current situation with the Avery Elementary School exemplifies the need for towns to reconsider the design and construction process of their public school buildings. Other case studies show that communities can reuse existing buildings instead of demolishing them. In order to improve the lifespan of these public buildings, the original design of the structures should consider features and design strategies to enhance the potential to repurpose the building in the future. Certain building materials offer advantages that can increase a building's lifespan and ameliorate the costs associated with modifying a building for reuse. Also, to ensure the longevity of the building, sustainable features should be considered in the design to reduce life-cycle costs for operations and maintenance. The team will use the researched information and design capacity requirements from the *Massachusetts State Building Code* to accomplish the project objective.

3.0 Analysis of the Design of a Public School Building

Before designing an original structural frame for the elementary school building, the team performed an analysis of the steel frame design that was originally designed for the building. The purpose of this analysis was to develop an understanding of the method used in the selection of member sizes. This analysis provided insight in regards to the adequacy of the members. It was used to determine the structural engineer's method for designing the framing system while maintaining a low cost. Typical bays from the second floor, third floor, and flat roof section were chosen to evaluate. A girder and a beam were analyzed from each of these bay systems. All beams and girders selected were simply supported with columns. LRFD load combinations were used to find the critical factored design loading acting on the steel elements. Typical columns from each floor were selected and evaluated as well as a typical connection used in the framing system.

3.1 Design Criteria and Assumptions

As input to the analysis of the existing design, Table C3-1 of ASCE *7-05* was used as a guide for design dead loads for insulation, ceiling, MEP systems, and metal decking. The ceiling was assumed to be a suspended steel channel system with a load of 2 psf, but a design load of 3 psf was used to account for error in assuming a value of 2 psf when the exact allowable dead load is unknown. The insulation was assumed to be fibrous glass because it is the highest design consideration weight for insulation at 1.1 psf and the design load was taken as 2 psf to provide a conservative assumption for the dead load of the insulation. The MEP system used the design load for *ASCE 7-05* allowance for mechanical duct with an added 1 psf for conservatism. The design load for the metal decking was taken as 3 psf to represent an 18 gage metal deck. The dead load for the concrete slabs was determined based on the given weight of 30.21 psf in the structural drawings (EDG, S0.01) A summary of the dead loads for the building are shown in Table 3.

Table 3: Design Dead Loads (EDG, S0.01)

Dead Loads	Amount (psf)
Insulation	2.00
Ceiling	3.00
MEP Systems	5.00
Concrete	30.21
Metal Decking	3.00
TOTAL DL	**43.21**

Table 3 presents a summary of the dead loads used in the design and Table 4 presents the live loads used in the design. The live load values were obtained from the structural drawings (EDG, S0.01).

Table 4: Live Loads (EDG, S0.01)

Designated Area	Live Load (psf)	Total (psf)
Classrooms	50+20	70
Reading Rooms	60+20	80
Corridors	100	100

Other specifications that were used for the structural analysis are shown in Table 5. All structural wide flange shapes consist of ASTM A992 with a strength of 50 ksi. The strength of the concrete is also provided with a minimum strength of 3,000 psi at 28 days (EDG, S0.01). The thickness of the concrete slab and steel decking are provided in the structural drawings as well (EDG, S0.01).

Table 5: Specifications of Concrete, Steel and Metal Decking (EDG, S0.01)

Specifications Used for Calculations		
Fy	50	ksi
f'c	3.00	ksi
Concrete Weight	145	pcf
thickness of concrete slab	2.5	in
thickness of steel deck	2	in

The snow load for the roof was determined to be 45 psf (EDG, S0.01). The load combinations that were used in the structural analysis were as followed:

$$\boldsymbol{For\ the\ roof}: \boldsymbol{U = 1.2D + 1.6S}$$

$$\boldsymbol{For\ the\ floors}: \boldsymbol{U = 1.2D + 1.6L}$$

The provided design criteria and assumptions were consistently used in the evaluation of the beams, girders, columns, base plates, footings, and connections.

3.2 Structural Frame Model

In order to better visualize the current structural framework, a three-dimensional model was created using *Revit Structure 2012* based on the structural drawings provided by EDG. Through the model, the team members were able to easily visualize the overall layout and quickly determine length of members, which were used when analyzing the beams and girders. The structural drawings show all of the steel in the structure and provide details on how beams and girders are connected, as well as the base plates for the columns. The model created in *Revit* is a simplified version, as it only shows the primary structural steel members and not the secondary steel elements such as connections and base plates. It also does not include some of the beams or girders in the roof. Furthermore, the roof sections on the second and third floor were drawn as flat roofs instead of sloped roofs to simplify the drawing. Figure 15 shows a two-dimensional layout of the first floor generated in *Revit*. Through this drawing, the locations of

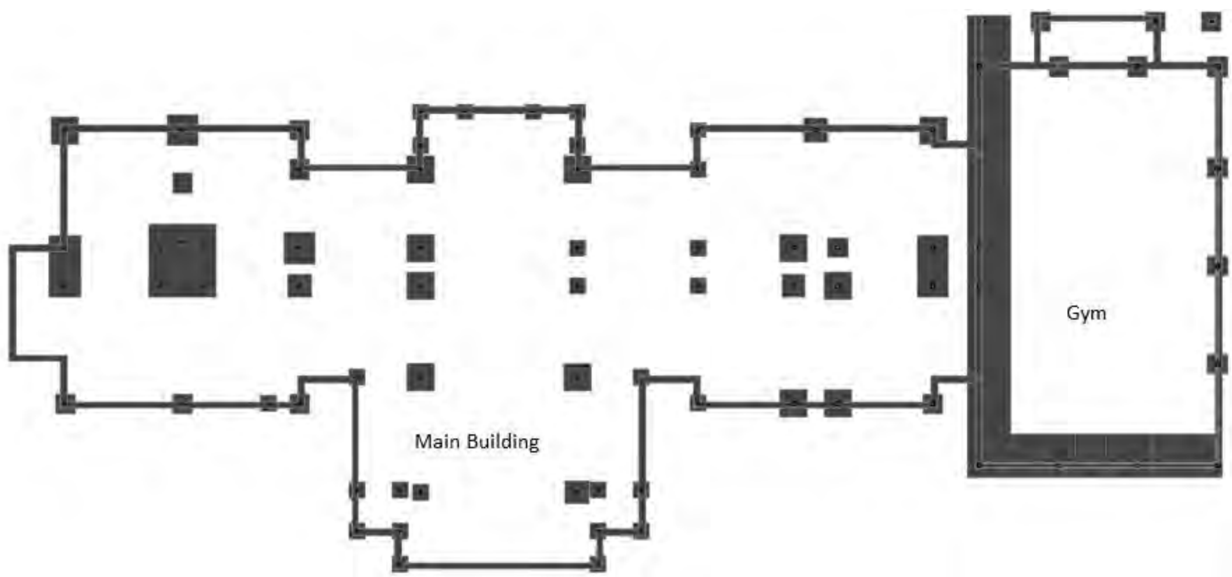


Figure 15: First Floor Layout

the columns (which are identifiable by the square footings) can be seen as well as the general footprint of the building.

Figure 16 shows the second floor layout of the main building. Highlighted in blue are the locations of the vertical bracing that provide lateral load resistance. There are four vertical frames in each of the north-south and west-east directions and, and they are located in each of the three floors. Although the team did not analyze the lateral bracing for the current design, an awareness of their locations was important to determine possible relocation of the bracing in the alternate design.

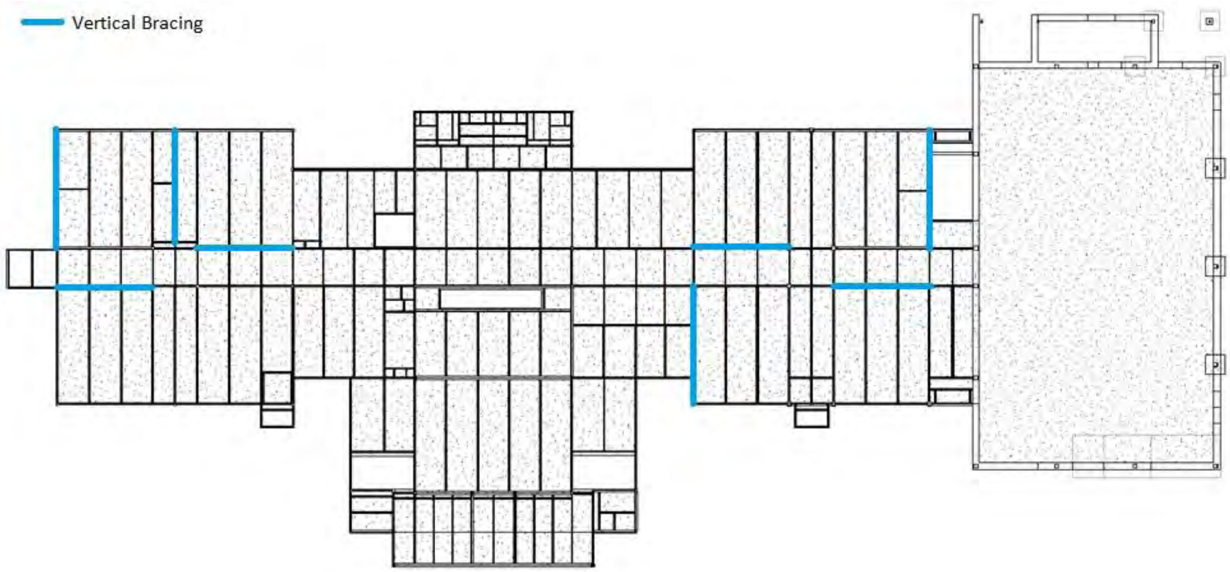


Figure 16: Second Floor Layout and Vertical Bracing Location

Figure 17 shows the third floor layout and highlighted in green is the area in each of the floors that will be redesigned. Since the team planned on removing or minimizing the number of interior columns, the highlighted area was determined to be the area that would be most affected. So, when performing the analysis of the beams, girders and columns, the team focused on analyzing representative members within this area.

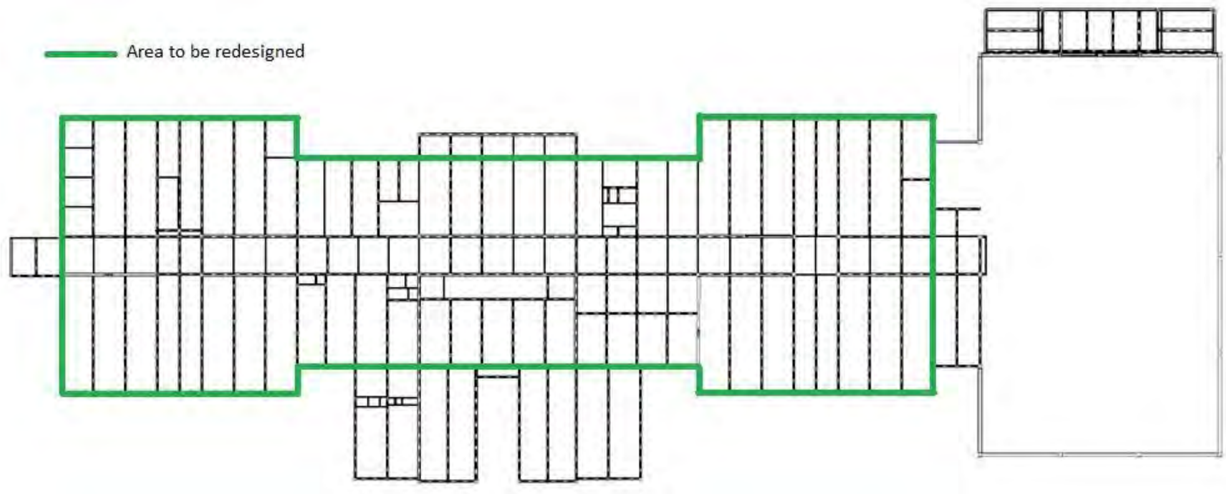


Figure 17: Third Floor Layout with Area to be Redesigned Highlighted

Figure 18 shows the Mechanical Penthouse Floor Layout, and the boundary of the Mechanical Penthouse is highlighted in orange. Adjacent to the penthouse are roof areas which are subject to snow drifting against the sidewalls of the penthouse. As a result, these areas will have to be designed for a higher snow load.

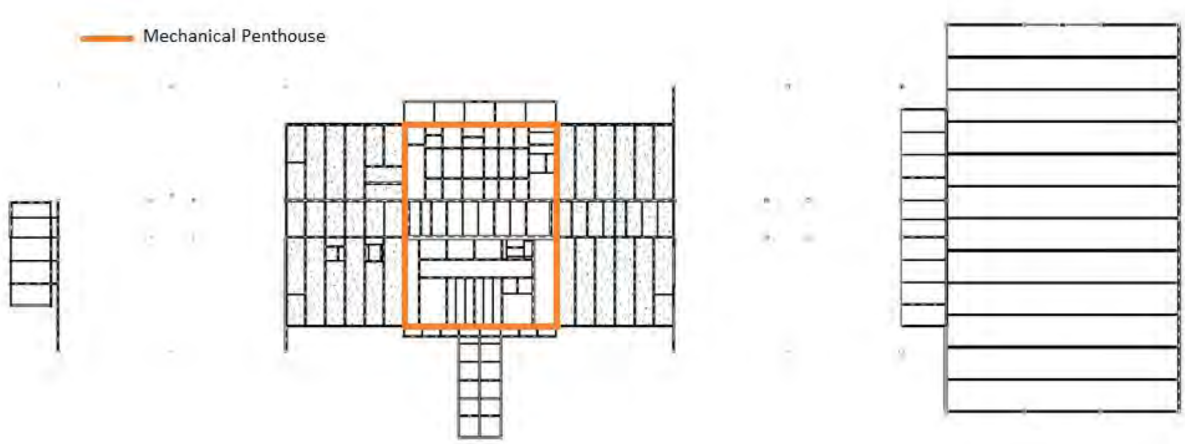


Figure 18: Mechanical Penthouse Floor Layout

Figure 19 shows the south elevation of the building, while Figure 20 and Figure 21 show three-dimensional views of this model.

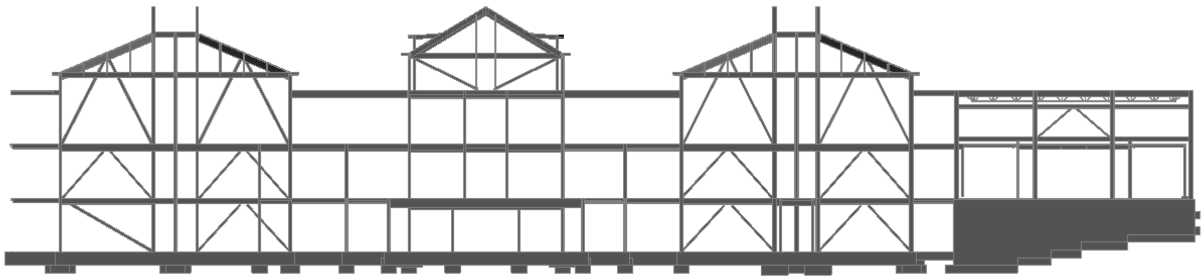


Figure 19: South Elevation View of Current Design Model

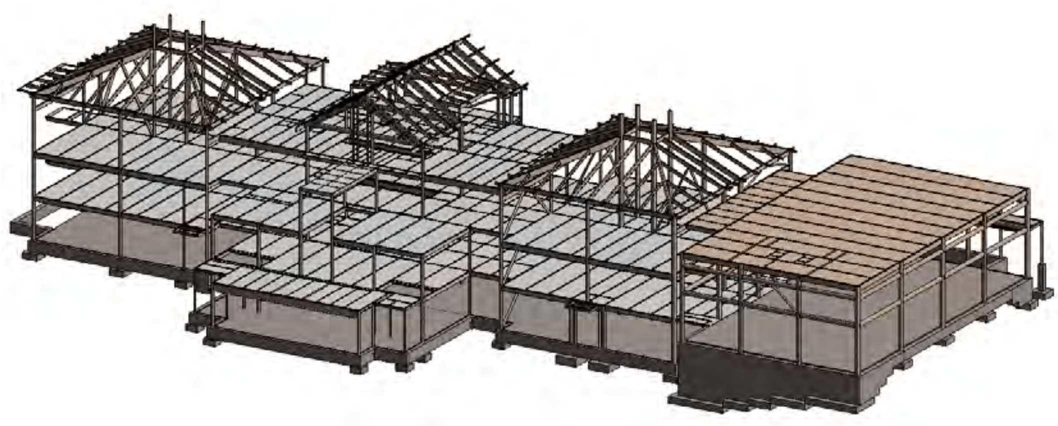


Figure 20: Three-dimensional View of Current Design Model 1

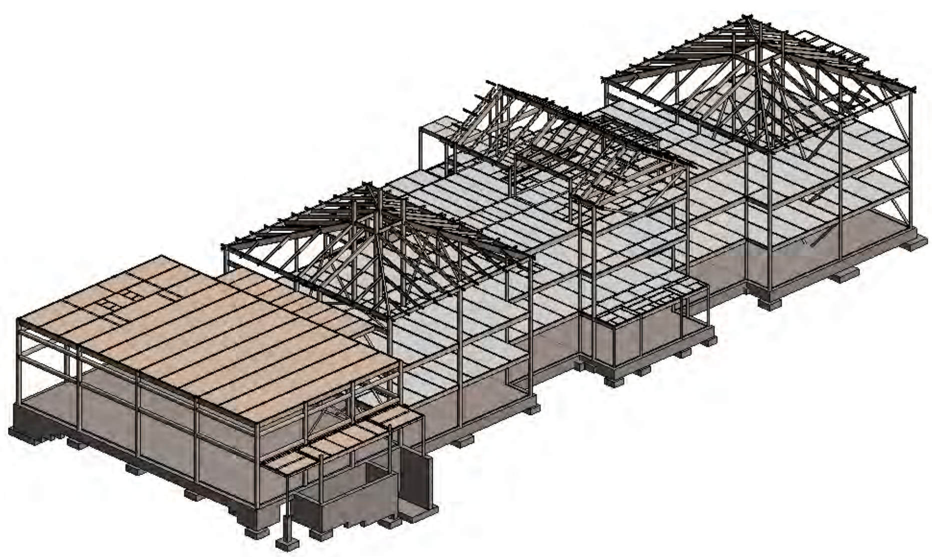


Figure 21: Three-dimensional View of Current Design Model 2

3.3 Beam Analysis

A typical beam was selected to analyze from the second floor, the third floor, and the flat roof section. The number of shear studs used for composite construction of each beam was provided in the structural drawings. *Excel* spreadsheets containing the strength and deflection calculations for each beam size can be found in Appendix B.

3.3.1 Methodology

Using the given loadings in Tables 3 and 4, the self-weight of the steel beam, and LRFD load factors, the governing design moment was determined. For serviceability performance during construction, the deflection due to the weight of the wet concrete and the steel beam was calculated using the moment of inertia value for the beam size in the equation and comparing it to the allowable deflection. The allowable deflection was taken as $\frac{1}{360}$ of the span length because this value is accepted as the most deflection that can occur without causing damage in the underlying plaster (McCormac, 2008). The service live load deflection was also calculated using the interpolated value for I_x from Table 3-21 in the *AISC Manual* and compared to a deflection value of $\frac{1}{360}$ of the span length. The applied shear was checked for adequacy by comparing it to the shear capacity value for the selected beam size obtained in Table 3-6 of the *AISC Manual*. The deflection due to loading was then calculated using the equation:

$$\Delta = \frac{5w_{u\_unfactored}L^4}{384EI} * 1728$$

After the deflection of the beam was calculated, the moment capacity number of shear studs for the beam was obtained from the structural drawings (EDG, S0.01) and used to determine the composite moment capacity. The amount of studs and the value of Q_n according to Table 3-20 of the *AISC Steel Manual* were used to determine the ΣQ_n value in Table 3-20 of the *AISC Steel Manual*. This required value of ΣQ_n was compared to the available values in Table 3.19. The next lowest value was used as the ΣQ_n and the Y_1 value and PNA were determined based on the selected ΣQ_n in Table 3.19. The moment capacity was determined by interpolation using Table 3-19 and the calculated value for Y_2. The moment of inertia was also determined by interpolation using Table 3-20 and the calculated value for Y_2. After the studs were evaluated, the spacing was determined based on the amount of studs, the length of the beam, and the rib size of the steel decking. The adequacy of the spacing was then checked against *AISC Manual, 13th edition*, section I3.2d (6).

3.3.2 Results

Table 6 and Table 7 below display the results of the beam analyses. The details of the calculations can be found in the spreadsheets presented in Appendix D. Table 5 displays the specifications of the beam according to the drawings with a calculated value for the stud spacing, assuming a uniform spacing. It also displays the calculated service live load deflections and comparison against their allowable deflections. The spacing of the studs was also evaluated with the *AISC Manual* section I3.2d (6) to assess for acceptable stud spacing.

Table 6: Selected Beams with Amount of Shear Studs and Deflections

Floor	Beam Size	Amount of Shear Studs	Stud Spacing (in)	Beam Length (ft)	Loading Deflection (in)	Allowable Deflection (in)	Deflection Due to Live Load (in)
Second	W16 x 31	12	16	28.67	0.36	0.96	0.22
Third	W12 x 16	10	18	20.00	0.29	0.67	0.17
Roof	W16 x 31	16	18	20.00	0.09	0.67	0.42

The following table displays the calculated design moment, interpolated moment capacity, calculated design shear, construction deflection, and the allowed shear capacity according to the *AISC Steel Manual.*

Table 7: Applied Moments, Shear and Deflection Due to Wet Concrete of the Beam

Floor	Size	Applied Moment (k*ft)	Moment Capacity (k*ft)	Deflection due to Wet Concrete	Allowable Deflection Due to Wet Concrete	Applied Shear (k)	Allowable Shear Capacity (k)
Second	W16 x 31	86.96	760	0.23	1	11.60	55.9
Third	W12 x 16	37.83	128	0.18	1	7.37	30.2
Roof	W16 x 31	29.73	276	0.06	1	5.57	81

3.3.3 Conclusion

After analyzing three typical beams that were used in the design of the structure, the team was able to draw some conclusions on the method for selecting beams. Each beam that was chosen was found to be quite adequate when compared to the allowable deflection of 1/360 of the span length. The team determined that by calculating for a small deflection, the structural design was able to use partial composite beams and significantly decrease the amount of shear studs needed. This saved the accumulated cost that would have been incurred for large amounts of shear studs. Moreover, it appears that the number of shear studs specified by the structural designer was chosen because of spacing requirements according to the *AISC Manual, 13th edition,* section I3.2d (6). There was a consistent method used in the selection of each beam size that was evaluated. The structural designer picked very adequate beams for the applied load and moment so that less shear studs could be used and the cost of shear studs would be decreased.

3.4 Girder Analysis

A typical girder was selected to analyze from the second floor, the third floor, and the flat roof section. The number of shear studs used for composite construction of each girder was provided in the structural drawings. *Excel* spreadsheets containing the strength and deflection calculations for each girder size can be found in Appendix C.

3.4.1 Methodology

The girders were evaluated in a manner similar to the beam evaluation. Using the given loadings in Tables 3 and 4, the self-weight of the steel girder, and LRFD load factors, the governing design moment was determined. For serviceability performance during construction, the deflection due to the weight of the wet concrete and the steel girder was calculated using the moment of inertia value for the girder size in the equation and comparing it to the allowable deflection. The allowable deflection was taken as 1 inch for the girders since $\frac{1}{360}$ of the span length is larger than 1 inch for every girder that was analyzed. The service live load deflection was also calculated using the interpolated value for I_x from Table 3-21 in the *ASIC Steel Manual* and compared to a deflection value of 1 inch. The applied shear was checked for adequacy by comparing it to the shear capacity value for the selected beam size obtained in Table 3-6 of the *AISC Steel Manual*. The deflection due to loading was then calculated using the equation

$$\Delta = \frac{5w_{u\_unfactored}L^4}{384EI} * 1728$$

After the deflection of the girder was calculated, the moment capacity number of shear studs for the girder was obtained from the structural drawings (EDG, S0.01) and used to determine the composite moment capacity. Some differences in process include the determination of the amount of shear studs. The amount of shear studs as provided in the drawings for each girder was used to determine the ΣQ_n value. The value for Q_n for the girders differs from the Q_n for the beams. The Q_n value was determined from Table 3-21 of the *AISC Steel Manual*. Since the girders are parallel to the deck, and the $w_r/h_r > 1.5$, a value of 21.0 was selected to use for Q_n. After the Q_n value was selected, the process of evaluation was similar to the process described for the beam analysis. The capacity and deflection were checked for adequacy and the spacing of the shear studs was determined by considering the length of the girder, the ribbing of the steel deck, and the amount of shear studs. The spacing was also checked against the spacing regulations according to *AISC Manual* section I3.2d (6) and modified accordingly.

3.4.2 Results

The evaluation of the selected girders provided the results displayed in the tables below. Table 8 displays the sizes of the selected girder with the provided amount of shear studs and length along with deflections. The spacing of the shear studs was also calculated and adjusted according to *AISC Manual* section I3.2d (6).

Table 8: Selected Girder Shear Studs with Calculated Spacing and Deflection

Floor	Size	Amount of Shear Studs	Stud Spacing (in)	Girder Length (ft)	Loading Deflection (in)	Allowable Deflection (in)	Deflection Due to Live Load (in)
Second	W30 x 90	28	16	40.00	0.92	1	0.49
Third	W24 x 68	28	18	40.00	0.96	1	0.35
Roof	W24 x 55	20	18	31.00	0.48	1	0.71

Table 9 displays the calculated applied moment to the selected girders as well as the calculated shear.

Table 9: Applied Moment and Shear and Wed Concrete Weight Deflection

Floor	Size	Applied Moment (k*ft)	Moment Capacity (k*ft)	Deflection due to Wet Concrete	Allowable Deflection Due to Wet Concrete	Applied Shear (k)	Allowable Shear Capacity (k)
Second	W30 x 90	973.73	1439	0.21	1	95.21	212
Third	W24 x 68	654.37	1228	0.56	1	63.81	133
Roof	W24 x 55	228.64	686	0.27	1	28.48	126.00

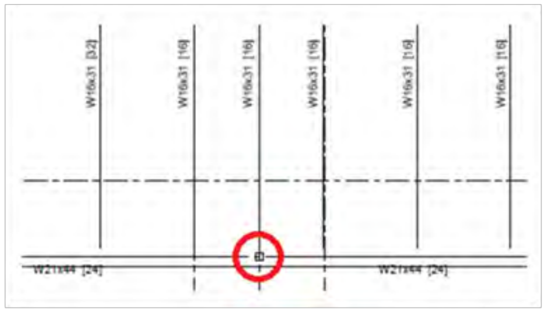
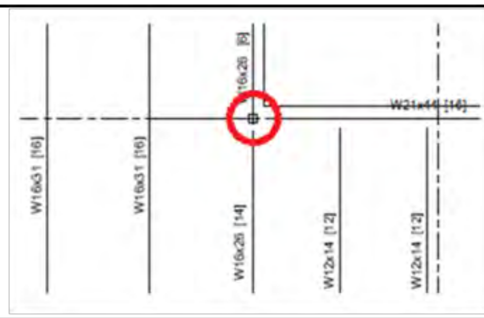
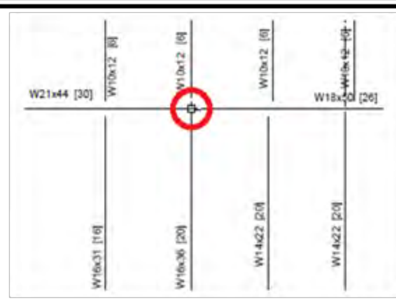
3.4.3 Conclusion

After analyzing the girders, it was found that the girder sizes were picked in order to maintain an allowable deflection. The applied moment was well under the moment capacity of the selected girders, but the deflections of the girders due to the applied loading were just less than 1 inch.

3.5 Column Analysis

The team chose four columns to analyze based on location. Column S-3 is an exterior column, G-5 is a corner exterior column, L-5 and L-16 support the corridor, and J-19 is an interior column in Area B. All of these columns are shown in Table 10.

Table 10 Partial Drawings of Column Locations

<table>
<tr><th>Column</th><th>Partial Drawing</th></tr>
<tr><td>S-3</td><td></td></tr>
<tr><td>G-5</td><td></td></tr>
<tr><td>L-5</td><td></td></tr>
<tr><td>L-16</td><td></td></tr>
<tr><td>J-19</td><td></td></tr>
</table>

3.5.1 Methodology

The tributary area for each column was determined by mapping the distance halfway between the column in question and the nearest columns. The beam and girder weights in this area were added to the dead load acting on the column, along with the weights for the metal decking, concrete slab, insulation, ceiling, and MEP systems. The live load varied depending on where a given column was located. For example, the live load for the corridors is greater than the live load for the classrooms as indicated in Table 4.

The effective length of the columns was the story height, or 14 feet for the first and second floors and 18 feet for the third floor, since they were assumed to be braced by the beam and girder framing at each story level. The first check is for the available critical stress which is due to the effective length and column section properties. This stress is then compared to column strength due to the effective length, grade of steel, applied loads, and interpolation between given critical stresses and effective lengths. The design loads are then compared to the column strength to ensure the column is capable of supporting such loads. The last check is for the available strength which can be determined from the critical stress in the column which is compared to the factored loading acting on the column. After this process is completed for the third floor, the load acting on the third floor column must be added to the loads acting on the second floor column, and so on.

3.5.2 Results

Table 11 displays the adequacy checks for the five columns analyzed. It shows the factored load increasing from the third floor to the first floor. The columns were named by the column lines in the structural drawings, consistent with the names in.

Table 11: Column Analysis Results

Column	Floor	Size	P_u (k)	$\Phi_c F_{cr}$ (ksi)	$\Phi_c P_n$ (k)	Adequacy Check
S3	3rd	HSS 8x8x5/16	60	34.10	269	Yes
S3	2nd	HSS 8x8x5/16	138	34.10	269	Yes
S3	1st	HSS 8x8x5/16	212	34.10	269	Yes
G5	3rd	HSS 8x8x5/16	47	34.10	269	Yes
G5	2nd	HSS 8x8x5/16	109	34.10	269	Yes
G5	1st	HSS 8x8x5/16	169	34.10	269	Yes
L5	3rd	HSS 8x8x5/16	53	34.10	269	Yes
L5	2nd	HSS 8x8x5/16	110	34.10	269	Yes
L5	1st	HSS 8x8x5/16	182	34.10	269	Yes
J16	3rd	HSS 8x8x5/16	44	34.10	269	Yes
J16	2nd	HSS 8x8x5/16	118	34.10	269	Yes
J16	1st	HSS 8x8x5/16	192	34.10	269	Yes
L19	3rd	HSS 8x8x5/16	45	34.10	269	Yes
L19	2nd	HSS 8x8x5/16	109	34.10	269	Yes
L19	1st	HSS 8x8x5/16	174	34.10	269	Yes

3.5.3 Conclusion

EDG chose to use HSS 8x8x5/16 columns for the entire structure. This is most likely for constructability purposes. The team analyzed five columns on all three floors. The HSS 8x8x5/16 proved to be adequate for all columns, which leads to the conclusion that the designer chose the largest load at the first floor level, established an acceptable column size, and then that one size was used for the entire building.

The resources used for this analysis included the structural drawings and Tables 1-12 and 4-22 from the *AISC Steel Manual 2005*. Values from each of these tables were used in equations for the effective length and determining the available critical stress.

3.6 Connection Analysis

The structural design by EDG provides typical connection details; however, it is noted in the drawings that the "General Contractor will coordinate all connections with precast sub-contractor and the structural steel sub-contractors." Therefore, it was the responsibility of the structural steel sub-contractor to design the connections. Using the provided information, the team designed a typical connection. The structural drawings note that "composite steel beams and girders shall be designed using the reaction "R" modified by the magnification factor as Follows:

For Steel Section Depth = "D"
RC = Required Connection Capacity for Composite Beams
For "D" Greater than or equal to 24in., RC = 1.5R
For "D" Greater Than or Equal to 21in. but less than 24in, RC = 1.75R
For "D" Greater Than or Equal to 14in., but less than 21in., RC = 2.0R
For "D" Less Than 14in. RC=2.25R"

3.6.1 Methodology

In the General Notes section, it is stated that all connections shall be double angle connections with ¾" diameter A325-N Bolts. With this information, the team designed a typical girder-to-column connection. A W24 x 68 Girder to H8x8x1/2 Column connection (Figure 22) was investigated since this was a connection that took on a considerable amount of load. The following steps were used to design this typical connection.

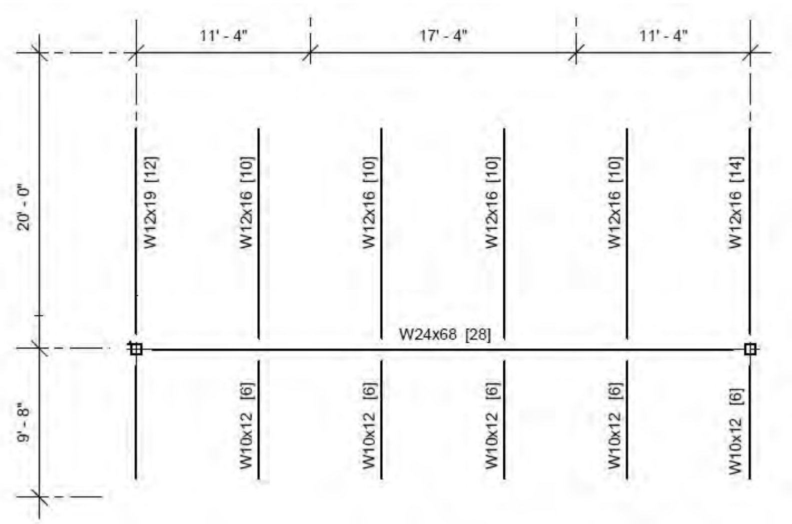


Figure 22: Layout of the W24x68 Girder

The beam web's shear capacity was calculated to ensure it would be greater than the design shear. The design shear (V_u) was first calculated using equation $V_u = W_u/2$, where Wu is the total factored design load acting on the girder. Then V_u was multiplied by R=1.75 since the depth of the girder was less than 24", but greater than 21".The R value is a factor of safety incorporated into the design of connections. V_u was compared to the beam shear capacity, $\varphi V_n = \varphi * 0.6 * F_y\ d * t_w$. Next the shear capacity of the connection bolt was used. The Equation $\varphi R_n = 2\varphi F_v * A_b$ determined the shear capacity of the bolt for the condition of double shear. Using this equation to solve for the number of bolts "n", $n = V_u / \varphi R_n$.

The layout of the connection was then defined. From Table J3.4 of the *AISC Steel Manual*, for a ¾" bolt there needs to be a minimum of 1 ¼ "distance from the bolt to the sheared edged and 1" minimum distance from the bolt to the rolled edge. Also, according to the *AISC Steel Manual*, the length of the angle has to be equal to at least half the depth of the beam web (T), but less than the depth "T".

To calculate the required angle thickness, three limit states, tearing, bearing, and shear rupture were analyzed to determine which limit state would govern the thickness of the angle. After determining the minimum thickness of the angle, Table 1-7 (pg. 44) from the *AISC Steel Manual* denotes the appropriate size for the angle.

3.6.2 Results

The team determined that a <2L 3 ½" x 3 ½" x ¼" was adequate, however, EDG's design notes that the connection will use a minimum ½" thick plate. The structural engineers may have required this detail as a factor of safety or as a common practice in their design process. Therefore, an adequate angle connection would be <2L 3 ½" x 3 ½" x ½", with 5 bolts spaced at 3" (Figure 23). This connection would be an adequate connection throughout the building.

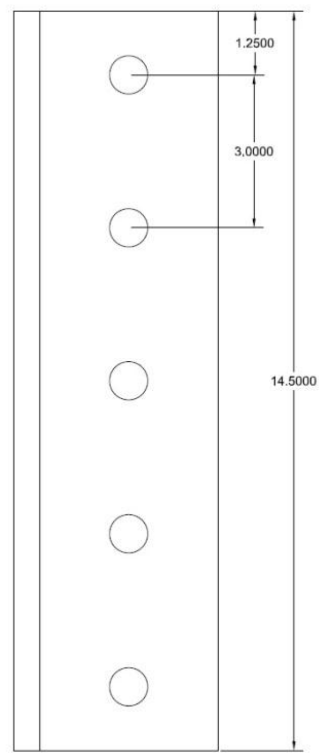


Figure 23: Designed Typical Connection

3.7 Base Plate Analysis

The base plate is the connection between the column and footing. The structural drawings made some specifications for the base plates; however, most of the designs were dependent on the specific column and footing sizes. Base plates for columns S-3, G-5, L-5, J-16, and L-19 were analyzed.

3.7.1 Methodology

The factored loading is taken from the column analysis for the first floor column. The required base plate area must be calculated and checked against the specified size. Usually the base plate will end up being a square for easy of constructability, which is what happened in this case. The bearing strength of the concrete must be checked against the factored loading. The required base plate thickness was then calculated based on plate bending and rounded up to the nearest ¼". The base plate area and thickness are defined in the form of PL t x B x N, and compared to the base plate used by EDG.

3.7.2 Results

Table 12 shows the actual base plate size compared to the required base plate size. It also shows the corresponding factored loads, bearing capacity, and the calculated thickness.

Table 12: Base Plate Size Requirements for Various Columns

Column	Actual Base Plate Size	Minimum Required Base Plate Size	P_u	$\Phi_c P_p$	t_{req}
S-3	PL 1" x 14" x 14"	PL ¾" 9" x 9"	204	247.86	.703"
G-5	PL 1" x 14" x 14"	PL ¾" 8" x 8"	163	195.84	.707"
L-5	PL 1" x 14" x 14"	PL ¾" 8" x 8"	175	195.84	.732"
J-16	PL 1" x 14" x 14"	PL ¾" 8.5" x 8.5"	185	221.085	.709"
L-19	PL 1" x 14" x 14"	PL ¾" 8" x 8"	168	195.84	.718"

3.7.3 Conclusion

The base plate sizes are specified with the column schedule on the EDG's structural drawings (EDG, S1.21). The size depends on the size of the column, the column's design load, and the footing properties.

The footing and base plate for the five columns previously stated were analyzed for the existing building. The footing area and column area were determined in order to find the minimum required base plate areas. These calculated areas and plate thicknesses are less than the values that EDG used in their final design; therefore, the given base plate areas are adequate. The bearing strength of the concrete proved to be greater than the factored loading acting on the column. The minimum required base plate areas determined would be too small for this structure. The HSS 5/16 x 8 x 8 columns need to rest on the base plate, along with the anchor bolts. Lastly, the thickness was determined. The final base plate design was a PL ¾" x 9" x 9". All the dimensions were sufficient without being over-adequate which proves EDG was economical and chose the best option for the base plates.

3.8 Footing Analysis

The footings provide the support for the entire structure, which makes them a crucial part of the analysis. The compressive strength of the concrete footings is 3,000 psi, and the areas are specified on the footing schedule on the structural drawings. The drawings also indicate a soil bearing pressure of 6000 psf. The area of the footing must be large enough to keep the ratio of loading to area below a soil pressure of 6000 psf. The thickness of the footings is decided upon by use of *ACI 318-05 Building Code Requirements for Structural Concrete and Commentary*.

3.8.1 Methodology

The required footing areas were established using an allowable stress approach to the soil and the superimposed loads from the columns. The size of the footing can be found on the structural drawings, which can be used to find the footing area. Using the equation $f_u = {}^{P}/_{A}$, the bearing pressure can be solved. The bearing pressure was checked to be less than the soil bearing capacity of 6000 psf.

3.8.2 Results

Table 13 contains the bearing pressure for each footing analyzed in kips per square foot. The design load and area of footing are also included in the table because they were used to calculate the bearing pressure.

Table 13: Determined Design Loading for Column Footings

Column	Calculated Design Load	Actual Footing Area	Calculated Bearing Pressure
S-3	148.87 kips	25 ft^2	5.95 ksf
G-5	118.48 kips	25 ft^2	4.74 ksf
L-5	138.43 kips	36 ft^2	3.85 ksf
J-16	139.84 kips	36 ft^2	3.88 ksf
L-19	121.04 kips	64 ft^2	1.89 ksf

3.8.3 Conclusion

The bearing capacity as specified on the structural drawings is 3Tsf, or 6 ksf. The results show the highest calculated bearing pressure is 5.95 ksf, which is less than the allowed 6 ksf. All the footings the team analyzed proved to be adequate. The footing sizes vary depending on the load they are supporting. For example, footing “FA”, as noted in the structural drawings, is supporting 5 columns that are close together. It was most likely not possible to fit 5 footings in such a small area, so EDG decided to design one, large 18’8” x 17’4” combined footing to accommodate all 5 columns. All footings in Area A are 2’ in depth, while a few of the footings in Area B were 3’ in depth. Section 1806.1 of the *Massachusetts State Building Code* (date) states: “All permanent supports of buildings and structures larger than 100 square feet (9.3 m^2) in area or ten feet (3 m) in height shall extend to a minimum of four feet (1.2 m) below finished grade except when erected upon sound bedrock or when protected from frost”. The exterior footings in the original plan are 3.5 feet below grade while the interior columns are 1.5 feet below grade. Without the Geotechnical Report, the team could not determine if the building was on bedrock, however it is unlikely since it is located on top of a hill. This building location most likely protects the footings from frost and freeze-thaw conditions.

