Mental Health in a New WPI Residence Hall

A Major Qualifying Project Report
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This report represents the work of one or more WPI undergraduate students submitted to the faculty as evidence of completion of a degree requirement. WPI routinely publishes these reports on the web without editorial or peer review.

Abstract

Worcester Polytechnic Institute (WPI) has seen its undergraduate student population increase in recent years while on-campus housing options remain limited for upperclassmen. Like many educational institutions, WPI's community is challenged with transitioning from an isolated campus environment to a more personal, pre-pandemic scene. With students witnessing fellow classmates' mental health suffer, something must be done to prevent feelings of isolation and stress. This project proposes a new residence hall be built on WPI's campus that provides adequate social spaces and amenities for students, while also maximizing WPI's investment through modular construction methods. Structural and architectural components of this residence hall were designed with social spaces playing an important role. Furthermore, the mechanical systems of the building incorporate sustainable, renewable energy. If WPI adopts this concept of a residence hall, undergraduates will benefit from increased housing options and community spaces while the university can maximize its investment through cost-efficient construction methods.

Executive Summary

Through modular construction, Worcester Polytechnic Institute (WPI) can adopt a new residence hall that fosters student mental well-being with a building design that contains mental health considerations. The methodology, results, deliverables, and conclusions are provided in this report.

Methods

After extensive research, several design processes were developed to create a methodology. Architectural aspects of the residence hall were designed to help promote mental health and wellness for the students within. The determination of what exactly can promote mental health is analyzed in this section, via published works and collected data. This led the team to specific architectural design choices as discussed in the deliverables. Once the architectural plan was set, the structural and sustainability elements of the residence hall were designed accordingly. The design of these components considered local site conditions and specified minimum loadings in accordance with ASCE 7 and ACI 318 14. Virtual models of these components were developed along with hand calculations to support and confirm all the design choices.

Deliverables

The deliverables consist of handwritten design calculations for the structural columns, beams, and foundation. This section also contains the software outputs such as the project schedule, money timeline graph, structural beam design, architectural design, dynamic analysis, floor plans, and renderings. A project risk assessment along with energy costs and calculations are also deliverables included in this section. The project cost economics including the project cost of $44,382,194, a timeline of 201 days, and a return on investment of about 7 years are also included in the deliverables. Each of the mentioned items make up the physical products of the project and are what the results are based on.

Results

The first major takeaway from this project is that using modular construction techniques, this residence hall will be built for less than traditionally constructed buildings, while still offering lots of sustainability. Along with the savings from the construction, money will be pumped and circulated through the local economy in Worcester as there will be nearly 500 residents, which is beneficial not only for WPI but the city of Worcester itself. The residence hall will also feature amenities that will reduce stress and anxiety for those living inside, promoting social interaction while also giving a feel of independent adult living.

Conclusions

This project was based on two different theses. First, methods of modular construction led to reduced construction costs, a shorter project timeline, and a quicker return on investment. Second, the consideration of mental health is included in the design of the residence hall to increase student well-being, sense of community, social connectedness, and relaxation. After analyzing the deliverables, considering the results of the project and their impact it was determined both thesis statements are true.

Introduction

With an increasing student population and limited space on campus, Worcester Polytechnic Institute (WPI) is faced with the unique challenge of presenting appealing on-campus living options for enrolled students, particularly after the completion of their first year. Many upperclassmen are displaced to off-campus housing in apartments or fraternity and sorority houses since the process of finding suitable housing on campus can be difficult and unpredictable. Furthermore, with recent demands for action regarding students' mental health, WPI needs a living space that houses a large community of upperclassmen while also providing services that make living as a community member more enjoyable.

The goal of this project is to design a residence hall for upperclassmen students that maximizes the capacity of the building space while also focusing heavily on community interaction, mental health focused amenities, and overall quality of living conditions to promote positive mental health on campus. The new design of the building should appeal to both prospective residents (students) as well as WPI administration. By providing more beds for students, the university should benefit financially while also being able to promote a positive, safe space for students to gather and be social outside of classes.

This project strives to better not only the community but surrounding areas as well. When evaluating design options and the existing location, several aspects must be considered. Therefore, the design must preserve the existing landscape and body of water, Salisbury Pond, currently located adjacent to the proposed building site: WPI's Townhouse property. In consideration and by request of neighboring residents, the architectural design must also match the aesthetic of other houses and buildings in the area. Lastly, interior design will play a paramount role to create a residence that will foster a safe space for students and improve mental health. Likewise, sustainable and efficient usage of energy must also be taken into account while constructing a new building in the Worcester community. Renewable energy and sustainability will be incorporated into this project to minimize the building's environmental footprint, specifically through roof-mounted solar panels.

This project's main deliverable is the design of a modularly constructed residence hall with other traditionally constructed components, as well as the construction plan, phasing, cost, schedule, and process. The design and construction outline may be used as an inspiration for WPI in future years to accommodate the increasing student population. The results include an architectural design of the building with floor plans and supplemental site plans involving a parking lot and landscaping components. Alongside the rendered model of the building will be a design plan for the main, prefabricated (or modular) portion of the building, identified as a "mod" or "mods". This includes the structural steel design of the mods, concrete floor slabs, and the connections from one mod to another. Furthermore, the design of the building's shear wall and concrete foundation has been included, which will be traditionally constructed components.

Furthermore, the construction plan, project schedule, and costs have all been precisely estimated following the modular and traditionally built portions of the building and will be paired with the design of the building as another main deliverable. Using prefabrication methods and modular construction, aspects including the project's timeline and on-site costs have been significantly reduced to maximize the university's potential investment and minimize risk involving such large-scale construction projects. Finally, a dynamic analysis and an overall plan for sustainable systems to help power and heat the building that has been created. Mechanical components that will be used to achieve this include solar panels and hydronic heating that were designed for this prospective residence hall.

If WPI adopts a new, sustainable residence hall to accommodate its increasing student population, the use of modular construction will significantly reduce financial and logistical risks of construction and provide a faster, more efficient project timeline. Furthermore, the incorporation of mental health considerations and amenities into a new residence hall will allow WPI’s student population to socialize and relax, all while providing options for non-first-year students with limited housing choices outside of other campus residences.

Background

This section begins with discussing WPI’s present-day situation regarding housing, both on and off campus, while also exploring the university's need for a social residence hall that considers student mental health. On the other hand, various components of engineering and designing a residence hall will each be entertained and explained. Applicable topics regarding construction project management (CPM) will be discussed as well, to introduce what must be prepared for this report. Other design aspects regarding building a residence hall that will supplement these sections include sustainable energy, heating and ventilation, and building limitations.

WPI’s Current Housing Situation

Worcester Polytechnic Institute (WPI) is a university that has grown and transformed increasingly since its inception in 1865. As a school with a relatively small campus, WPI must make efficient use of its available space despite an increasing student population. One crucial issue that has become ever more evident in recent years is the university’s lack of housing, particularly on campus. Despite recent efforts by WPI to subdue this issue by renting a hotel and overtaking neighboring universities' housing to provide more living space for students, WPI is still struggling to comfortably house all students, more specifically upperclassmen. For most, residence halls within proximity to campus are popular living options for first and even second-year students. The Morgan, Daniels, Riley, Institute, and Founders Halls traditionally house students during their first year (‘First Year’) at WPI. In an email communication with Amy Beth Laythe of WPI Residential services, it was found that, on average, 97% of first-year students live in provided residence halls on campus. When expanded to the entire undergraduate student body, only 50% of students live on campus (excluding fraternity and sorority houses). This leaves over 40% of the undergraduate class to find housing off campus in apartments or to commute from home if nearby. With the number of students at WPI steadily increasing, off-campus living at nearby apartments has become more competitive and limited as more students are vying to live in a set number of apartments. Although approximately 500 beds have been added to the campus since 2013, the student population has increased by about 16% (*Enrollment by WPI Institutional Research*, 2023). If the upward trend in population continues in the years to come, housing options will continue to be even more restricted for upper-class undergraduates. (A. Laythe, personal communication, January 24, 2023). See *Figure 1* below:

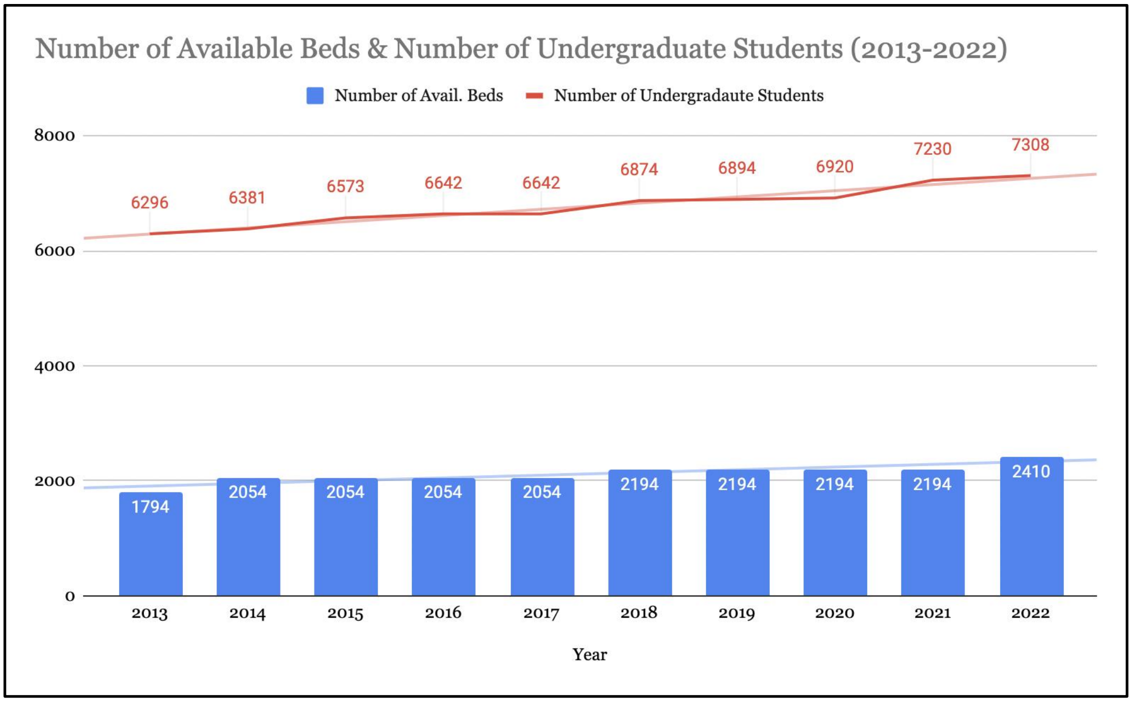


Figure 1: WPI student enrollment and available beds in university-provided residence halls and apartments, from 2013 to 2022.

When trying to find housing, upperclassmen are often left with limited options, especially amongst on-campus residences. With minimal choices, students may feel obligated to hastily pick a living situation that is less than ideal for their personal preferences and/or needs. Whether it be a further distance from campus buildings, a higher cost in exchange for convenience, or simply a living space that suits a resident's preferred living style, WPI students deserve more options and variety when it comes to on-campus housing. Any solution to WPI's on-campus housing shortage must appeal to both the student population as well as the school's administration and staff.