4.0 Structural Alternate Design

In order to provide the town of Dedham with a public building that can be easily modified to fit their future needs, an alternate structural design for the building was prepared as an example. This alternative minimizes the restrictions on the architectural design by providing an open-layout concept structural design.

Structural columns were placed mostly in the exterior frame of the building in order to provide open space for a flexible architectural design. When interior columns were needed they were strategically placed next to permanent elements such as elevator shafts and stairwells where obstruction of the architectural layout is minimal. This approach enhances the opportunity for future use of the space because the architectural layout can be changed throughout the lifespan of the building without the restriction of many interior structural elements. The same approach was used when designing the lateral load resisting system. The necessary bracing was placed in strategic locations so that it does not obstruct a possible architectural layout. This chapter discusses a proposed alternate structural design to encourage the reuse of the building and the approach the structural engineer would take when dealing with each structural element. The layout of the building was divided into three sections, displayed in Figure 24, to organize the analysis of the structural systems. The following chapters will reference this layout to describe the structural design and analysis.

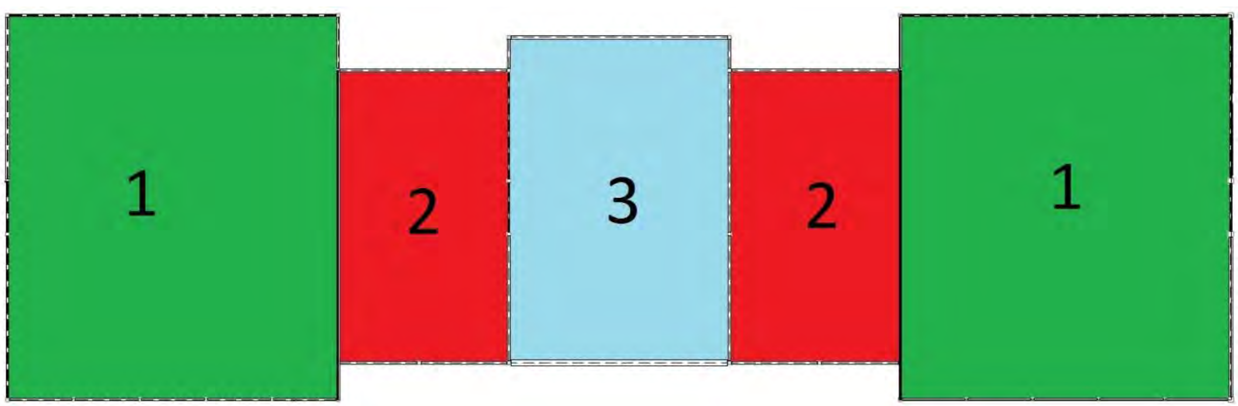


Figure 24: Avery School Architectural Layout Divided into Separate Section

4.1 Design Criteria and Assumptions

The dead loads presented in Table 14 are the same dead loads used in the current design of the Avery School. The alternative designs account for the same dead loads because the concrete slab, metal decking, MEP (mechanical, electrical, and plumbing) systems, ceiling, and insulation systems were not expected to change from the original design.

Table 14: Dead Loads used for Avery Design

Dead Loads	Amount (psf)
Insulation	2
Ceiling	3
MEP Systems	5
Concrete	30.21
Metal Decking	3
TOTAL DL	**43.21**

The current Avery School design accounts for certain live loads that are necessary for the building to operate as a school. The flooring system must account for an increased live load in order for the building to be able to serve functions other than an elementary school in the future. The live loads accounted for are determined based on the singular purpose of each designation area of the building. Table 15 displays the designated area and the design live load for each area that was stated in the structural drawings (EDG, S0.01) when designing the framing. When the team designed the framing system for the redesign of the building, the various live loads were considered, but the 70 psf live load that is considered for the classrooms was increased to 80 psf in order to allow more flexibility in the reuse of this space.

Table 15: Live Loads of Each Designated Area

Floor Live Loads (psf)	
Classrooms	70
Reading Room	80
Corridors	80
Gymnasium	100
Stairs/Lobby	100
Storage	125
Mechanical Penthouse	150

4.2 Exploring Possibilities for Framing Systems

Various options were explored for the structural design of the elementary school building. Since the structural floor framing must account for a dead load of approximately 44 psf and a live load of 80 psf to 100 psf, the flooring system must be built up in comparison to the framing for the existing school. In order to provide adequate framing for the floor as well as maintain an open floor plan, innovative structural systems were explored. When evaluating the structural systems, the flooring area of dimensions 60' x 69' 8" (displayed in Figure 25 as green) was the test area for each structural system. This was used as the testing area because it is the

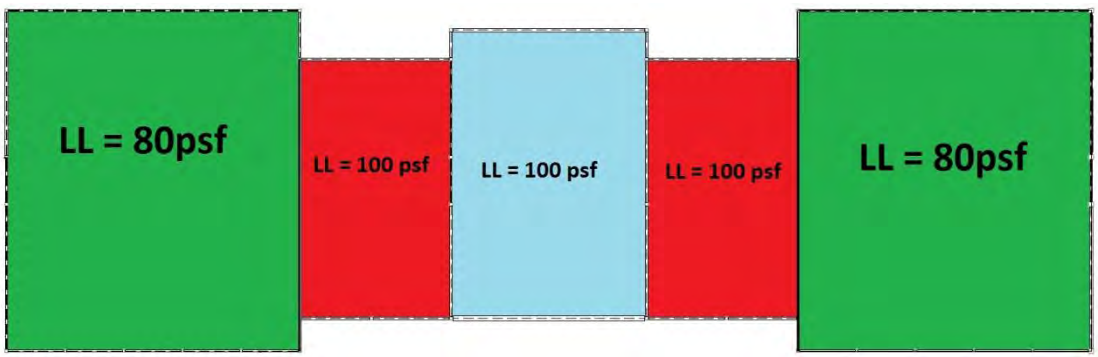


Figure 25: Design Live Loads

largest area of open space for the framing system to accommodate. Once 80 psf was found to be the maximum live load that could be designed for used in such a large area, 80 psf was determined to be the uniform design live load. However, when designing the middle section of the building (denoted in red and blue) it was found that since the span was not as large, this area could be designed for a live load of 100 psf. Therefore, two live loads were accounted for in the design based on the span restraints.

The first approach to designing a structural system that supports the distributed load without interior columns included the process of simply increasing the sizes of all of the beams and girders. This was an unsuccessful approach due to the increased weight of the required member sizes. For instance, in order to support the weight of the beams and their tributary loading, the depth of the girders would have to increase beyond 3-1/2 ft. The depth of horizontal construction between each ceiling and the overlying floor was targeted to stay at 3-1/2 ft. in order to remain consistent with the original architectural design and building height. In order to

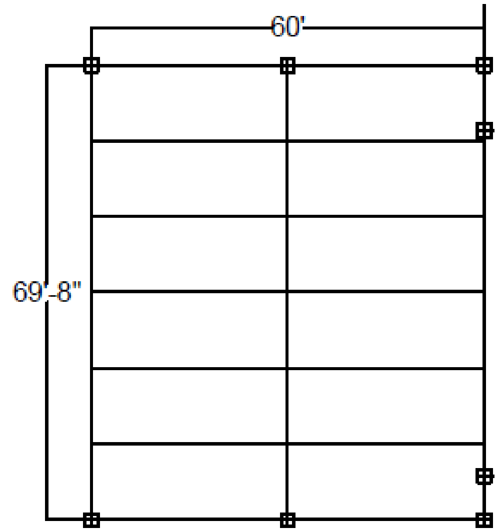


Figure 26: Layout of Beam and Girder System

account for the weight of the beams in the layout presented in Figure 26, the girder sizes had to be larger than a W40, exceeding the girder depth limitation. Therefore, any attempt to use rolled steel girders that span the full 69'-8", would result in exceeding the depth restraint.

4.2.1 Evaluation of a Beam Grid System

A beam grid system was evaluated as a second possibility for an open space framing system. The beam grid system was considered because it is able to carry loads through a grid system of welded beams without the need for interior columns. The applied loads would act perpendicular to the beam system, and the load path would distribute through the beams via shear and bending effects. This system would provide a rectangular floor area of open space that is sufficiently supported. Unfortunately, the shape of the Avery school does not allow for this system to be successfully used. The beam grid system was analyzed to be applicable for short spans. The design of a beam grid system in a 69'-8" by 60' area would require very large beam and girder sizes. An analysis found that the girder size needed for this design would be one of the largest sizes found in the AISC Steel Manual and this system was disregarded because of the unreasonably high expected cost of this system.

4.2.2 Evaluation of a Staggered Truss System

The staggered truss system was also explored as a possible framing system that would provide open space as well as support for large loads. An explanation of the staggered truss system can be found in the background section. The staggered truss system was able to provide open space for rectangular floor areas. The trusses that are built into the exterior walls would allow for the elimination of interior structural elements that would affect the floor layout. However, since the truss spans the floor height, it would not allow for the central corridor on the second floor of the elementary school. This would cause a very significant change in the layout of the classrooms and would disrupt the architectural design too much to be considered as a viable option. This system works best for taller, narrow buildings so that the trusses will not disrupt the usable space.

4.3 Successful Use of Truss Girders

An explored structural option for the 60' x 69'-8" area (section 1 in Figure 24) in the second and third floor was the use of truss girders that span the 69'-8". Various truss girders were evaluated using the *Vulcraft Joist Girder Catalog*. The applied loading in pounds per linear foot was determined by accounting for the dead load and live load and spacing of the truss girders. The dead loads and live loads were totaled and multiplied by the truss girder spacing or tributary width of 6 feet as exemplified in Table 16.

Table 16: Design Loads in Linear Foot on Truss Girder

Dead Loads				
Insulation	2	lb/ft^2	12	lb/ft
Ceiling	3	lb/ft^2	18	lb/ft
MEP Systems	5	lb/ft^2	30	lb/ft
Concrete	30.2	lb/ft^2	181.25	lb/ft
Metal Decking	3	lb/ft^2	18	lb/ft
Girder wt	2.8	lb/ft^2	42	lb/ft
	46.01	**TOTAL DL**	301.25	lb/ft
		Factored	361.5	lb/ft
Live Loads				
Floors	90	lb/ft^2	540	lb/ft
		TOTAL LL	540	lb/ft
		Factored	864	lb/ft

The live loads and dead loads were then factored according to the following load combination equation (Vulcraft, 2010).

$$U = 1.2D + 1.6L$$

The total factored loading was found to be 1225.5 lb/ft. After this value was determined, the Vulcraft Standard LRFD Load Table for longspan steel joists, LH-series, was consulted. The layout of this design can be seen in Figure 27. Therefore, this truss girder was found to be adequate to support the dead loads such as the concrete slab and decking as well as the live load of 80 psf if spaced every 6 ft. The clear span of 69'-8" was considered, and a joist girder was selected. The joist designation of 44LH17 was selected. This truss girder can hold a total factored loading of 1235 lb/ft for a 70' span. The Vulcraft catalog recommends the 44LH17 when considering the general span that is needed. This size was sufficient for an applied factored loading of 1225.5 lb/ft. It has an approximate weight in pounds per linear foot of 47 lbs/ft and the depth of the truss is 44 inches. This depth is above the target depth of 42 inches; therefore, it must be noted that the ceiling height should be reduced 2 inches. The truss girders will not be composite and will be cambered 2 inches as recommended for this joist girder by the Vulcraft catalog. Properties of the 44LH17 can be found in Table 17. The deflection was calculated according to Vulcraft's method which used the following equation (Vulcraft, 2010):

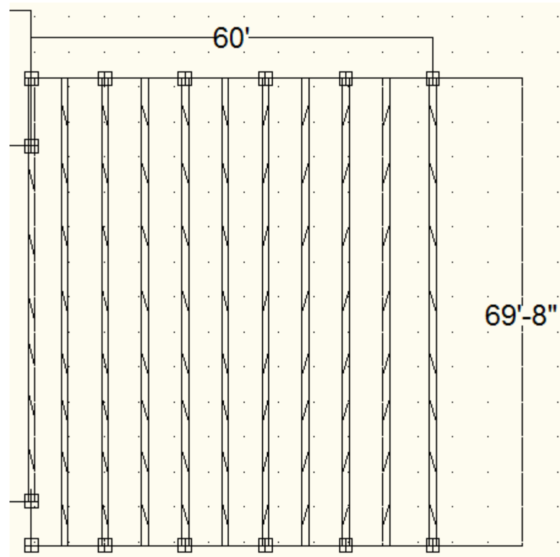


Figure 27: Girder Layout for Exterior Bays

$$\frac{25.88 * Dead\ Load * clear\ span^2}{E * 1000 * I}$$

The deflection of the 44LH17 used in the second and third floor was calculated to be 1.5 inches. This deflection will be offset with the 2" camber.

Table 17: Properties of the 44LH17

Joist Designation	Weight in lbs/linear foot	Depth in inches	Clear Span, ft	Safe Load plf	I, moment of inertia of joist	WLL*	Span	E, ksi
44LH17	47	44	70	1235	4251.258239	450	69.66667	29000

*produces a deflection of $\frac{1}{360}$ of the span

Truss girders were also used on the roof in the 60' x 69'-8" area of the roof. The load combination equation used for this area is:

$$1.2D + 1.6S$$

The same size steel joist truss was used as in the floor designs, even though the live load of 80 psf was decreased to a snow load of 45 psf. Because of the limits on span for the joist truss, the 44LH17 must be used. Using the factored load combination, an applied load of 793.5 lbs/ft was calculated. This is significantly under the safeload limit of 1235 lbs/ft for the 44LH17.

4.4 Beam and Girder Design

Structural floor designs for Sections 2 and 3 were designed using girders and beams instead of the truss girders applied in section 1 due to the architectural limitations further discussed in the chapter. As a result, a different design approach was used for these two sections.

4.4.1 Methodology

Column locations defined the layouts for the bays which were assigned with letters as illustrated in Figure 29. Establishing bay sizes for sections "2" and "3" determined beam and girder designs and constraints.

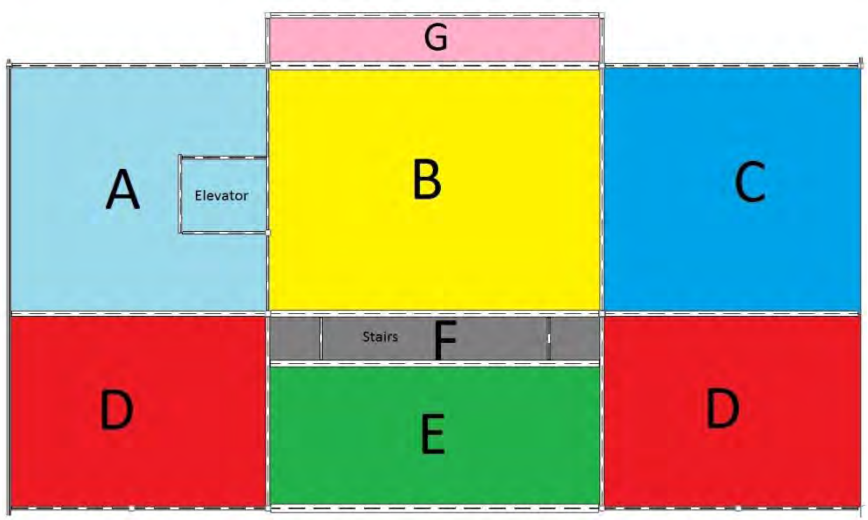


Figure 28: Bay Layouts for Sections 2 and 3

Investigation and selection of beam sizes were conducted using *Microsoft Excel* spreadsheets that organized and generated repeated calculations. After entering beam spacing and design loads, the maximum factored moment (M_u) was calculated providing a base to choose a beam size with an adequate moment capacity. Once the properties of the beam, gathered from Table 1-1 of the *AISC Steel Construction Manual*, were input into the spreadsheet, several capacity checks were calculated, including deflection due to wet concrete and beam weight, deflection during construction, and shear capacity. If the beam were to fail any of these limits, the beam size was increased. Using the advantages of composite construction, the composite capacity and elastic moment of inertia of the beam were interpolated using Table 3-19 and Table 3-20 from *AISC Steel Construction Manual*. Targeting the lowest possible location of the plastic neutral axis (PNA) decreased the number of studs required for the composite construction, subsequently lowering the cost of the design. Since most of the beams were governed by deflection during construction, the elastic moment of inertia determined the size of the beam and the location of the PNA. Several variations of beam sizes and spacing were explored to establish the most economical and practical beam and girder design. Refer the reader to the appropriate appendix for the work. For further information, refer to Appendix I: Beam Design Sample Spreadsheet.

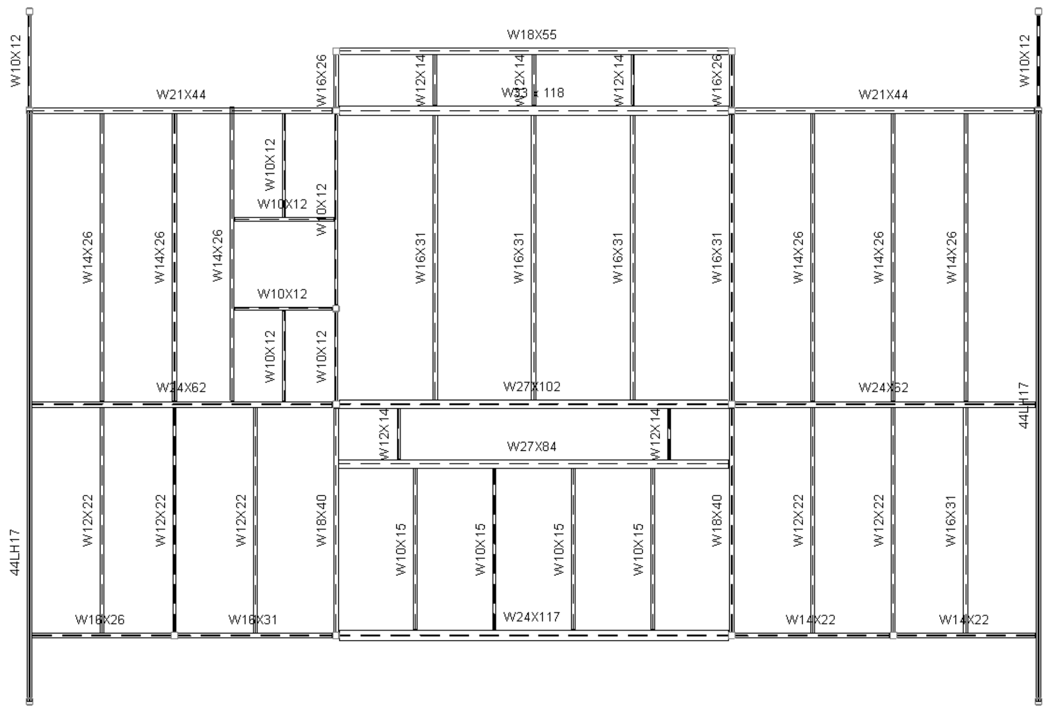
4.4.2 Results

Beams and girders were imported into *Revit Structure* after their sizes were determined. The following figures, Figure 29, Figure 30, and Figure 31 represent the structural floor design layouts for sections 2 and 3 (second and third floor), floor design for the Mechanical Penthouse and roof design for section 2, and the roof design for the Mechanical Penthouse respectively.

Discussion of Second and Third Floor Design

Keeping the location of the mechanical penthouse on top of the third floor aptly influenced the design of the structural layout referred to as section "3". Since the mechanical penthouse carries a significant amount of live load (150 psf), it was deemed impractical to eliminate all of the columns that supported this room. Therefore, columns P8 and P13 remained to support the mechanical penthouse. Column J8 was not removed because it is located adjacent to the elevator shaft, which would remain if the building layout was modified in the future. Similarly, columns L-8 and L-13 remained because they are located by the stairs and it was assumed that the location of the stairs would not change during repurposing. Therefore, interior columns J-5, L-5, J-16, L-16, and J-13 were eliminated for the alternate design. Due to the architectural layout that limited the structural design, the team decided it was most practical to design sections 2 and 3 with beams and girders instead of the long span truss girders.

Figure 29: Structural Floor Design for Sections 2 and 3



Discussion of Mechanical Penthouse

Lacking adequate information about the mechanical systems located in Mechanical Penthouse prevented sufficient structural analysis for the floor framing system. Therefore, with the exception of eliminating column J-13 as a support, EDG's original steel design for the Mechanical Penthouse was incorporated into the alternate design. Subsequently, excluding column J-13 required the girder, outlined in Figure 30, to carry a significantly increased amount of load, including the two girders connecting to column J-13. The structural program *MDSolids* was used to model and analyze the effects of the various applied loads in order to design a girder with adequate capacity. It was determined a W21 x55 would be adequate to carry the extra load.

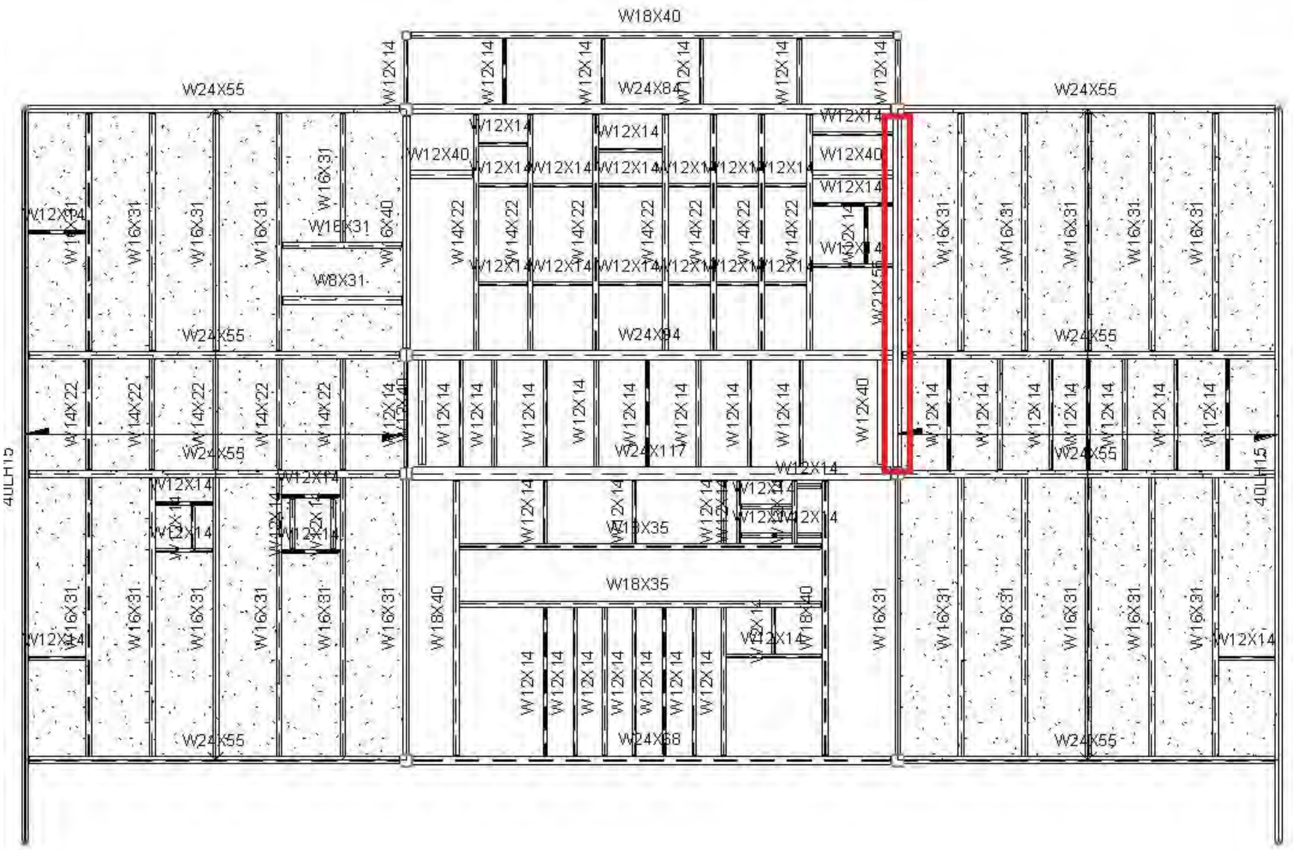


Figure 30: Structural Floor Design For Mechanical Penthouse and Roof Design for Sections 2 and 3

Discussion of Roof Designs

As a result of eliminating interior columns in section 1, the roof design was required to be flat. The roof design for sections "1" incorporate the same truss girder layout presented on the second and third floor, but decreased their size to support a reduced live load of 45 psf. The roof design above section "2", which supports mechanical components, also features the original structural design by EDG. The roof of the Mechanical Penthouse was redesigned in order to adequately support a flat roof, depicted in Figure 31.

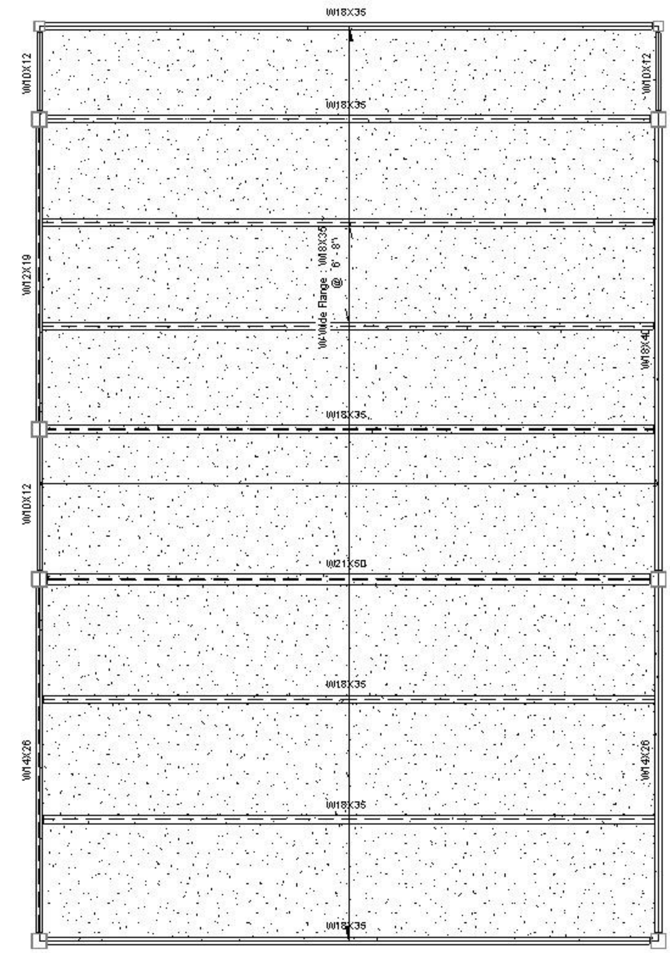


Figure 31: Mechanical Penthouse Roof Design

4.5 Gravity Column Design

This section discusses the design of the gravity columns. To keep consistent with EDG practice, the column sizes were standardized to promote constructability needs rather than adopting a least weight solution. This standardization tended to produce oversized columns in several instances. One column size was defined for Areas A and C (areas not supporting the Mechanical Penthouse) due to the symmetry in the bay systems, and another column size was defined for Area B (the area supporting the Mechanical Penthouse.

4.5.1 Methodology

The columns were split into two groups depending on the loads acting on them. The first group consisted of the columns in Area B that support the Mechanical Penthouse. These columns supported a live load of 125 psf on the Mechanical Penthouse floor, and an assumed

100 psf live load for the floors underneath. Also, these are the only columns that would be supporting a fourth floor. The second group consisted of the columns in the sections to either side of the Mechanical Penthouse. The difference in loads acting on the columns in these sections was sufficient to justify the use of a second column size. Ideal constructability is using only one column size; however, in this case using two would save a lot of money.

The column with the greatest combination of load and tributary area in the Mechanical Penthouse section was L-13, and it was S-17.7 for the areas not supporting the Mechanical Penthouse. These columns will require the largest column capacity, which would be the limiting factor. When designing the columns, the load on the more heavily loaded first floor column was used to design the column stack extending to all three floors. This load in pounder per square foot was multiplied by the tributary area, which is the box created by using the distance halfway between columns. The next step was to check the column's design capacity and make sure it was greater than the factored load. Various HSS column sizes were tested for the best choice column. The area and radius of gyration were both obtained from Table 1-12 in the *AISC Steel Construction Manual*. The critical stress was determined, from which the load capacity for the column was calculated.

A W-shape section was also investigated for use in the design. The HSS 5x5x$^3/_8$ is 22.37 pounds per linear foot. A W10x19 is the smallest section that is not slender for a 50 ksi yield strength. This column proved to be not even close to the capacity needed to support the gravity loads, therefore a much bigger and heavier column would be needed. Since the unit cost of steel is by the pound, the HSS column would be much more cost efficient. The HSS column was chosen since it is the lower cost option of the two shapes, and since both shapes follow the same constructability method. Also, HSS columns are sometimes used as exposed columns, so the use of this shape gives the architect more options for building aesthetics.

4.5.2 Results

Two columns were selected for use in the new design. HSS 5 x 5 x ½ columns proved to be adequate for the sections of the building that are not supporting the Mechanical Penthouse. The capacity for this size column with effective length of 14-feet is just over 165 kips. The largest load acting on the column, which is on the first floor column, is 128 kips. Therefore, this size is very adequate for the loading it is under.

The columns under the Mechanical Penthouse were under a much higher loading. HSS 12 x 12 x 5/8 columns provided 875 kips of capacity. The first floor column has a factored load of 764 kips acting on it. This column size was chosen for the three interior columns and the exterior columns between column lines 8 and 13.

The difference between the two column sizes is substantial. This is because truss girders were used in the sections that are not supporting the Mechanical Penthouse. Truss girders are much lighter than the rolled beam sections that would be necessary to support the same load that is carried by the truss girders. Also, the columns are spaced 12-feet apart on the outside of the building, so the tributary area is only about 200 square feet. These factors result in minimal load acting on the columns. On the other hand, the columns supporting the Mechanical Penthouse support five times the load of the other columns. The truss girders were not used in this section, which created large and heavy wide flanged beams and girders. These columns take on a lot of loading, so the column size had to be much greater than the previously discussed columns.

4.6 Connection Design

Double angle connections were called for in original structural plans presented by EDG. The team adopted this practice and applied double angle connections in the alternate design. Incorporating lateral bracing to resist lateral loads permitted the application of simple connections instead of providing a more costly rigid frame system.

4.6.1 Methodology

Spreadsheets were created in *Microsoft Excel* following the same analysis procedure presented in section 3.6. The structural analysis program *MDSolids* to solve for reaction forces for complex situations to save time. Three separate connection scenarios were analyzed to ensure the structural design would be adequately supported. These scenarios are listed as follows:

- 2 Girder-to-Column Connections (labeled B3 and C2)
- 1 Beam-to-Girder Connection (labeled C1)

The locations of the connections are outlined below in Figure 32.

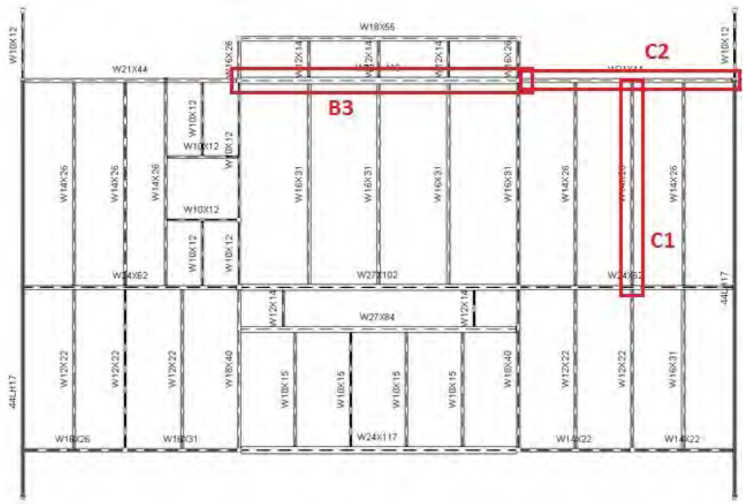


Figure 32: Locations of Analyzed Connections

4.6.2 Results

Table 18 displays the three connections that were designed and analyzed.

Table 18: Connection Summary

Connection Summary							
Connection Type	Item	Angle	# of Bolts	Bolt Spacing	Rolled Edge Distance	Sheared Edge Distance	Total Length (L)
Girder-Column	B3	<2L 3½ x 3½ x 1/4	8	3 in.	1 in.	1.25 in.	23.5 in.
Girder-Column	C2	<2L 3½ x 3½ x 1/4	4	3 in.	1 in.	1.25 in.	11.5 in.
Beam-Girder	C1	<2L 3½ x 3½ x 1/4	4	3 in.	1 in.	1.25 in.	11.5 in.

The connection of Girder B3 to its supporting columns was analyzed because of the significant amount of load it carries to the columns. The end reactions of this girder was calculated at 232.50 kips, resulting in the use of eight bolts spaced at 3 inches. Girder C2 was chosen to be analyzed because there were several other girders that carried similar loads as beam C1 was chosen for similar reasons. Calculations require beam C1 to have a minimum of 3 bolts spaced 3 inches, however, increasing the bolts to 4 spaced at 3 inches will provide the connection for C1 with the same geometry at C2. Therefore, this connection can be applied to most of the girders of and beams in the system promoting more efficient constructability and cost.

4.7 Base Plates and Footing Design

The footings and base plates were designed in the same practice as the columns. For constructability purposes, and to save money on the concrete formwork for the footings, there was a typical footing size and a typical base plate size established for each of the two column sections that were designed.

4.7.1 Methodology

After the columns were designed, a footing was designed to support the column, as well as anchor the structure to the subsoil. The only information needed for a footing area design is the soil bearing capacity and the factored load. The soil bearing capacity (f_p) was given in the structural drawings as 3 tsf, or 6 ksf. The unfactored loading (P) was then found throughout the column stack for each of the two columns. Finally, the equation $A = P / f_p$ was used to find the required area of the footing (A). After the area is found, the square root gives the minimum square dimension for the footing. For constructability purposes, the footing dimensions were rounded up to the nearest 6".

The next part of footing design is the thickness. The team determined the thickness of the concrete so as to eliminate the use of stirrup steel. Specifically, the punching shear and beam shear capacities of the footing were defined to exceed the factored design load. The punching shear is the force the footing must counteract from the column that wants to break through the footing. The critical section for evaluating the punching shear is a distance of half the effective depth to the steel in the footing in all directions from the column edge. The beam shear is due to maximum shear force of the soil forcing the footing up and the column forcing the footing down. The beam shear is considered critical at a distance equal to the effective depth away from the edge of the column. Whichever shear force required the thickest footing determined the final footing thickness.

In order to keep the footing from bending under the pressure of the soil, reinforcing dowels are used toward the bottom of the footing. The moment was found using the soil bearing capacity and various dimensions of the footing as designed. The dimensions along with a sketch can be found in Appendix N. From this, the minimum steel area required is determined. The size of reinforcement bars are determined based on the amount of steel needed. Lastly, a development length into the footing past the column edge is found using a Table from *Design of Concrete Structures*. (Nilson et. all, 2009)

Once the overall footing size was determined, the base plate was designed. First, the column properties, such as the thickness and outside dimensions, are needed to find the base and height of the column. A combination of equations using the compressive strength of the concrete, the factored design load at the base of the column, and the area of the footing resulted in a trial dimension for the base plate. The base plate should be a square dimension if possible. The trial plate size must be checked for bearing strength and finally the required plate thickness. The bearing strength was found using the equation $\phi_c P_p = \phi_c (0.85f'_c) A_1$. The capacity must be greater than the factored load acting on the column and base plate. Lastly, the thickness was determined using the greatest of three equations that use the base plate dimensions.

4.7.2 Results

As previously mentioned, two footing sizes were selected to use as standard sections for the proposed design of the school. The footing under column S-17.7 is to be a square spread footing that measures 4-feet on either side. The footings are 2-feet thick, with a distance of 20.5" to the center of gravity of the steel. This footing has a beam shear capacity of 171.8 kips, and a punching shear capacity of 161.7 kips, both of which exceed the factored design loading of 128 kips exerted by column S-17.7. The loads call for 11 No. 4 reinforced steel dowels to be used in each direction. These dowels will extend 11" past the edge of the column.

The footing supporting column L-13 was designed to be a 9'6" square spread footing with a thickness of 4-feet. This allowed a capacity of 826 kips of beam shear, and 834 kips of punching shear, both of which are greater than the loading of 764 kips for column L-13. 17 No. 9 bars were used 3" above the bottom of the footing in each direction. The development length for this scenario is 47".

The 9'6" square spread footing is very large compared to what the current design used for footings. However, this footing is sustained the loads from many of the columns that were removed from the original design. When mapped on the structural drawings, the footings are spaced fairly equally, which confirmed the footing size is reasonable. The 4' square footing is very comparable to the footings designed by EDG, which makes sense since the loads are similar to the columns in the existing design. The same size footing is to be reused as often as possible, since the formwork used for cast in place concrete is very expensive. It is recommended to pour the footings strategically to not lose too much time in the construction phase. Also, this way the formwork can be reused many times.

The two base plates that were designed proved to be as different as the footings to which they would be connected. The base plate for L-13, the column stack supporting the Mechanical Penthouse, was designed to be 1.75" thick, and 17" square around the edges. This base plate would be about 2.5" larger than the column in all directions. The base plate for column S-17.7 is 1" thick and 7" square.

The PL 1" x 7" x 7" base plate is a reasonable size base plate for the size of the column and the load acting on it. The base plate dimensioned PL 1.75" x 17" x 17" is much bigger than the other base plate, however it is transferring much more load than the smaller plate.

4.8 Lateral Force Resisting System Design

As structural engineers design structural frames to support gravity loads, a building must also be able to withstand lateral loads, which consist of wind and earthquake loads. Lateral Force Resisting Systems must therefore be included in the steel frame of the building. The design of these systems began once the preliminary sizes for the beams, girders and columns were determined. Two types of framing systems were initially considered – a braced frame and an unbraced or rigid frame. Since the current building already has braced frames, the project team determined that this type of bracing would be suitable for the alternate design. There is a variety of bracing configurations such as K braces and single diagonal braces. For the alternative design, a single diagonal brace was chosen as opposed to a K brace, which is what Avery currently has. The project team chose this type of bracing due to the smaller loading applied to each frame. With lesser loads, the axial compression and tensile forces on the bracing decreased. Furthermore, a diagonal brace used less material than a K brace, which in turn will decrease the steel costs.

Braces should be located along the center of mass of the building and should be symmetrical to limit torsion. However, since the number of interior columns in the building has been minimized, the possible locations of the braces were restricted. The architectural layouts, such as wall and window placement, were also considered when deciding the bracing locations, but were not the governing factor in the final decision because the priority was to have a building design that is structurally sound. Therefore, if some brace locations will cause a need for architectural changes, those changes will then be made. As a result, most of the braces will be located along the exterior walls of the building in the North-South and West-East directions. The location of these braces will cause a need in changing the location of windows. The locations of BF-1 and BF-2 are the only brace locations that remained the same as the current building. There are four frames in the North-South direction, and there are eight frames in the West-East direction. The increase in number of frames is due to the shorter span of the BF-4 frame.

4.8.1 Methodology

This following section describes the methodology for determining the lateral loads acting on the structure and the *RISA* analysis that was used.

Lateral Loads

To determine the design for the Lateral Force Resisting Systems, the lateral wind and earthquake loads were determined first. The wind load was determined by using Method 1: Simplified Procedure addressed in Section 6.4 of *ASCE 7-05*. There were different factors used in calculating the wind load, and these factors depended on various assumptions or criteria such as the location of the building and the wind pressure. The equation used to determine the wind load was: $P_s = I \lambda K_{zt} p_{s30}$.

Table 19 shows the factors used to determine the wind loads, and the final determined wind load (in psf). Most of the values of these factors were provided in the structural drawings.

Table 19: Wind Load Factors

Factor	Value	Important Assumption (s)	Source & Supporting Table and/or Figure	Code
Basic Wind Speed, V, (mph)	100	Dedham	Table 1604.10	Mass. Bld Code
Importance Factor, I*	1.15	Category III Building	Table 6-1	ASCE 7-05
Height and Exposure Factor, λ	1.13	Exposure B, Avg height 46ft	Figure 6-3	ASCE 7-05
Topographic Factor, K_{zt}	1	Flat Ground	Figure 6-4	ASCE 7-05
Simplified Design Wind Pressure, p_{s30}, (psf)	15.9	100 mph wind, Exposure B Category, End Zone of Wall	Figure 6-2	ASCE 7-05
Wind Load, p_s (psf)	**20.66**			

The earthquake loads were determined using provisions addressed in Section 11 and Section 12 of *ASCE 7-05*. The determination of the earthquake load is complex as it requires the use of different equations to determine coefficients that depend on different criteria such as site class, occupancy level and location. These coefficients were then used in other equations required to determine the earthquake load. Similar to the treatment of wind loads, most of the factors necessary for calculating earthquake forces were provided in the structural drawings. These were the factors that remained the same due to the same soil conditions and occupancy level. Therefore, the project team only had to determine the factors that changed as a result of the change in structural design. These factors include, but are not limited to, the total weight of the building (sum of the structural steel weight and dead loads) and the height of the highest level of the building. These main factors then affected the values of other coefficients, which were calculated. For example, the main factor difference between the current and alternate design was the total base shear of the building. The earthquake loads were determined as a proportion of the base shear, which is calculated by multiplying the seismic response coefficient to the total weight of the building, which was the sum of the weight of the structural steel and the dead loads on the alternative design. Table 20 shows the factors used to determine the earthquake load.