The Importance of Mental Health on College Campuses

The world is not yet far removed from the lasting profound societal impacts of the COVID-19 global pandemic. Regardless of demographics, people worldwide were affected in some form or another by the health and safety precautions required during such a volatile time as a pandemic. Broadly speaking, one population subset that experienced extreme consequences from the COVID-19 outbreak was college students. Students were forced to adjust quickly to different isolated learning, living, and social environments. Directly resulting from the social isolation safety precautions, interaction among the university community was significantly hindered. As mentioned above, this was mainly due to isolation and social distancing practices to minimize contact and disease transmission. While these circumstances were not ideal, students felt the impacts of isolation, and their mental health was negatively affected in turn (Ewing et. al., 2021, p. 20). Social presence among students is vitally important to their lived experience at a university. However, student-to-student interaction can also have drastic effects on learning ability as well as mental health (Chang et. al., 2022). Specifically, in today's mix of online and in-person learning, it has been discussed that:

> "Research has shown that high levels of social presence may promote satisfaction, academic self-efficacy, self-regulated learning ability and learning performance (Lim et al., 2021)" (Chang et. al., 2022, p. 516).

All things considered, social presence and student interaction directly impact a student's well-being on a college campus.

In response to such issues involving mental health and college students, social interaction and positive spaces must be encouraged in everyday student life. And one area that can make a positive difference in overall student wellness is the space in which they live. A residence hall provided by the university could create a positive impact on a student's college experience and promote student wellness and interaction amongst their peers. Gina Harris (2022) discusses the importance of mental health when considering the design of a residence hall, "According to the World Health Organization, mental health promotion should include actions that create living conditions and environments that support mental health and allow people to adopt and maintain healthy lifestyles." To specifically design a space that promotes such values, certain details and aspects must be included in the design. Harris further describes in her article, "Designing with Mental Health in Mind", that other architects in this field provide potential options to be implemented in a residence hall. Architect and author, Ben Channon, outlines several components to promoting mental well-being, including ample quality and quantity of lighting for each space, involvement of plants, water, and wildlife, selective color choice, calm spaces for comfort and leisure, open (or airy) spaces, and an overall sense of home (Channon, 2019). In an interview with Paula Fitzpatrick, WPI's Director of the Center for Well-Being, she echoes many of the same sentiments when it comes to striving for positive sociability in university buildings. Dr. Fitzpatrick stressed a need for common spaces both inside and outside of a student's living quarters such as seating areas and common rooms that allow for a congregation of members of the community. Visibility to the outdoors and color choice also appear to be areas of interest that Dr. Fitzpatrick described when conversing about mental health. Curved walls and large glass windows were also suggested as potential design implementations by Dr. Fitzpatrick (Fitzpatrick, personal communication, 2022).

WPI is one university that has felt the underlying ramifications of suffering student mental health and several tragedies involving student suicide and depression have rocked the school's community in recent years. Whether it be an overwhelming sense of pressure at such a rigorous institution or a lack of social interaction and positive spaces, WPI's student population would benefit from additional consideration for mental health issues. Several student accounts express the feeling of uncertainty and grief for their fellow classmates (Dalmia, 2022) and different groups on campus have organized and rallied around the cause for action on the university's behalf to combat depression and feelings of being alone (*WPI Students Rally for Mental Health Awareness,* 2021). Whether it be a WPI or any university throughout the world, maintaining student mental health is a priority and must be considered for residence halls or buildings alike.

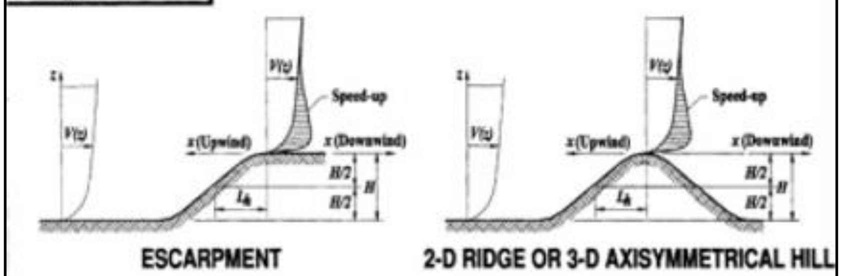
Building Materials

Structural Steel:

Structural steel has been cemented as modern material for large superstructures over the last century. The strength capacity for steel is very high for its respective weight, which in turn makes it an ideal construction material. Due to its lack of high compressive strength, steel is often used in tandem with concrete to form a phenomenon known as composite action, as concrete in turn has immense compressive strength. This material combination of concrete and steel, in turn, is both strong under tension and compression creating an ideal building material for most scenarios. (Cruickshank, 2016). In the United States, ASTM (American Society for Testing and Materials) is used to regulate and unify standards regarding the grade of steel. These standards set guidelines for classifying, evaluating, and specifying the material, chemical, mechanical, and metallurgical properties of the different types of steels (ASTM, 2022). Two popular grades of steel include ASTM A36 and ASTM A576. They are commonly used for the frame of larger structures (Leeco, 2020). Structural Steel is widely used in the New England Area due to its high resistance to weather conditions. Although steel has high inherent resistivity to the elements, primers are normally applied to reduce the corrosion that may occur (ScienceDirect, 2015).

Loading (from ASCE 7):

Structural steel is a system of columns and beams designed to hold building-specific loads and materials (such as concrete for the flooring) as required by design and specifications. There are several different types of loading, in both the lateral direction and vertical direction (which are also called gravity loads) that must be considered when designing structural steel. Earthquake and wind loads are the two lateral loads that the residence hall was designed to withstand, and these loads are calculated from different factors that are taken from ASCE 7, *Minimum Design Loads for Buildings and Other Structures*. The determination of the wind loads is based on 4 factors that affect how much of a lateral force the wind creates. The first 3 are topographical factors K1, K2, and K_3 that account for wind coming off sloped terrain and into the walls of the building. These 3 factors can be determined in ASCE 7 section 26.8, page 251, as shown in *Figure 2*. These three factors are determined by the height of the slope (H) over the length of the slope (L_H) For the purposes of our project, excavation will create a flat border of terrain around the building so these factors will not alter the wind loading off our structure. The fourth factor we need to consider for the wind is K_e, the ground elevation factor. K_e is found in section 26.9 and can be calculated as shown in *Figure 2*.



Topographic Multipliers for Exposure C

H/L_h	K_1 Multiplier 2-D Ridge	K_1 Multiplier 2-D Escarp.	K_1 Multiplier 3-D Axisym. Hill	x/L_h	K_2 Multiplier 2-D Escarp.	K_2 Multiplier All Other Cases	z/L_h	K_3 Multiplier 2-D Ridge	K_3 Multiplier 2-D Escarp.	K_3 Multiplier 3-D Axisym. Hill
0.20	0.29	0.17	0.21	0.00	1.00	1.00	0.00	1.00	1.00	1.00
0.25	0.36	0.21	0.26	0.50	0.88	0.67	0.10	0.74	0.78	0.67
0.30	0.43	0.26	0.32	1.00	0.75	0.33	0.20	0.55	0.61	0.45
0.35	0.51	0.30	0.37	1.50	0.63	0.00	0.30	0.41	0.47	0.30
0.40	0.58	0.34	0.42	2.00	0.50	0.00	0.40	0.30	0.37	0.20
0.45	0.65	0.38	0.47	2.50	0.38	0.00	0.50	0.22	0.29	0.14
0.50	0.72	0.43	0.53	3.00	0.25	0.00	0.60	0.17	0.22	0.09
				3.50	0.13	0.00	0.70	0.12	0.17	0.06
				4.00	0.00	0.00	0.80	0.09	0.14	0.04
							0.90	0.07	0.11	0.03
							1.00	0.05	0.08	0.02
							1.50	0.01	0.02	0.00
							2.00	0.00	0.00	0.00

Elevation above sea level, z (ft)	Ground elevation factor, K_e
0	1.00
1,000	0.96
2,000	0.93
3,000	0.90
4,000	0.86
5,000	0.83
6,000	0.80
10,000	0.70

$$K_e = e^{-0.0000362z}$$

where z = elevation above sea level (in ft)

Figure 2: The chart on the top is where topographical factors K_1 K_2 and K_3 can be determined. The chart on the bottom shows how the ground elevation factor K_e is calculated.

Gravity loads, or vertical loads, are considered next when looking at structural steel design. There are four main groups of vertical loads that the structure was designed to hold: dead loads, live loads, roof live loads, and snow loads. All the required loads are found in ASCE 7. The first type of vertical load are dead loads, which are the natural load of all the materials used to make a structure. Since these loads come strictly from the materials used to make the building, the dead loads for the building can be strictly defined. In the design metal decking, concrete used to make the floors, and solar panels were accounted for as dead loads. Using given information about each material the weight of each material can be calculated and added together to create our total dead load. The second type of load is the live load, which is the weight of goods and people in a building, and these are subject to change. Since these loads can change, depending on what type of structure is built and what it is being used for, table 4-1 in ASCE 7 has a list of minimum live loads for different building types (as shown in *Figure 3*).

Residential	
One- and two-family dwellings	
Uninhabitable attics without storage	10 (0.48)[l]
Uninhabitable attics with storage	20 (0.96)[m]
Habitable attics and sleeping areas	30 (1.44)
All other areas except stairs	40 (1.92)
All other residential occupancies	
Private rooms and corridors serving them	40 (1.92)
Public rooms[a] and corridors serving them	100 (4.79)

Figure 3: This shows the part of table 4-1 of ASCE where the minimum live loads for residential structures are defined.

The third type of vertical load is the roof live load or the weight of temporary objects on the roof (such as people or goods). The minimum roof live load is defined in the same location in ASCE, table 4-1, as seen in *Figure 4,* and based on the design for the roof of the residence hall, the minimum roof load is determined based on said design.

Roofs		
Ordinary flat, pitched, and curved roofs	20 (0.96)[n]	
Roofs used for roof gardens	100 (4.79)	
Roofs used for assembly purposes	Same as occupancy served	
Roofs used for other occupancies	[o]	[o]
Awnings and canopies		
Fabric construction supported by a skeleton structure	5 (0.24) nonreducible	300 (1.33) applied to skeleton structure
Screen enclosure support frame	5 (0.24) nonreducible and applied to the roof frame members only, not the screen	200 (0.89) applied to supporting roof frame members only
All other construction	20 (0.96)	

Figure 4: This figure shows the section of table 4-1 that defines the minimum roof live loads for a structure based on the characteristics of the roof.

The last vertical load that we needed to account for in our design is the snow loads, as we are in a part of the country where snow is very common. Snow loads are just loads that can be potentially created on the roof by snow, and we need to design our steel to hold a certain amount of snow. To calculate the roof snow load (for a flat roof in our case) we need to find three factors within ASCE 7 chapter 7, the thermal factor C_t, the exposure factor C_e, and the importance factor I_s. C_e and C_t are defined in tables 7-2 and 7-3 as seen in *Figure 5* and the importance factor, I_s, is defined in table 1.5-1 and are based on the risk categories mentioned earlier.