Table 20: Earthquake Load Factors

Name/Description	Factor	Value	Source
Seismic Response Coefficient	C_S	0.0867	Struct. Dwgs
Redundancy Factor	ρ	1	Section 12.3.4
Spectral Response Coefficient	S_{DS}	0.208	Struct. Dwgs
Effect of Dead Load (k)	D	3,279	Calculated
Earthquake Force on building (k)	E	425	Calculated
Height at highest level of building (ft)	h_n	51.75	
Building Period Coefficient	C_t	0.028	Table 12.8-2
Building Period Coefficient	x	0.8	Table 12.8-2
Period	T_L	0.66	Equation 12.8-7
Exponent related to structure period (sec)	k	1.11	Calculated
Total Base Shear (k)	V	289	Calculated
Horizontal Seismic Force Component (k)	$Q_E = V$	289	Calculated

Once the wind loads and earthquake loads were determined, the distribution of these loads in each of two perpendicular directions was calculated. The wind force acting in each direction was determined by multiplying the pressure by the total wall surface area in that direction. The total surface area only included the wall area in the main building. The gym area of the school was assumed to act as a separate entity. Although the frames are of different heights, the total wind force in the North-South direction and the West-East direction was evenly distributed between the four and eight frames, respectively. The wind force on each level was calculated based on the tributary height of the frame. For frames BF-2 and BF-3 that had four elevated levels, it was assumed that the wind forces at third level would include those from above it to simplify the wind force calculations. Therefore, the wind force was still applied on first three levels. The total earthquake force on each level was determined as a portion of the total base shear, based on the height and total weight of each level. This force was evenly distributed between the frames in each direction as well. The wind and earthquake forces applied on each level for each type of frame is shown in Table 21. In the table, "N-S" and "W-E" indicates the direction of the frame. Refer to Appendix O for tables showing the information used to calculate the different, individual lateral forces.

Table 21: Wind Force and Earthquake Force on Each Level of Each Frame

BF-1 N-S			
Level	**Avg. Tributary Height**	**Wind Force (k)**	**Earthquake Force (k)**
Roof	9.34	**10.6**	**42.63**
Third Level	16.34	**18.6**	**19.51**
Second Level	14	**16.0**	**10.12**
First Level	7	**8.0**	-
TOTAL	46.68	53.2	72.3

BF-2 N-S			
Level	**Avg. Tributary Height**	**Wind Force (k)**	**Earthquake Force (k)**
MEP	11.67	**13.3**	**42.63**
Third Level	14	**16.0**	**19.51**
Second Level	14	**16.0**	**10.12**
First Level	7	**8.0**	-
TOTAL	46.67	53.3	72.3

BF-3 N-S			
Level	**Avg. Tributary Height**	**Wind Force (k)**	**Earthquake Force (k)**
MEP	11.67	**13.3**	**42.63**
Third Level	14	**16.0**	**19.51**
Second Level	14	**16.0**	**10.12**
First Level	7	**8.0**	-
TOTAL	46.67	53.3	72.3

BF-4 W-E			
Level	**Avg. Tributary Height**	**Wind Force (k)**	**Earthquake Force (k)**
Roof	9.34	**2.8**	**42.63**
Third Level	16.34	**4.9**	**19.51**
Second Level	14	**4.2**	**10.12**
First Level	7	**2.1**	-
TOTAL	46.68	14.0	72.3

RISA Analysis

After the wind and earthquake forces were determined, the computer software *2D RISA* and *Microsoft Excel* spreadsheets were used to perform and organize the structural analysis of each frame without a diagonal brace. With *RISA,* a model of each frame was first created.

After the 2D models were created, the lateral loads were applied, as well as the total dead loads (distributed loads and point loads), live loads, snow loads and roof live loads. Through *RISA*, the axial, shear and bending moments due to each of the six types of loading were determined. Each frame was also analyzed for its total lateral deflection. Each loading system was applied and analyzed separately so that it could be substituted into the load combinations through the *Exce*l spreadsheets developed. From *ASCE 7-10*, the team determined three LRFD load combinations that had the greatest effect on the design of the frame. These load combinations are:

LC 4: U = 1.2D + 1.0W + 0.5L + 0.5S
LC 5: U = 1.2D +1.0E +0.5L + 0.2S
LC 7: U = 0.9D + 1.0E

Load combinations (LC) 4 and 5 were chosen because they take into account the wind or earthquake forces, which produce the greatest reaction forces. With these combinations as well, the original load combination has a live load factor of 1. However, since the live load for most of the building is less than or equal to 100 psf, the load factor for the live load is permitted to be equal to 0.5 (ASCE, 2010). Load combination 7 was chosen to assess the possibility of uplift.

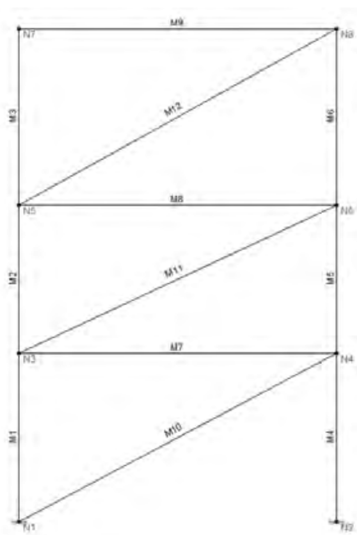


Figure 33: BF-1 *RISA* Model

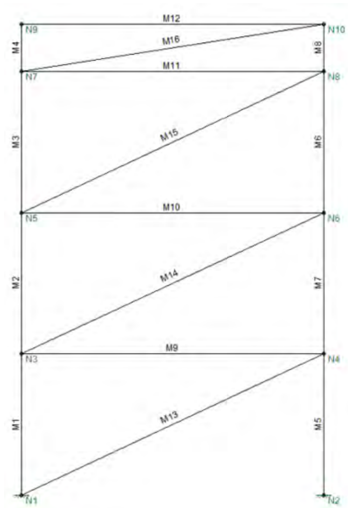


Figure 34: BF-2 *RISA* Model

After the initial frames were analyzed, the diagonal frame was placed in the model. Figure 33 and Figure 34 show the *RISA* models of BF-1 and BF-2 frame, respectively. BF-1 has the same member labels as BF-4, while BF-2 has the same member labels as BF-3. However the dimensions are different. A summary of the story heights, span and girder sizes for each frame are shown in Table 23. For BF-1, BF-2 and BF-4, their initial column design was an HSS5x5x1/2 as determined in Chapter 4. As for BF-3, its initial column design was an HSS12x12x5/8.

Table 22: Dimensions of Each Frame and its Girders' Sizes

Top Level of Bracing	Story Height (ft)				Girder Size			
	BF-1	BF-2	BF-3	BF-4	BF-1	BF-2	BF-3	BF-4
Roof	18.67	4.67	9.75	18.67	W16x31	W16x31	W16x26	W16x31
MEP	-	14	14	-	-	W16x36	W16x31	-
Third level	14	14	14	14	W16x36	W18x46	W18x40	W16x36
Second Level	14	14	14	14	W16x36	W21x62	W18x40	W16x36
Span of Frame (ft)	30	30	23.33	12				

With the bracing included, another structural analysis of the axial forces in each frame due to the different loads was performed to determine the minimum required axial compression force in each diagonal bracing. Through this, the project team was able to choose a preliminary square HSS size and include its size and dead loads in the *RISA* models. Another structural analysis was performed, following the steps described earlier, and lateral checks were performed. All lateral deflections had to be less than the limiting ratio of H/400. H refers to the overall building height when the total lateral deflection is being calculated. When just one story is being calculated, the height of the specific story is used for H. If the deflections exceeded this, a larger bracing size was used.

Since the lateral loads have been taken into account, and there are additional dead loads applied on the column due to the weight of the bracing, the adequacy of the column designs had to be checked. Therefore, a second-order analysis was required and the Story Stiffness Method was an approximate way to archive this. Through this method, the column analysis was performed by checking for the required tensile strength P_r and required flexural strength M_r. The three load combinations stated earlier were used in this method. With the column analysis, only the members on the first level were examined because the lowest-tier columns support the largest the loads. Finding the column that experiences the largest combined axial and bending compression allows for a conservative design of the remaining beams. The column analysis required determining the following values: column loading effects (P_{nt}, M_{nt}), the lateral deflection, Amplifier B_1, required 2^{nd}-order strength (P_r and M_r) and the governing Interaction Equation, which was dependent on the ratio of P_r/P_c. Since frame is a concentric braced frame, it can be idealized as a truss, and therefore, there will be no column moments developed. Hence, the lateral sway through the multiplier B_2 does not have to be taken to account. As for the girders, they did not need to be checked for adequacy since no additional gravity loads were added to them.

4.8.2 Results

Through the structural analysis performed by using *RISA* and *Microsoft Excel* spreadsheets, four lateral frame designs were determined. Refer to Appendix P to see the axial, shear, moment results from *RISA* and the total reactions calculated based on the load combinations. From the results, it was determined that load combination 5: U=1.2D+1.0E+0.5L+-.2S produced the greatest reaction forces, which means that the earthquake loads had a greater effect on the building than the wind loads. Furthermore, the results showed that the inner members (M4 for BF-1 and BF-4, and M5 for BF-2 and BF-3), experienced the greatest amount of axial compressive loading and bending moment.

After the bracing sizes were determined, the adequacy of the first level columns was checked through the Story Stiffness Method. It was known that the initial column designs were overdesigned, therefore it was expected that most, if not all of the columns, would be adequate to support the lateral loads. This was true for all the frames except BF-4, which required a larger column size. A summary of the lateral framing design is shown in Table 23 .P_r is the required axial compressive strength of each brace while P_c is the design axial compressive strength of the member.

Table 23: Lateral Resisting Frames Design

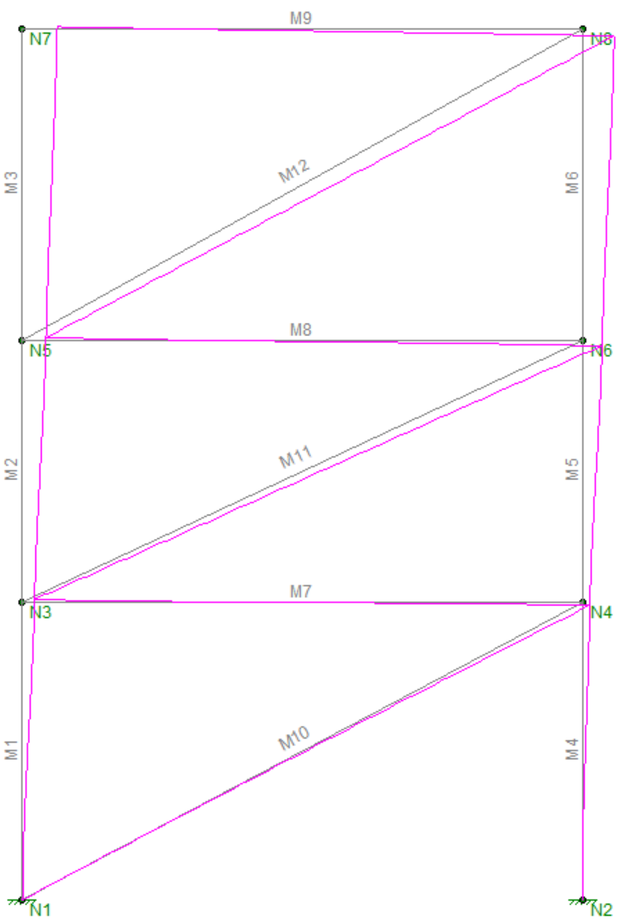
BF-1 Brace and Column Design					
Level	**Length**	**Pr (k)**	**Brace Size**	**ΦPn (k)**	**Column Size**
Third Level	35.34	48.55	HSS 7x7x1/2	101.30	HSS5x5x1/2
Second Level	33.11	68.06	HSS 7x7x5/8	115.23	HSS5x5x1/2
First Level	33.11	81.61	HSS 7x7x5/8	115.23	HSS5x5x1/2

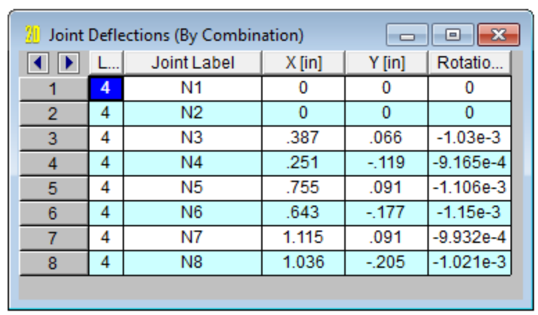
BF-2 Brace and Column Design					
Level	**Length**	**Pr (k)**	**Brace Size**	**ΦPn (k)**	**Column Size**
MEP	30.36	1.2	HSS 5x5x1/4	27.90	HSS5x5x1/2
Third Level	33.11	46.74	HSS 7x7x1/2	115.23	HSS5x5x1/2
Second Level	33.11	68.16	HSS 7x7x5/8	134.12	HSS5x5x1/2
First Level	33.11	79.39	HSS 7x7x5/8	134.12	HSS5x5x1/2

BF-3 Brace and Column Design					
Level	**Length**	**Pr (k)**	**Brace Size**	**ΦPn (k)**	**Column Size**
MEP	25.29	0.38	HSS 5x5x1/4	40.20	HSS12x12x3/8
Third Level	27.21	47.42	HSS 7x7x1/2	178.53	HSS12x12x3/8
Second Level	27.21	69.72	HSS 7x7x1/2	178.53	HSS12x12x3/8
First Level	27.21	79.58	HSS 7x7x1/2	178.53	HSS12x12x3/8

BF-4 Brace and Column Design					
Level	**Length**	**Pr (k)**	**Brace Size**	**ΦPn (k)**	**Column Size**
Third Level	22.19	36.16	HSS 7x7x1/2	242.67	HSS6x6x1/2
Second Level	18.44	47.25	HSS 7x7x5/8	358.82	HSS6x6x1/2
First Level	18.44	60.31	HSS 7x7x5/8	358.82	HSS6x6x1/2

With the lateral frame analysis, the lateral deflection check was the last step in determining the adequacy of the braces and columns. The deflection was determined through the use of *RISA*. An example of the *RISA* results for BF-1 is shown in Figure 35. The pink line shows the deflection of the frame due to the wind and earthquake load. The line has been magnified for easier view of the deflection. For all deflection diagrams, refer to Appendix O. The results of the lateral deflections are summarized in Table 24. As it can be seen in the table, all frames had deflections less than H/400, but the deflection for BF-4 was very close to the limiting ratio. However, since the ratio of H/400 is already a conservative limit, the size of the brace for this frame is adequate enough. The rest of the frames have deflections less than 1 in or slightly more. From the results, it can also be noted that left area of the frames (nodes N3, N5, N7 and N9), tend to have a slightly larger deflection than the right side. The observed differences on the left versus right of the frame reflect the axial shortening of the horizontal




Joint Deflections (By Combination)

	L...	Joint Label	X [in]	Y [in]	Rotatio...
1	4	N1	0	0	0
2	4	N2	0	0	0
3	4	N3	.387	.066	-1.03e-3
4	4	N4	.251	-.119	-9.165e-4
5	4	N5	.755	.091	-1.106e-3
6	4	N6	.643	-.177	-1.15e-3
7	4	N7	1.115	.091	-9.932e-4
8	4	N8	1.036	-.205	-1.021e-3

Figure 35: *RISA* Analysis of Lateral Deflection of BF-1

girder.

Table 24: Lateral Deflection Results of All Four Frames

Level	Node	Lateral Deflection [in]							
		BF-1	H/400	BF-2	H/400	BF-3	H/400	BF-4	H/400
Second Level	N3	0.388	0.42	0.227	0.42	0.306	0.42	0.345	0.42
	N4	0.251	0.42	0.288	0.42	0.215	0.42	0.313	0.42
Third Level	N5	0.756	0.84	0.536	0.84	0.592	0.84	0.781	0.84
	N6	0.643	0.84	0.587	0.84	0.52	0.84	0.756	0.84
Fourth Level	N7	1.115	1.40	0.821	1.26	0.82	1.26	1.353	1.40
	N8	1.036	1.40	0.821	1.26	0.764	1.26	1.338	1.40
Fifth Level	N9	-	-	0.867	1.40	0.881	1.551	-	-
	N10	-	-	0.867	1.40	0.88	1.551	-	-

Looking at the final design of all four frames, it can be seen that there are only three bracing sizes and three column sizes considered. The range of sizing was minimized because it would be more cost effective to have a small range of member sizing and will result to faster fabrication. With this in mind, the project team minimized the bracing sizing to three, causing some members to be very conservative. All the diagonal braces could have been an HSS7x7x5/8, but smaller sizes were chosen to decrease the amount of steel.

4.9 Alternative Building Layout

While the different structural members and layouts were being determined, a three three-dimensional model of the alternative structural design was being created on *Revit Structure*. The last members to be included in the model were the diagonal bracing. The model of the current design was used as the base for the alternative. This allowed quicker changes or removal of members instead of developing it from scratch. Also, this ensured the project team that the areas that were supposed to stay the same actually did. This was important when extracting material data to be used of the cost estimate, which will be addressed in the next chapter. **Error! Reference source not found.** to Figure 42 show the floor layouts, elevation views, and three-dimensional views of the building. The structural floor layout of the MEP level isn't shown since the location and sizing of most of the beams and girders remained the same.

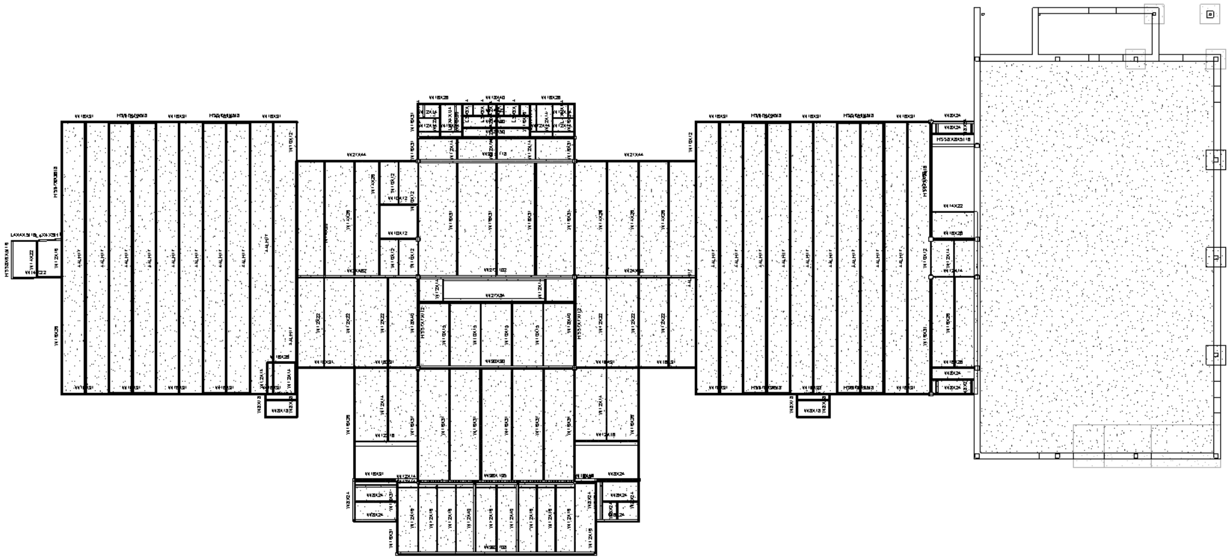


Figure 36: Second Floor Structural Layout

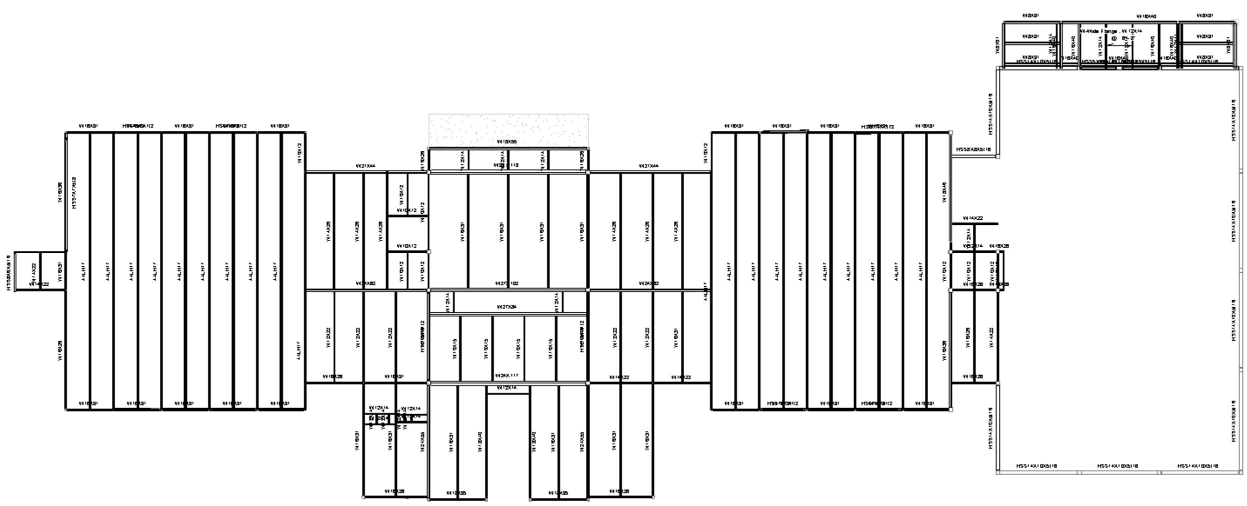


Figure 37: Third Floor Structural Layout

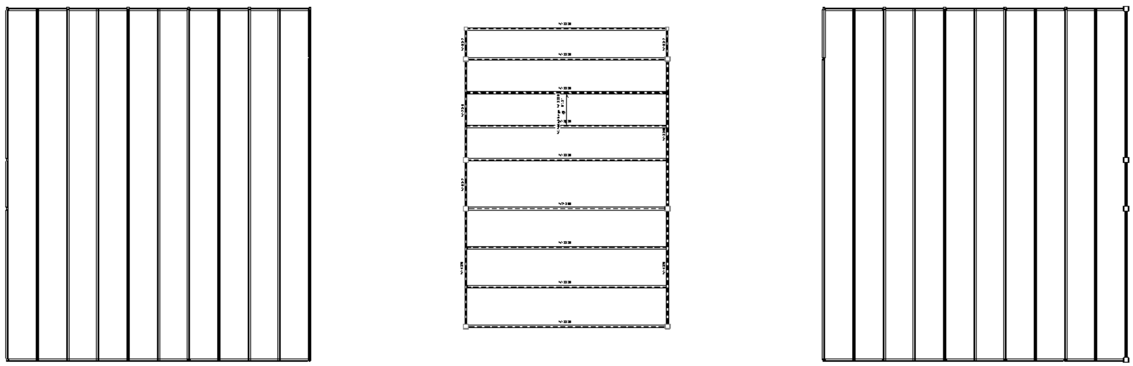


Figure 38: MEP Roof and Roof Structural Layout

Figure 39: South Elevation View

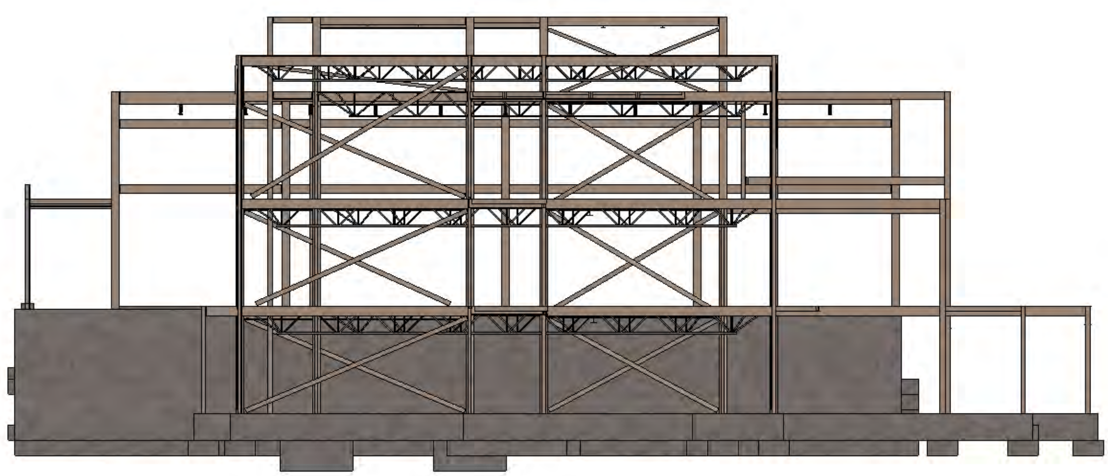


Figure 40: West Elevation View

Figure 41: 3D Rendered View of South Elevation

Figure 42: 3D Rendered View of North Elevation

Through the three-dimensional images of the building, an easier visualization of the alternative building design can be made. From these images, and comparing them to the images of the current design in Chapter 3, one can see that the main differences between the two designs are the location of columns and braces and the flat roofs of the building.

5.0 Cost Estimate

The cost estimates for the current and alternative structural designs were completed as one of the project deliverables. Although the team knew the total cost estimate determined by Consigli for the steel structure, the team developed a second cost estimate for the current design based on the *Revit* model that was created in Chapter 3. The team determined their own estimate so that the same unit costs can be used for both the current design and alternative designs, which enables a reasonable cost comparison to be made. The steel construction for Avery began in early 2011 (Dedham Patch, 2012). As a result, the project team assumed Consigli's estimate was made for that year. To approximate the total cost of steel for the current and alternative designs, the team obtained the average unit costs found in the *RSMeans Construction Cost Data 2011*. Since the unit costs are also the average for the US, they were adjusted based on a location factor for Boston, which is the closest city to the town of Dedham.

For each size member in the book, there was a different bare cost (cost of materials, labor, and equipment) as well as a total cost that included overhead costs. The cost values for each member were placed into a summary sheet that included the beam type, total unit length and the different bare unit and total costs. *Revit Structure* was very useful in determining cost estimates as it has the ability to extract and quantify the different items in a three-dimensional model. Therefore, once most of the beams were placed in the model, the structural framework and column schedules were extracted. These schedules listed all the members in the model and included each member's length and unit weight. The schedule of structural members was then exported into an *Excel* spreadsheet in which the team determined the total quantity (in linear foot) of each member type, multiplied it by its respective unit costs, and then totaled member costs to calculate an estimated cost for the structural framework. The same method was used in determining the structural cost of the proposed alternative design that was discussed in Chapter 4.

5.1 Structural Cost Estimate of Current Design

Based on the payment requisition provided by Consigli, the total cost estimate for the structural steel for the existing Avery School was $1,150,000. However, in the payment requisition, the total tonnage of steel was not included. Therefore, the project team determined an approximate weight through the use of *Revit Structure* and determined a corresponding unit cost per pound, which is addressed later on in this section. Consigli's total estimate consisted of the cost estimates for the drawings, materials (raw steel), fabrication (shop labor), steel decking (material) and erection (steel and decking). A breakdown of all these categories in the building can be seen in Table 25 as well as the percentage of the total costs for each category. This data was obtained from Consigli's payment requisition and shows how the building was divided into twelve sections, or blocks. However, the team was not provided with any information on what areas of the building the blocks referred to. For the drawings cost, it was assumed that this included both the structural drawings and shop drawings.

Table 25: Consigli's Cost Estimate of Current Design (Consigli Construction, 2010)

	Drawings	Material (Raw Steel)	Fabrication (Shop Labor)	Deck (Material)	Erection (steel and Decking)	
Anchors/ Embedded/ Mobilization	$ 800.00	$ 8,000.00	-	-	$ 4,000.00	
Block 1	$ 5,400.00	$ 64,000.00	$ 30,400.00	$ 7,300.00	$ 18,000.00	
Block 2	$ 5,400.00	$ 64,100.00	$ 30,500.00	$ 7,300.00	$ 30,000.00	
Block 3	$ 3,000.00	$ 35,000.00	$ 16,700.00	$ 7,300.00	$ 20,000.00	
Block 4	$ 3,000.00	$ 33,000.00	$ 15,700.00	$ 7,300.00	$ 30,000.00	
Block 5	$ 2,600.00	$ 27,600.00	$ 13,200.00	$ 2,000.00	$ 20,000.00	
Block 6	$ 4,000.00	$ 47,700.00	$ 23,000.00	$ 3,800.00	$ 25,000.00	
Block 7	$ 2,400.00	$ 28,400.00	$ 13,500.00	$ 8,300.00	$ 29,000.00	
Block 8	$ 2,350.00	$ 27,600.00	$ 13,200.00	$ 8,300.00	$ 29,000.00	
Block 9	$ 4,550.00	$ 53,500.00	$ 25,500.00	$ 5,600.00	$ 15,000.00	
Block 10	$ 3,900.00	$ 45,500.00	$ 21,700.00	$ 5,600.00	$ 20,000.00	
Block 11	$ 2,300.00	$ 28,000.00	$ 13,300.00	$ 27,000.00	$ 20,000.00	
Block 12	$ 2,300.00	$ 27,600.00	$ 13,200.00	$ 8,300.00	$ 30,000.00	**TOTAL**
SUBTOTALS	$ 42,000.00	$ 490,000.00	$ 229,900.00	$ 98,100.00	$ 290,000.00	$ 1,150,000.00
Percentage of Total	3.7%	42.6%	20.0%	8.5%	25.2%	100.0%

In the *RSMeans* book, the unit bare costs are divided into material, labor and equipment costs. As described in *RSMeans,* the bare material costs are usually the "Invoice Cost" from the fabrication shop, and include the mill base price of the steel plus mill extras, transportation to the shop, shop drawings and detailing where warranted, shop fabrication and handling and a few other factors (RSMeans, 2011). Therefore, it can be assumed that the *RSMeans* total raw steel cost is almost equivalent to Consigli's material and fabrication cost. It is not assumed that they are the same since Consigli has a separate cost for drawings, while *RSMeans* include it in the raw material cost. Thus from the requisition, the drawings cost was removed from the total estimate, giving subtotal of $1,108,000 for the steel framework, which was used as a reference when the project team determined their own estimate. Consigli's total cost estimate for raw material and fabrication, which is $719,900, will also be compared to the total raw material cost determined from *RSMeans.*

Using *Revit Structure*, the project team was able to determine an approximate steel weight of the existing building by exporting the structural framework schedule and the column schedule and importing them into an *Excel* spreadsheet. In the schedules, they included the total length of every member in the building, as well as the weight per linear foot of the member. With the use of *Excel*, the total quantity of each type of member was determined and was multiplied by the weight per linear foot, giving a total building steel weight of 681,800 lbs or approximately 310 tons. A breakdown of this weight by member type is shown in Table 26. By dividing the steel weight by the total raw and fabrication cost determined by Consigli, a steel unit cost of $1.06 per pound or $2320 per ton was determined. Since the approximated steel weight does not

Table 26: Total Steel Weight of Current Design

Type	Weight (lbs)
W beams	447,441
HSS beams	43,727
HSS columns	129,178
HSS bracing	46,022
Joist Girders	15,432
L-Angles	809
TOTAL (lbs)	681,800
TOTAL (tons)	309.91

include all the steel in the building, the unit cost determined in this manner would be more than the actual unit cost. Nonetheless, this value was used and compared to the material unit cost obtained from *RSMeans.*

From the structural framework schedule and column schedule, the quantity of W beams (linear foot) and columns (weight) was determined. These values were then multiplied to the unit costs found in *RSMeans.* However, during the process of determining the costs per linear foot of each beam member, the team encountered a minor problem in which some of the beams used in the design were not present in the *RSMeans.* When this occurred, the project team decided to estimate the material cost per linear foot through interpolation, and used the formula below.

$$Cost\ per\ linear\ foot\ of\ Beam\ X = \frac{Cost\ of\ next\ beam\ size}{Weight\ of\ next\ beam\ size} * weight\ of\ beam\ X$$

For the labor and equipment cost, it usually increases after a certain beam size. Therefore, the unit labor and equipment cost chosen was the cost for the next beam size.

The total quantity in linear feet and the total bare costs for each beam type can be seen in Table 27, and the unit bare costs for each beam type can be seen in Appendix S. In this table, the

Table 27: Total Bare Costs for Different W Beams and Girders for Current Design

Beam Type	Total Quantity (ft)	Total Weight (lbs)	Total Bare Costs				Total Unit Bare Costs
			Material	Labor	Equipment	Total	
W6X12	64	768	$ 949.76	$ 282.88	$ 172.80	$ 1,405.44	$ 1.83
W8X10	304	3035	$ 3,763.40	$ 1,341.47	$ 819.45	$ 5,924.32	$ 1.95
W8X18	56	1013	$ 1,253.57	$ 248.63	$ 151.88	$ 1,654.07	$ 1.63
W8X24	172	4124	$ 5,069.58	$ 828.32	$ 506.96	$ 6,404.85	$ 1.55
W8X31	119	3699	$ 4,594.21	$ 575.17	$ 352.02	$ 5,521.40	$ 1.49
W10X12	564	6764	$ 8,369.91	$ 2,491.24	$ 1,521.80	$ 12,382.95	$ 1.83
W12X14	2227	31184	$ 38,590.22	$ 6,704.56	$ 4,098.47	$ 49,393.26	$ 1.58
W12X16	704	11257	$ 13,930.69	$ 2,117.75	$ 1,294.57	$ 17,343.00	$ 1.54
W12X19	210	3997	$ 4,905.68	$ 633.24	$ 387.10	$ 5,926.02	$ 1.48
W12X40	140	5619	$ 6,967.81	$ 459.37	$ 280.96	$ 7,708.14	$ 1.37
W14X22	1070	23541	$ 28,973.12	$ 2,867.68	$ 1,754.85	$ 33,595.65	$ 1.43
W14X30	68	2050	$ 2,528.21	$ 201.57	$ 122.99	$ 2,852.78	$ 1.39
W16X26	1556	40446	$ 49,780.16	$ 4,122.42	$ 2,520.12	$ 56,422.70	$ 1.40
W16X31	2960	91750	$ 113,947.30	$ 8,731.03	$ 5,327.41	$ 128,005.73	$ 1.40
W16X36	150	5400	$ 6,682.50	$ 498.00	$ 270.00	$ 7,450.50	$ 1.38
W16X40	352	14068	$ 17,409.65	$ 1,167.68	$ 713.97	$ 19,291.29	$ 1.37
W16X57	30	1710	$ 2,118.36	$ 104.70	$ 63.90	$ 2,286.96	$ 1.34
W18X35	370	12966	$ 16,114.58	$ 1,478.10	$ 666.81	$ 18,259.48	$ 1.41
W18X40	331	13251	$ 16,398.36	$ 1,321.81	$ 596.30	$ 18,316.47	$ 1.38
W18X46	344	15824	$ 19,608.00	$ 1,372.56	$ 619.20	$ 21,599.76	$ 1.37
W18X50	90	4500	$ 5,580.00	$ 378.00	$ 171.00	$ 6,129.00	$ 1.36
W18X55	40	2200	$ 2,720.00	$ 168.00	$ 76.00	$ 2,964.00	$ 1.35
W21X44	894	39321	$ 48,705.02	$ 321.72	$ 1,456.68	$ 50,483.42	$ 1.28
W21X55	87	4767	$ 5,881.68	$ 320.68	$ 141.27	$ 6,343.63	$ 1.33
W21X62	225	13930	$ 17,187.26	$ 831.28	$ 375.20	$ 18,393.73	$ 1.32
W24X55	818	44976	$ 55,607.00	$ 2,821.24	$ 1,275.69	$ 59,703.93	$ 1.33
W24X62	40	2480	$ 3,060.00	$ 138.00	$ 62.40	$ 3,260.40	$ 1.31
W24X68	40	2720	$ 3,360.00	$ 138.00	$ 62.40	$ 3,560.40	$ 1.31
W24X76	120	9120	$ 11,280.00	$ 414.00	$ 187.20	$ 11,881.20	$ 1.30
W24X84	40	3360	$ 4,160.00	$ 142.00	$ 64.00	$ 4,366.00	$ 1.30
W24X94	40	3760	$ 4,640.00	$ 142.00	$ 64.00	$ 4,846.00	$ 1.29
W24X117	80	9360	$ 11,600.00	$ 292.00	$ 132.00	$ 12,024.00	$ 1.28
W30X90	40	3600	$ 4,472.73	$ 127.60	$ 57.60	$ 4,657.93	$ 1.29
W30X108	51	5481	$ 6,800.50	$ 161.89	$ 73.08	$ 7,035.47	$ 1.28
W36X135	40	5400	$ 6,680.00	$ 131.20	$ 59.20	$ 6,870.40	$ 1.27
TOTAL NATIONAL AVG. (11')			$ 553,689	$ 44,076	$ 26,499	$ 624,264	
TOTAL BOSTON (11')			$ 566,424	$ 60,296	$ 36,251	$ 662,971	
ROUNDED TOTAL (11')			**$ 566,000.00**	**$ 60,000.00**	**$ 36,000.00**	**$ 663,000.00**	

	Material	Installation	Total
Boston Location Factor	102.3	136.8	117.7

calculated total unit cost for material, labor and equipment per pound of each W beam are also shown. Since the average unit cost of steel is usually in terms of dollars per pound, the project team determined the unit cost of each beam. This allowed the team to see the change in price as the size and weight of the beam increased. As seen in the table, the general trend was as the beam size increased, the total unit cost decreased. It also allowed unit costs of beams not found in *RSMeans* to be approximated. Since the total bare costs are based on a national average, the costs were adjusted based on a location factor for Boston. For the location factor for installation, it was assumed that installation included labor and equipment, therefore when adjusting these values, the factor of 136.8 was used. From the table, it can be seen that steel cost in Boston is higher than the national average.

For the columns, *RSMeans* unit costs were based on the price of material, labor and equipment per column of a certain height and size. For labor and equipment, it was assumed that the price would remain for any column height. The material unit cost per pound was approximated by dividing the total material cost by the total weight of the column. The unit costs are shown in Table 28.

Table 28: Unit Costs of HSS Columns

Type	Material ($/lb)	Labor ($/column)	Equipment ($/column)
HSS12X12X5/16	$ 1.21	$ 57.50	$ 36.00
HSS6X6X1/2	$ 1.34	$ 49.00	$ 30.00
HSS6X6X5/16	$ 1.34	$ 49.00	$ 30.00
HSS8X8X1/2	$ 1.25	$ 53.00	$ 32.50
HSS8X8X5/16	$ 1.25	$ 53.00	$ 32.50

To compare Consigli's estimate and the project team's estimate, the *RSMeans* unit cost per ton was also determined and is shown in Table 29. With this data, the cost of structural steel for the Avery School was compared to the average costs of other steel projects.

Table 29: 2011 Minimum and Maximum Bare Costs for Schools

Structural Steel Projects	2011 Bare Costs ($/ton)				Total Incl O&P
	Material	Labor	Equipment	Total	
Schools minimum	$ 2,250.00	$ 264.00	$ 119.00	$ 2,633.00	$ 3,050.00
Schools maximum	$ 3,275.00	$ 460.00	$ 209.00	$ 3,944.00	$ 4,650.00
Schools average	$ 2,762.50	$ 362.00	$ 164.00	$ 3,288.50	$ 3,850.00

Since the team could not determine whether the cost of steel for Avery is based on the minimum or maximum costs, the team determined the average cost per ton of steel by dividing the total cost of the W beams by the total weight of W beams that was determined earlier. This gave an average of $2740 per ton. Using this average value and multiplying it to the total weight of the steel structure, the team estimated the steel framework of the current design, to be approximately $ 850,000, as shown in Table 30. This estimate is 18% more than Consigli's cost estimate of raw material plus fabrication. The team recognized the possible reasons for this difference. With cost estimates, they can be done at the different phases of the project, therefore, the level of accuracy can vary. If the cost estimate was made on the completion of the design work, the accuracy may range from +15% to -10% (Oberlender, 2010). With an +18% difference from Consigli's cost, the *RSMeans* estimate seems reasonable. Also, another reason for the high unit costs could be the difference in interpretation as to what was considered raw cost, or fabrication cost.