Table 7-3 Thermal Factor, C_t	
Thermal Condition[a]	C_t
All structures except as indicated below	1.0
Structures kept just above freezing and others with cold, ventilated roofs in which the thermal resistance (R-value) between the ventilated space and the heated space exceeds 25 °F × h × ft²/Btu (4.4 K × m²/W).	1.1
Unheated and open air structures	1.2
Structures intentionally kept below freezing	1.3
Continuously heated greenhouses[b] with a roof having a thermal resistance (R-value) less than 2.0 °F × h × ft²/Btu (0.4 K × m²/W)	0.85

Table 7-2 Exposure Factor, C_e			
	Exposure of Roof[a]		
Terrain Category	Fully Exposed	Partially Exposed	Sheltered
B (see Section 26.7)	0.9	1.0	1.2
C (see Section 26.7)	0.9	1.0	1.1
D (see Section 26.7)	0.8	0.9	1.0
Above the treeline in windswept mountainous areas.	0.7	0.8	N/A

Figure 5: These two tables provide the thermal and exposure factors C_e and C_t that are used to calculate a snow load.

Once we have the thermal, exposure, and importance factors and the ground snow load, the roof snow load can be calculated. For the Worcester area, the ground snow load is 55 psf and is to be multiplied by the other factors as seen in more detail in *Figure 6* below.

The flat roof snow load, p_f, shall be calculated in lb/ft² (kN/m²) using the following formula:

$$p_f = 0.7 C_e C_t I_s p_g \qquad (7.3\text{-}1)$$

*Figure 6: Equation 7.3-1: Used to calculate the flat roof snow load*s.

The next type of lateral loading is earthquake loads and four site parameters can alter the earthquake loading. The first two parameters are S_{DS} and S_{D1}, defined in section 11.4.4 of ASCE 7. The third parameter is S_1 and it is defined in section 11.4.1. The last parameter is defined as transition period T_L and can be found in section 11.4.5 of ASCE 7. These four factors are based on geographic location and the risk category, a factor that affects the earthquake loading based on the purpose a building is serving. The residence hall will have a risk category 3 to determine the earthquake loads as building code states "occupancies with an occupant load of 50 or more resident care recipients but not having surgery or emergency treatment facilities" (*Digital Codes*, Building Codes 2021 of Illinois).

Dynamic Analysis:

Excitation is the action of making something vibrate. When a particular structure exhibits forced harmonic excitation, vibrations occur of equal frequency to the excitation. If the natural frequency of the structure is the same as the excitation frequency then the amplitude can increase drastically causing severe damage to the structure as well as endangering the lives of the inhabitants and those in proximity (Dahleh et al., 2003).

Vibration analysis of structures includes modeling the harmonic frequency of the structure and testing how it holds under a simulated seismic load. Understanding the natural frequency of the structure is critical when designing against excitation (Wang et al., 2020). As a result, degrees of freedom and modes must be taken into account. Degrees of freedom represent the number of point masses and the directions the masses can independently move in. Modes are the distinct natural frequencies in a structure that occur as a direct result of the point masses (Zhao, 2019). The first modes of the structure are the most important as the dynamic response is heavily impacted by the first couple of natural frequencies and their corresponding mode shapes. Designing with selected frequencies to avoid excitation can significantly improve the seismic performance of the structure and minimize the risk to human life (Wang et al., 2020).

Reinforced Concrete:

Concrete is a material that is developed by mixing Portland cement along with water, as well as fine and coarse aggregates to achieve a desired strength. Concrete is an important construction material because its strength is adjustable to the desired needs and, as a material, it holds very well under compression forces. Concrete also can be molded into any shape, which makes it applicable and useful for a variety of construction projects. Additionally, adding steel reinforcing bars (rebar) to reinforce a concrete structure gives it the ability to perform well in tension as well as compression (Yakut, 2004). These properties, along with concrete's durability and long lifespan, relative to other materials like wood and steel, are why reinforced concrete is a desired material for large-scale construction projects.

Construction Methods

Traditional Construction:

Traditional construction is the method used on most projects worldwide. Traditional construction methods have a wide variety of definitions and have been used to construct builds for hundreds of years. This type of construction consists of bringing materials to a project and assembling them on-site. The site is first excavated and made ready to pour concrete for the foundation. A foundation slab is poured, typically accompanied by rebar (steel) for strength. For larger projects grade beams (horizontal rebar columns) may be placed to provide support for braced frames to combat lateral loads from forces like the wind. Next, any set steel required is placed, plumbed, and secured. For multiple-story buildings, metal decking is placed for each floor to provide a level surface for concrete pours. Once the skeleton of the building is set, the envelope of the building is built followed by the interior. One major advantage of traditional construction methods is that it allows a building to be customized to the architect's or owner's vision. However, some drawbacks that come with traditional construction are that weather plays a larger factor in terms of time delays during the project. This project delivery method also typically has a longer project duration which can increase costs.

Modular Construction Methods:

Modular Construction is sections of buildings constructed off-site, with the same construction materials as any onsite traditional construction would use. These parts are then shipped to the construction site and installed in a very timely way that almost resembles putting a puzzle together. Modular units must be the same size or smaller than their corresponding shipping method. In most cases, the units will need to be able to fit on a truck to get to the job site, so the units should therefore be sized accordingly. Modular construction can be permanent (PMC) or reusable (RB). In PMC modular buildings are permanent, like any traditionally built building. Relocatable buildings are built in modular units and made so that the units can easily be taken out to be relocated or refurbished.

One very important factor to consider as a part of modular construction is the connections of beams and columns to each other, along with the connection of modular units to themselves. The success of a whole modular system root in the strength of these connections and may be the most important area to consider in terms of design. With that said there are lots of different ways that modular units can be interlocked successfully without losing strength. The most common way of connection is via bolting, and typically there is a primary column-to-column (CTC) bolted connection (Corfar & Tsavdaridis, 2022). One method of bolting is connection plates, and this can be used for vertical connections and horizontal connections. There would be predrilled holes in each of the columns or beams (depending on where and how you want to connect them modular units) and a metal plate would line up with the holes and then be bolted onto the two members, therefore preventing vertical and horizontal movement of the connected members (Ma et al., 2021). This type of connection in columns is known as a steel column splice and can be implemented horizontally. This principle also is not strictly limited to any certain beam shape.

To go into this idea a little more in-depth with strictly column connection, *Figure 7* illustrates a similar type of connection. In this case, the only addition is the in-build component which will go inside the two metal columns being connected and act as an interior connection component (Ma et al., 2021). Here long split bolts also go through the floor beam of the top modular unit and the ceiling beam of the bottom modular unit for added connectivity and stability (Ma et al., 2021).

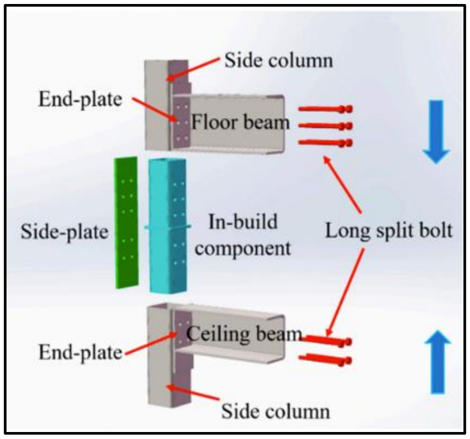
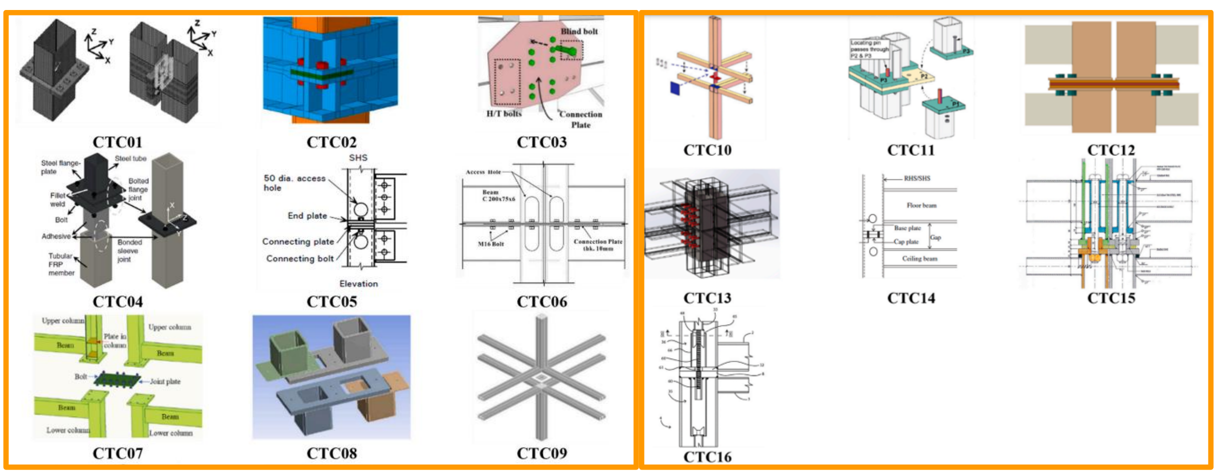


Figure 7: This figure shows a method of bolted connection between 2 modular columns (Ma et al., 2021).

CTC connections however are not the only way to connect modular units, as BTB (beam to beam) and FTF (fitting to fitting) connections are also both common ways to connect modular units. BTB connections are where beams are connected, most typically by some form of bolting (Corfar & Tsavdaridis, 2022). Fitting-to-fitting (FTF) connections are slightly different from the BTB and CTC connections. Beams and columns are joined together with a fitting, and these fittings are then connected, very similar to the way Legos work (Corfar & Tsavdaridis, 2022). Some of these fittings are bolted together, but other fitting connections have interlocking devices that connect to other fittings successfully without bolts (Corfar & Tsavdaridis, 2022). As modular construction becomes more and more popular, more and more innovative IMCs are being developed, and *Figure 8* below shows a variety of them (Corfar & Tsavdaridis, 2022).