Table 30: RSMeans Steel Raw Cost Compared to Consigli's Material and Fabrication Cost

Avg. Cost per ton	$	2,740
Consigli's Total Steel Cost	$	719,900.00
RS Means Total Steel Cost	$	850,000.00

For the steel structure, the total cost of the studs was also determined since the stud quantity will change in the alternative design. The stud unit cost was obtained from the *RSMeans* book using interpolation since the size that was used was not in the book. To estimate the number of studs, the maximum number of studs per beam type was determined and multiplied by the number of beams of that type. As a result, a total estimate of 8044 studs was determined. For more information on the breakdown of studs per beam type, refer to Appendix P. With a stud unit cost of $2.45 per stud, this gave a stud estimate $19,710.

5.2 Cost estimate of Alternative Design

Similar to the process for determining the cost estimate for the existing design, the cost estimate of the alternative design was determined through the use of *Revit Structure* and *Exce*l. The RS Means unit costs were already determined from the previous section, but the total unit costs of the W beams were still determined to see if there were any significant changes in the average cost. The breakdown of the total weight of the different members is shown in Table 31.

Table 31: Breakdown of Different Member Types in Alternate Design by Weight

Total Weight	
Type	**Weight (lbs)**
W beams	283,399
HSS beams	3,684
HSS columns	132,925
HSS bracing	41,039
Joist Girders	207,181
L-Angles	809
TOTAL (lbs)	669,037
TOTAL (tons)	304.11

Comparing this to the approximated weight of the initial design, the project team's alternate design was only about 20,000 pounds lighter, which is 2% of the initial weight. This was different to the project team's initial expectation of having a heavier design. However, the alternate design is lighter due to having less W beams, which were replaced with joist girders. As a result, the steel framework is a lot lighter. From the W beams data, the total raw cost was determined to be approximately $371,500 as shown in Table 32. The average cost for the W beams was $2880 per ton, which is $140 per ton more than the average determined for the current estimate.

Table 32: Total Cost of W Beams in Alternative Design

For ALL W Beams	Total Unit Bare Costs ($ per ft)				Total Incl O&P
	Material	Labor	Equipment	Total	
TOTAL National Avg (11')	$ 363,054.90	$ 25,427.35	$ 14,989.35	$ 404,927.44	$ 461,623.61
TOTAL Boston (11')	$ 371,405.16	$ 26,012.18	$ 15,334.11	$ 414,240.77	$ 472,240.95
Rounded Total Steel Cost	$ 371,500.00	$ 25,500.00	$ 15,000.00	$ 412,000.00	$ 465,000.00

With the same unit costs from the current design, the total cost of the raw steel and studs were determined. Table 33 shows the total quantity of the steel and studs and the final cost estimate of $846,000, which is only $4,000 less than the initial design. The cost estimate was

Table 33: Total Cost of Raw Steel and Studs for Alternate Design

	Total Quantity	Unit Cost	Total Cost
Steel	304 tons	$ 2,730.00	$ 829,920.00
Studs	6520 studs	$ 2.45	$ 15,974.00
Total			$ 845,894.00
Rounded Total Cost			$ 846,000.00

limited as it was based on the material unit cost of steel, allowing a significant decrease in costs. However, it should be noted that there are also other factors that are considered in the total price of steel such as drawings, labor and equipment. With these factors, the cost estimate of the steel would actually be higher.

6.0 Demolition vs. Repurposing

Naturally, the value and performance of buildings decay over time; therefore, its performance is examined to determine the extent of the building's lifecycle. Parties invested in the life cycle, whether they are private investors, local government, or commercial industries, are then faced with the challenging decision of demolition or reuse.

Limited land availability and the demand for new sustainability performances increase the need to demolish obsolete buildings (Shipley, 2010; Russel, 2001; Spector 2003). Demolition also promotes new construction fit with modern mechanical systems which attracts investment. Demolition cost estimation for the new Avery School was completed to grasp the extent of the demolition costs and affects. The cost estimating program *CostWorks* (RSMeans, 2011), a computer software database of construction costs gathered from *RSMeans*, was used to estimate the cost of demolition. This program roughly estimates the demolition of the new Avery School to be approximately \$630,000 in 2010. The adjusted future cost of demolition after a 50 year period, assuming an average inflation of 3%, is then calculated by using the future value formula:

$$Future\ Value = Present\ Value * (1 + inflation\ rate)^n$$

Therefore, the estimated cost for the demolition of the new Avery School after a life cycle of 50 years was estimated at \$2,800,000. Table 34 displays the breakdown costs for the demolition in 2010 dollars.

Table 34: Demolition Cost Breakdown

Estimated Cost for Demolition			
Quantity	**Unit**	**Description**	**Ext. Total O&P**
1320	C.Y.	Steel Disposal	$ 16,473.60
1661	C.Y.	Masonry Disposal	$ 20,064.88
331	Ton	Dump Charges for Steel	$ 32,769.00
2000	Ton	Dump Charges for Masonry	$ 198,000.00
200	L.F.	Foundation and Footing Disposal	$ 3,110.00
1385	Ton	Dump Charges for Foundations	$ 137,115.00
23444	SF Flr.	Selective demolition: gutting building interior	$ 230,923.40
Total Cost in Cost for 2010			$ 638,455.88
Total Cost in 50 years			$ 2,798,930.58

The amounts for steel and concrete foundations were gathered from material schedules generated by *Revit Structure*. The quantity of masonry was estimated by multiplying the approximate perimeter of the building by its average height. This estimate was slightly reduced to account for windows, doors, etc. Additionally, it should be noted that the demolition costs significantly vary from project-to-project since certain elements of the building can be salvaged or recycled, which was not accounted for in this estimate. The amount of recycled material from the deconstruction of a building greatly varies; however, according to the EPA, about 50% of generated waste from Construction and Demolition (C&D) derives from demolition (EPA, 2008). Although improved deconstruction methods have increased the recycle of materials in the past few years, the current trend suggests that demolition and new construction will gradually reduce as renovation increases to meet improved standards and the reduction of energy consumption (Kohler and Yang, 2007; Shipley, 2010).

Steadily rising energy costs will further increase the cost of material transportation, a significant expense of the demolition. Disposal charges are also increasing due to limited space in landfills and higher standards in recycling (EPA, 2008). According to Table 34, approximately 58% of the cost of the demolition is material dump charges. Increasing recycled material will partially alleviate these high costs; however, leaving the superstructure intact to be reused will completely diminish these expenses. According to a building reuse study in Canada, a developer quotes that they have saved between 10% and 12% by choosing to reuse buildings instead of new construction, and another source claimed that reusing a building can cut construction costs by 22% because the superstructure is still intact (Shipley et al, 2006). Flexible design also provides a quicker and easier way to adapt to changing demands, such as an increase in population or demand for commercial office space. Reconfiguration of space is a more effective solution than relocation (Fuster et al, 2006). The ability to swiftly change a building's function will provide savings in time and expenses, which increase productivity.

Mechanical systems are elements of a building that have a shorter life span. Therefore, to ensure the reusability of a building, it is critical that these subsystems can be easily replaced or refurbished in the future. The building envelope and finishes are also a challenge for reusability. Although they do not affect the structural integrity of the building, their deterioration lowers values of the building and its surroundings. Therefore, ongoing maintenance of the building's finishes is required for the building to be successfully reused. The rest of super structure has a long life span that requires little to no maintenance at all.

The trend and support for reuse is exemplified by the Town of Dedham. The town wants to avoid demolition of the old Avery School. According to the Avery Re-use Committee, established in 2010, the Town wanted to renovate the building to serve for community purposes and avoid substantial demolition costs (Avery Re-use Committee, 2012). Various uses were discarded, such as a Town Hall, Police Station, Senior Citizen Housing, and private housing, because of structural and spatial limitations. The Town also wanted a smooth/fast transition to avoid degradation of the vacant property that would have negative effects on the surrounding area. As a result, the Committee decided that they will convert the old Avery School into an Arts and Community Center, remaining a town-owned asset that will have minor maintenance costs (Avery Re-use Committee, 2012). If reuse was considered in the original construction of the Avery School, then they would not have to address these current issues.

7.0 Utilizing Structural Systems for an Alternate Building Size

After designing an alternate structural design for the current Avery School building, the team found it necessary to analyze other structural framing systems that would anticipate the reuse of a building. The Avery School building has a specific shape that was followed during the redesign of the structure. Models displaying the building's shape can be found in Chapter 4. During the redesign, the team determined that the shape and size of the building limits the structural framing schemes that can be applied to this building. The various-sized sections of the building were aggregated in a pattern that would not allow for the use of the staggered truss system or beam grid system. The team found that the ideal shape for a building to be designed cost efficiently for reusable open space is a rectangle with square bays. Therefore, a rectangular building design is used in this chapter in order to explore the application of other structural framing systems.

The rectangular bays of approximately 60' x 69' located in the wings of the Avery School (as identified as Section 1 in Figure 24) were used as test sections for the alternate framing systems. In order to perform the alternate framing systems, the sections were changed to be 60' x 60' square areas. This square shape is ideal for the beam grid system and the staggered truss system. Background information on the beam grid system and staggered truss system can be referred to in 2.5.4 and 2.5.5. These systems are able to provide large areas of open space as well as the joist girders that were utilized in the design provided in Chapter 4. The team performed an analysis using these framing systems in order to provide a cost comparison of alternate framing systems.

The framing systems were evaluated and compared based on the amount of open space they provide, the loading they can withstand, and the initial cost of the steel construction. The ideal framing system would provide a large amount of open space, withstand a high live load and have a low initial cost of steel construction. Cost is a major factor when considering the framing system, but open space is the most important factor. Therefore a system that has open space and can be constructed at a low cost is considered a successful system, given that the design live load is at least 70 psf. A live load of 80 psf would allow for the open space to be used for classroom space, offices, hotel rooms, corridors, and other common uses. Refer to Table 40 for design live loads for specified use of the space.

7.1 Beam Grid System Application

The first structural system that was explored is the beam grid system. An explanation of the functionality of the beam grid system can be found in Chapter 2 Background. In brief, the beam grid system allows for a distribution of applied forces through a grid of rigidly connected beams. The grid transfers the applied loads through various load paths to the girders that are placed along the exterior walls of the building. The load is then transferred from the girders into the columns.

$$w_u = 1.2DL + 1.6LL = 1.2\left(0.04321\frac{k}{ft^2}\right) + 1.6\left(0.1\frac{k}{ft^2}\right)$$

$$w_u = 1.2DL + 1.6SL = 1.2\left(.0432\frac{k}{ft^2}\right) + 1.6\left(0.045\frac{k}{ft^2}\right)$$

Since the beam grid system works like a two-way slab, the tributary area of each beam is a triangular shape on each side of the beam. Therefore, each beam supports a smaller area than found in conventional one-way beam-and-slab construction, allowing for a smaller beam size to be used. Figure 43 displays the tributary area of the framing system generated by *Scia*. The beams are shown as black lines and the boundary of the tributary area for each 10-foot beam are shown as red lines.

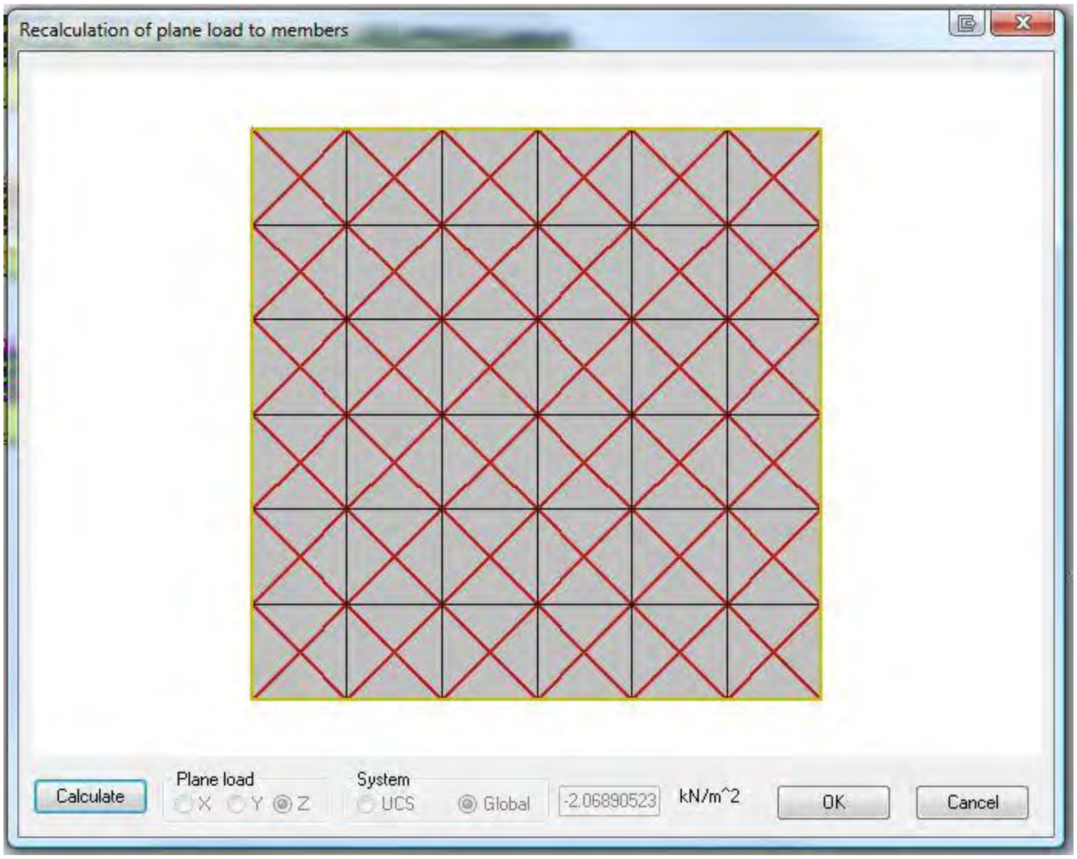


Figure 43 Tributary Area for Beam Grid System

The beam grid system was designed using *Revit Structure*, *Autodesk Robot*, and *Scia*. *Revit Structure* was used to build the model of the structure and apply the loads. *Robot* and *Scia* were used to analyze the structure. In order to obtain minimum deflection in the girders and beam grid, *Scia* was used to test beam sizes and find deflections. The beam grid system used a uniform beam size throughout the grid. Using *Scia*, this beam size was determined to be W36 x 135 spaced every 10 feet. This was determined by changing the beam sizes in *Scia* until a minimal deflection and moment of the beam system were obtained. A 10-foot section of the grid system was cut and evaluated using *Excel* spreadsheets. It was found that a 10-foot beam with the triangular tributary area would be forced composite design. The forced composite beams were found to have the plastic neutral axis located in the flange of the beam and would require 28 shear studs spaced every 4 inches. The beam grid system is an effective system because the beams are connected by welding each beam in the system together, forcing the beams to act as one whole unit. Therefore, at each node 4 beams come together and need to be welded in order to obtain a rigid connection.

A W44 x 335 section was determined to be used as the size for the girders. This member needs to be such a large size because each girder is supporting ¼ of the weight of the bay. The girders were not composite members; therefore shear studs were not used. The deflection of the girders was minimized to account for the deflection of the beams. In order to offset a deflection in the girders, a camber of 3 inches is recommended. Therefore, when the beam grid system deflects within the bay, the whole deflection will be within the limits. Using *Scia*, a total deflection of the grid system was estimated to be 3.375 inches without the consideration of the enhanced stiffness provided by composite construction (due to *Scia* program restraints). With the specified camber for the girders, the entire grid system is expected to deflect less than ½". A model of the deflections can be seen in Figure 44.

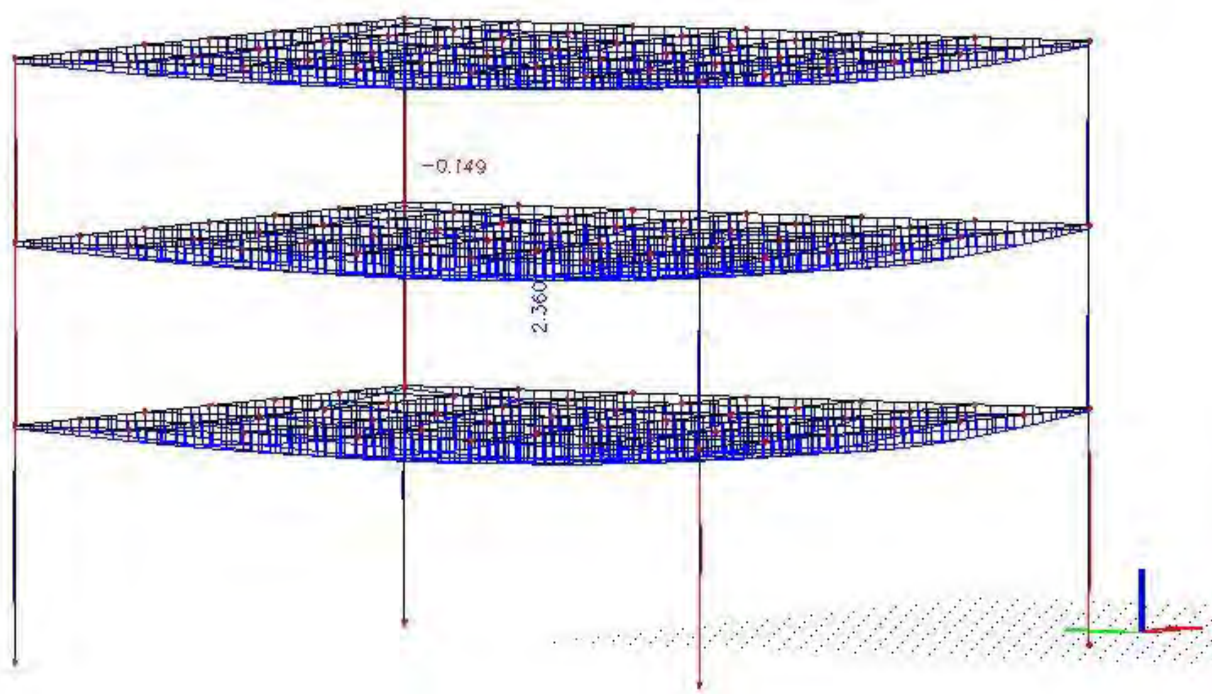


Figure 44: Model of the deflections for Beam Grid System

7.1.1 Column and Footing Design for Beam Grid System

There are four columns in the beam grid design, one column at each corner of the grid. Therefore, each exterior column needs to account for ¼ of the area, 900 square feet. The beam grid system uses many members spaced closely together, therefore the column must support a large weight due to the high density of steel beams. In order to provide sufficient strength while limiting the overall dimensions, each column was designed as an HSS 12 x 12 x 5/8. These are large columns, but only 4 are needed for a 60' x 60' area, with no interior columns. In order to support these columns, a footing size of 9.5' x 9.5' x 3' is recommended. The floor plan is displayed in Figure 45 below.

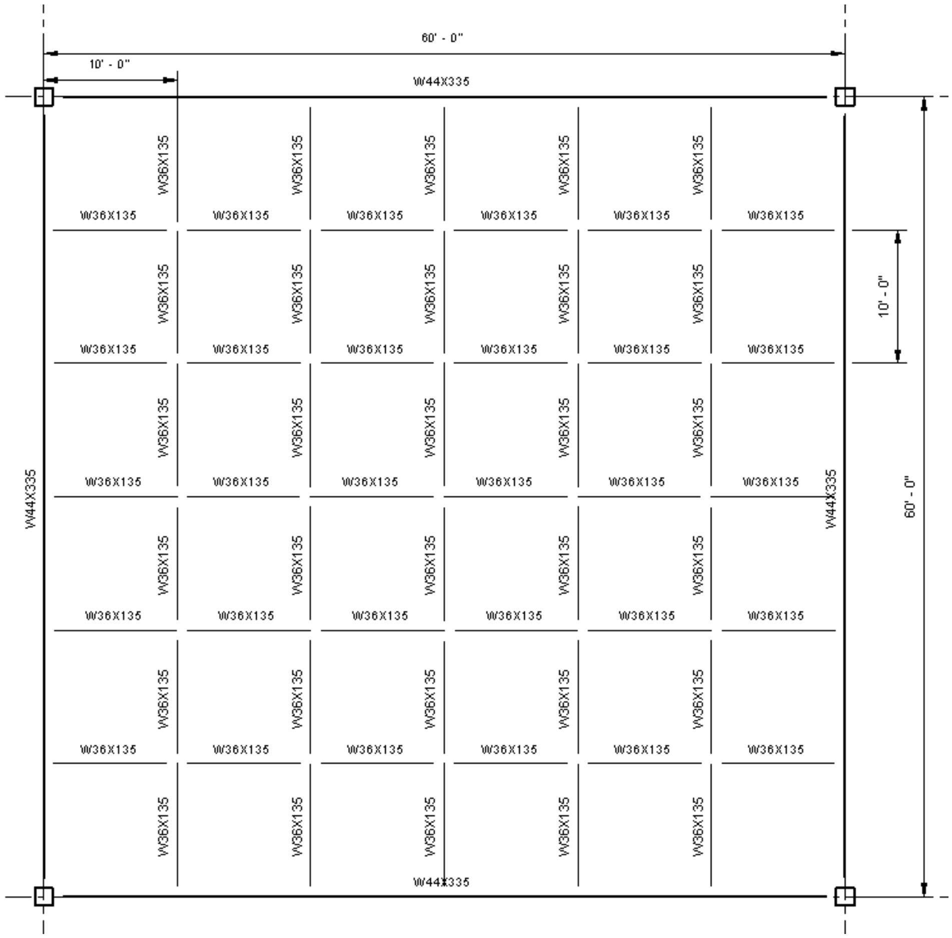


Figure 45: Floor Plan Design for Grid System

7.1.2 Cost Analysis of Beam Grid System

Since the beam grid system uses a large amount of steel members that have a high unit weight (lb. /ft.), the system's cost of steel is very high. The cost analysis of the structure was performed by creating a schedule of the beams, girders and columns in *Revit Structure* and exporting the schedule to *Excel*. Then the cost of each structural member was determined according to data provided in *RS Means 2011* for steel construction including material, labor, and equipment. The steel cost estimate for the 60'x60' section of the structure was found to be approximately $884,000. This cost estimate does not consider the cost of welding that is needed at each connection. Because there is a large amount of welding needed in this system due to the moment resisting continuity at the beam connections, the cost will increase. The system is effective in providing a large amount of open space, but expensive to produce. In order to reduce the cost of on-site welding, a possibility would be to fabricate the grid in larger pieces at the shop beforehand.

7.1.3 Comparison to the Alternate Design with Open Web Joists

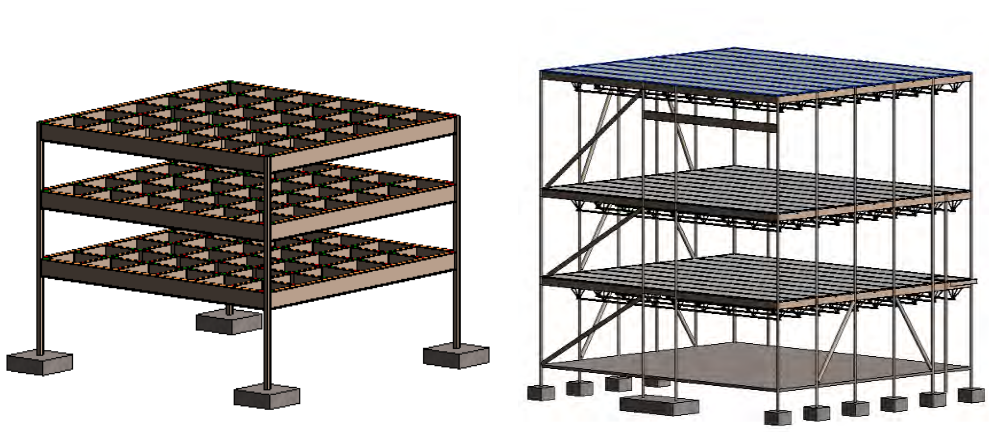


Figure 46: Model of Beam Grid System Design and Joist Girder System Design

The total cost of $883,875.77 for the beam grid systems was compared to the cost of the team's design that utilizes Vulcraft steel joist girders presented in Chapter 4 of the report. Figure 46 displays the *Revit* models for the two systems. Each system provides three floors in the cost estimate and does not include the concrete slabs in the estimate. The 60' x 70' bay size designed in Chapter 4 has an estimated cost of approximately $129,000. This is significantly less than the beam grid system because it does not require as much steel.

Since there is a large amount of steel in the beam grid system, it is not recommended for an area of 60' x 60'. The cost comparison is not favorable, although it is an effective structural system. If this system was applied to a smaller area such as a 30' x 30', smaller members would be adequate and the beam grid system could effectively provide open, reusable space.

7.2 Staggered Truss System Application

The second structural system that was explored is the staggered truss system. Chapter 2 Background provides a description of this structural system. The staggered truss system consists of alternating trusses on the exterior walls of the frame as displayed in Figure 47. This system allows for open interior space of the building since most of the supports are along the exterior walls of the framing.

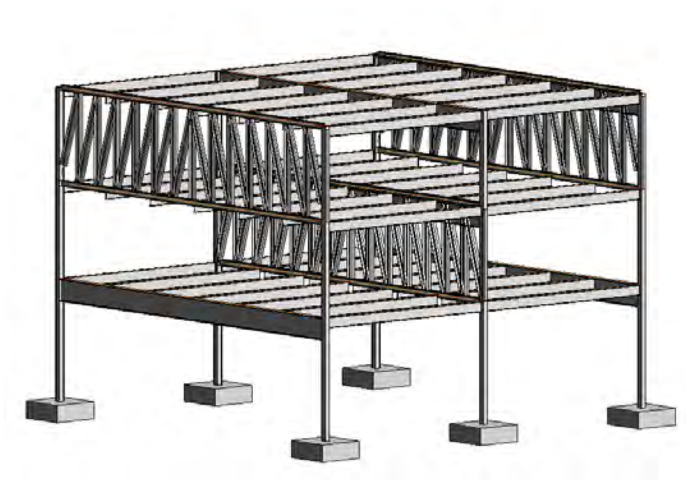


Figure 47: 3D Model of Staggered Truss System

7.2.1 Designing the Trusses and Evaluating the Axial Forces

In order to provide a design of the truss used in the staggered truss system, *RISA* was used to evaluate the axial forces in the members. After experimenting with different designs, a truss design was selected that allowed for a uniform member size throughout the truss. This design caused tension in the bottom chord and compression in the top chord. The range of axial forces in the members was minimal. Therefore, a member size that accounted for the largest axial force was selected and used throughout the truss. A structural analysis of the design was performed in *RISA* with the applied loads acting on the building. Since the spacing between the exterior trusses is 60 feet, each exterior truss has a tributary width of 30 feet. The interior truss was considered to account for a span of 30 feet on each side; therefore, it has a tributary width of 60 feet. The load combinations used were:

Exterior truss: $w_u = 1.2DL + 1.6LL = 1.2\left(1.30\frac{k}{ft}\right) + 1.6\left(3.0\frac{k}{ft}\right) = 1.56\frac{k}{ft} + 4.8\frac{k}{ft}$

Interior truss: $w_u = 1.2DL + 1.6LL = 1.2\left(2.59\frac{k}{ft}\right) + 1.6\left(6.0\frac{k}{ft}\right) = 3.11\frac{k}{ft} + 9.6\frac{k}{ft}$

These trusses span a length of 60 feet and alternate on each floor. Their height spans the height from floor to floor which was taken as 14 feet. The member sizes of the truss were selected in accordance with the maximum axial force found in a member of the truss. All member sizes of the truss were selected to be uniform with the maximum force as the determining factor. *RISA* was used to evaluate the truss design with the applied design loads. Figure 48 show the evaluated truss design from *RISA*.

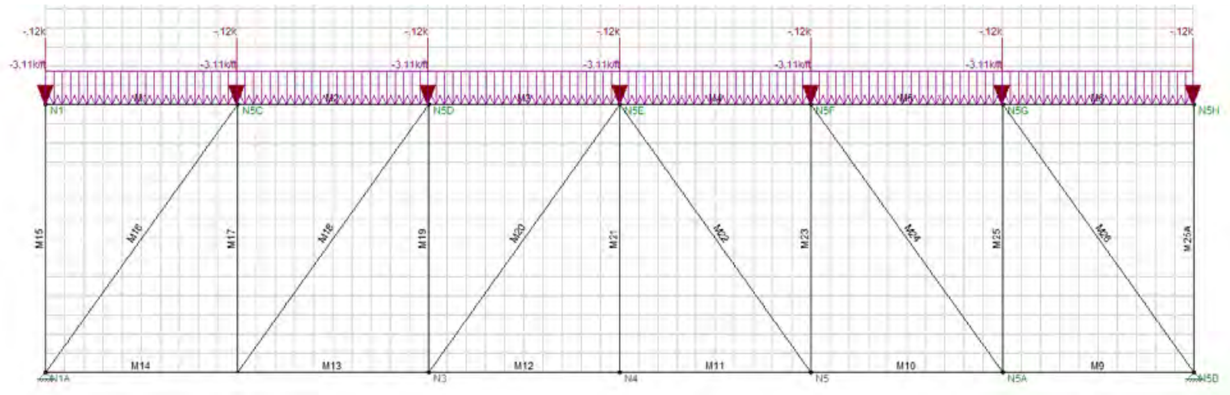


Figure 48: Truss with Applied Dead Load

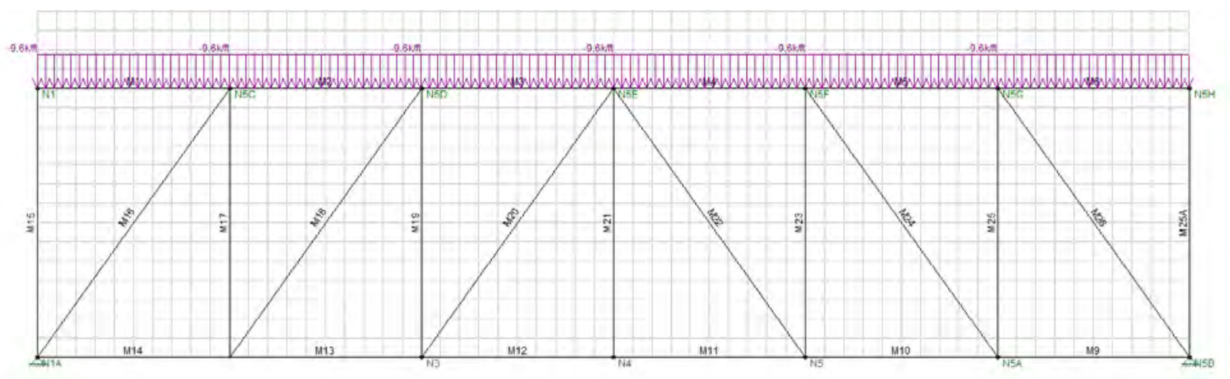


Figure 49: Truss with Applied Live Load

When the member forces were determined in *RISA*, members 26 and 16, as shown in the above figures, were found to have the largest axial force of 195 k for the exterior truss and 389 kips for the interior truss. Therefore one member size that accounted for maximum axial force was used for all members. The member size was chosen based on KL = (1.0) (18 feet). Since the longest members, the diagonal members, have a length of 17.502 ft., a length of 18 ft. was used. A member size was determined using Table 4-1 of the AISC Steel Manual. A size of W10 x 39 was chosen for the exterior truss members because it has a strength in axial compression of 216 kips which is greater than the maximum axial force of 196 kips at a length of 18 feet. Therefore the trusses were designed with uniform W10 x 39 members. The interior truss was composed of W10 x 54 members because they have an axial compression strength of 423 kips at a length of

18 feet. The deflections of the members of the trusses were evaluated in *RISA* and the total deflection of the truss was found to be 0.69 inches for the interior truss and 0.35 inches for the exterior trusses. This was found by finding the displacement of node N4, the middle node on the bottom chord where the maximum deflection occurs. The deflections were found to be minimal and within the allowance of 1 inch. The deflection of the truss is displayed in Figure 50.

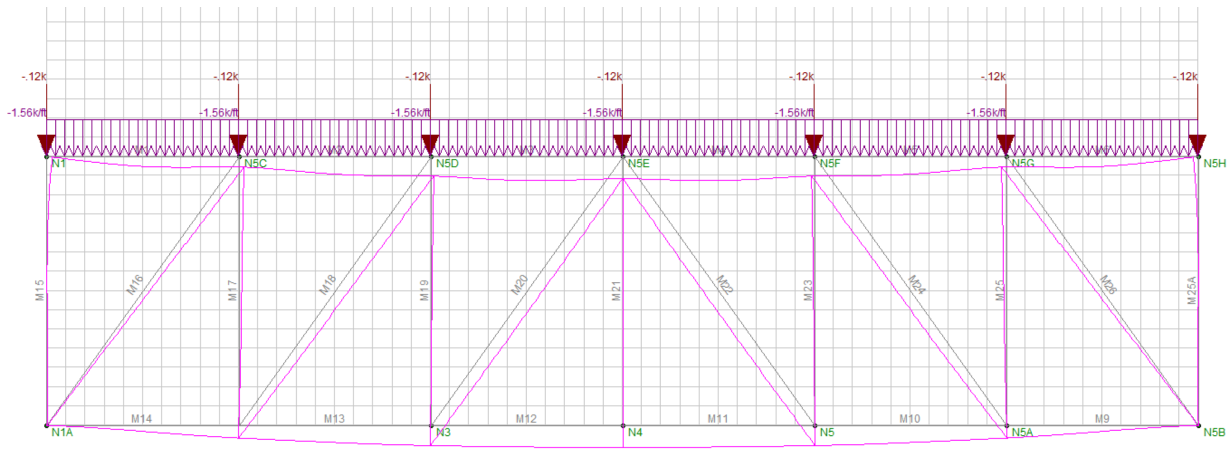


Figure 50: The Exaggerated Deflection of the Truss

Beams were designed to support the flooring for the staggered truss system. The beams are spaced every 10 feet and connect to the truss at the joints. The beam size chosen was a W18 x 40 acting in partial composite with 18 studs. The deflection of the beam was found to be 0.96 inches, which is within the 1 inch allowable limit. The moment capacity of 387 k-ft. was found to be very adequate for the applied moment of 125 k-ft. The deflection of the beam was the determining factor when selecting the beam size. A uniform beam size and spacing was used for all the floors and the roof. The girders on the exterior were designed as W40 x 183 with an allowable deflection of 1.19 inches and a recommended camber of 2 inches. The deflection was the design factor for the girders because the moment capacity is very adequate at 4024 k-ft for an applied moment of 1529 k-ft. The floor plan for the second floor is displayed below in Figure 51.

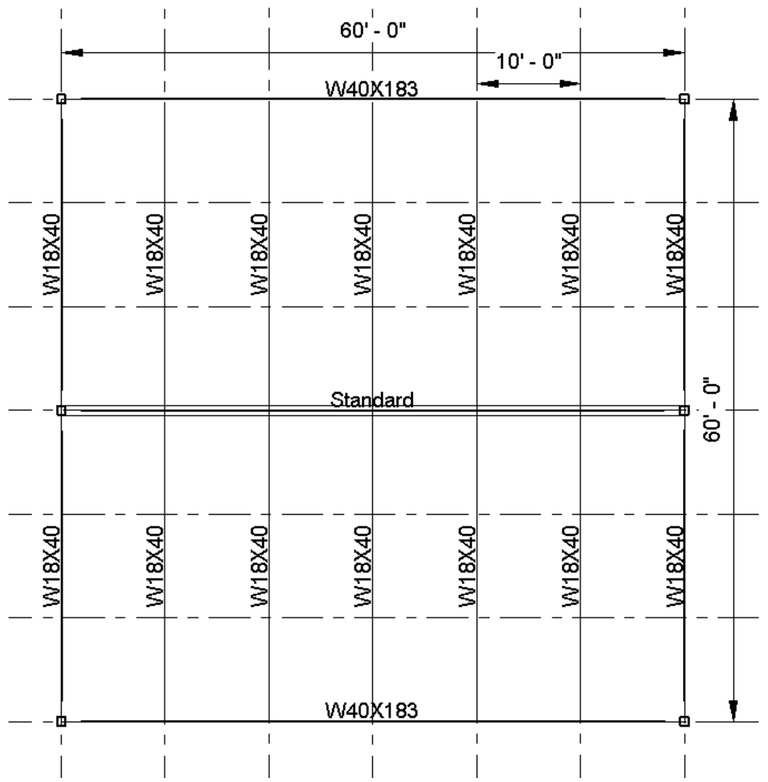


Figure 51: Floor Plan of Second Floor Staggered Truss System

7.2.2 Column and Footing Design for Staggered Truss System

There are six columns in the staggered truss system, which are located around the exterior of the building. The corner columns account for an area of 900 square feet and the middle columns account for 600 square feet. In order to design for one column size, the size of the exterior columns was used throughout the design. The staggered truss system consisted of minimum flooring framing since 30-foot beams are spaced at 10 feet. In order to provide sufficient support while limiting the column dimensions, each column was designed as an HSS 10 x 10 x 5/8 with a footing size of 9.5’ x 9.5’ x 3’.

7.2.3 Cost Analysis of Staggered Truss Design

The total cost of the steel in the staggered truss system design was estimated to be approximately $373,000. This value is significantly less than the estimated $800,000 cost for the beam grid system. Refer to Appendix U for the details of the cost estimate for each framing element. This design uses significantly less materials and requires smaller columns for supports.

But this system is still more expensive than the design using steel joist girders presented in Chapter 4.

7.2.4 Comparison to the Alternate Design with Open Web Joists

Figure 52: Comparison of Staggered Truss System and Steel Joist Girder System

Figure 52 displays the models of the staggered truss system and the steel joist girder system. The staggered truss system is an effective framing system to support a 100 psf live load with a minimal deflection while limiting the expenditures for structural materials. But it does not provide full open space like the design with the steel joists. The steel joist design accounts for a live load of 80 psf, whereas the staggered truss system provides for 100 psf live load. But when comparing cost, the design with the steel joist girders is approximately half the cost of the staggered truss system. The design utilizing steel joists had an estimated cost of approximately $129,000 whereas the staggered truss system has an estimated cost of $373,000. In conclusion, the staggered truss system accounts for a higher live load than the steel joist design system, but it is more expensive and does not provide full open space on each floor.

7.2.5 Conclusion

In summary, these framing systems are adequate, but the cost differs greatly. If the designer is mindful of the cost of construction, then the current design provided in Chapter 4 should be utilized for the framing system. Because the system uses open web steel joists, the cost is significantly less than the other designs. A unit cost in $/sq. ft. of floor area is provided in Table 35 for ease of comparison.

Table 35: Unit Cost Comparison

System	Unit Cost (cost/sq.ft.)
Beam Grid System	$ 81.84
Staggered Truss System	$ 34.51
Current Design	$ 10.25

8.0 Comparison of Building Materials

Many materials are used in the construction of buildings. In public and commercial buildings, such as the Avery Elementary School, concrete or steel usually make up the structural frame of the building. This chapter investigates why steel was chosen as the primary material used in the Avery Elementary School. The team investigated the sustainability advantages and disadvantages of steel and concrete, since sustainability in the construction sector has been a concern. The use of the buildings and all construction-related activities generate more than 40% of all CO2 emissions, use about 40% of the produced energy and consume more than 40% of the material resources used in the society. (U.S. Department of Energy, 2008) The fire and building safety codes are also very important considerations since the building will be primarily used as a school.

8.1 Concrete Construction

The use of concrete structures has advantages and disadvantages. This section will go through major topics that designers are concerned about when deciding which construction material to use for a structure. These topics include sustainability, aging effects, health hazards, and the ability to resist fire.

8.1.1 Sustainability in Concrete

Concrete was initially used as a primary building material because rocks, limestone and clay, the raw materials of cement, are the most bountiful resources in Earth's crust. The least cost way to build has always been to use whatever resources are readily available to use, and construction is no different. However, in developing countries the depletion of these natural resources has been a major problem and is expected to worsen in the near future. (Sakai, 2000)

Another major concern with the use of concrete is the CO_2 emission levels that are given off during the process of cement production. The actual making of cement emits a large amount of CO_2 through the processing of limestone and energy. The transport of cement materials and concrete alone contribute a significant portion of the total CO_2 emitted in this process. "Concrete is usually produced by mixing its components after transporting them to a plant, and is then transported to construction sites. Light oil and electricity are used in these processes, accounting for around 25% of the CO_2 emitted overall in cement production" (Sakai, 2000).

8.1.2 Aging of Concrete

Concrete structures are mostly constructed with reinforced steel. Depending on the structure, it can be with steel dowels and stirrups, or prestressed steel tendons that span through the concrete members. According to a report by Professors Oliva and Cramer at the University of Wisconsin, shrinkage and creep directly affect the length of concrete members over time. As the concrete members shorten, so do their respective tendons, which lead to a loss of prestress forces.