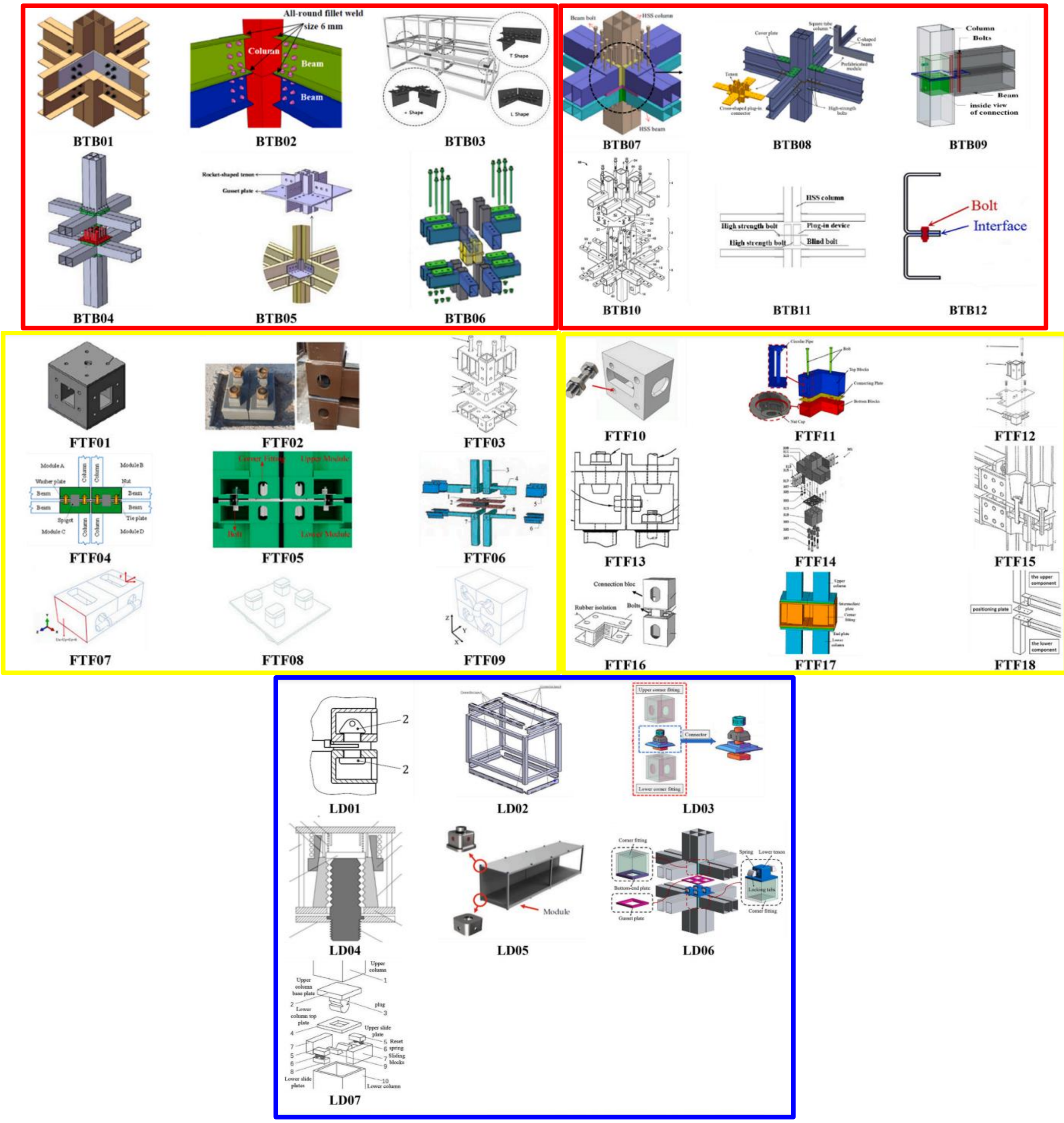


Figure 8: This Figure shows various BTB, CTC, FTF, and LD (FTF connections with Locking devices) (Corfar & Tsavdaridis, 2022).

As mentioned above CTC connections are the most widely used method to connect modular units. Most often these connections are composed of column splices and columns of upper and lower modular units bolted together. This type of connection transfers the vertical loading independently and "some of these systems provide vertical and horizontal connectivity independently of each other through separate tie and endplates (CTC01–04, CTC14, CTC16), more advanced systems provide vertical and horizontal load transfer simultaneously by utilizing an intermediate plate between the columns' endplates (CTC05–12, CTC15)" (*Figure 2*) (Corfar & Tsavdaridis, 2022). Systems that create a simultaneous vertical and horizontal load transfer, for example in CTC 10 (*Figure 2*) increase shear stiffness and stability of the connections and thus these types of systems are extremely beneficial. However, these types of systems are composed of more elements. Looking at CTC 10 (*Figure 2*) the IMC has a gusset plate and an end base plate, and although “the beneficial effect of the additional [connection elements], inspection operations of the internal bolted connection are severely obstructed” (Corfar & Tsavdaridis, 2022).

Similarly, BTB connections are connections between ceiling and floor beams that are “engineered to closely fill the gaps between modules (mods), optimizing the unusable space between each of the floor and ceiling cassettes” (Corfar & Tsavdaridis, 2022). In BTB connection systems, either flanges or webs of inter-module beams are bolted together. Some of the BTB connections can be on the simpler side such as BTB 08 and some connections can be more complex and have more parts such as BTB10 (Corfar & Tsavdaridis, 2022). Although these connections are very efficient in transferring load, they are very difficult to assemble and disassemble as the BTB connections are generally very interconnected (*Figure 2*) (Corfar & Tsavdaridis, 2022).

A study was conducted to determine the best types of IMCs based on the fittings shown above in *Figure 8*. The IMCs were ranked individually for three categories, strength, constructability, and manufacturability, and the FTF IMCs scored the highest, followed by the BTB IMCs.

Fittings for the lower mods are to be bolted to the fittings of the upper mods and a gusset plate is placed in between them. (Deng et al., 2022). These vertically connected corner fittings are made with bolt clamps through an operating hole as shown in *Figure 10* (Deng et al., 2022). Each fitting has an inner plate that the vertical bolts are fastened to (Deng et al., 2022). As a modular unit is being stacked on top of another the bolt is placed in the upper fitting and guided through the gusset plate and into the fitting of the modular unit below it, and then fastened (Deng et a., 202l) (*Figure 9*). This type of connection can also be coupled with other connections (e.g. shear connections) for added stability (Deng et al., 2022). The fitting-to-fitting (FTF) connections have been subjected to testing and perform better than CTC and BTB connections on average (Corfar & Tsavdaridis, 2022). These connections can also be called fully prefabricated liftable connections (FPLCs) that are prefabricated and welded to the hollow square structural (HSS) columns, and able to be hoisted by a crane (Deng et al., 2022) as shown in *Figure 9*.