Creep is the increase in strain with time due to a sustained load. When the stressed anchored tendons are initially released, their forces pull the concrete toward the middle of the beam causing a sustained compressive stress and shorten it very quickly. This phenomenon is call elastic strain, which happens immediately. The other strain that is present in a prestressed member is the creep strain, which takes place over a long period of time. The final effects of creep include the deflection of beams and slabs as well as loss of prestress. (Oliva, 2008)

Shrinkage is sorted into two categories, plastic shrinkage and drying shrinkage. Plastic shrinkage occurs during the first few hours after placing fresh concrete in the formwork. The water tends to bleed out and puddle on the surface, and this loss of water results in shrinking of the concrete. This shrinkage is mostly apparent in floor slabs since they have a large surface area. Drying shrinkage is the decrease in the volume of a concrete element when it loses moisture by evaporation. It occurs after the concrete has already attained its final set. Shrinkage in general is not a reversible process, even with the use of additives in the concrete. (Oliva, 2008)

8.1.3 Health Hazards of Concrete

Even though it’s not common, construction workers are at risk for many health hazards when exposed to Portland cement, the most common ingredient in concrete. Cement mortar has also been known to cause health issues. Hazardous materials in wet concrete and mortar include calcium oxide which is corrosive to human tissue, crystalline silica which is abrasive to the skin and can damage lungs, and chromium that can cause allergic reactions. (Sahai, 2001)

Skin contact can cause some major skin irritation. The hazards of wet cement are due to its caustic, abrasive and drying properties. If skin comes in contact with wet cement for a short period of time, the irritation is minimal. However, prolonged exposure could result in alkaline compounds, such as calcium oxide, penetrating and leaving third degree burns, or skin ulcers. The most common time for this to happen is when wet cement gets trapped between skin and boots, gloves, or clothing. (Sahai, 2001)

Allergic reactions may also develop from working with cement for a long time. A "significant percentage of all workers using cement will develop an allergy to chromium, with symptoms ranging from a mild rash to severe skin ulcers." (Sahai, 2001) Hexavalent chromium is a very dangerous chemical in cement since it not only can lead to the development of skin ulcers from an allergic reaction, but it can cause a respiratory allergy, occupation asthma.

Eye contact with cement and inhalation of cement both have the possibility of severely hurting somebody who is exposed to cement dust. Airborne dust can irritate the eyes, causing redness, chemical burns, or in the worst cases, blindness. Inhaling dust caused by sanding, grinding, or cutting concrete can lead to an often fatal lung disease called silicosis. Some studies have connected crystalline silica exposure and lung cancer. Concrete is a material that poses many risks for those that install it on the jobsite. (Sahai, 2001)

8.1.4 Fire Safety and Concrete

Concrete is a great material for fire resistance. It is a good thermal insulator with a thermal conductivity of 1-3 W/m.k., so it delays heat transmission (Jacobs, 2007). According to the European Concrete Platform, "concrete is non-combustible and it has a low rate of temperature rise across a section, which means that in most structures concrete can be used without any additional fire protection" (Jacobs, 2007). Also, concrete would do a lot to confine a fire. In many cases, concrete can safely perform for several hours in a standard fire test when properly designed. Spalling, or chipping or flaking along a concrete surface, is concrete's normal response to high temperatures. This event drastically reduces the strength capacity of concrete, and is the cause for concrete members to fail in a fire. Concrete receives its fire safety ratings by how long a mixture can avoid spalling, which is especially important in school construction. As much time as possible is desired to allow the occupants, usually children, to evacuate the building before spalling takes over and the concrete loses strength. Therefore, design codes include the effect of spalling for schools as well as other occupancy levels. (Jacobs, 2007)

Concrete can successfully demonstrate the ability to protect the building and its occupants since it retains its loadbearing capacity, protects people from harmful smoke and gases, shields people from heat, and facilitates intervention by firefighters. The obvious goal with fire resistance is for the building to remain stable during a fire. Concrete has the ability to do this when certain design criteria and maintenance practices are observed, which makes concrete a very appealing material to use for fire resistance. (Jacobs, 2007)

8.2 Steel Construction

Steel structures have advantages and disadvantages to their use. This section will go through major topics that designers are concerned about when deciding which construction material to use for a structure.

8.2.1 Sustainability in Steel

Steel has high recyclability, durability and other factors that make the use of steel in permanent structures very appealing. As mentioned in Chapter 2, 28% of the steel going into buildings today is recycled steel. Also, when a steel building is taken down, 66% of that steel is recycled (Emerson, 2005). If steel is being reused after a building is dismantled, fewer natural resources will be used to fill the demand of steel. Also, the recyclability results in less waste.

In the Northeast, steel is readily available. The transport is not much longer than it would be for concrete. This is unique to the northeast region, and if you go to the Midwest this could be very different. There is an almost constant demand for steel throughout the whole region for new construction, so the supply and demand is much higher than other construction materials. (Emerson, 2005)

8.2.2 Aging of Steel

Steel is a very durable metal alloy, and unlike concrete, it does not experience creep under normal temperature conditions. This makes for a very long-lasting structure with minor aging effects. If the steel is exposed to the elements, rusting can occur which severely decreases the strength of the material. In the Avery Elementary School, the steel in the exterior framing is protected by a brick veneer around the building envelope. This veneer plus a reliance on flashing and gaskets protect the steel from any water penetration from rain or snow. As long as rusting is avoided, the steel can stay strong for a long period of time. Upon erection there are no immediate strength losses or outside forces needed to strengthen the steel in place. High strength steel can be used when necessary, but this does not affect the structure down the road.

8.2.3 Health Hazards of Steel

Steel construction is a dry and lean process. A steel frame consists of many members that are usually connected on-site by steel bolts. This makes for immediate use, and there is no wait for the frame to settle or dry. Also, when the steel is delivered on site, there is no need to cut it or sand it down; it is typically delivered ready to install. This reduces the dust that construction workers may breathe in with other building materials that need cutting and sanding, which avoids negative health effects to the lungs.

The fumes from welding and cutting are the greatest health hazards for ironworkers if the steel is coated with lead-based paint. Welding is necessary at all connections, and cutting is most common when driving piles into the foundation (IHSA 2005). According to Infrastructure Health and Safety Association (IHSA) (or CSAO), "lead poisoning can occur when you inhale or ingest lead dust and fumes during burning or welding of steel structures coated with lead-containing paints" (IHSA, 2005). Symptoms of lead poisoning vary from nausea and vomiting to convulsions or seizures in the more serious cases. Lead-based paint is mostly seen in older structures such as highway bridges. Since the building under investigation will primarily be an elementary school, there will not be lead pain used in the structure. Therefore, these health effects are not pertinent to this project; however they are still serious health risks for ironworkers working with steel in other settings. (IHSA, 2005)

8.2.4 Fire Safety and Steel

According to the U.S. Department of Commerce Technology National Institute of Stands and Technology (NIST), there are many methods to protect steel structures from fire. When the fire heats up the steel, it eventually becomes very weak and fails. One method is the use of insulation to protect the steel from these elevated temperatures. A traditional insulation method is encasing the steel members in concrete. The concrete will delay heat transmission to the steel elements. However, this is not always the best insulator since concrete can add a lot to the dead weight of the structure. An inexpensive way to protect structural elements is by applying a fireproofing spray. The advantages of this method include easy application especially to detailed features such as connections and bolts, quick installation, and a durable coating material. The fact that the spray can be applied to unpainted steel also makes for a lower cost option. Some disadvantages include a wet and messy installation, a chance of over spraying, poor aesthetics since the spray is exposed, and a tough management of quality control. (Goode, 2004)

There are other insulation methods such as applying a coat of intumescent paints, but these methods are costly. A method that is very applicable to the buildings column is filling the hollow steel members with water or concrete to take the heat from the steel element in the case of a fire. However, concrete and water will add weight to the load the column is supporting, so these methods must be taken into account during the design process. (Goode, 2004)

8.3 Conclusion

Neither concrete nor steel is a perfect building material. They both have their own problems, with steel typically having poor fire safety ratings, and concrete having issues with both sustainability and health and safety. According to a report done by Joakim Widman under SB International (Widman, 2005), several of the problems with sustainable construction can be solved by using the strategies shown in Table 36.

Table 36: Summary of issues of environmental concern and their common relationships to construction, (Widman, 2005)

Issue	Construction Relation	Possible to Improve Through
Embodied Energy	Refining of raw materials to construction products. Recycling	Building system optimization; Recycling; Reuse;

	saves about 70%	Prefabrication
Operational Energy	Includes 85-95% of the life-cycle energy usage in a building; Increased life-cycle thinking	Thermal efficiency; Low-energy equipment; Airflow control; Lean construction
Transports	Truck transports are emission sources; Increased trade means more transports	Light structures; Optimized logistics; Local products
Raw Materials and Water	Construction business uses much material; Virgin materials needed for production	Recycling; Reuse; Material efficient structures
Emissions	CO2 emissions from raw material refining; Affection on environmental effects	Efficient use of energy and materials; Recycling
Recycling and Reuse	Unique recyclability; Societal forces are pushing towards increased reuse and recycling	Life-cycle design; Use of recyclable materials; Standardization
Waste and Land-Use	Construction business is a waste generator; Light and industrial construction favorable.	Recycling; Reuse; Lean construction; Prefabrication
Indoor Environment	Unwanted water in structures; Comfort parameters specified in regulations	Dry materials; Airflow control; Judicious design

The above table, taken directly from Widman's report, offers several strategies for improvement on the selected sustainability issues. Many of the strategies apply only to steel structures (4), a few apply to steel and concrete structures (3), and one applies only to concrete structures. Only steel construction could solve the operational energy, emissions, recycling and reuse, and indoor environment issues. Steel and concrete would both be able to solve parts of embodied energy, raw materials and water, and waste and land-use issues. Transport is the only issue that can only be solved with concrete construction. Most, if not all, of the improvements suggested in the table can be achieved through the use of steel as the main construction material.

Since the building in study is located in the Northeast, structural steel is the obvious choice to make when deciding on which construction material is best for this project. However, steel has many areas for improvement that should be considered when using steel as the main construction material. Future study is recommended to investigate the use of steel and concrete in other regions of the country.

9.0 Geothermal Heating and Cooling Systems

To increase the lifespan of a building and promote more sustainable operations, the building must be properly maintained, which includes maintaining its different building systems. Since a high percentage of school building's energy consumption is due to heating and cooling (U.S. Environmental Protection Agency, 2006) energy efficient HVAC systems must be used. As a result, the project team chose to investigate the use of geothermal energy as a source for HVAC systems. As described in Chapter 2 Background, one of the ways geothermal energy could be obtained is through the use of geothermal heat pumps (GHP), also known as ground source heat pumps (GSHP). In order to choose an appropriate GHP and design its system, a variety of factors such as soil conditions and location will need to be taken into account. Since the initial cost of a GHP system is also much more than traditional HVAC systems, a life-cycle cost analysis should be performed to determine the long-term benefits of such a system. Through this chapter, the project team determined the conditions required for each type GHP system, the advantages and disadvantages of geothermal heat pumps, how to determine a life-cycle cost analysis and the feasibility of this green technology for Avery Elementary School and in the Northeast.

9.1 Types of Geothermal Heat Pumps

There are four different types of GSHP systems, which are either a closed-loop or open-loop system. The closed loop systems uses fluid in the pipes to absorb or relinquish heat from the ground and can either be a vertical, horizontal, or pond/lake system. As for the open-loop system, it uses ground water directly as its heat exchange fluid to heat/cool the heat pumps. Therefore, this system utilizes pipes as an in-take and an outlet system. Each type of system has its own advantages and disadvantages, and the most suitable GSHP system for each building will depend on the spatial, hydrological and geotechnical conditions of the building area.

9.1.1 Horizontal Closed Loop Systems

Horizontal closed-loop systems are most applicable for buildings (particularly residential ones) that have a sufficient amount of land available. For this type of system, required land area ranges from 1500-3000 square feet per ton of heating/cooling and depends on soil properties and earth temperatures (ISWD, 2004). This information is summarized in Table 37 to be able to

Table 37: General Characteristics of the Different GHP Systems

Type of GHP Systems	Typical Pipe Length per ton of heating/cooling	Location of Loop system	Required Surface Area or Water Volume
Horizontal Closed Loop	400-600 feet	Loops installed 4 to 10 feet below ground	1500-3000 sf per system
Vertical Closed Loop	400-600 feet	Loops located in boreholes 200 to 300 feet in the ground	150-300 sf per borehole
Surface Water Closed Loop	Varies	Loops located in water body, at least 8 feet below water surface	Pond size is 10 to 50 tons per acre
Open-Loop	Varies	Column wells are 1000 to 1500 feet deep	Column well spacing is 200 to 600 feet

(McQuay, 2002), (Geo4VA, 2011), (Energy Savers, 2011)

easily compare the general characteristics of each type of system. As shown in the table, each horizontal loop ranges from 400 to 600 feet and it also depends on the earth properties (ISWD, 2004). The loops can run straight and parallel to the ground, or can be in a slinky shape such as shown in Figure 53. The horizontal loops are usually installed in trenches 4 to 10 feet deep (Geo4VA, 2011). However, trenches beyond five feet may require the use of retaining walls for support, which would increase installation costs. As mentioned before, the ground temperature at least 10 feet below the surface is usually at a constant temperature of 50° to 60°F. Therefore, the shallow depths of the horizontal loops place them in a location where the ground temperature would naturally change with seasons. This means that this type of system would not be ideal for cold climate regions such as the Northeast. If it were in warmer climates though, this system is ideal as it is easier to install the either vertical or pond/lake systems. (McQuay, 2002)

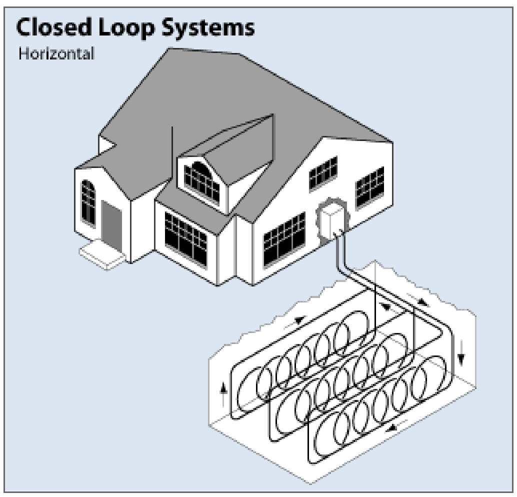


Figure 53: Horizontal Closed Loop System with a Slinky Method for the Looping Pipe from Energy Savers

9.1.2 Vertical Closed-Loop Systems

Vertical loop systems are usually used when there is a limited amount of land available. An example of this system is shown in Figure 54. As shown in Table 37, the installation of this system requires drilling boreholes 100 to 400 feet into the ground, and then the pipes are extended into the boreholes (Energy Savers, 2011). The advantage of this type of system as opposed to the horizontal loop is that it requires less piping and is ideal for when the disruption of the landscape needs to be minimized. With the vertical loop system, since the pipes go deep into the ground, in the Northern climates, operating loop temperature can range from 35°F to 90°F (McQuay, 2002).

For this type of system, the borehole layout is important for an efficient geothermal design. Each borehole requires 150 to 300 square feet of surface area per system ton to avoid heat dissipating between each other or into the ground (ISWD, 2004). By having a sufficient core volume, heat can dissipate from the bore hole without having a longer term effect on the average ground temperature. The most effective borehole layout is one that is located around the perimeter of a land area instead of at the core. This system can also be located under paved areas such as parking lots. (McQuay, 2002)

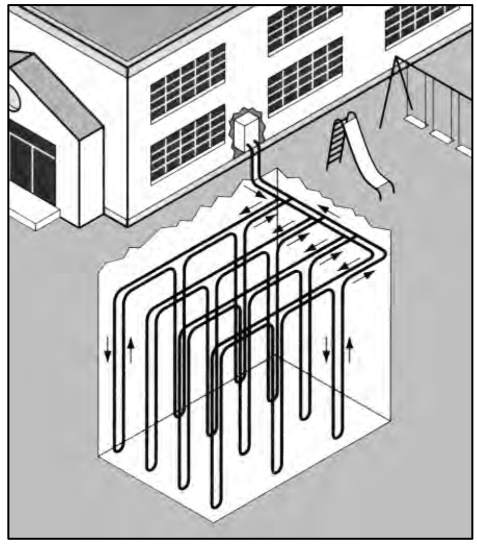


Figure 54: Vertical Loop GSHP design from Geo4VA

9.1.3 Surface Water Closed Loop Systems

Pond/lake systems, also called surface water loop systems, require a pond or lake as the name suggests. With these systems, the body of water is used as a heat sink or for heat storage. An example of this is shown in Figure 55. This type of system requires ponds that are at least 10 to 12 feet deep and could even go up to 25 feet deep to have a constant average temperature in the system (McQuay, 2002). The pond sizes also vary from 10 to 50 tons per acre (McQuay, 2002). The typical operating range for the pond/lake system is 35°F to 87°F, and so the efficiency of this system would depend on the weather. This system is suitable for buildings in hot-climate regions instead of cold-climates where the body of water won't freeze (McQuay, 2002). Furthermore, this type of system may be the lowest cost option if there is an adequate water body near the building (Energy Savers, 2011)

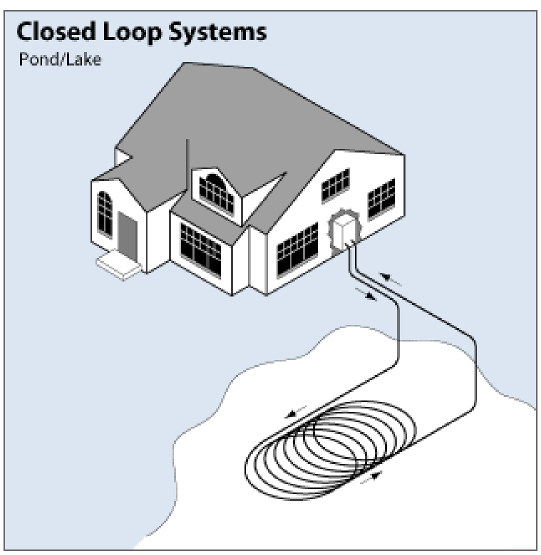


Figure 55: Surface Water Closed Loop System from Energy Savers

9.1.4 Open-Loop System

Open-loop systems directly use ground water to heat/cool the heat pumps in a building. Therefore, this type of system requires sufficient ground water to support the heating/cooling loads of a building. The ground water can be obtained through a single, double or multiple well systems. These systems work as a pump and reinjection system in which the ground water flows into the heat pump, circulates into the building, and then discharges back into a body of water or a drainage field. Figure 56 shows an example of a double-well system. Since ground water is being used in the pump system, it is important to have filtration systems as the water can have elements that may have negative effects on the heat pump system. This system can also use a lake as a water source if it is available and the water temperature is 40°F in order to avoid freezing. According to Smart Energy, open-loop systems are an ideal system for the Northeast (Smart Energy, 2002). However, they are fully dependent on the availability of aquifers in the area.

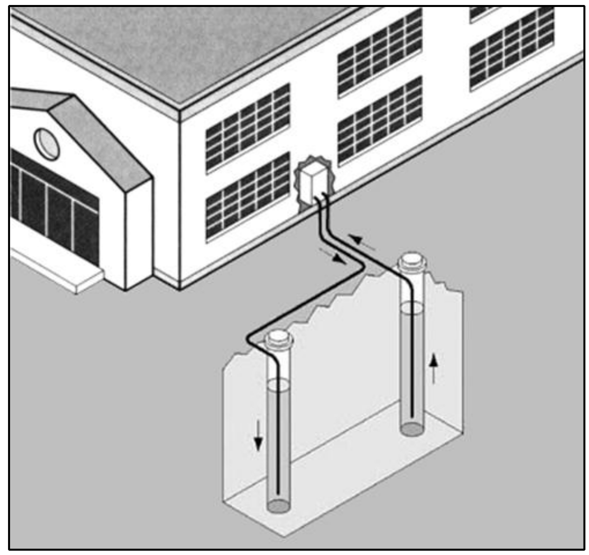


Figure 56: Double-well Open Loop with Groundwater Production and Injection Wells from Geo4VA.

9.1.5 Choosing and Designing a GSHP system

The first step in determining the feasibility of a geothermal system is testing for the thermal conductivity of the ground. Even though geothermal systems can be located in almost any locations, the designer needs to know the ground temperature as well as soil conditions (geological factor) in order to optimize the design of the loop system. Since the purpose of the loop system is to be able to absorb or relinquish heat from or to the ground, the loop system must be located at a level below grade that has a sufficient ground temperature. As mentioned earlier, the spatial factors would also affect the feasibility of the type of system. With open-loop systems its feasibility would then depend on the hydrological factors of the site: the depth of the ground water and its quality must be determined as well as the soil conditions. If this type of system is chosen, the designer has the option of using extraction/diffusion wells or stand column wells.

Local codes must also be considered in the design as drilling will affect surrounding environments.

Once the type of system is chosen, the design of the different subsystems/equipment must be determined. It is important to determine a suitable design because if it isn't designed correctly, the system can actually use or produce more heat than necessary. The most important factor in this decision is to know the specific application of the system. Will it be used for heating, cooling or both? These loads will affect the size and length of the pipes in the loop system. With the sizing and number of units of geothermal heating equipment, this will also depend on the heating and cooling loads of a building. These loads vary with the size of the building and its use. When designing for current buildings, actual studies can be made to determine the loads. However, with new construction, the future loads are harder to determine since actual data cannot be measured. Instead, computer software applications can be used to create models of the different calculated loads.

Other factors that need to be determined in the design of a system are 1) loop flow rates and temperatures 2) piping details 3) pumping design 4) estimating loop fluid volume and 5) estimating pipe pressure, all of which depend on the depth and length of the pipes. These factors need to be considered because it will affect the sizing and location of the loops in the system and will also affect the heating/cooling capability of the GHP. Lastly, the feasibility of the system would also depend on if the availability of contractors to install the systems and their costs. An efficient design of a geothermal system is complex and calls for professional expertise as it requires different earth connection, which was described in Chapter 2 Background. Various studies and tests will need to be made. Nonetheless, these systems, once designed and implemented can be a worthy investment.

9.2 Advantages and Disadvantages of GSHP systems

Geothermal energy is one of the cleanest forms of alternative energy and has several advantages. This type of energy was chosen to be investigated as opposed to other renewable energies because it is not fully weather-dependent and is applicable for most locations. With geothermal heating and cooling systems, the main advantage is the reduction of dependence on fossil fuels to power HVAC systems and actively regulate temperature. GHP systems draw about 80% of its energy from its surroundings with the remaining being attained from electricity (Egg

& Howard, 2011). Thus, it is very energy efficient. GHPs are also applicable for new and retrofit construction. An example of a school replacing its traditional heating systems to a GHP system is described in the next section. With heating and cooling systems, schools have experience in efficient systems that sometimes cool or heat a building, when the climate is already cold/hot. Through a GHP system, heating and cooling loads can be distributed in the building based on need. A GHP system is also advantageous to traditional heating or cooling systems as it encompasses all the components of standard indoor climate control: heating, cooling, humidity control, zoning, air quality, air changes and so on (Egg & Howard, 2011).

Another advantage of this system is that the components of the HVAC system, particularly in living spaces, are easily accessible which increases the convenience factor and helps ensure that the systems are maintained on a timely basis. A smaller mechanical room is required, providing more usable space in a building. The equipment is flexible as well in a way that it can easily be subdivided or expanded to fit building remodeling or additions. The efficiency of a system will depend on how it is maintained as well. Compared to boilers and air-forced heating systems, GHPs are also quiet and reliable.

Geothermal systems have many advantages, but also a few drawbacks. The main one is the installation costs. GHPs usually cost twice as much to install as traditional systems, which causes reluctance for building owners. Also, GHPs use one-third of its heating energy from electricity, which is generated from combustion of fossil fuels (EcoHeat Solutions, 2009). However, looking in the long-term usage, this system will provide significant energy savings, and thus cost savings.

9.3 Life-Cycle Cost Analysis

The installation cost of GHP systems is much more than traditional systems. Therefore, if there is a limited budget for a building design, owners would want to know the length of the pay-back period for the system. Pay-back period is the amount of time it takes for the total cost savings to equal the initial cost for installation and is based on present value. Therefore, time value of money is not taken into account. In general, payback period for GHP is 5-10 years (Energy Savers, 2011). To determine a life-cycle cost analysis, various aspects regarding the building and the HVAC system must be determined. This section briefly describes those aspects and how it can be determined.

The first step in calculating and analyzing the life-cycle cost is determining the actual energy consumption and its costs with a traditional system, as well as its maintenance costs. This will then be used as a benchmark for energy savings with the GHP system. After this benchmark is determined, the actual costs related to the GHP can be calculated. The first aspect to be determined is the installation cost of the system. This will vary on the type of system, and on the contractor that would be used. Installation time will depend on soil conditions, length and depth of pipe, and the construction equipment required. With horizontal systems for example, a typical installation can be completed in one or two days (International Heat Pump Association, 2011).

The next step in the cost analysis is determining the projected energy savings. This requires knowing the peak heat and cooling loads of the building, which was also required when designing the system, and the load capacity of the GHP system. Average heating loads for similar projects could be used as a preliminary estimate. According to Smart energy, an average geothermal system in the Northeast would operate annually about 2000-2600 hours for heating, and 400-500 hours for cooling, resulting in an annual total of 2400-3100 hours (Smart Energy, 2011). However, in order to establish an accurate life-cycle cost analysis, an accurate assessment of the loads for the building design must be defined. After this information is obtained and the different costs are calculated, comparisons between initial and long-term costs can be calculated.

With so many factors being considered in a cost analysis, various software applications have been developed and can be used. One software is *BLCC5* (US Department of Energy, 2011), which is a program developed by the National Institute of Standards and Technology (NIST). This program can perform a life-cycle analysis of building and building components, and is useful for comparing alternate designs that have high initial costs but lower operating costs. Through this program, different GHP systems can be compared to determine which type of system would be most economical. (Kalin, Walker, Macaluso, 2011). A simpler software is *RETScreen 4* (Natural Resources Canada, 2011) which is an *Excel*-based project analysis software by Natural Resources Canada. This software contains the average ground temperature of different cities and simply requires the user to input the different aspects of a building site. It also already has different GHP systems available in its database, which could just be input into the spreadsheet to determine if the system would meet the required loads of the building. Through these software applications, an accurate cost-analysis can be determined.

9.4 Case Study

In New England, there are numerous schools that already use GSHP to obtain geothermal energy and use it as a source of heating and cooling. One example is Hasting School in Westborough MA. In 1997, Hasting School was the first successful 100% geothermal school in New England (Water Energy Distributors, 2000). Hasting School, a 72,000 square foot facility built in 1970, originally had an all-electric heating and no cooling facility. In 1996, the school had a 200-ton GSHP installation which consisted of six standing columns wells (SCWs) for earth coupling. The geothermal heat pump system feeds through the wells; therefore, this is an example of a vertical closed loop system. Water Energy Distributors created a report about the school's use of geothermal energy to familiarize the readers with the ease of installing column wells and geothermal heat pumps, as well as to discuss the benefits the school has experienced from the systems (Water Energy Distributors, 2000).

According to the report, the system chosen for the retrofit was a "water-to-water GeoExchange system with twenty centralized modular ten-ton water-to-water heat pumps [which] fed a new two-pipe building-wide distribution system with a two-pipe distribution" (Water Energy Distributors, 2000). For the six boreholes, they were placed approximately 75 feet from one another and laid out in a linear array. The generous spacing was required to insure little thermal transfer between the wells. If they were spaced less than 50 feet apart, then a thermal analysis would have been needed (Water Energy Distributors, 2000). The wells were located in an overgrown strip of land the school owned, which is why the installation of the wells was feasible. The geothermal heat pumps extracted water from the bottom of the wells and returned it to the top, feeding into the building. The pumps were located in a 20ft x 40ft mechanical room in the building and were able to act as a boiler, producing heated water, or as a chiller producing cold water (Water Energy Distributors, 2000).

The conversion from electric-heat systems to geothermal heat pumps has resulted in a significant decrease in energy consumption, thereby reducing operating costs. Since the installation of the system, the school has reported an average of $75,000 savings per year, which already includes the addition of air conditioning that the school previously lacked (Water Energy Distributors, 2000). From actual electric bills, the school decreased its electric use for heating by 70%. Besides lower operational costs, the geo-exchange heat pump systems have provided lower maintenance costs, maximum design flexibility and easier exhaust air recovery, just to name a few benefits.

9.5 Conclusion

Based on the research conducted in this chapter and examination of the Hasting School case study, the project team concluded that the use of geothermal heating and cooling systems is feasible for Avery Elementary School and in the Northeast in general. Due to land constraints and the cold climate, the vertical closed system or the open loop system seem to be most applicable. Location of the GHP system will also have more flexibility with new construction than retrofit as it can be located under paved areas, decreasing remodeling costs. The actual design for the system will require examining the soil conditions of the land. Currently, the Avery School has a high-efficiency boiler heating system as a means to decrease its energy consumption. However, in the future, GHP systems can be considered when it is time to update the systems.

10.0 Checklist to Measure Ease of Repurposing a Building

The goal of the checklist is to promote sustainability in a similar fashion to the Leadership in Energy and Environmental Design (LEED) rating system. This checklist will address sustainability in terms of the reusability of a building. The findings of the project provide a basis of what the final checklist should entail. The following checklist will assist structural engineers to design a building that can be easily repurposed. As the project team mentioned, there are many factors that contribute to making a building easy to repurpose. The checklist is intended to promote decisions in the design phase that would consider the building's lifecycle. It will encourage the designer to consider more than the immediate use of the building, and can be used to assess the structural design of the building for future use. The final Repurposing for New Construction Checklist can be used as a complimentary evaluation to the LEED Checklist that specifically focuses on the reuse of a building.

Table 38: Mockup of Checklist for Repurposing

Repurposing for New Construction Registered Project Checklist

This checklist provides a reusability rating system for the new construction. It will be used to evaluate the aspects of the construction that affect the potential reuse of the construction. The construction will be rated according to four categories. Each category consists of attributes that will directly affect the reuse of the building. Note: longevity of the construction is considered in order to promote lasting, versatile constructions. Also, it should be noted that this checklist is a template for a building located in the northeast area of the United States. The point system for other locations varies from this system depending on available resources.

Project Name: The Avery Elementary School
Project Location: Dedham, Massachusetts
Project Contractor: Consigli Construction
Project Score: 15 of a possible 18 points

Points

5	4	3	2	1	0	
						Structural Layout of the Building
						Placement of Permanent Structures
					X	**Stairwells:** if located adjacent to an exterior wall, score 1, if located in interior space, score 0.
					X	**Elevators:** if located adjacent to an exterior wall, score 1, if located in interior space, score 0.
						Areas of Open Bays
		X				**Smallest area of open space**: 3600sqft, score 5; 2500sqft, score 4; 1600 sqft, score 3; 900sqft, score 2; under 900sqft, score 0.
						Health and Safety of the Building
						Health Hazards of Construction Materials
			X			Steel framed, score 2; concrete framed, score 0.
						Fire Safety of Construction Materials
				X		Steel framed, score 1; concrete framed, score 2.
						Lifespan of the Building
						Aging of Materials
				X		Steel framed, score 1; concrete framed, score 1.
						Building's Compliance with Code
						Design for Versatile Occupancy Levels
	X					**Average design live load of floor space:** 100-125psf, score 5; 80-99psf, score 4; 60-79psf, score 3; 50-59psf, score 2; under 50psf, score 0.
					X	**Average design live load of roof:** If the design live load of the roof is equal to or greater than the live load design for the floor(s); score 1; if not, score 0.

Levels of Reusability		
Platinum	100%	highest possible score
Gold	85%	and above
Silver	75%	and above
Bronze	60%	and above

The checklist offers a total of 18 points. Using 18 points as a perfect score, each level was devised based on possible accumulation of points. Various levels of performance were established based on the LEED levels of performance. The performance levels are defined as the percentage of points received out of the total amount. The project will receive Platinum Level if it scores 100%, Gold Level if it scores 85% and above, Silver Level if it scores a 75% and above, and a Bronze if it scores 60% and above. The following sections explain the rationale behind the allocation of points as well as the project team's reasons for including the different factors in the checklist.

10.1 Layout of the Building

The layout of the building should be designed to give the building owner as many options as possible to change the use of the floor space. This can be done through a strategic placement of permanent structures as well as large open areas.

10.1.1 Placement of Permanent Structures

For full points, permanent structures, such as elevator shafts and stairwells, should be designed in an area that will not impede the open floor plan. In the design of the New Avery Elementary School, the stairwell and elevator shaft are in the middle of the central bay, which makes it difficult and costly for the central bay to become an open floor space. If these elements of vertical travel are moved toward the exterior walls, the open space of the floor plan will not be as affected. Locating the permanent structures on an exterior wall will earn the project a 1 on the checklist, while an interior location will receive 0 points.

10.1.2 Areas of Open Bays

Ideally, the area of open bays should be the size of the bay from column to column. However, that is not always possible. Depending on the structure of the bay and the size of members the designer is willing to use, interior columns would need to be added toward the middle of the floor plan. This scenario should be kept to a minimum to promote flexibility, and fewer columns gain increased points on the checklist. The point distribution for different areas of open space are: 5 points for an area of 3600 ft^2, 4 points for an area of 2500 ft^2, 3 points for an area of 1600 ft^2, 2 points for an area of 900 ft^2, and 0 points for an area less than 900 ft^2.

10.2 Health & Safety of the Building

The provisions of the *Massachusetts State Building Code* are written primarily to protect the health and safety of the occupants of a building. However, this checklist also promotes decisions that contribute to the health and safety of the individuals who will be responsible for the erection of the building.

10.2.1 Health Hazards of Construction Materials

Since there are very few health hazards that come from steel construction, the use of steel earns the project 2 points. The biggest concern is the inhalation of the fumes from welding and cutting steel. On the other hand, concrete is a very dangerous material to work with, which is why a concrete project will receive a score of 0. These hazards range from minor skin irritation to a very serious chemical burns.

10.2.2 Fire Safety of Construction Materials

When it comes to fire safety ratings, steel again received an average rating. Steel itself is a metal which is very negatively impacted by high temperatures, but with proper fire resistance strategies it can be protected. Building codes will require fire resistance protection for steel for certain building heights, areas, and occupancies, so it's already part of the building process, which is why the use of steel will receive a score of 1 for this section. When it comes to fire safety ratings, concrete received 2 points. Concrete is a naturally fire resistant material that does not require additional fire protection.

10.3 Lifespan of the Building

The lifespan of the building is a crucial section of the check list. A building that lasts longer will have a greater chance of having the opportunity to be repurposed in the future.

When it comes to the aging of steel structures, steel scored an average 1 point. While the durability of steel is a very positive characteristic, corrosion can still occur over time. Steel is a material that can last a very long time; however other materials have been known to last longer than steel, which is why it received an average rating.

Concrete received a 1 for the aging category. Depending on the type of concrete construction, some structures will have more severe aging effects than others. Concrete spanning large lengths are typically reinforced with prestressed tendons. These tendons suffer immediate and long-term losses, such as creep and shrinkage.

10.4 Design for Versatile Occupancy Levels

According to the *Massachusetts States Building Code*, the building will have to meet certain criteria in order for the structure to be an operating facility, no matter what the use. A section of the checklist analyzed the uniformly distributed live load designed for the building. The *International Building Code* adopts the minimum uniformly distributed live loads from *ASCE-7*, Chapter 4. Table 39 narrows this list of live loads into occupancies that would be most capable of converting from one use to another.

Table 39: Minimum Uniformly Distributed Live Loads

Occupancy or Use	Pounds Per Square Foot
Hospitals	
Operating rooms, laboratories	60
Patient Rooms	40
Corridors above first floor	80
Office Buildings	
Lobbies and first-floor corridors	100
Offices	50
Corridors above first floor	80
Hotels	
Private rooms and corridors serving them	40
Public rooms and corridors serving them	100
Manufacturing	
Light	125
Schools	
Classrooms	40
Corridors above first floor	80
First-floor corridors	100
Stores	
Retail	
First floor	100
Upper floors	75
Wholesale, all floors	125

Buildings that are designed with a substantial live load, such as a light manufacturing building, can be converted into any type of use with minimal alterations to the structural system of the building. Due to the large bay systems and open floor space of a factory, these buildings have beneficial structural features that promote adaptable reuse. On the other hand, it would be impractical to reuse a multistory school, designed with a live load of 40 psf, as an office that requires 50 psf live load because it would be expensive and difficult to increase the structural capacity of the existing structure. Quickly converting into a new use will decrease the time of transfers between owners and occupants. With the consideration of reuse in mind during early stages of design, an adaptable structural design will be able to accommodate future demands.

To determine how many points would be awarded to the design, the live loads were evaluated according to how many different occupancies they can serve. The use of design live loads in the range from 100 psf to 125 psf are adequate for almost any use, which will earn 5 points, while design live loads from 40 psf to 49 psf are only adequate for a small range of uses, so this case receives a score of 0. The rest of the values are evaluated in a similar manner and are displayed in Table 40.

Table 40: Live Load Earned Points

Live Load (psf)	Earned Points
100-125	5
80-99	4
60-79	3
50-59	2
Below 50	0

Some buildings are designed with various applied live loads, such as the team's design model for the new Avery school. The redesign applied a live load of 80 psf. in two thirds of the building, while one third of the building was designed with 100 psf. To account for this variation, the live load was multiplied by the ratio of the building area to which it was applied. Therefore, if a live load of 100 psf was applied to 1/3 of the building and 80 psf was applied to 2/3 of a building, then 100 psf would be multiplied by 0.33 and 80 psf would be multiplied by 0.67. The products of these values are then added together, which in this case amounts to a final value of 86 psf. Consequently, the team's redesign would receive 4 points for the applied live loads. Additionally, if the roof live load exceeds 45psf, then the structural design would receive an additional point. This additional point will promote the addition of another floor in the future or the addition of a sustainable element such as a garden on the roof.

11.0 Conclusion of Report and Recommendations

In summary, the team met the goals of the Major Qualifying Project by evaluating a current design of a steel-framed building, providing an alternate design that anticipates the reuse of the building, researching further aspects of the building that promote reuse, and drawing conclusions and recommendations based on the findings of the project. By evaluating the current design of the Avery Elementary School located in Dedham, Massachusetts, the team was able to draw conclusions about the structural engineer's strategy for designing each member of the building. The alternate design provided a real-world scenario to research and utilized various framing tools that would provide a structural design with large areas of open space. Further research on the building allowed the team to develop an understanding of demolition costs, the possible structural framing materials of a building, several structural framing designs using steel, and the sustainability of using a geothermal heating system. The research and analysis performed allowed the team to develop sufficient understanding to design a mock-up of a checklist that encourages structural engineers to consider the reusability of the structure.

The project encourages future studies in order to develop a deeper understanding of designing for reuse. First, further study can be done on other possible framing systems that allow for open space. This MQP focused on typical framing of beams and girders with the use of open web joist girders. It also explored the beam grid system and staggered truss system as options for framing. However, more structural systems can be researched and applied. The comparison of steel and concrete allows for further studies to investigate alternative building materials in respect to reusability. Further study can be performed on the use of geothermal energy to improve the longevity of the building. Since the design of a geothermal heat pump system was beyond the scope of this project, future work can consist of determining a suitable GHP design and calculating its life-cycle cost analysis.

The checklist presented in the MQP can provide a framework for a system that encourages new building designs to consider the adaptability of the structure. The checklist is a basic guideline and rating system for the reusability of buildings. There is potential to build off of this guideline in order to produce an accurate, elaborate rating system for new construction. The use of this checklist will promote structural engineers and contractors to consider the entire lifespan of the structure and aim to improve the disposability of future buildings.

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13.0 Appendices

Table of Contents

Appendix A: Proposal

Introduction

Abandoned public buildings create problems for communities. The presence of vacant buildings can create costly problems that are a drain on the town's budget and impede the development of neighborhoods (Palmer, 2008). According to a survey conducted in 2004, most U.S. public school buildings are abandoned after 60 years of use because they can no longer meet the needs of the occupants (O'Connor, 2004). This was usually due to the change in student population since the school was built - a result of "the baby-boom echo, immigration, and migration" (Lewis, 2000). In 2005, a study found that 22% of the public schools were within 5% of their capacity and another 10% exceeded the capacity of the building. Furthermore, it was discovered that in order for these towns to alleviate overcrowding, 78% of the schools have used portable classrooms, 53% have turned non-classroom space into classrooms, and 35% have built new permanent buildings or additions (Chaney, 2007). Therefore, the school buildings require costly funds in order to adapt to the changing population and prevent an accumulation of abandoned buildings.

Many towns own school buildings that are in need of renovations or will face abandonment. A study conducted by the U.S. Department of Education in 1999 found that 1 in 4 schools possessed an onsite building that was "in less than adequate condition" (Lewis, 2000). 4-6% of schools reported that they had buildings that were in poor condition and 1-2% reported that their buildings needed to be replaced because of non-operational conditions or substantial substandard performance (Lewis, 2000).

Due to limited funds and short lifespan of buildings, local governments have few options to provide their community with adequate public school facilities. Therefore, there is a critical need to sustainably extend the life cycle of a building to efficiently serve the public.

Problem Statement

Towns frequently need to provide funding for costly building renovations and modifications to their current public school buildings. Sometimes the town cannot fund the renovations needed to maintain their current school building and resort to funding a new construction as an alternate solution. However, the previous school building is then left abandoned with few options of reuse due to its limited functionality. This is a recurring cycle for most towns as they constantly face the need to abandon and tear down a school building and incur the cost of a new construction.

Objective

The purpose of this project is to design a public school building that can be adapted to the community's needs without costly renovations. This project will use the design for the Avery Elementary School in Dedham, Massachusetts as a case study. The Avery School is a new construction that is to replace the previous outdated Avery School building. Following the building layout of the new school, provided by Dore and Whittier Architects, Inc., this project will consist of a structural design that can be easily repurposed in the future. This design focuses on permitting a more versatile architectural layout. Furthermore, the proposed design will integrate sustainable features, such as an HVAC system that increases the building's energy efficiency. As a result, it will increase the building's lifespan and operate with low maintenance costs. A cost estimation of the construction and design will be prepared and compared to the current cost of the school. After evaluating this example, the team will determine a list of recommendations for the structural and sustainable design of school buildings that can be followed to achieve the construction of a building that is most advantageous to the community.

Scope of Work

This project will cover topics in the areas of structural design, sustainability and cost analysis. The team will work as independent investigators to develop a design that is suitable for repurposing. The preliminary floor plan for the new Avery Elementary School will be treated as a base for the structural design and analysis of the existing building. A 3-dimensional model will be created to enhance the visual understanding of the building layout. The team will primarily focus on a structural steel system, with a secondary focus on implementing different sustainable design strategies for increasing energy efficiency. The deviations will include determining the changes necessary to make the building as versatile as possible by limiting the amount of structural walls in the middle of the floor plan. This will be done using exterior columns and various truss systems, as well as determining ways to increase the building's energy efficiency. After the necessary changes are made, the team will reevaluate the design loads acting on the structure. A cost analysis will be performed to determine payback periods for alternative designs chosen as well as comparing the life cycle costs of the building. Since the building is for a specified initial use, the team will investigate any changes that will need to be made during the repurposing process. Since the building is owned by the Town of Dedham, multiple future uses will be investigated. Based on the cost and performance comparisons of structural design, recommendations will be made to the Town of Dedham and Consigli Construction Company, Inc. in the form of a checklist that can be utilized for future public buildings.

Methodology

Table 1: Methodology Chart

Task	Plan of Action	Resources Used
Structural Analysis of Existing Design		
REVIT model of structural design	Create 3D model based on Dore and Whittier's structural drawings	REVIT
RISA structural analysis of loads acting on frame	Input structural frame and design loads	RISA, Dore & Whittier Structural Drawings
Roof Analysis	Analyze the trusses used	Dore & Whittier Structural Drawings, Excel
Floor Analysis	Analyze the beams, girders, and columns	Dore & Whittier Structural Drawings, Excel
Typical Footing Analysis	Analyze the typical footing	Dore & Whittier Structural Drawings, Excel
Base Plate and Connection Analysis	Analyze the beam,girder, and column connections	Dore & Whittier Structural Drawings, Excel
Structural Analysis Review	Group confirms calculations	
Structural Design for Repurposing		
Determine Occupancy Level	Choose a level that will allow more reuse options in the future	Massachusetts Building Codes/ International Building Codes
Determine Column Layout	Find the longest span while keeping a reasonable and cost effective beam size	Microsoft Excel
Determine whether truss girders or open web joists will be used	Calculate which will require the least amount of steel while allowing an flexible floorplan	Microsoft Excel, RISA
Roof Design	Design roofing system	Microsoft Excel
Floor Designs	Design flooring system by analyzing different layouts	Microsoft Excel

Typical Footing Design	Design typical footing to withstand new column layout	Microsoft Excel
Base Plate and Connection Design	Design steel connections	Microsoft Excel
Structural Design Review REVIT model of structural design	Group confirms calculations	
Cost Estimation of Structural Design		
Determine average unit costs used by Consigli	Determine unit prices through requisition provided by Consigli	Consigli's Requisition
Determine overall cost of proposed structure	Use unit prices used by Consigli for material, labor and construction cost. Or use unit prices found in RS Means	*RS Means Square Foot Costs*
Compare and Contrast Estimation to Consigli's estimation	Determine areas of significant differences, and then determine why	
Sustainable Strategies for increasing energy-efficiency		
Determine applicable HVAC systems that can be applied to design	Research geothermal systems, building envelope, dual-paned systems etc.	
Examine case studies with determined systems/design	Determine the benefits of the design	
Determine pay-back period of implementing systems	Determine average cost of maintenance with and w/o sustainable strategy	Microsoft Excel
Develop a Rating System for Building Re-Purposing		
Create a template checklist for a newly constructed building	use LEED checklists as examples to make a checklist for re-purposing	Microsoft Excel
Construct a point system that rewards a certain number of points for each aspect	use the cost estimation comparison of the Avery School to develop point worth	
Develop levels of rating for the building according to the possibility of re-purposing	figure the amount of points that make a building gold, silver, or bronze	

Deliverables

By following the proposed methodology, the group will create several deliverables for this project, which consist of the following:

- 3D Model of current Avery School Design
- Structural Alternatives for increasing open space
- Cost estimates of these alternatives
- 3D Model of Structural Redesign
- Checklist for new constructions to measure the ease of repurposing a building

Conclusion

Designing for repurposing is a promising solution for extending the life cycle of buildings. This process will save long term cost by avoiding new construction, demolition, and complicated renovation. Areas, whether urban or rural, will be able to make smooth transitions with their stock of public buildings to keep up with a changing environment. The design of Avery Elementary School serves as a foundation for this project. Evaluating structural design elements and cost analysis will address the opportunity of incorporating this repurposing strategy to the sustainable development of buildings. By assessing the possibility of structurally re-designing an elementary school, this project addresses a common problem with a practical, sustainable solution.

Schedule

Table 2: Schedule

	A Term							B Term							C Term							Author
Week	1	2	3	4	5	6	7	1	2	3	4	5	6	7	1	2	3	4	5	6	7	
Define Scope of Work																						ALL
Proposal																						ALL
Background Research																						ALL
Submittal #1: Final Proposal																						
Structural Analysis of Existing Design																						**JS**
REVIT model																						EF
RISA Analysis																						OR
Roof and Floor Analysis																						JS
Footing, Base Plate and Connections Analysis																						AC
Group Review																						ALL
Submittal #2: Structural																						
Structural Design for Repurposing																						**AC**
Determine Column Layout and Truss System																						OR
Roof and Floor Design																						JS
REVIT model																						EJ
Footing, Base Plate and Connections Design																						AC
Group Review																						ALL
Submittal #3: Alternate Structural Design																						
CAD Drawings																						**EF**
Existing Design																						EF
Design for Repurposing																						OR
Submittal #4: CAD Drawings																						
Sustainable Strategies for Energy Efficiency																						**EF**
Determine applicable HVAC systems to be used																						EF
Examine case studies																						JS
Determine costs/pay-back period																						OR
Submittal #5: Sustainable Strategies Section																						
Cost Estimation of Structural Design																						**OR**
Determine average unit costs used by Consigli																						EF
Determine overall cost of proposed structure																						OR
Compare and Contrast Costs																						AC
Submittal #5: Cost Estimation Summary																						
Develop a Rating System for Building																						**OR**
Create a template checklist																						OR
Construct point system																						JS
Develop levels of rating																						AC
Submittal #6: Rating System Checklist																						
Write Report																						**ALL**
Submittal #7: Final Report																						

Appendix B: Beam Analysis Sample Spreadsheet

Floor	Item	Size	Shear Studs	Length (ft)	ØbMpx	Ix	Zx	A	d	bf/2tf	h/tw
Second	Beam	W16 x 31	16	28.67	203	375	54	9.13	15.9	6.28	51.6

CHECK:					
Deflection	Δ	0.36	<	1	inches
Capacity	ØbMp	86.96	<	760	k*ft

Load Combination: U = 1.2D + 1.6L + 0.5(Lr/S)		
Spacing:	4.5	ft
Length:	28.67	ft
Beam wt	31	lbs/ft

Mu	83.14	k*ft		
Mu w/beam	86.96	k*ft		
DL	0.19	k/ft		
DL w/beam	0.23	k/ft		
LL	0.36	k/ft		
SL	-	k/ft		
1.2DL	0.23	k/ft		
1.2DL w/beam	0.27	k/ft		
1.6LL	0.58	k/ft		
0.5SL	-	k/ft		
wu factored	0.81	k*ft		
wu w/beam	0.85	k*ft		
wu unfactored	0.55	k*ft		
wu w/beam un	0.59	k*ft		
E	29000	ksi		
be	54	in	86	in
			54	in

Interpolated Capacity	
ØbMn	**760**
Y2 low	2.5
Y2 high	3
ØbMn low	740
ØbMn high	769

Studs		**3/4" size**	
#of studs according to drawings:			16
requires:	2ΣQn	**275.20**	k
Table 3.21	Qn	17.2	k
requires:	ΣQn	137.6	k
Table 3.19	ΣQn	164	
Table 3.19	PNA	6	
Table 3.19	Y1	1.99	
given	Ycon (t)	4.5	in
calculated	a required	**3.32**	in
calculated	Y2	**2.84**	in

Stud Spacing			
The drawings specify for the studs to be "spaced evenly along beam"			
L =	28.67		
# of studs	16		
spacing:	1.79	ft	
I3.2d(6)	3/4" size	0.75	
2.5*thick of f	1.1	> dia of stud	**OK**
thick of flang	0.440	Table 1-1	
max spacing	20	>	21.50
final spacing	**16.00**	**inches**	

Deflection Due to Wet Concrete + Beam Weight		
Weight	166.94	plf
Moment	17.15	k*ft
C1	161	
Δ	0.23	< 1 in

Deflection Due to Live Load		
1/360 span	0.96	
C1	161	
M_L	36.98	ft*k
Ix	**858**	in^4
Ix low	826	in^4
Ix high	872	in^4
ΔL	0.22	< 0.96 in

Shear Check		
Vu	11.60	k
ØVn	55.9	k
	11.60	< 55.9 k

Deflection Check		
Deflection	**0.36**	**< 1 in**

Appendix C: Girder Analysis Sample Spreadsheet

Floor	Item	Size	Shear Studs	Length (ft)	ØbMp	Ix	Zx	A	d	bf/2tf	h/tw
Second	Girder	W30 x 90	28	40	1060	3610	283	26.4	29.5	8.52	57.5
Interpolated Values:					1439	7416					

CHECK:			
Deflection	Δ	0.92	< 1 in
Capacity	ØbMp	973.73	< 1439 k*ft

Interpolated Capacity	
ØbMn	**1439**
Y2 low	3.5
Y2 high	4
ØbMn low	1430
ØbMn high	1440

Studs		3/4" size	
#of studs according to drawings:			28
# of studs required for full composite:			
requires:	2ΣQn	**588.00**	k
Table 3.21	Qn	21	k
requires:	ΣQn	294	k
Table 3.19	ΣQn	329	k
Table 3.19	PNA	BFL	
Table 3.19	Y1	0.61	
given	Ycon (t)	4.5	in
calculated	a required	**1.08**	in
calculated	Y2	**3.96**	in
calculated	# of studs	28	

Load Combination: U = 1.2D + 1.6L + 0.5(Lr/S)		
Spacing:	22.88	ft
Length:	40.00	ft
Girder wt	90	lbs/ft

Mu	952.13	k*ft	
Mu w/girder	973.73	k*ft	
DL	1.53	k/ft	
DL w/girder	1.62	k/ft	
LL	1.83	k/ft	
SL	-	k/ft	
1.2DL	1.83	k/ft	
1.2DL w/girder	1.94	k/ft	
1.6LL	2.93	k/ft	
0.5SL	-	k/ft	
wu factored	4.76	k*ft	
wu w/girder	4.87	k*ft	
wu unfactored	3.36	k*ft	
wu w/girder un	3.45	k*ft	
E	29000	ksi	
be	120	in	120 in
			274.5 in

Deflection Due to Wet Concrete + Girder Weight		
Weight	781.02	lbs/ft
Moment	156.20	ft*k
C1	161	
Δ	0.21	< 1 in

Deflection Due to Live Load		
1/360 span	1.33	
C1	161	
M_L	366.00	ft*k
Ix	**7416**	
Ix low	7400	
Ix high	7610	
ΔL	0.49	< 1.33 in

Dead Load of Beams		
Size	Weight	Amount
W16x31	31	4
	L =	28.67
W14x22	22	4
	L =	17.08
Total Weight	106	lb/ft
Beam Spacing	4.5	ft
Weight	23.5556	psf
wr	6	in
hr	3	in
wr/hr	2	in
Table 3.21		

Check Shear		
Vu	95.21	k
ØVn	212	k
	95.21	< 212 k

Check Deflection		
Deflection	**0.92**	**< 1 in**

Stud Spacing			
The drawings specify for the studs to be "spaced evenly along beam"			
L =	40.00		
# of studs	28		
spacing:	1.43	ft	
I3.2d(6)	3/4" size		
2.5*thick of f	1.525	> dia of stud	**OK**
thick of flange	0.610	Table 1-1	
max spacing	20 in	>	17.14
final spacing:	**16.00**	**inches**	
rib deck	2" so even number of inches		

Appendix D: Column Analysis Sample Spreadsheet & Hand Calculations

Column Size:	HSS 8X8X5/16		
		UNITS	Resource
KL	14	ft	
Loads			
P_d	16.52	kips	
P_L	28.30	kips	
P_u	174	kips	
Section Properties			
A	8.8	in^2	Table 1-1
r_x	3.130	in	
r_y	3.130	in	
Capacity Interpolation			
KL/r_y actual	53.67	-	
KL/r lower	53.000	-	Table 4-22
$\Phi_c F_{cr}$ lower	34.300	ksi	
KL/r upper	54.000	-	
$\Phi_c F_{cr}$ upper	34.000	ksi	
$\Phi_c F_{cr}$ actual	34.099	ksi	
$\Phi_c P_n$	268.837	kips	

Fy	**50.00**	**ksi**
f'c	**3.00**	**ksi**
Dead Loads Floor		
metal decking	1061.10	lb
concrete deadwt	10684.69	lb
insulation	707.40	lb
ceiling	1061.10	lb
MEP systems	1768.50	lb
FLOOR BEAMS + Girders	1240.000	lb
Critical Area	353.7	ft^2
TOTAL DEAD LOAD	**16522.79**	**lb**
TOTAL LIVE LOAD	**80.00**	**psf**
spacing	5.00	ft
concrete deadwt	145.00	pcf
thickness of metal deck	2.00	in
thickness of metal deck	0.17	ft
thickness of concrete	2.50	in
thickness of concrete	0.21	ft
10% of Concrete Load	1.10	plf

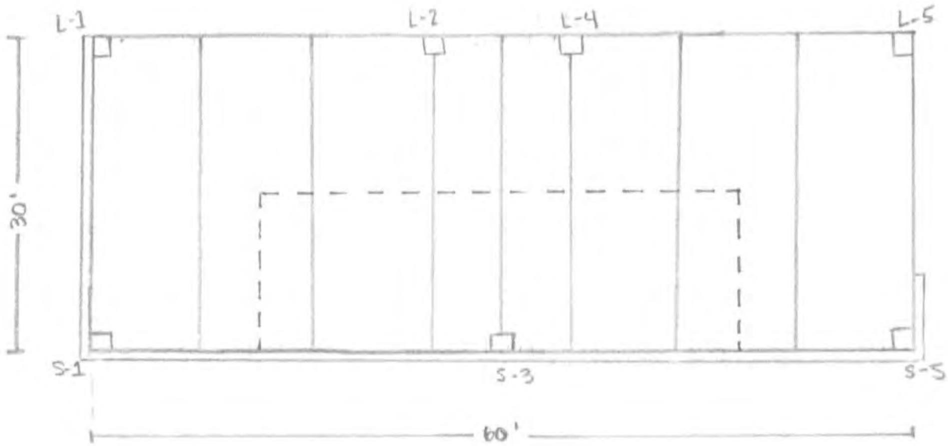
Changes with Beam Size
Calculates by itself
Changes with KL

Column S-3 Analysis Hand Calculations

Assumption: Roof has same layout as 3rd Floor.

3rd Floor

Determine Tributary Area:



$A_{tr} = 15' \times 30' = 450\ ft^2$

– : Boundary for tributary area.

Determine Loading:

Dead Load

metal decking: $3\ psf \times 450\ ft^2 = 1350\ lb$
concrete deadweight: $145\ pcf \times 2.5" \times 1/12" = 30.21\ psf$
$30.21\ psf \times 450\ ft^2 = 13595\ lb$
insulation: $2\ psf \times 450\ ft^2 = 900\ lb$
ceiling: $3\ psf \times 450\ ft^2 = 1350\ lb$
MEP: $5\ psf \times 450\ ft^2 = 2250\ lb$
Beam weight: $(15' \times 31\ lb/ft)\,5 = 2325\ lb$
Girder weight: $(15' \times 44\ lb/ft)\,2 = 1320\ lb$

Total DL = 23090 lb = 23.09^k

$LL = 20\ psf \times 450\ ft^2 = 9000\ lb = 9^k$

$SL = 45\ psf \times 450\ ft^2 = 20250\ lb = 20.25^k$

$P_u = 1.2(23.09) + 1.6(20.25) = 60.1^k$

Column Capacity:

Section Properties of HSS 8x8x5/16

$A = 8.76 \text{ in}^2$
$r = 3.13 \text{ in}$

$$\frac{KL}{r} = \frac{14 \times 12}{3.13} = 53.67$$

Interpolate $\phi_c F_{cr}$ using Table 4-22, and $F_y = 46$ ksi

KL/r	$\phi_c F_{cr}$
53	34.3
53.67	x
54	34.0

$$x = \left[\frac{\left(\frac{54 - 53.67}{54 - 53}\right)}{34.3 - 34.0} - 34.0 \right] - 1 = 33.9 \text{ ksi}$$

$$\phi_c P_n = 0.9\,(33.9)(8.8 \text{ in}^2) = 268.5^k$$

Check: $268.5^k > 60.1^k$ ∴ column is adequate.

2nd Floor

$DL = 23.09^k$
$LL = 70 \text{ psf} \times 450 \text{ ft}^2 = 31.5^k$

$P_u = 1.2(23.09) + 1.6(31.5) + 60.1$
$= 138.2^k$

Check: $268.5^k > 138.2^k$ ∴ column is adequate

1st Floor

$DL = 23.09^k$
$LL = 31.5^k$

$P_u = 1.2(23.09) + 1.6(31.5) + 138.2$
$= 216.3^k$

Check: $268.5^k > 216.3^k$ ∴ column is adequate

Appendix E: Connection Analysis Hand Calculations

ANGLE CONNECTION 1

Given: Girder W24x68
$F_u = 65 ksi$, $F_y = 50 ksi$, span = 40 ft, Trib width = 14.83
$A = 20.1 in^2$, $d = 23.7 in$, $t_w = 0.415 in$, $h/t_w = 52$
$T = 20.75$

Loading

(deadload of Penthouse)(Trib width) + (weight of girder) + (weight of beams)

$$\text{Dead load} = \frac{(106.2 psf)(14.83') + (68 lb/ft) + (50 lb/ft)}{1000}$$

$$= 1.69 k/ft$$

$$\text{Live Load} = ((70 psf)(14.83 ft))/1000$$

Per Plans

$$= 1.04 k/ft$$

$$W_u = 1.2D + 1.6L$$
$$= 1.2(1.69 k/ft) + 1.6(1.04 k/ft) = 3.69 k/ft$$

$$V_u = \frac{W_u L}{2} = \frac{(3.69 k/ft)(40 ft)}{2} = 73.8^k$$

Double angle connection w/ 3/4" ϕ A325-N Bolts
- Per Drawings

$$h/t_w = 52 in \le 2.24\sqrt{\frac{E}{F_y}} = 53.95 \Rightarrow \phi = 1.0$$

$$\phi V_n = (\phi = 1.0)(0.6 \times 50 ksi)(d = 23.7")(0.415")$$

$$\phi V_n = 295^k \ge 73.8^k \quad \underline{ok}$$

number of required bolts

Bolt strength $F_{nv} = 48 ksi$ (AISC Table J3.2)

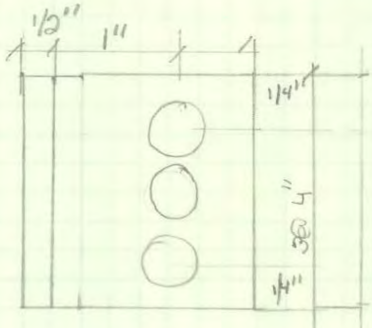
$$\phi R_n = 2(0.75)(F_v = 48 ksi)(A_b = \pi/4 (3/4)^2) = 31.8^k/\text{bolt}$$

$$n = \frac{V_u}{\phi R_n} = \frac{73.8^k}{31.8^k} = 2.32 \Rightarrow \text{use 3 bolts.}$$

From Table J3.4

for 3/4" bolt => min 1 1/4" from sheared edge.
for 3/4" bolt => min 1" from rolled edge

2



$\frac{T}{2} \leq L < T$

$10.375 \leq 10.5 < 20.75 in$

$L = 10.5$

For constructability, AISC pg 1-46 for 2" workable gage, use 3 1/2" angle leg

Calculating Angle Thickness $F_u = 58 ksi$, $F_y = 36 ksi$

Tearing/Bearing

$\phi R_n = (0.75)(1.2 L_c t F_u) \leq (0.75)(2.4 d_b t F_u)$

Tearing between bolts $L_c = 4".0 - d_e = 4" - (3/4 + 1/8") = 3.125"$

Tearing edge bolts $= L_c = 1.25 - 1/2 d_e = 1.25" - 0.5(3/4 + 1/8) = 0.83$

$\phi R_n = (0.75)(1.2)(3.125")t F_u = 2.813\ t F_u$ (between bolts)

$\phi R_n = (0.75)(1.2)(0.83")t F_u = 0.731\ t F_u$ (edge bolts)

For bearing => $(\phi = .75)(2.4 \times 3/4")t F_u = 1.35 t F_u$

Top 2 bolts are governed by tearing while bottom bolt is governed by bearing

$\phi R_n = 2(1.35 t F_u) + 1(0.731) t F_u = 3.43 t F_u$

$2(198.4 t) \geq 73.8^k$

$t \geq 0.186 in$

Shear rupture on net area of angle

$\phi R_n = 2(\phi = 0.75)(0.6 F_u)(L - n d_e) t \geq V_u$

$\phi R_n = 2(0.75)(0.6(58 ksi))(10.5 - 3(3/4" + 1/8)) t \geq 73.8^k$

$\phi R_n = 384.98 t \geq 73.8^K$

$t \geq 0.1917 in$ $\boxed{\text{governs}}$

$\phi R_n = 2(1.0)(0.6)(F_y) L t \geq V_u$

$= 2(1.0)(0.6)(38 ksi)(10.5 in) t \geq V_u$

$= 478.8 t \geq 73.8$

$t \geq 0.1541$

3

Therefore, from table 1-7 p1-44

$\boxed{\text{use } \angle\, 2L\ 3\tfrac{1}{2}'' \times 3\tfrac{1}{2}'' \times \tfrac{1}{4}''}$

Appendix F: Base Plate Analysis Sample Spreadsheet & Hand Calculations

Base Plate Size	PL 1" x 14" x 14"	
Footing Size	8' x 8'	
P_u	168	k
f'_c	3	ksi
Φ_c	0.6	
A_2	9216	in^2
Assume sqrt(A_2/A_1)	2.0	
A1	54.90	in^2
Δ	0.53	in
N	7.94	in
	use 8"	
B	6.91	in
	use 7"	
For constructability, make base plate 8"x8"		

Column Properties		
Size	HSS 8x8x5/16	
t	0.291	in
b	7.13	in
h	7.13	in
b/t	24.50	
h/t	24.50	
A	50.84	in^2
Base Plate Properties		
B	8	in
N	8	in

Check bearing Strength of Concrete		
$\Phi_c P_p$	195.84	k
	> P_u = 168k, OK.	
Required Plate Thickness		
m	0.613	in
n	1.148	in
n'	**1.783**	in
	use n' for l	
t_{req}	0.718	in
Needed	PL 3/4"x8"x8"	
Actual	PL 1"x14"x14"	
	OK	

Base Plate Analysis Hand Calculations

Verifying PL 1" × 1-2 × 1'-2" base plate for column S-3.

$t = 1"$
$B = 1'2" = 14"$
$N = 1'2" = 14"$
$A = 196\ in^2$
Footing Size = 5' × 5' × 2'
$P_u = 212.^k$
$f'_c = 3\ ksi$
Column Size: HSS 8 × 8 × 5/16

$A_2 = (5' \times 12") \times (5' \times 12") = 3600\ in^2$

Determine Required Base Plate Area

$A_1 = BN$

* Assume $\sqrt{A_2/A_1} = 2.0$

$$A_1 = \frac{P_u}{\phi_c (0.85 f'_c)\sqrt{A_2/A_1}}$$

$$= \frac{212}{0.6(0.85 \times 3)(2.0)} = 69.3\ in^2$$

for HSS 8 × 8 × 5/16:

$\frac{b}{t} = 24.5 \times (t = 0.291)$, $b = 7.13\ in$

$\frac{h}{t} = 24.5 \times (t = 0.291)$, $h = 7.13\ in$

$Area = (7.13)^2 = 50.8\ in^2 < 69.3\ in^2$ ✓

Check Bearing Strength of concrete

$$\phi_c P_p = \phi_c\, 0.85 f'_c A_1 \sqrt{A_2/A_1}$$
$$= 0.6(0.85 \times 3)(69.3)(2.0)$$
$$= 212.1^k > 212^k$$ ✓

Compute Required Base Plate Thickness

↳ using $B = N = 14"$

$$m = \frac{N - 0.95d}{2} = \frac{14 - (0.95 \times 7.13)}{2} = 3.61\ in$$

$$n = \frac{B - 0.8b}{2} = \frac{14 - (0.8 \times 7.13)}{2} = 4.15\ in$$

$$n' = \frac{\sqrt{db}}{4} = \frac{\sqrt{7.13^2}}{4} = 1.78\ in$$

Appendix G: Footing Analysis Hand Calculations

Footing Analysis Hand Calculations

Verifying 5'×5' footing area for footing supporting Column S-3.

Calculated Design Load.

1st Floor: $P_1 = 19.44^K + 31.50^K = 50.94^K$

2nd Floor: $P_2 = 23.09^K + 31.50^K = 54.59^K$

3rd Floor: $P_3 = 23.09^K + 20.25^K = 43.34^K$

$P_T = P_1 + P_2 + P_3 = 148.87^K$

Footing Area (from Structural Drawings)

$A = 25\ ft^2$

Bearing Capacity (from Structural Drawings)

$f_p = 3\ tsf = 6\ ksf$

Capacity Check:

$f_p \geq P_T/A$

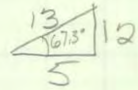
$6^K/ft^2 \geq 148.87^K/25ft^2$

$6^K/ft^2 \geq 5.95^K/ft^2$ ✓

Appendix H: Pitched Roof Analysis Hand Calculations

Roof Beam anlysis

Roof slope = 5/12



$$\text{Span} = \frac{24'4''}{\cos(67.3^\circ)} = 63'$$

Tributary width = 9'8" = 9.67'

Dead load

metal decking = 3.00 lb/ft²
Shingles (Asphalt) = 2.00 lb/ft²
Insulation = 2.00 lb/ft²
Ceiling = 3.00 lb/ft²
MEP System = 5.00 lb/ft²
Wt of beam = 22 lb/ft²

Total Dead = 17.2 psf

$$17.2\,psf\left(\frac{5}{13}\right) = 6.62\,psf \times 63' = .417\,k/ft$$

Snow load

$$45\,psf\left(\frac{5}{13}\right) = 17.3\,psf$$

$$(17.3\,psf)(63') = 1090.4\,lb/ft$$
$$1.09\,k/ft$$

Factored Load

$$1.2D + 1.6S = W_u$$

$$1.2(0.417\,k/ft) + 1.6(1.09\,k/ft) = W_u$$

$$2.24\,k/ft = W_u$$

$$M_u = \frac{W_u L^2}{8} = \frac{(2.24\,k/ft)(63')^2}{8} = 1{,}111\,k\text{-}ft$$

$$\text{Required } Z_x = \frac{M_u}{\phi F_y} = \frac{(1111\,k\cdot ft)}{(0.9)(50\,ksi)} = 24.69\,in^3$$

for W14X22 $Z_x = 33.2\,in^3 > 24.69\,in^3$

Therefore the W14X22 beam is adequate.

Also Note:
LTB (internal torsion Buckling) is avoided because of lateral support by roof deck.

$\frac{b_f}{2t_f} = 7.46 \le 9.2$ ∴ Flange local buckling is avoided

$\frac{h}{t_w} = 53.3 \le 90.5$ ∴ web local buckling is avoided.

Appendix I: Beam Design Sample Spreadsheet

Floor	Item	Size	Shear Studs	Length (ft)	ØbMpx	Ix	Zx	A	d	bf/2tf	h/tw
Second	Beam	W12 x 22	26	23.33	140	156	29.3	6.48	12.3	4.74	41.8

CHECK:			
Deflection	Δ	0.93	< 1 in
Capacity	ØbMp	118.34	< 163 k*ft

Load Combination: U = 1.2D + 1.6L + 0.5(Lr/S)		
Spacing:	8.08	ft
Length:	23.33	ft
Beam wt	22	lbs/ft

Mu	116.54	k*ft		
Mu w/beam	118.34	k*ft		
DL	0.35	k/ft		
DL w/beam	0.37	k/ft		
LL	0.808	k/ft		
SL	-	k/ft		
1.2DL	0.42	k/ft		
1.2DL w/beam	0.45	k/ft		
1.6LL	1.29	k/ft		
0.5SL	-	k/ft		
wu factored	1.71	k*ft		
wu w/beam	1.74	k*ft		
wu unfactored	1.16	k*ft		
wu w/beam un	1.18	k*ft		
E	29000	ksi	70	in
be	70.00	in	97.00	in

Deflection Due to Wet Concrete + Beam Weight		
Weight	266.18	lbs/ft
Moment	18.12	ft*k
C1	161	
Δ	0.39	< 1 in

Deflection Due to Live Load		
1/360 span	0.78	
C1	161	
M_L	55.01	ft*k
ΔL	0.38	< 0.778 in

Shear Check		
	19.98	< 36.6 k
Vu	19.98	k
ØVn	36.6	k

Deflection Check		
Deflection	0.55	< 1 in
	0.93	< 1 in

FULL COMPOSITE			
Studs		**3/4" size**	
calculated	2ΣQn	648	k
Table 3.20	Qn	17.20	k
calculated	ΣQn	324	k
Table 3.20	PNA	0	
Table 3.20	Y1	0	
given	Ycon (t)	4.5	in
calculated	a required	1.82	in
calculated	Y2	3.59	in
Interpolated	Ix	**496**	
Table 3.20	Ix low	458	
Table 3.20	Ix high	490	
Interpolated	**ØbMn**	**249**	
Table 3.19	Y2 low	3.00	
Table 3.19	Y2 high	3.5	
Table 3.19	ØbMn low	235	
Table 3.19	ØbMn high	247	
PARTIAL COMPOSITE			
Studs		**3/4" size**	
calculated	2ΣQn	162	k
Table 3.20	Qn	17.20	k
Table 3.19	ΣQn	81	k
Table 3.19	PNA	7	
Table 3.19	Y1	3.04	
given	Ycon (t)	4.5	in
calculated	a required	1.82	in
calculated	Y2	3.59	in
Interpolated	Ix	**292**	
Table 3.20	Ix low	277	
Table 3.20	Ix high	290	
Interpolated	**ØbMn**	**163**	
Table 3.19	Y2 low	3.00	
Table 3.19	Y2 high	3.5	
Table 3.19	ØbMn low	159	
Table 3.19	ØbMn high	162	

Stud Spacing		
The drawings specify for the studs to be "spaced evenly along beam"		
FULL		
L =	23.33	
# of studs	38	
spacing:	0.61	ft
PARTIAL		
L =	23.33	
# of studs	10	
spacing:	2.33	ft

Appendix J: Girder Design Sample Spreadsheet

Floor	Item	Size	Length (ft)	A	d	bf	tf
All	PH girder	W14 x 26	23.33	7.69	13.9	5.03	0.42

CHECK:			
Deflection	Δ	0.95	< 1 in
Capacity	ØbMp	173.56	< 227.84 k*ft

Account for Wt of Beams		
Total	70.000	lb/ft

Load Combination: U = 1.2D + 1.6L + 0.5(Lr/S)		
Spacing:	40.00	ft
Half Spacing:	20.00	ft
Length:	23.33	ft
Beam wt	26	lbs/ft
spacing for load	20.00	ft
Mu w/girder	**173.56**	**k*ft**
DL w/girder	0.93	k/ft
LL	0.900	k/ft
1.2DL w/girder	1.11	k/ft
1.6LL	1.44	k/ft
wu w/girder	2.55	k/ft
wu w/girder un	1.83	k/ft
E	29000	ksi
be	35	in
Interior Girder?	NO	YES/NO

Interior Girder	
70	in
480.00	in
Exterior Girder	
35	in
140.00	in

PARTIAL COMPOSITE		
Studs		**3/4" size**
calculated	2ΣQn	348 k
Table 3.21	Qn	17.20 k
Table 3.20	ΣQn	174 k
Table 3.20	PNA	BFL
Table 3.20	Y1	-0.034
given	Ycon (t)	4.5 in
calculated	a required	4.31 in
calculated	Y2	2.35 in
Interpolated	Ix	**444**
Table 3.20	Ix low	437
Table 3.20	Ix high	459
Interpolated	**ØbMn**	**228**
Table 3.19	Y2 low	2.00
Table 3.19	Y2 high	2.5
Table 3.19	ØbMn low	223
Table 3.19	ØbMn high	230

Deflection Due to Wet Concrete + Girder Weight		
Weight	1234.33	lbs/ft
Moment	84.00	ft*k
C1	161	
Δ partial	0.64	< 1 in

Evaluate for Y1		
C	507	k
T	279	k
507	>	279
C>T	falls in flange	
ybar	-0.034045726	in
Mn=Mp	2105.498108	in-k
	175.4581757	ft-k
ΦMn	157.9123581	ft-k
	in web	

LL Deflection		
1/360 span	0.78	
C1	161	
M_L	61.25	ft*k
ΔL partial	0.47	< 0.778 in

Stud Spacing		
The drawings specify for the studs to be "spaced evenly along beam"		
PARTIAL COMPOSITE		
L =	23.33	
# of studs	22	
spacing:	1.06	ft
Stud Spacing	1	ft

Shear Check		
	29.75	< 52.4 k
Vu	29.75	k
ØVn	52.4	k
Check Deflection		
Partial	0.95	< 1 in

Appendix K: Column Design Sample Spreadsheet

Column Size:	HSS 5 x 5 x 1/2		
		UNITS	Resource
KL	14	ft	
Loads			
P_d	12.74	kips	
P_L	21.00	kips	
P_u	79	kips	
Section Properties			
A	7.88	in^2	Table 1-1
r_x	1.820	in	
r_y	1.820	in	
Capacity Interpolation			
$KL/r_{y\ actual}$	92.31	-	
KL/r lower	92.000	-	Table 4-22
$\Phi_c F_{cr}$ lower	23.400	ksi	
KL/r upper	93.000	-	
$\Phi_c F_{cr}$ upper	23.100	ksi	
$\Phi_c F_{cr}$ actual	23.308	ksi	
$\Phi_c P_n$	165.298	kips	
Column Weight			
Weight/foot	28.3	lb/foot	
Total weight	396.2	lb	

Fy	**50.00**	**ksi**
f'c	**3.00**	**ksi**
Dead Loads Floor		
metal decking	630.00	lb
concrete deadwt	6343.75	lb
insulation	420.00	lb
ceiling	630.00	lb
MEP systems	1050.00	lb
FLOOR BEAMS + Girders	3662.000	lb
Critical Area	210	ft^2
TOTAL DEAD LOAD	**12735.75**	**lb**
TOTAL LIVE LOAD	**100.00**	**psf**
spacing	6.00	ft
concrete deadwt	145.00	pcf
thickness of metal deck	2.00	in
thickness of metal deck	0.17	ft
thickness of concrete	2.50	in
thickness of concrete	0.21	ft
10% of Concrete Load	1.10	plf

Changes with Beam Size
Calculates by itself
Changes with KL

Appendix L: Connection Design Sample Spreadsheet

Given					
Girder Size	W33 x 118				
Fu	65	ksi	Span	40	ft
Fy	50	ksi	Trib Width	17.84	ft
A	34.7	in^2	tw	0.55	in
d	32.9	in	h/tw	54.5	
T	29.625	in			

Loading		
Dead Load	1.99	k/ft
Live Load	5.71	k/ft
Wu	11.52	k/ft
Vu	232.50	k

Type of Angle Connection		
Double Angle Connection		
Diameter	0.75	in
A 325 - N Bolts		

Shear Capacity			
ϕ=	0.9		
ϕVn	635.1345	≥	232.5

Number of Required Bolts		
ϕ=	0.75	
nFV	48	ksi
ϕRn	31.8	k/bolt
n	7.31	Bolts
Use	8	Bolts

Connection Layout			
	Min		Max
Rolled Edge Distance	1	1	3
Min Sheared Edge Dist.	1.25	2	3
Minimum Bolt Spacing	2.25	3	6
	T/2 ≤	L	< T
	14.81	25	29.63

*Permissible edge distances cannot be greater than 6"

Calculating Angle Thickness

Fu	58	ksi
Fy	36	ksi

Tearing/Bearing		
Tearing b/w Bolts		
Lc =	2.13	in
ϕRn	1.91	tFu
Tearing Edge Bolts		
Lc =	1.56	in
ϕRn	1.41	tFu
For Bearing		
ϕRn	1.35	tFu

Bolt(s) governed by tearing	8
Bolt(s) governed by bearing	0

Tearing/Bearing		
t ≥	0.186	in
Shear Rupture on net area of angle		
ϕRn	939.6	in
t ≥	0.247	in
ϕRn	972	in
t ≥	0.239	in

t ≥	0.247	governs
t =	1/4	in

Connection Summary		
<2L 3 1/2 x 3 1/2 x 1/4		
Number of Bolts	8	
Bolt Spacing	3	in
Rolled Edge Distance	1	in
Sheared Edge Distance	2	in
Total Length	25	in

Appendix M: Base Plate Design Sample Spreadsheet

Base Plate Size		
Footing Size	9.5' x 9.5'	
P_u	763	k
f'_c	3	ksi
Φ_c	0.6	
A_2	12996	in^2
Assume $\mathrm{sqrt}(A_2/A_1)$	2.0	
A1	249.35	in^2
Δ	0.77	in
N	16.56	in
	use 17 in	
B	15.06	in
	use 16"	
For constructability, make base plate 17"x17"		

Column Properties		
Size	HSS 12 x 12 x 5/8	
t	0.581	in
b	10.28	in
h	10.28	in
b/t	17.70	
h/t	17.70	
A	105.75	in^2
Base Plate Properties		
B	17	in
N	17	in

Check bearing Strength of Concrete		
$\Phi_c P_p$	884.34	k
	> P_u = 763k, OK.	
Required Plate Thickness		
m	3.615	in
n	**4.387**	in
n'	2.571	in
	use n for l	
t_{req}	1.771	in
Final Design	**PL 1.75" x 17" x 17"**	

Appendix N: Footing Design Spreadsheet & Hand Calculations

Column L13		
		Units
Soil bearing Capacity	6	ksf
Unfactored Load	524.6	kips
Required Area of Footing	87.43	sq. ft.
Footing Width	9.35	ft
Use 9.5' x 9.5'		

Column S17.7		
		Units
Soil bearing Capacity	6	ksf
Unfactored Load	90.86	kips
Required Area of Footing	15.14	sq. ft.
	3.89	ft.
Use 4' x 4'		

Footing Design Hand Calculations

① Footing Area for footing supporting column S-17.7

Design Loads:

1st Floor: $P_1 = 12.74^k + 21.00^k = 33.74^k$

2nd Floor: $P_2 = 12.74^k + 21.00^k = 33.74^k$

3rd Floor: $P_3 = 12.74 + 9.45^k = 22.19^k$

$P_T = P_1 + P_2 + P_3 = 89.67^k$

$f_p = 6\ ksf$

$f_p = \frac{P_T}{A} \rightarrow A = \frac{P_T}{f_p}$

$A_{min} = \frac{89.67^k}{6\ ksf} = 14.95\ ft^2$

$\sqrt{14.95\ ft^2} = 3.86'$ footing dimension

↳ use $\underline{4' \times 4'}$

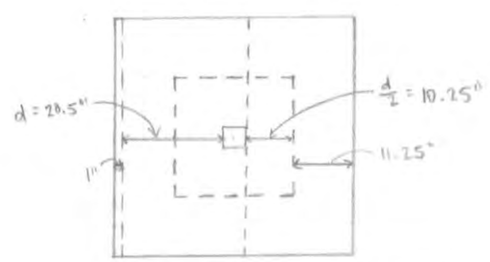
② Footing Thickness

Punching Shear:

$\phi V_c = \phi 4 \lambda \sqrt{f'c}\, b_o d \geq V_u$

Try $d = 20.5"$

$b_o = 4' \times 12"/_1 = 48"$



$\phi V_c = (\phi = 0.75)\, 4\, (\lambda = 1.0) \sqrt{(f'c = 3000\ psi)}\, (b_o = 48")(d = 20.5") \geq 128{,}000\ lb$

$161{,}687\ lb > 128{,}000\ lb$

∴ 2' thickness works for Punching Shear

Beam Shear:

$$\phi V_c = \phi 2 \lambda \sqrt{f'_c}\, bd \geq V_u$$

Try $d = 20.5''$
$b = 4(5'' + 20.5'') = 102''$

$$\phi V_c = (\phi = 0.75)\, 2\, (\lambda = 1.0) \sqrt{(f'_c = 3000\ psi)}\, (b = 102'')(d = 20.5'') \geq V_u = 128{,}000\ lb$$

$171{,}793\ lb > 128{,}000\ lb$

∴ 2' thickness works for Beam Shear

* Since the force in the concrete is greater than the factored shear force in the footing, no stirrup steel is needed for reinforcement.

③ Design of reinforcing bars

$$M_u = 6.0\ ksf \times 4' \left(\frac{1.79^2}{2}\right) 12 = 461.4\ \text{in-kips}$$

$$A_s = \frac{461}{0.9 \times 60 \times 19.5} = \frac{461}{1053} = 0.43\ in^2$$

$$A_{s,min} = \frac{3\sqrt{3000}}{60{,}000} \times 48 \times 20.5 = 2.69\ in^2$$

but not less than...

$$\frac{200}{60{,}000} \times 48 \times 20.5 = \boxed{3.28}\ (\text{governs})$$

Use 11 #4 bars in each direction.

From Table A.11 in Design of Concrete Structures by Nilson et al, the development length is 11".

Appendix O: Lateral Loads Spreadsheets

Wind Load using Method 1: Simplified Procedure				
Factor	**Value**	**Important Assumption (s)**	**Source & Supporting Table and/or Figure**	**Code**
Basic Wind Speed, V, (mph)	100	Dedham	Table 1604.10	Mass. Bld Code
Importance Factor, I*	1.15	Category III Building	Table 6-1	ASCE 7-05
Height and Exposure Factor, λ	1.13	Exposure B, Avg height 46ft	Figure 6-3	ASCE 7-05
Topographic Factor, K_{zt}	1	Flat Ground	Figure 6-4	ASCE 7-05
Simplified Design Wind Pressure, p_{s30}, (psf)	15.9	100 mph wind, Exposure B Category, End Zone of Wall	Figure 6-2	ASCE 7-05
Wind Load, p_s (psf)	**20.66**			

*Also provided in architectural drawings
ps30 = simplified design wind pressure for Exposure B at h=30 ft and I=1.0
ps = λ*Kzt*I*ps30

Adjustment Factor for Building Height and Exposure, λ

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

Source: ASCE 7-05

North-South Elevation					
Section	Quantity	Height	Width	Total Surface Area (SF)	Total Wind Force (k)
A	2	46.67	60	5600	116
B	2	42	30	2520	52
C	1	51.75	40	2070	43
			TOTAL	10190	211

Average Height (ft)	46.3
# of Frame Systems	4
Total Wind Force per Frame System (k)	52.75

West-East Elevation					
Section	Quantity	Height	Width	Total Surface Area (SF)	Total Wind Force (k)
A'	1	14	18	252	5.2
B'	1	42	22.6	949	19.6
C'	1	46.67	6.7	313	6.5
D'	1	51.75	59	3053	63.1
E'	1	46.67	4	187	3.9
F;	1	14	4.7	66	1.4
			TOTAL	4820	99.7

Average Height (ft)	41.9
# of Frame Systems	8
Total Wind Force per Frame System (k)	12.5

SURFACE AREA PER FLOOR			
Assembly Description	Area	Area	Comments
Second Floor			
Floor Construction	4180 SF	4180	2nd Level
Floor Construction	1564 SF	14564	2nd Level
Floor Construction	2239 SF	2239	2nd Level
Floor Construction	1643 SF	1643	2nd Level
Floor Construction	1815 SF	1815	2nd Level
Floor Construction	4180 SF	4180	2nd Level
	SUBTOTAL	28621.00	
Floor Construction	1194 SF	1194.00	2nd Level - Roof
Floor Construction	347 SF	347.00	2nd Level - Roof
	SUBTOTAL	1541.00	
	TOTAL	30162.00	
Third Floor			
Floor Construction	4180 SF	4180.00	3rd Level
Floor Construction	1564 SF	1564.00	3rd Level
Floor Construction	2239 SF	2239.00	3rd Level
Floor Construction	1643 SF	1643.00	3rd Level
Floor Construction	4180 SF	4180.00	3rd Level
Floor Construction	4180 SF	4180.00	3rd Level
	SUBTOTAL	17986.00	
Floor Construction	1815 SF	1815	3rd Level - Roof
	SUBTOTAL	1815	
	TOTAL	19801.00	
Roof			
Floor Construction	293 SF	293.00	Roof
Floor Construction	1643 SF	1643.00	Roof
Floor Construction	1643 SF	1643.00	Roof
Floor Construction	4180 SF	4180.00	Roof
Floor Construction	4180 SF	4180.00	Roof
	SUBTOTAL	8360.00	
Floor Construction	2122 SF	2122	Roof Level - MEP
Floor Construction	240 SF	240	Roof Level - MEP
	SUBTOTAL	2362.00	
	TOTAL	10722.00	
Floor Construction	2360 SF	2360	MEP Roof
	TOTAL	2360	

SURFACE AREA PER FLOOR	
Level	Area (ft^2)
Second Level Floor	16219
Second Level Roof	1496
TOTAL	17715
Third Level Floor	14197
Third Level Roof	1815
TOTAL	16012
Roof	14974
Roof-MEP	2362
TOTAL	17336
Total Floor Area	45390
Total Roof Area	5673
BUILDING TOTAL	51063

The table below shows the different factors used to determine the earthquake loads which were calculated or determined by using provisions addressed in Section 11 and Section 12 of *ASCE 7-05*.

Name/Description	Factor	Value	Source
Seismic Response Coefficient	C_S	0.0867	Struct. Dwgs
Redundancy Factor	ρ	1	Section 12.3.4
Spectral Response Coefficient	S_{DS}	0.208	Struct. Dwgs
Effect of Dead Load (k)	D	3,279	Calculated
Earthquake Force on building (k)	E	425	Calculated
Height at highest level of building (ft)	h_n	51.75	
Building Period Coefficient	C_t	0.028	Table 12.8-2
Building Period Coefficient	x	0.8	Table 12.8-2
Period	T_L	0.66	Equation 12.8-7
Exponent related to structure period (sec)	k	1.11	Calculated
Total Base Shear (k)	V	289	Calculated
Horizontal Seismic Force Component (k)	$Q_E = V$	289	Calculated

These factors were then used to determine the earthquake forces for each level which is shown in the table below and uses equations found in Section 11 and Section 12 of *ASCE 7-05* as well.

Earthquake Forces for Each Level in North-South Frames								
Level	Avg. Story Height	Height	Weight (kips)	WiHi^k	Cvx	EQ Force per level (k)	EQ Force per Frame	Story Shear (k)
Roof	18.67	46.67	8.26	588.32	0.59	170.51	42.63	0
3	14	28	6.79	274.29	0.27	78.03	19.51	170.51
2	14	14	7.48	139.99	0.14	40.46	10.12	248.54
1	0	0	-	0	0	0	0	289
Total	46.67	-	22.53	1002.6	1	289	72.26	289

Earthquake Forces for Each Level in West-East Frame								
Level	Avg. Story Height	Height	Weight (kips)	WiHi^k	Cvx	EQ Force per level (k)	EQ Force per Frame	Story Shear (k)
Roof	18.67	46.67	8.26	588.32	0.59	170.51	21.31	0
3	14	28	6.79	274.29	0.27	78.03	9.75	170.51
2	14	14	7.48	139.99	0.14	40.46	5.06	248.54
1	0	0	-	0	0	0	0	289
Total	46.67	-	22.53	1002.6	1	289	36.12	289

BF-1 N-S			
Level	**Avg. Tributary Height**	**Wind Force (k)**	**Earthquake Force (k)**
Roof	9.34	**10.6**	**42.63**
Third Level	16.34	**18.6**	**19.51**
Second Level	14	**16.0**	**10.12**
First Level	7	**8.0**	**-**
TOTAL	46.68	53.2	72.3

BF-2 N-S			
Level	**Avg. Tributary Height**	**Wind Force (k)**	**Earthquake Force (k)**
MEP	11.67	**13.3**	**42.63**
Third Level	14	**16.0**	**19.51**
Second Level	14	**16.0**	**10.12**
First Level	7	**8.0**	**-**
TOTAL	46.67	53.3	72.3

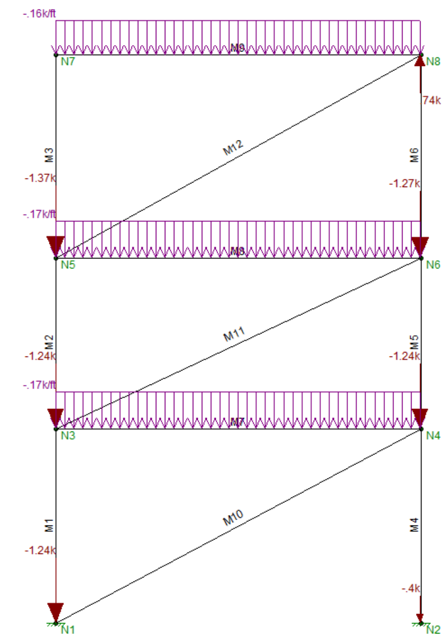
BF-3 N-S			
Level	**Avg. Tributary Height**	**Wind Force (k)**	**Earthquake Force (k)**
MEP	11.67	**13.3**	**42.63**
Third Level	14	**16.0**	**19.51**
Second Level	14	**16.0**	**10.12**
First Level	7	**8.0**	**-**
TOTAL	46.67	53.3	72.3

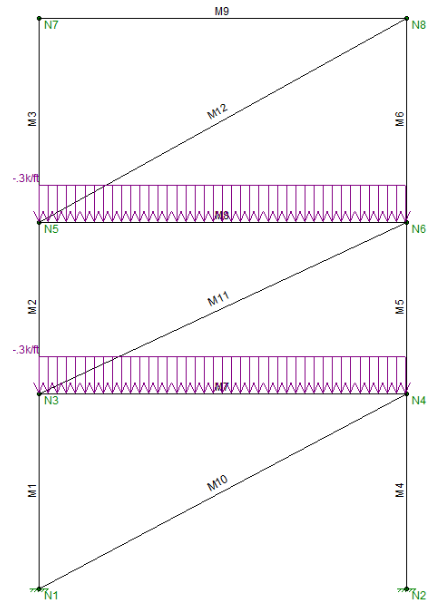
BF-4 W-E			
Level	**Avg. Tributary Height**	**Wind Force (k)**	**Earthquake Force (k)**
Roof	9.34	**2.8**	**21.31**
Third Level	16.34	**4.9**	**9.75**
Second Level	14	**4.2**	**5.06**
First Level	7	**2.1**	**-**
TOTAL	46.68	14.0	36.1

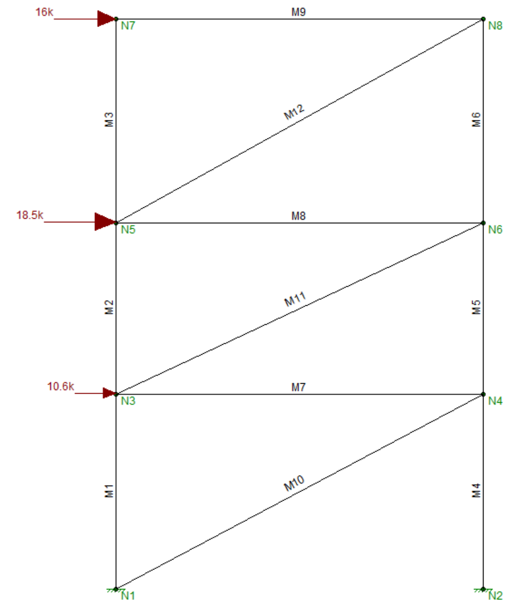
**NOTE: The total wind force per frame is higher than the calculated wind force due to using the avg. height as opposed to total height.

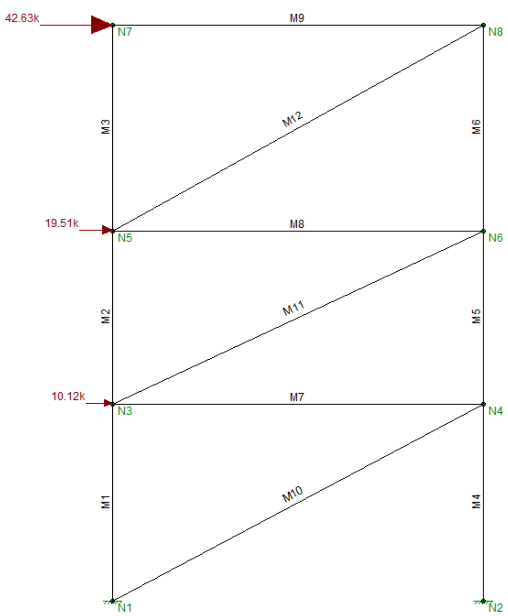
Appendix P: Sample of RISA Load Models and RISA Results

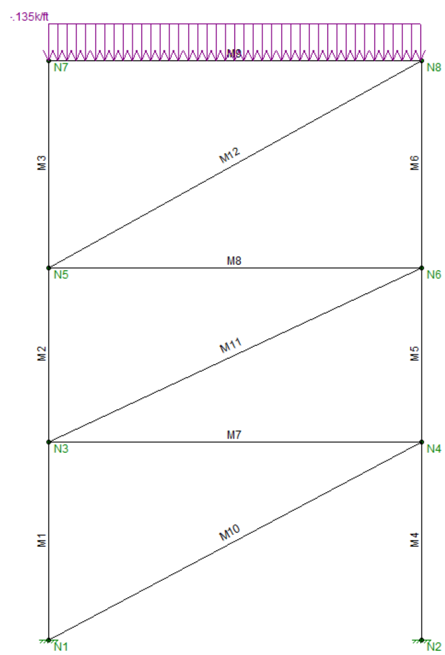
The following RISA diagrams show the dead loads, live loads, wind loads, earthquake loads, snow loads and roof live loads of Frame 1. For frames 2-4, the loading diagrams looked similar with just different magnitudes

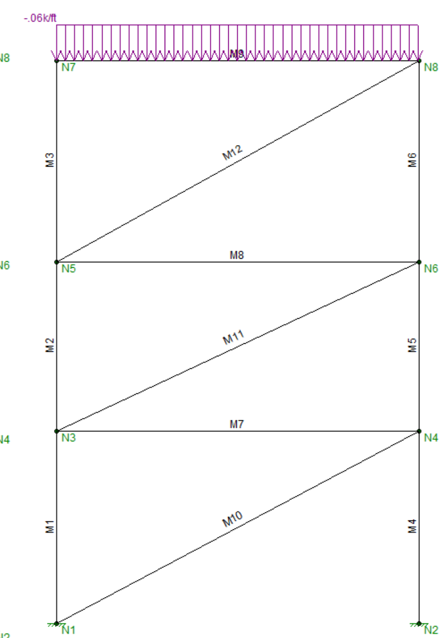




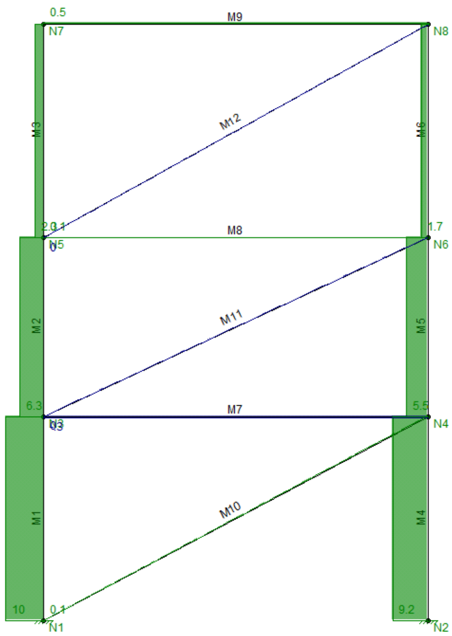


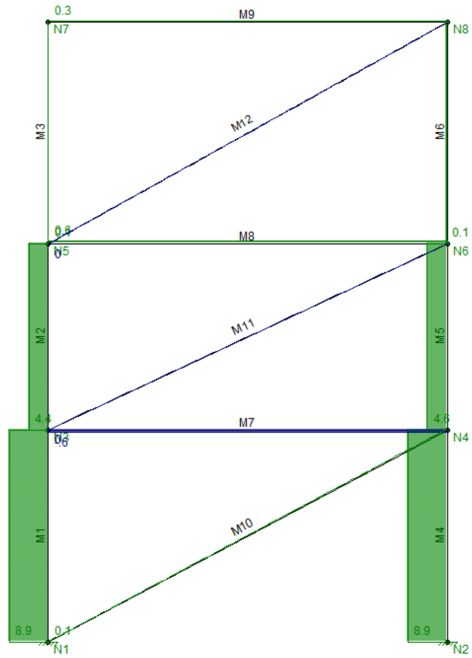


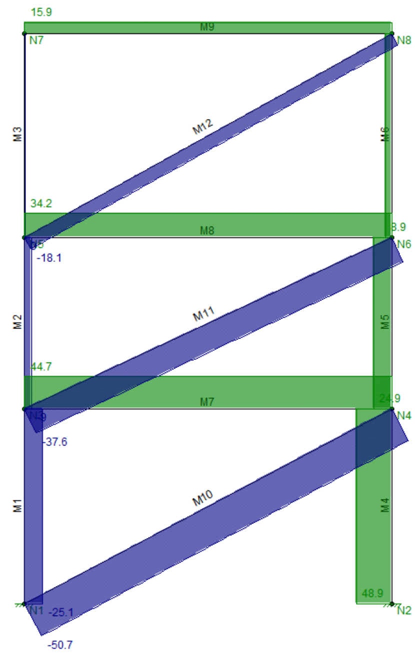


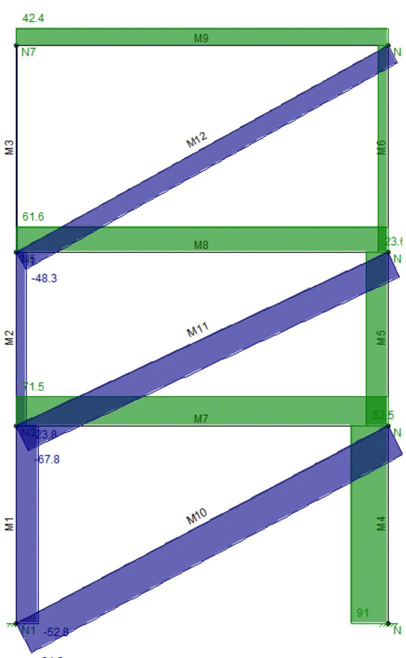


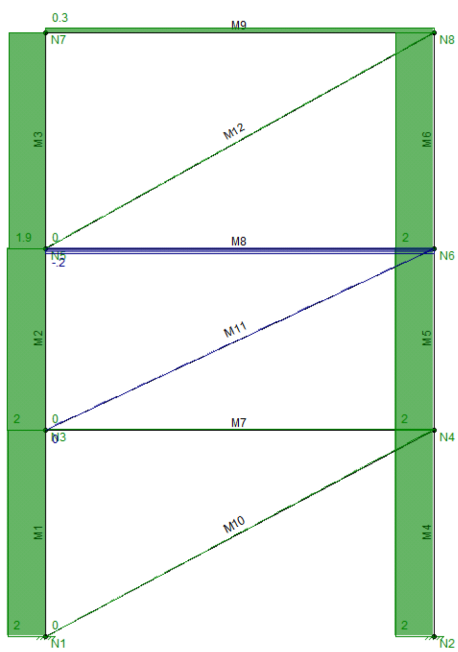
By order, the following RISA diagrams show the axial reactions due to the dead loads, live loads, wind loads, earthquake loads, snow loads and roof live loads of Frame 1.

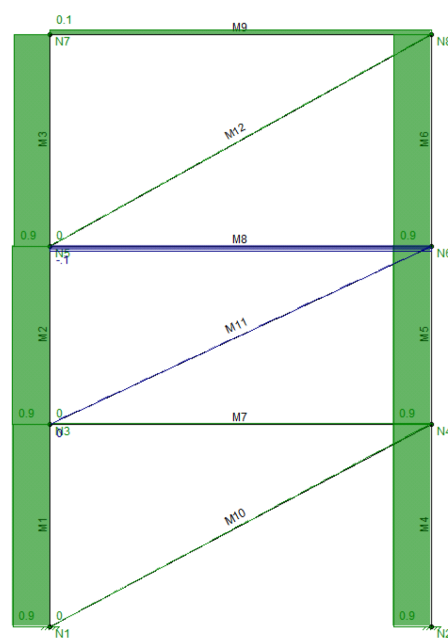




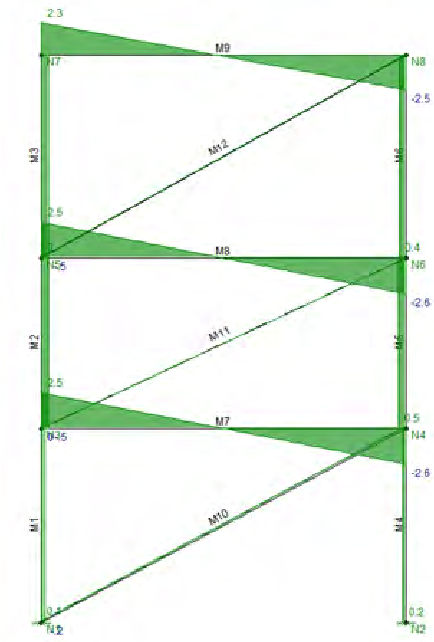


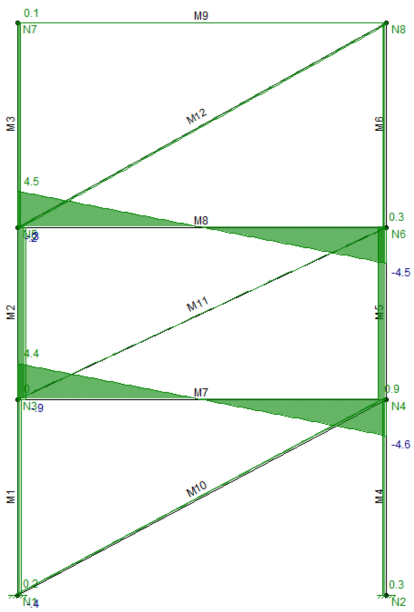


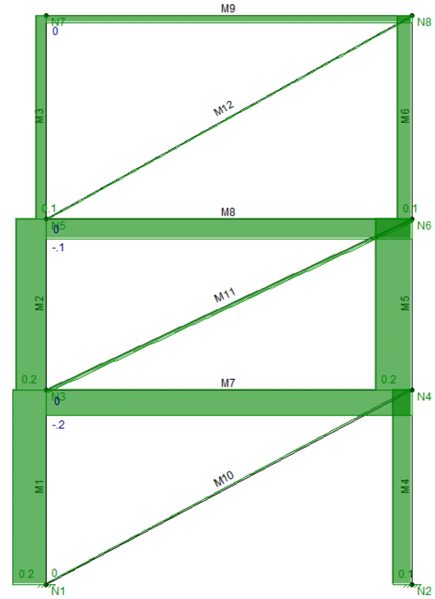


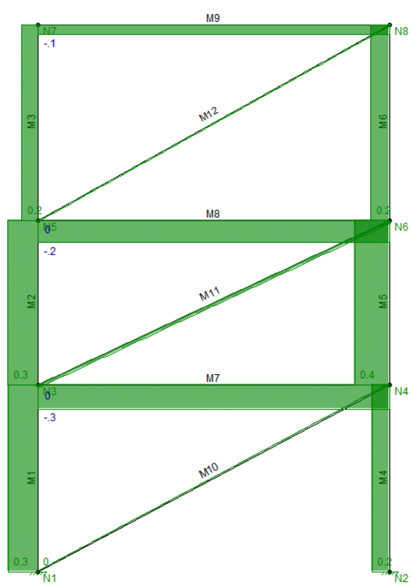


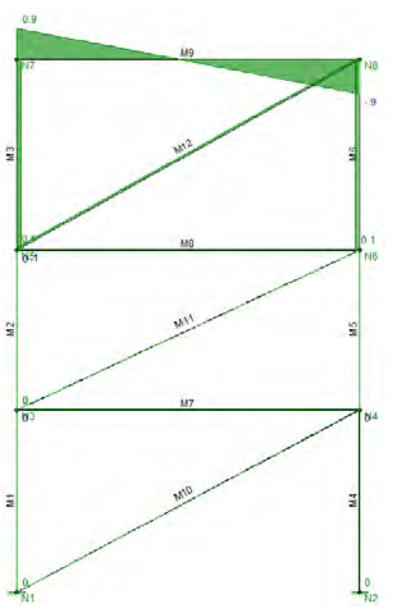
By order, the following RISA diagrams show the shear reactions due to the dead loads, live loads, wind loads, earthquake loads, snow loads and roof live loads of Frame 1.

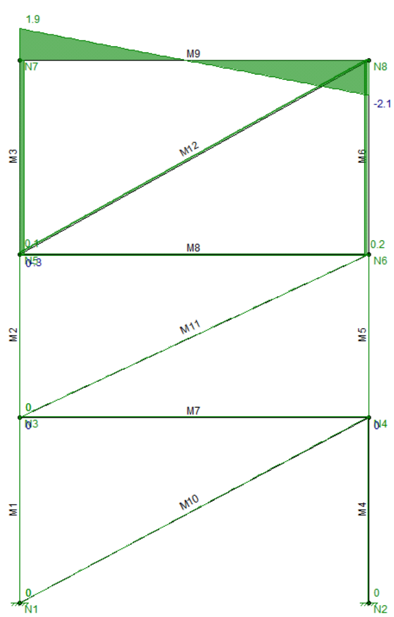




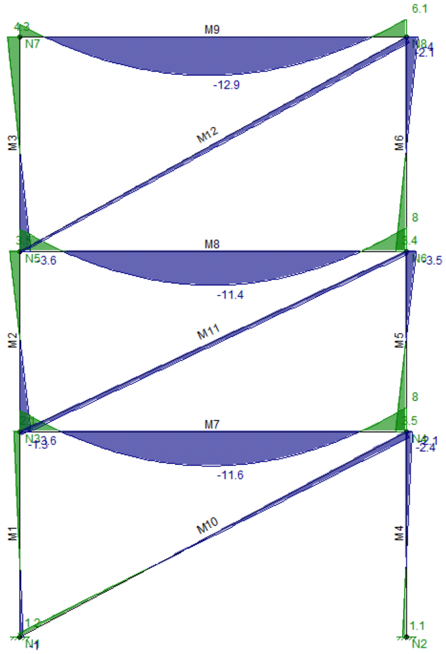


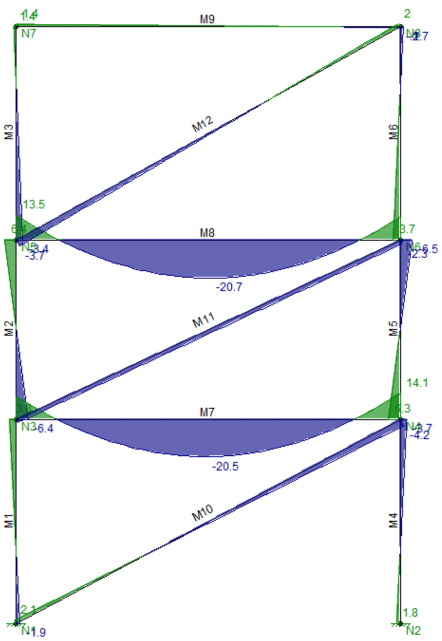


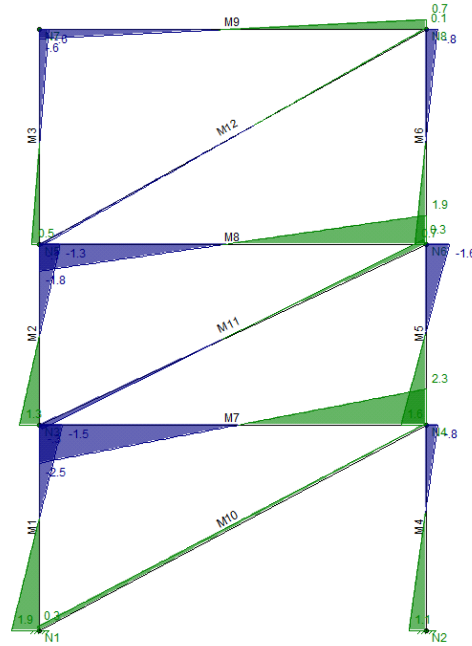


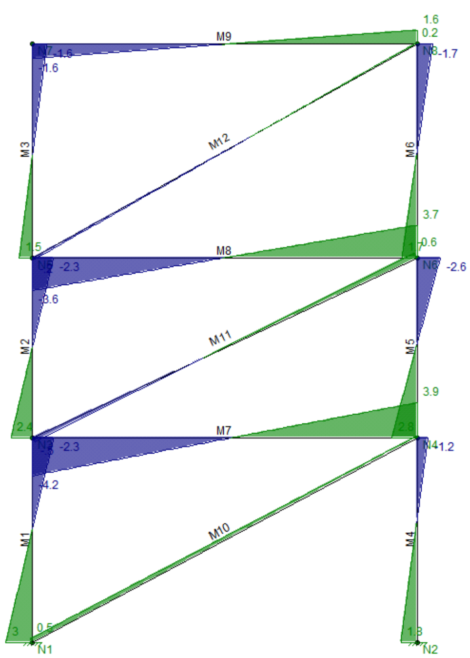


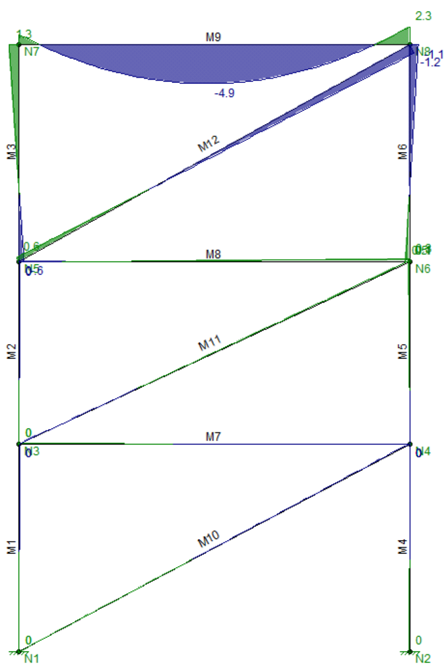
By order, the following RISA diagrams show the moment reactions due to the dead loads, live loads, wind loads, earthquake loads, snow loads and roof live loads of Frame 1.

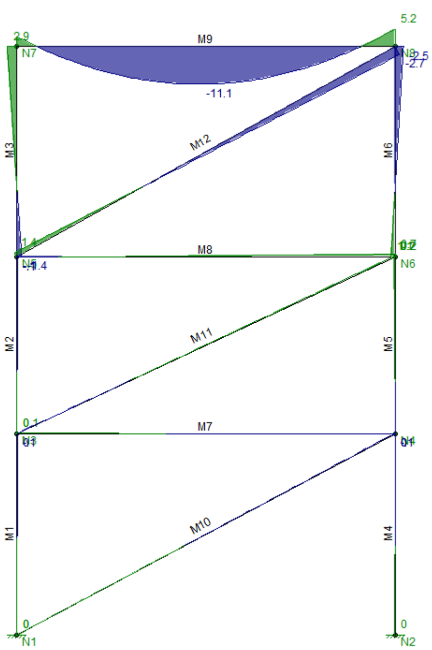








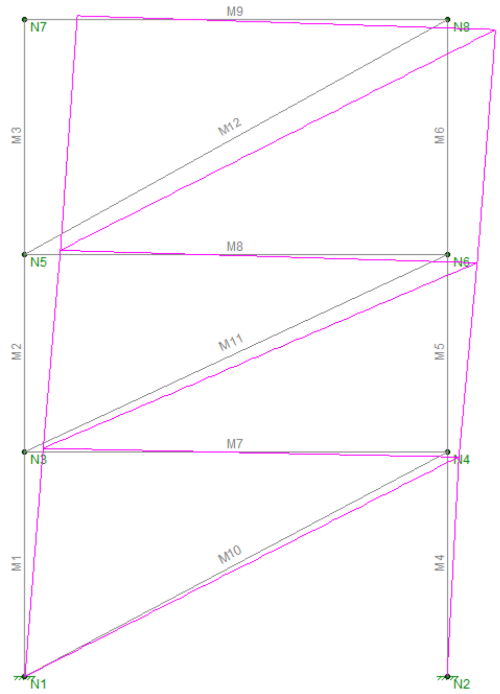




The following diagrams show the deflection results for BF-1, BF-2, BF-3 and BF-4 due to the combined wind and earthquake loads. The deflected frame is represented by the pink line, and the magnitude of the deflection was increased in order to better visualize the deflection.

Joint Deflections (By Combination)

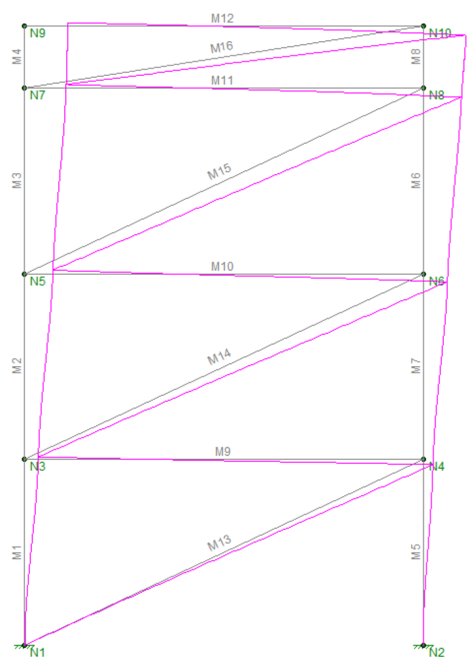
	L...	Joint Label	X [in]	Y [in]	Rotatio...
1	4	N1	0	0	0
2	4	N2	0	0	0
3	4	N3	.387	.066	-1.03e-3
4	4	N4	.251	-.119	-9.165e-4
5	4	N5	.755	.091	-1.106e-3
6	4	N6	.643	-.177	-1.15e-3
7	4	N7	1.115	.091	-9.932e-4
8	4	N8	1.036	-.205	-1.021e-3



RISA Analysis of Lateral Deflection of BF-1

Joint Deflections (By Combination)

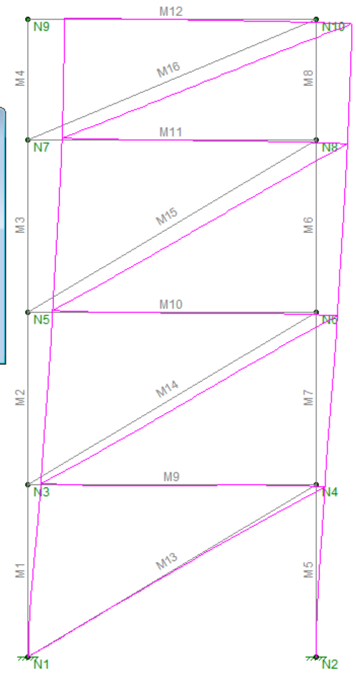
	L...	Joint Label	X [in]	Y [in]	Rotatio...
1	4	N1	0	0	0
2	4	N2	0	0	0
3	4	N3	.306	.063	-6.289e-4
4	4	N4	.227	-.113	-5.797e-4
5	4	N5	.637	.087	-8.822e-4
6	4	N6	.554	-.176	-9.008e-4
7	4	N7	.935	.088	-7.609e-4
8	4	N8	.871	-.2	-1.228e-3
9	4	N9	.974	.088	-6.069e-4
10	4	N10	.975	-.2	-1.256e-3



RISA Analysis of Lateral Deflection of BF-2

Joint Deflections (By Combination)

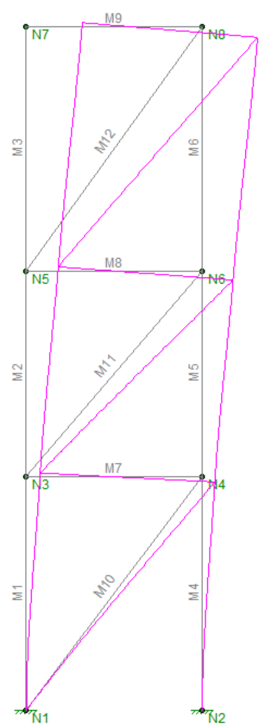
	L...	Joint Label	X [in]	Y [in]	Rotatio...
1	4	N1	0	0	0
2	4	N2	0	0	0
3	4	N3	.305	.031	-1.555e-3
4	4	N4	.215	-.054	-1.3e-3
5	4	N5	.592	.043	-1.195e-3
6	4	N6	.52	-.084	-1.301e-3
7	4	N7	.821	.043	-8.161e-4
8	4	N8	.764	-.096	-1.066e-3
9	4	N9	.881	.043	-3.643e-4
10	4	N10	.881	-.096	-8.845e-4



RISA Analysis of Lateral Deflections of BF-3

Joint Deflections (By Combination)

	L...	Joint Label	X [in]	Y [in]	Rotatio...
1	4	N1	0	0	0
2	4	N2	0	0	0
3	4	N3	.284	.067	-1.494e-3
4	4	N4	.258	-.121	-1.442e-3
5	4	N5	.653	.092	-2.015e-3
6	4	N6	.632	-.179	-1.993e-3
7	4	N7	1.147	.092	-2.104e-3
8	4	N8	1.134	-.208	-2.182e-3



RISA Analysis of Lateral Deflections of BF-4

Appendix Q: Lateral Force Resisting System Design Sample Spreadsheets with RISA Analysis Spreadsheets

LOADINGS		
Dead Load	43.21	psf
Live Load	80	psf
Snow Load	45	psf
Roof Live Load	20	psf

LOAD COMBINATIONS	
LC 4:	U = 1.2D + 1.0W + 0.5L + 0.5S
LC 5:	U = 1.2D +1.0E +0.5L + 0.2S
LC 7:	U = 0.9D + 1.0E

F1 LOADINGS									
Level	**Beam Size**	**ominal Wt. (lb/f**	**DL (k/ft)**	**Total DL (k/ft)**	**LL (k/ft)**	**WL (k)**	**SL (k/ft)**	**EQ L (k)**	**Lr (k/ft)**
Roof	W16x31	31	0.12963	0.16	-	10.6	0.14	42.63	0.06
Third Level	W16x36	36	0.12963	0.17	0.3	18.6	-	19.51	-
Second Level	W16x36	36	0.12963	0.17	0.3	16	-	10.12	-

Column Level	Length (ft)	Nominal Wt. (lb/ft)	Total Wt. per column k)
Third Level	18.67	28.3	0.53
Second Level	14	28.3	0.40
First Level	14	28.3	0.4

Brace Level	Length (ft)	Size	Nominal Wt. (lb/ft)	Total Wt. (k)
Third Level	35.34	HSS 7x7x1/2	41.91	0.74
Second Level	33.11	HSS 7x7x5/8	50.6	0.84
First Level	33.11	HSS 7x7x5/8	50.6	0.84

Member	Section	AXIAL Forces [k] due to								
		Dead Load	Live Load	Wind Load	EQ Load	Snow Load	Roof Live Load	LC 4	LC 5	LC 7
M1	1	10.03	8.847	-25.046	-52.772	2.024	0.899	42.52	69.64	61.80
	2	10.03	8.847	-25.046	-52.772	2.024	0.899	42.52	69.64	61.80
M2	1	6.286	4.412	-8.951	-23.792	2.03	0.902	19.72	33.95	29.45
	2	6.286	4.412	-8.951	-23.792	2.03	0.902	19.72	33.95	29.45
M3	1	2.34	0.065	-0.026	-0.066	1.933	0.859	3.83	3.29	2.17
	2	2.34	0.065	-0.026	-0.066	1.933	0.859	3.83	3.29	2.17
M4	1	9.227	8.921	48.967	91.099	2.024	0.899	65.51	107.04	99.40
	2	9.227	8.921	48.967	91.099	2.024	0.899	65.51	107.04	99.40
M5	1	5.528	4.584	24.926	52.572	2.027	0.901	34.87	61.90	57.55
	2	5.528	4.584	24.926	52.572	2.027	0.901	34.87	61.90	57.55
M6	1	1.68	0.145	8.866	23.623	1.982	0.881	11.95	26.11	25.14
	2	1.68	0.145	8.866	23.623	1.982	0.881	11.95	26.11	25.14
M7	1	-0.3	-0.39	44.814	71.786	0.004	0.002	45.37	72.34	72.06
	2	-0.3	-0.39	44.814	71.786	0.004	0.002	45.37	72.34	72.06
M8	1	0.064	0.397	34.314	61.783	-0.138	-0.061	34.66	62.09	61.84
	2	0.064	0.397	34.314	61.783	-0.138	-0.061	34.66	62.09	61.84
M9	1	0.47	0.209	15.958	42.517	0.166	0.074	16.71	43.22	42.94
	2	0.47	0.209	15.958	42.517	0.166	0.074	16.71	43.22	42.94
M10	1	0.061	0.104	-50.857	-81.485	0.003	0.001	50.98	81.61	81.54
	2	0.061	0.104	-50.857	-81.485	0.003	0.001	50.98	81.61	81.54
M11	1	-0.017	0.004	-37.763	-68.034	-0.01	-0.004	37.79	68.06	68.05
	2	-0.017	0.004	-37.763	-68.034	-0.01	-0.004	37.79	68.06	68.05
M12	1	-0.003	-0.079	-18.196	-48.5	0.043	0.019	18.26	48.55	48.50
	2	-0.003	-0.079	-18.196	-48.5	0.043	0.019	18.26	48.55	48.50

Member	Section	SHEAR Forces [k] due to :								
		Dead Load	Live Load	Wind Load	EQ Load	Snow Load	Roof Live Load	LC 4	LC 5	LC 7
M1	1	-0.196	-0.245	0.136	0.216	0.001	0	0.49	0.57	0.39
	2	-0.196	-0.245	0.136	0.216	0.001	0	0.49	0.57	0.39
M2	1	-0.507	-0.632	0.132	0.234	-0.002	0	1.06	1.16	0.69
	2	-0.507	-0.632	0.132	0.234	-0.002	0	1.06	1.16	0.69
M3	1	-0.47	-0.209	0.042	0.113	-0.166	-0.074	0.79	0.81	0.54
	2	-0.47	-0.209	0.042	0.113	-0.166	-0.074	0.79	0.81	0.54
M4	1	0.199	0.24	0.085	0.136	0.001	0	0.44	0.50	0.32
	2	0.199	0.24	0.085	0.136	0.001	0	0.44	0.50	0.32
M5	1	0.495	0.635	0.15	0.259	-0.005	-0.002	1.06	1.17	0.70
	2	0.495	0.635	0.15	0.259	-0.005	-0.002	1.06	1.17	0.70
M6	1	0.444	0.236	0.054	0.123	0.14	0.062	0.77	0.80	0.52
	2	0.444	0.236	0.054	0.123	0.14	0.062	0.77	0.80	0.52
M7	1	2.518	4.431	-0.119	-0.2	0	0	5.36	5.44	2.47
	2	-2.582	-4.569	-0.119	-0.2	0	0	5.50	5.58	2.52
M8	1	2.536	4.556	-0.086	-0.169	-0.038	-0.017	5.43	5.50	2.45
	2	-2.564	-4.444	-0.086	-0.169	-0.038	-0.017	5.40	5.48	2.48
M9	1	2.34	0.065	-0.026	-0.066	1.933	0.859	3.83	3.29	2.17
	2	-2.46	0.065	-0.026	-0.066	-2.117	-0.941	4.07	3.47	2.28
M10	1	0.107	0.206	0.013	0.021	0	0	0.24	0.25	0.12
	2	0.107	0.206	0.013	0.021	0	0	0.24	0.25	0.12
M11	1	-0.007	0.003	-0.006	-0.01	-0.003	-0.001	0.02	0.02	0.02
	2	-0.007	0.003	-0.006	-0.01	-0.003	-0.001	0.02	0.02	0.02
M12	1	0.048	-0.196	-0.002	0	0.13	0.058	0.22	0.18	0.04
	2	0.048	-0.196	-0.002	0	0.13	0.058	0.22	0.18	0.04

Member	Section	Moment Forces [ft-k] due to :								
		Dead Load	Live Load	Wind Load	EQ Load	Snow Load	Roof Live Load	LC 4	LC 5	LC 7
M1	1	-1.021	-1.29	1.178	1.898	0.011	0.005	3.05	3.77	2.82
	2	2.109	2.63	-0.993	-1.552	-0.012	-0.006	4.84	5.40	3.45
M2	1	-3.59	-4.46	0.932	1.679	-0.006	-0.002	7.47	8.22	4.91
	2	3.514	4.39	-0.916	-1.6	0.024	0.011	7.34	8.02	4.76
M3	1	-3.578	-2.38	0.339	0.93	-0.933	-0.415	6.29	6.60	4.15
	2	4.25	1.10	-0.353	-0.946	1.839	0.817	6.92	6.96	4.77
M4	1	1.078	1.29	0.763	1.242	0.008	0.004	2.71	3.18	2.21
	2	-2.105	-2.55	-0.59	-0.926	-0.008	-0.003	4.40	4.73	2.82
M5	1	3.483	4.40	1.076	1.868	-0.023	-0.01	7.47	8.25	5.00
	2	-3.451	-4.49	-1.031	-1.753	0.053	0.023	7.44	8.15	4.86
M6	1	3.417	2.60	0.442	1.023	0.755	0.335	6.22	6.57	4.10
	2	-3.978	-1.34	-0.459	-1.031	-1.584	-0.704	6.69	6.79	4.61
M7	1	7.043	9.55	-1.847	-3.134	0.022	0.01	15.08	16.36	9.47
	2	7.992	11.60	1.732	2.864	0	0	17.12	18.26	10.06
M8	1	7.539	11.33	-1.224	-2.525	-0.51	-0.227	16.19	17.34	9.31
	2	7.965	9.63	1.345	2.546	0.62	0.276	16.03	17.04	9.71
M9	1	4.25	1.10	-0.353	-0.946	1.839	0.817	6.92	6.96	4.77
	2	6.062	-0.86	0.435	1.036	4.591	2.04	10.43	9.66	6.49
M10	1	1.236	2.36	0.381	0.646	0.015	0.01	3.05	3.31	1.76
	2	-2.404	-4.65	-0.066	-0.07	-0.014	-0.01	5.28	5.28	2.23
M11	1	-1.343	-2.46	-0.077	-0.097	-0.029	-0.01	2.93	2.94	1.31
	2	-1.096	-2.55	0.128	0.23	0.082	0.04	2.76	2.83	1.22
M12	1	-0.447	-4.55	-0.031	-0.004	1.468	0.65	3.58	3.11	0.41
	2	-2.083	2.19	0.024	-0.005	-3.006	-1.34	5.12	4.20	1.88

Joint	Joint Reactions in X-direction (k) due to:						
	Dead Load	Live Load	Wind Load	EQ Load	Snow Load	Roof Live Load	EQ+W
N1	0.199	0.24	-44.981	-72.07	0.001	0	-117.05
N2	-0.199	-0.24	-0.119	-0.19	-0.001	0	-0.31

Joint	Joint Reactions in Y-direction (k) due to:						
	Dead Load	Live Load	Wind Load	EQ Load	Snow Load	Roof Live Load	EQ+W
N1	11.393	9.079	-48.935	-91.049	2.025	0.9	-139.98
N2	9.627	8.921	48.935	91.049	2.025	0.9	139.98

Joint	Joint Reactions in MZ-direction (k) due to:						
	Dead Load	Live Load	Wind Load	EQ Load	Snow Load	Roof Live Load	EQ+W
N1	0.215	1.067	2.16	3.499	-0.006	-0.003	5.66
N2	1.078	1.29	1.109	1.803	0.006	0.003	2.91

Joint	Deflections due to:						H/400
	Wind Load		Earthquake Load		Total Lateral Loads		
	X [in]	Y [in]	X [in]	Y [in]	X [in]	Y [in]	X [in]
N1	0.00	0.00	0.00	0.00	0.00	0.00	0.00
N2	0.00	0.00	0.00	0.00	0.00	0.00	0.00
N3	0.15	0.02	0.24	0.05	0.39	0.07	0.42
N4	0.09	-0.04	0.16	-0.08	0.25	-0.12	0.42
N5	0.28	0.03	0.48	0.06	0.76	0.09	0.84
N6	0.24	-0.06	0.41	-0.12	0.64	-0.18	0.84
N7	0.38	0.03	0.73	0.06	1.12	0.09	1.40
N8	0.36	-0.07	0.68	-0.14	1.04	-0.21	1.40

Key
Values used in Column Anlaysis
Maximum compressive values in each brace

Appendix R: Lateral Loads Column Analysis

Hand Calculations

Column Analysis of Braced Frame

Load combination $U = 1.2D + 1.0W + 0.5L + 0.5S$

Member to analyze: M1, column size: HSS 5×5×1/2

$L = 14$ ft $\quad I_x = 26$ in^4 $\quad A = 7.88$ in^2

$P_{nt} = 1.2D + 1.0W + 0.5L + 0.5S$

$P_{nt} = 1.2(10.03) + 1.0(25.046) + 0.5(8.847) + 0.5(2.024) = 42.52k$

$M_{nt} = 1.2D + 1.0W + 0.5L + 0.5S$

$M_{nt} = 1.2(2.109) + 1.178 + 0.5(2.627) + 0.5(0.012) = 5.03k$

$\Sigma H = 0.136 + 0.216 = 0.352\,k$

$\Delta H = 1.12$

Amplifier B1

$M1 = 1.2(\underset{moment}{DL\ min}) + 0.5(\underset{moment}{LL\ min}) + 0.5(\underset{moment}{SL\ min})$

$M1 = 1.2(1.62) + 0.5(1.29) + 0.5(0.01) = 1.88$

$M_2 = M_{nt}$ (since no M_{lt}) $\therefore M_2 = 5.03$

Reverse curvature

$P_{e1} = \cancel{264}\ \dfrac{\pi^2 EI}{(KL)^2} = \dfrac{\pi^2(29000)(26)}{(1 \times 14 \times 12)^2} = 264\,k$

$C_m = 0.6 + 0.4\left(\dfrac{M_1}{M_2}\right) = 0.6 + 0.4\left(\dfrac{1.88}{5.03}\right) = 0.75$

$P_r = P_{nt} = 42.52k$

$B1 = \dfrac{C_m}{\left(1 - \frac{P_r}{P_{e1}}\right)} = \dfrac{0.75}{\left(1 - \frac{42.52}{264}\right)} = 0.89$ since $B1 < 1$ use $B1 = 1$

Req. 2nd order values

$P_r = B_1 P_r = (1)(42.52) = 42.52\,k$

$M_{rx} = B_1 M_r = (1)(5.03) = 5.03\,k\text{-ft}$

$P_c = 173\,k$

$P_r/P_c = \dfrac{42.52}{\cancel{57.03}\ 173} = 0.246$ since $\dfrac{P_r}{P_c} > 0.2$ use interaction equation H1-1a

$\dfrac{P_r}{2P_c} + \left(\dfrac{M_{rx}}{M_{cx}} + \dfrac{M_{ry}}{M_{cy}}\right) = \dfrac{42.52}{2(173)} + \left(\dfrac{5.03}{45} + \dfrac{0}{45}\right) = 0.23 < 1$ good

Sample Column Analysis Spreadsheets

BF-1 Column Analysis of M1 and M4 with Lateral Loads applied on North Side

From ASCE 7-10

LOAD COMBINATIONS	
LC 4:	U = 1.2D + 1.0W + 0.5L + 0.5S
LC 5:	U = 1.2D +1.0E +0.5L + 0.2S
LC 7:	U = 0.9D + 1.0E

Frame/Member		BF-1/M1			BF-1/M4		
Load Combination		LC 4	LC 5	LC 6	LC 4	LC 5	LC 6
Columns Size		HSS 5x5x1/2	HSS 5x5x1/2	HSS 5x5x1/2	HSS 5x5x1/2	HSS 5x5x1/2	HSS 5x5x1/2
Column Values	L (ft)	14	14	14	14	14	14
	Ix (in^4)	26	26	26	26	26	26
	A (in^2)	7.88	7.88	7.88	7.88	7.88	7.88
Floor Values	Total Dead Load	10.45	10.45	10.45	10.45	10.45	10.45
	Total Live Load	9.23	9.23	9.23	9.23	9.23	9.23
Axial Forces	DL (k)	10.03	10.03	10.03	9.227	10.03	10.03
	LL (k)	8.847	8.847	8.847	8.921	8.847	8.847
	WL (k)	25.046	25.046	25.046	48.967	25.046	25.046
	EQ L (k)	52.772	52.772	52.772	91.099	52.772	52.772
	SL (k)	2.024	2.024	2.024	2.024	2.024	2.024
	Lr L (k)	0.899	0.899	0.899	0.899	0.899	0.899
Moment Forces	DL (k-ft)	2.109	2.109	2.109	2.105	1.043	1.043
	LL (k-ft)	2.627	2.627	2.627	2.55	1.291	1.291
	WL (k-ft)	1.178	1.178	1.178	0.763	1.178	1.178
	EQ L (k-ft)	1.898	1.898	1.898	1.242	1.898	1.898
	SL (k-ft)	0.012	0.012	0.012	0.008	0.011	0.011
	Lr L (k-ft)	0.006	0.006	0.006	0.004	0.005	0.005
Column Load effects	Pnt (k)	42.52	69.64	61.799	65.51	69.64	61.799
	Mnt (k-ft)	5.03	5.7447	3.7961	4.57	3.7973	2.8367
Lateral Deflection	ΣH (k)	0.352	0.352	0.352	0.409	0.409	0.409
	ΔH (in)	1.12	1.12	1.12	1.12	1.12	1.12
Amplifier B1	DL min moment (k-ft)	1.02	1.02	1.02	1.08	1.08	1.08
	LL min moment (k-ft)	1.29	1.29	1.29	1.29	1.29	1.29
	SL min moment (k-ft)	0.01	0.01	0.01	0.01	0.01	0.01
	M1 (k-ft)	1.88	1.88	1.88	1.94	1.94	1.94
	M2 (k-ft)	5.03	5.74	3.80	4.57	3.80	2.84
	Single or Reverse?	Reverse	Reverse	Reverse	Reverse	Reverse	Reverse
	Pe1 (k)	264	264	264	264	264	264
	Cm	0.75	0.73	0.80	0.77	0.80	0.87
	Pr (k)	42.52	69.64	61.80	65.51	69.64	61.80
	B1	0.89	0.99	1.04	1.02	1.09	1.14
	B1 > 1?	No	No	Yes	Yes	Yes	Yes
	B1	1.00	1.00	1.04	1.02	1.09	1.14
Req. 2nd-order strength values	Pr (k)	42.52	69.64	64.39	67.13	76.14	70.54
	Mrx (k-ft)	5.03	5.74	3.96	4.68	4.15	3.24
Interaction Equations	Pc (k)	173	173	173	173	173	173
	Pr/Pc	0.246	0.403	0.372	0.388	0.440	0.408
	Use Equation	H1-1a	H1-1a	H1-1a	H1-1a	H1-1a	H1-1a
	Mry (k-ft)	0	0	0	0	0	0
	Mcy (k-ft)	45	45	45	45	45	45
	Mrx (k-ft)	5.03	5.74	3.96	4.68	4.15	3.24
	Mcx (k-ft)	45	45	45	45	45	45
	Value	0.23	0.52	0.45	0.30	0.52	0.47
	Value < 1?	Yes	Yes	Yes	Yes	Yes	Yes

Key:
Obtained from AISC Manual
Calculated based on other values
Obtained from Spreadsheets

Appendix S: Cost Estimate Data of Current Design Spreadsheets

Beam Type	Weight (lb/ft)	Total Quantity (ft)	Total Weight (lbs)	Unit Bare Costs ($/ft) Material	Labor	Equipment	Total	Total Incl O&P	Total Bare Costs Material	Labor	Equipment	Total	Total Unit Bare Costs
W6X12	12	64	768	$ 14.84	$ 4.42	$ 2.70	$ 21.96	$ 24.80	$ 949.76	$ 282.88	$ 172.80	$ 1,405.44	$ 1.83
W8X10	10	304	3035	$ 12.40	$ 4.42	$ 2.70	$ 19.52	$ 24.00	$ 3,763.40	$ 1,341.47	$ 819.45	$ 5,924.32	$ 1.95
W8X18	18	56	1013	$ 22.29	$ 4.42	$ 2.70	$ 29.41	$ 33.43	$ 1,253.57	$ 248.63	$ 151.88	$ 1,654.07	$ 1.63
W8X24	24	172	4124	$ 29.50	$ 4.82	$ 2.95	$ 37.27	$ 44.00	$ 5,069.58	$ 828.32	$ 506.96	$ 6,404.85	$ 1.55
W8X31	31	119	3699	$ 38.50	$ 4.82	$ 2.95	$ 46.27	$ 53.50	$ 4,594.21	$ 575.17	$ 352.02	$ 5,521.40	$ 1.49
W10X12	12	564	6764	$ 14.85	$ 4.42	$ 2.70	$ 21.97	$ 27.00	$ 8,369.91	$ 2,491.24	$ 1,521.80	$ 12,382.95	$ 1.83
W12X14	14	2227	31184	$ 17.33	$ 3.01	$ 1.84	$ 22.18	$ 25.38	$ 38,590.22	$ 6,704.56	$ 4,098.47	$ 49,393.26	$ 1.58
W12X16	16	704	11257	$ 19.80	$ 3.01	$ 1.84	$ 24.65	$ 29.00	$ 13,930.69	$ 2,117.75	$ 1,294.57	$ 17,343.00	$ 1.54
W12X19	19	210	3997	$ 23.32	$ 3.01	$ 1.84	$ 28.17	$ 31.95	$ 4,905.68	$ 633.24	$ 387.10	$ 5,926.02	$ 1.48
W12X40	40	140	5619	$ 49.60	$ 3.27	$ 2.00	$ 54.87	$ 61.20	$ 6,967.81	$ 459.37	$ 280.96	$ 7,708.14	$ 1.37
W14X22	22	1070	23541	$ 27.08	$ 2.68	$ 1.64	$ 31.40	$ 35.54	$ 28,973.12	$ 2,867.68	$ 1,754.85	$ 33,595.65	$ 1.43
W14X30	30	68	2050	$ 37.00	$ 2.95	$ 1.80	$ 41.75	$ 48.00	$ 2,528.21	$ 201.57	$ 122.99	$ 2,852.78	$ 1.39
W16X26	26	1556	40446	$ 32.00	$ 2.65	$ 1.62	$ 36.27	$ 42.00	$ 49,780.16	$ 4,122.42	$ 2,520.12	$ 56,422.70	$ 1.40
W16X31	31	2960	91750	$ 38.50	$ 2.95	$ 1.80	$ 43.25	$ 49.00	$ 113,947.30	$ 8,731.03	$ 5,327.41	$ 128,005.73	$ 1.40
W16X36	36	150	5400	$ 44.55	$ 3.32	$ 1.80	$ 49.67	$ 56.25	$ 6,682.50	$ 498.00	$ 270.00	$ 7,450.50	$ 1.38
W16X40	40	352	14068	$ 49.50	$ 3.32	$ 2.03	$ 54.85	$ 62.50	$ 17,409.65	$ 1,167.68	$ 713.97	$ 19,291.29	$ 1.37
W16X57	57	30	1710	$ 70.61	$ 3.49	$ 2.13	$ 76.23	$ 84.65	$ 2,118.36	$ 104.70	$ 63.90	$ 2,286.96	$ 1.34
W18X35	35	370	12966	$ 43.50	$ 3.99	$ 1.80	$ 49.29	$ 56.50	$ 16,114.58	$ 1,478.10	$ 666.81	$ 18,259.48	$ 1.41
W18X40	40	331	13251	$ 49.50	$ 3.99	$ 1.80	$ 55.29	$ 63.50	$ 16,398.36	$ 1,321.81	$ 596.30	$ 18,316.47	$ 1.38
W18X46	46	344	15824	$ 57.00	$ 3.99	$ 1.80	$ 62.79	$ 71.50	$ 19,608.00	$ 1,372.56	$ 619.20	$ 21,599.76	$ 1.37
W18X50	50	90	4500	$ 62.00	$ 4.20	$ 1.90	$ 68.10	$ 77.50	$ 5,580.00	$ 378.00	$ 171.00	$ 6,129.00	$ 1.36
W18X55	55	40	2200	$ 68.00	$ 4.20	$ 1.90	$ 74.10	$ 84.50	$ 2,720.00	$ 168.00	$ 76.00	$ 2,964.00	$ 1.35
W21X44	44	894	39321	$ 54.50	$ 0.36	$ 1.63	$ 56.49	$ 68.00	$ 48,705.02	$ 321.72	$ 1,456.68	$ 50,483.42	$ 1.28
W21X55	55	87	4767	$ 67.86	$ 3.70	$ 1.63	$ 73.19	$ 82.06	$ 5,881.68	$ 320.68	$ 141.27	$ 6,343.63	$ 1.33
W21X62	62	225	13930	$ 76.50	$ 3.70	$ 1.67	$ 81.87	$ 92.50	$ 17,187.26	$ 831.28	$ 375.20	$ 18,393.73	$ 1.32
W24X55	55	818	44976	$ 68.00	$ 3.45	$ 1.56	$ 73.01	$ 82.50	$ 55,607.00	$ 2,821.24	$ 1,275.69	$ 59,703.93	$ 1.33
W24X62	62	40	2480	$ 76.50	$ 3.45	$ 1.56	$ 81.51	$ 92.00	$ 3,060.00	$ 138.00	$ 62.40	$ 3,260.40	$ 1.31
W24X68	68	40	2720	$ 84.00	$ 3.45	$ 1.56	$ 89.01	$ 100.00	$ 3,360.00	$ 138.00	$ 62.40	$ 3,560.40	$ 1.31
W24X76	76	120	9120	$ 94.00	$ 3.45	$ 1.56	$ 99.01	$ 11.00	$ 11,280.00	$ 414.00	$ 187.20	$ 11,881.20	$ 1.30
W24X84	84	40	3360	$ 104.00	$ 3.55	$ 1.60	$ 109.15	$ 122.00	$ 4,160.00	$ 142.00	$ 64.00	$ 4,366.00	$ 1.30
W24X94	94	40	3760	$ 116.00	$ 3.55	$ 1.60	$ 121.15	$ 136.00	$ 4,640.00	$ 142.00	$ 64.00	$ 4,846.00	$ 1.29
W24X117	117	80	9360	$ 145.00	$ 3.65	$ 1.65	$ 150.30	$ 167.00	$ 11,600.00	$ 292.00	$ 132.00	$ 12,024.00	$ 1.28
W30X90	90	40	3600	$ 111.82	$ 3.19	$ 1.44	$ 116.45	$ 129.09	$ 4,472.73	$ 127.60	$ 57.60	$ 4,657.93	$ 1.29
W30X108	108	51	5481	$ 134.00	$ 3.19	$ 1.44	$ 138.63	$ 154.00	$ 6,800.50	$ 161.89	$ 73.08	$ 7,035.47	$ 1.28
W36X135	135	40	5400	$ 167.00	$ 3.28	$ 1.48	$ 171.76	$ 191.00	$ 6,680.00	$ 131.20	$ 59.20	$ 6,870.40	$ 1.27
Total	-	14435	447441				TOTAL NATIONAL AVG. (11')		$ 553,689	$ 44,076	$ 26,499	$ 624,264	
							TOTAL BOSTON (11')		$ 566,424	$ 60,296	$ 36,251	$ 662,971	
							ROUNDED COST		$ 566,000.00	$ 60,000.00	$ 36,000.00	$ 663,000.00	

	Material	Installation	Total
Boston Location Factor	102.3	136.8	117.7

Key
Value was interpolated or estimated

Weight Breakdown of different structural members

HSS Beams			
Type	**Total Quantity (LF)**	**Weight (lb/ft)**	**Total Weight (lbs)**
HSS5X5X5/16	46.56	19	885
HSS6X4X5/16	55	19.1	1051
HSS6X6X3/8	24.33	27.4	667
HSS7X7X1/2	44.49	41.9	1864
HSS8X6X5/16	28.5	27.6	787
HSS8X8X5/16	23.67	31.8	753
HSS12X12X5/16	246.17	48.8	12013
HSS12X6X5/16	101.88	36.1	3678
HSS14X10X5/16	290.5	48.9	14205
HSS16X8X5/16	160	48.9	7824
		TOTAL	43727

HSS Bracing			
Type	**Total Quantity (LF)**	**Weight (lb/ft)**	**Total Weight (lbs)**
HSS5X5X5/16	83.84	19	1593
HSS6X6X3/8	254.16	27.4	6964
HSS7X7X1/2	425.83	41.9	17842
HSS8X8X1/2	74.52	48.7	3629
HSS8X8X5/8	270.63	59.1	15994
		TOTAL	46022

Joist Girders			
Type	**Total Quantity (LF)**	**Weight (lb/ft)**	**Total Weight (lbs)**
36LH11	670.94	23	15432
		TOTAL	15432

*Note: In structural dwgs, only says 36LHSP1. Based on the length of the girder of almost 61ft, it was assumed the size of the girder was 36LH11, which was the biggest 36 joist girder possible for that length

L-Angle Members			
Type	**Total Quantity (LF)**	**Weight (lb/ft)**	**Total Weight (lbs)**
L3X3X1/4	12.33	4.9	60
L4X4X1/4	21.5	6.6	142
L4X4X5/16	25.29	8.2	207
L5X5X5/16	38.8	10.3	400
		TOTAL	809

HSS Columns			
Type	**Total Quantity (LF)**	**Weight (lb/ft)**	**Total Weight (lbs)**
HSS12X12X5/16	1075.33	48.8	52476
HSS6X6X1/2	79	35.1	2773
HSS6X6X5/16	381.21	23.3	8882
HSS8X8X1/2	150	48.7	7305
HSS8X8X5/16	1815.79	31.8	57742
		TOTAL	129178

Table below shows the approximate number of studs per beam

Family Type	Number of Beams	Approximate number of studs per beam	Total number of studs
W-Wide Flange: W10X12 Count	59	6	354
W-Wide Flange: W12X14 Count	162	12	1944
W-Wide Flange: W12X16 Count	26	14	364
W-Wide Flange: W12X19 Count	1	12	12
W-Wide Flange: W12X40 Count	5	10	50
W-Wide Flange: W14X22 Count	45	20	900
W-Wide Flange: W14X30 Count	1	16	16
W-Wide Flange: W16X26 Count	33	8	264
W-Wide Flange: W16X31 Count	107	16	1712
W-Wide Flange: W16X36 Count	3	20	60
W-Wide Flange: W16X40 Count	5	16	80
W-Wide Flange: W16X57 Count	1	18	18
W-Wide Flange: W18X35 Count	6	20	120
W-Wide Flange: W18X40 Count	11	30	330
W-Wide Flange: W18X46 Count	12	16	192
W-Wide Flange: W18X50 Count	3	26	78
W-Wide Flange: W18X55 Count	1	24	24
W-Wide Flange: W21X44 Count	21	30	630
W-Wide Flange: W21X55 Count	3	30	90
W-Wide Flange: W21X62 Count	9	30	270
W-Wide Flange: W24X117 Count	2	34	68
W-Wide Flange: W24X55 Count	12	20	240
W-Wide Flange: W24X62 Count	1	34	34
W-Wide Flange: W24X68 Count	1	28	28
W-Wide Flange: W24X76 Count	3	26	78
W-Wide Flange: W24X84 Count	1	30	30
W-Wide Flange: W24X94 Count	1	30	30
W-Wide Flange: W30X90 Count	1	28	28
		Total	8044
		Total Cost	19707.8

NOTE: For simplification, the largest stud size was chosen for all beams. Therefore, the stud cost would be an overestimate

Appendix T: Cost Estimate Data of Alternative Design Spreadsheets

Beam Type	Weight (lb/ft)	Total Quantity (ft)	Total Weight (lbs)	Unit Bare Costs ($/ft) Material	Labor	Equipment	Total	Total Incl O&P	Total Unit Bare Costs ($/ft) Material	Labor	Equipment	Total	Total Unit Bare Costs ($
W8X18	18	56.25	1012.5	$ 22.29	$ 4.42	$ 2.70	$ 29.41	$ 33.43	$ 1,253.57	$ 248.63	$ 151.88	$ 1,654.07	$ 1.63
W8X24	24	171.85	4124.4	$ 29.50	$ 4.82	$ 2.95	$ 37.27	$ 44.00	$ 5,069.58	$ 828.32	$ 506.96	$ 6,404.85	$ 1.55
W8X31	31	119.33	3699.23	$ 38.50	$ 4.82	$ 2.95	$ 46.27	$ 53.50	$ 4,594.21	$ 575.17	$ 352.02	$ 5,521.40	$ 1.49
W10X12	12	251.6	3019.2	$ 15.00	$ 4.42	$ 2.70	$ 22.12	$ 27.00	$ 3,774.00	$ 1,112.07	$ 679.32	$ 5,565.39	$ 1.84
W10X15	15	136.67	2050.05	$ 18.50	$ 4.42	$ 2.70	$ 25.62	$ 31.00	$ 2,528.40	$ 604.08	$ 369.01	$ 3,501.49	$ 1.71
W12X14	14	1105.59	15478.26	$ 17.50	$ 3.01	$ 1.84	$ 22.35	$ 25.38	$ 19,347.83	$ 3,327.83	$ 2,034.29	$ 24,709.94	$ 1.60
W12X16	16	179.46	2871.36	$ 19.80	$ 3.01	$ 1.84	$ 24.65	$ 29.00	$ 3,553.31	$ 540.17	$ 330.21	$ 4,423.69	$ 1.54
W12X19	19	20	380	$ 23.00	$ 3.01	$ 1.84	$ 27.85	$ 31.95	$ 460.00	$ 60.20	$ 36.80	$ 557.00	$ 1.47
W12X22	22	256.67	5646.74	$ 27.00	$ 3.01	$ 1.84	$ 31.85	$ 37.00	$ 6,930.09	$ 772.58	$ 472.27	$ 8,174.94	$ 1.45
W12X40	40	140.48	5619.2	$ 49.60	$ 2.83	$ 1.73	$ 54.16	$ 61.20	$ 6,967.81	$ 397.84	$ 242.75	$ 7,608.40	$ 1.35
W14X22	22	369.59	8130.98	$ 27.00	$ 2.68	$ 1.64	$ 31.32	$ 35.54	$ 9,978.93	$ 990.50	$ 606.13	$ 11,575.56	$ 1.42
W14X26	26	403	10478	$ 32.00	$ 2.68	$ 1.64	$ 36.32	$ 42.00	$ 12,896.00	$ 1,080.04	$ 660.92	$ 14,636.96	$ 1.40
W14X30	30	28.33	849.9	$ 37.00	$ 2.95	$ 1.80	$ 41.75	$ 48.00	$ 1,048.21	$ 83.57	$ 50.99	$ 1,182.78	$ 1.39
W16X26	26	512.67	13329.42	$ 32.00	$ 2.65	$ 1.62	$ 36.27	$ 42.00	$ 16,405.44	$ 1,358.58	$ 830.53	$ 18,594.54	$ 1.40
W16X31	31	2053.75	63666.25	$ 38.50	$ 2.95	$ 1.80	$ 43.25	$ 49.00	$ 79,069.38	$ 6,058.56	$ 3,696.75	$ 88,824.69	$ 1.40
W16X36	36	150	5400	$ 44.55	$ 3.32	$ 1.80	$ 49.67	$ 56.25	$ 6,682.50	$ 498.00	$ 270.00	$ 7,450.50	$ 1.38
W16X40	40	144.67	5786.8	$ 49.50	$ 3.32	$ 2.03	$ 54.85	$ 62.50	$ 7,161.17	$ 480.30	$ 293.68	$ 7,935.15	$ 1.37
W18X35	35	409.06	14317.1	$ 43.50	$ 3.99	$ 1.80	$ 49.29	$ 56.50	$ 17,794.11	$ 1,632.15	$ 736.31	$ 20,162.57	$ 1.41
W18X40	40	285.58	11423.2	$ 49.50	$ 3.99	$ 1.80	$ 55.29	$ 63.50	$ 14,136.21	$ 1,139.46	$ 514.04	$ 15,789.72	$ 1.38
W18X46	46	30	1380	$ 57.00	$ 3.99	$ 1.80	$ 62.79	$ 71.50	$ 1,710.00	$ 119.70	$ 54.00	$ 1,883.70	$ 1.37
W18X50	50	40	2000	$ 62.00	$ 4.20	$ 1.90	$ 68.10	$ 77.50	$ 2,480.00	$ 168.00	$ 76.00	$ 2,724.00	$ 1.36
W18X55	55	40	2200	$ 68.00	$ 4.20	$ 1.90	$ 74.10	$ 84.50	$ 2,720.00	$ 168.00	$ 76.00	$ 2,964.00	$ 1.35
W21X44	44	449.33	19770.52	$ 54.50	$ 0.36	$ 1.63	$ 56.49	$ 68.00	$ 24,488.49	$ 161.76	$ 732.41	$ 25,382.65	$ 1.28
W21X50	50	40	2000	$ 62.00	$ 3.60	$ 1.63	$ 67.23	$ 76.00	$ 2,480.00	$ 144.00	$ 65.20	$ 2,689.20	$ 1.34
W21X55	55	29.67	1631.85	$ 68.00	$ 3.70	$ 1.63	$ 73.33	$ 82.06	$ 2,017.56	$ 109.78	$ 48.36	$ 2,175.70	$ 1.33
W21X62	62	30	1860	$ 76.50	$ 3.70	$ 1.67	$ 81.87	$ 92.50	$ 2,295.00	$ 111.00	$ 50.10	$ 2,456.10	$ 1.32
W24X55	55	306.58	16861.9	$ 68.00	$ 3.45	$ 1.56	$ 73.01	$ 82.50	$ 20,847.44	$ 1,057.70	$ 478.26	$ 22,383.41	$ 1.33
W24X62	62	124	7688	$ 76.50	$ 3.45	$ 1.56	$ 81.51	$ 92.00	$ 9,486.00	$ 427.80	$ 193.44	$ 10,107.24	$ 1.31
W24X68	68	40	2720	$ 84.00	$ 3.45	$ 1.56	$ 89.01	$ 100.00	$ 3,360.00	$ 138.00	$ 62.40	$ 3,560.40	$ 1.31
W24X84	84	40	3360	$ 104.00	$ 3.55	$ 1.60	$ 109.15	$ 122.00	$ 4,160.00	$ 142.00	$ 64.00	$ 4,366.00	$ 1.30
W24X94	94	40	3760	$ 116.00	$ 3.55	$ 1.60	$ 121.15	$ 136.00	$ 4,640.00	$ 142.00	$ 64.00	$ 4,846.00	$ 1.29
W24X117	117	80	9360	$ 145.00	$ 3.65	$ 1.65	$ 150.30	$ 167.00	$ 11,600.00	$ 292.00	$ 132.00	$ 12,024.00	$ 1.28
W27X84	84	80	6720	$ 104.00	$ 3.22	$ 1.45	$ 108.67	$ 121.00	$ 8,320.00	$ 257.60	$ 116.00	$ 8,693.60	$ 1.29
W27X102	102	80	8160	$ 126.00	$ 3.33	$ 1.51	$ 130.84	$ 145.00	$ 10,080.00	$ 266.40	$ 120.80	$ 10,467.20	$ 1.28
W30X90	90	40	3600	$ 112.00	$ 3.19	$ 1.44	$ 116.63	$ 152.73	$ 4,480.00	$ 127.60	$ 57.60	$ 4,665.20	$ 1.30
W30X108	108	50.75	5481	$ 134.00	$ 3.19	$ 1.44	$ 138.63	$ 154.00	$ 6,800.50	$ 161.89	$ 73.08	$ 7,035.47	$ 1.28
W33x118	118	80	9440	$ 146.00	$ 3.26	$ 1.47	$ 150.73	$ 168.00	$ 11,680.00	$ 260.80	$ 117.60	$ 12,058.40	$ 1.28
W36X135	135	40	5400	$ 167.00	$ 3.28	$ 1.48	$ 171.76	$ 191.00	$ 6,680.00	$ 131.20	$ 59.20	$ 6,870.40	$ 1.27
Total	-	8455	290676					TOTAL NATIONAL AVG. (11')	$ 359,976	$ 26,175	$ 15,475	$ 401,626	
								TOTAL BOSTON (11')	$ 368,255	$ 35,808	$ 21,170	$ 425,233	
								ROUNDED COST	$ 368,000.00	$ 36,000.00	$ 21,000.00	$ 425,000.00	

	Material	Installation	Total
Boston Location Factor	102.3	136.8	117.7

Key
Value was interpolated or estimated

HSS Beams			
Type	**Total Quantity (LF)**	**Weight (lb/ft)**	**Total Weight (lbs)**
HSS6X4X5/16	5.5	19.1	105
HSS8X6X5/16	7.9	27.6	218
HSS8X8X5/16	11.83	31.8	376
HSS12X12X5/16	22.38	48.8	1092
HSS12X6X5/16	22.16	36.1	800
HSS14X10X5/16	22.35	48.9	1093
		TOTAL	3684

HSS Bracing			
Type	**Total Quantity (LF)**	**Weight (lb/ft)**	**Total Weight (lbs)**
HSS5X5X1/4	80.94	15.6	1263
HSS5X5X5/16	52.6	19	999
HSS7X7X1/2	409.23	41.9	17147
HSS7X7X5/8	427.48	50.6	21630
		TOTAL	41039

Joist Girders			
Type	**Total Quantity (LF)**	**Weight (lb/ft)**	**Total Weight (lbs)**
36LH11	670.94	23	15432
40LH15	1492.41	42	62681
44LH17	2746.13	47	129068
		TOTAL	207181

*Note: In structural dwgs, only says 36LHSP1. Based on the length of the girder of almost 61ft, it was assumed the size of the girder was 36LH11, which was the biggest 36 joist girder possible for that length

L-Angle Members			
Type	**Total Quantity (LF)**	**Weight (lb/ft)**	**Total Weight (lbs)**
L3X3X1/4	12.33	4.9	60
L4X4X1/4	21.5	6.6	142
L4X4X5/16	25.29	8.2	207
L5X5X5/16	38.8	10.3	400
		TOTAL	809

HSS Columns			
Type	**Total Quantity (LF)**	**Weight (lb/ft)**	**Total Weight (lbs)**
HSS5X5X1/2	558.67	28.3	15810
HSS6X6X1/2	802.67	35.1	28174
HSS6X6X5/16	318.71	23.3	7426
HSS8X8X5/16	365.38	31.8	11619
HSS12X12X5/8	372.75	93.1	34703
HSS12X12X5/16	721.17	48.8	35193
		TOTAL	132925

Type	Total Number of Studs
W10X12	228
W10X15	64
W12X14	792
W12X16	20
W12X19	46
W12X22	100
W12X40	400
W14X22	150
W14X26	584
W16X26	396
W16X31	1966
W16X40	62
W18X35	332
W18X40	264
W18X50	26
W18X55	24
W21X44	184
W21X50	46
W21X55	46
W24X117	64
W24X55	36
W24X62	232
W24X68	40
W24X84	30
W24X94	30
W27X102	128
W27X84	60
W30X90	28
W33 x 118	140
TOTAL	**6518**

Appendix U: Cost Estimation Spreadsheets for Alternate Design Strategies

Beam Grid System	
W-Wide Flange: W44X335	
total length (ft)	720
total weight (lb)	241200
total cost	$ 364,809.60
total factored cost	$ 430,475.33
total studs	0
total cost with studs	$ 430,475.33

W-Wide Flange: W36X135	
total length (ft)	1800
total weight (lb)	243000
total cost	$ 135,344.00
total factored cost	$ 364,818.24
total studs	5040
total cost with studs	$ 369,858.24

HSS-Column: HSS12X12X5/8	
total length (ft)	192
total cost	$ 31,116.00
total factored cost	$ 76,234.20
total studs	0
total cost with studs	$ 76,234.20

Location Factor:	1.18
cost per stud:	2.45
Cost / square-foot:	$ 81.84
TOTAL	$ 883,875.77

Staggered Truss System	
W-Wide Flange: W10X39	
total length (ft)	1070.083333
total weight (lb)	41733.25
total cost	$ 52,187.96
total factored cost	$ 61,581.80
total studs	0
total cost with studs	$ 61,581.80

W-Wide Flange: W10X54	
total length (ft)	535.0416667
total weight (lb)	28892.25
total cost	$ 45,457.14
total factored cost	$ 53,639.43
total studs	0
total cost with studs	$ 53,639.43

W-Wide Flange: W18X40	
total length (ft)	1260
total weight (lb)	28892.25
total cost	$ 43,783.43
total factored cost	$ 51,664.45
total studs	756
total cost with studs	$ 53,516.65

W-Wide Flange: W40X183	
total length (ft)	300
total weight (lb)	915
total cost	$ 120,000.00
total factored cost	$ 141,600.00
total studs	570
total cost with studs	$ 142,996.50

HSS: HSS10x10x1/2	
total length (ft)	192
total cost	$ 15,768.00
total factored cost	$ 18,606.24
total studs	0
total cost with studs	$ 18,606.24

Location Factor:	**1.18**
cost per stud:	**2.45**
Cost / square-foot:	**$ 34.51**
TOTAL	**$ 372,724.41**