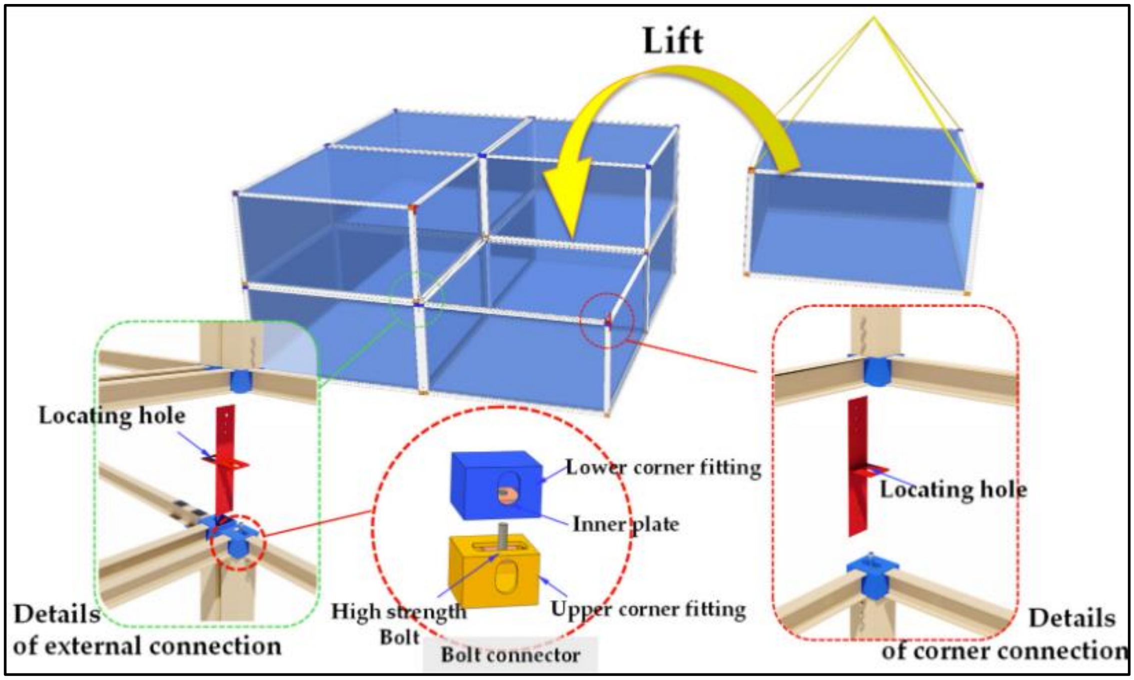


Figure 9: This figure shows how the corner fittings connect vertically and horizontally so that the modular units can be stacked on top of each other. The figure also shows how the vertical units can be hoisted by crane (Deng et al., 2022).

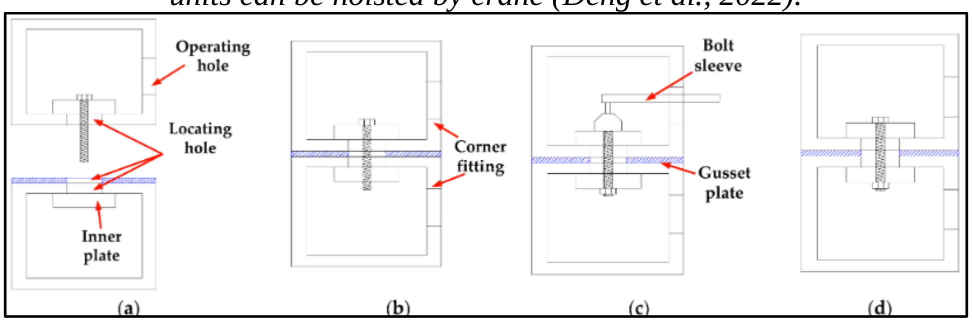


Figure 10: This figure shows how the corner fittings are bolted to each other (Deng et al., 2022).

Design Limitations:

To understand more about the desired project site for a potential residence hall, our team met with Nick Palumbo of WPI Facilities. Palumbo is the Director of Design and Construction for all construction work on WPI's campus. In a conversation with the team, he helped shed light on two main design limitations that came with the WPI Townhouse project site. The first limitation is the proximity of the site to a nearby wetland, Salisbury Pond located in Institute Park. The wetland ordinance restricts where new construction is allowed. That being said, two of the six townhouse units are situated on land that cannot be rebuilt. All in all, renovating these buildings and repurposing them is a goal for WPI. (Palumbo, personal communication, 2022). Another limitation that comes with proximity to a wetland is the strength of the soil of the site or its bearing capacity. This project seeks to incorporate as much residential parking as possible, so an underground parking garage might be ideal to accommodate this. If the soil's bearing capacity is lower, then considerations may have to be made for digging the parking garage into the natural gradient of the project site, even possibly leaving it partially exposed to the elements. Though this would not be ideal for covered parking, it would allow for our goal of maximized parking to be achieved.

Construction Project Management Methods

When considering the construction project management aspects for a new building, two major parts need to be discussed: the project schedule and the project budget. One way to increase the efficiency of a project is to reduce the timing of construction. Through modular construction, the schedule will be significantly shortened, as modular construction reduces construction time on site by 30-50%. (*7 Key Reasons to Explore Modular Construction*, 2022) This is because all the major construction components are prefabricated off-site and then shipped to the job site, ready for installation. While modular construction is a viable option, more detailed and unique portions of a building may still involve traditional construction, thereby adding to the project schedule. Nonetheless, the process of installing modular components of a building can overlap with the construction of traditionally constructed sections, resulting in a minor time loss, if any at all. Once all prefabricated elements are installed modularly, tasks including mod placement and connection, as well as other details such as lighting, painting, furnishing, and landscaping are done on-site.

In a modular construction project, the majority of the budget consists of the prefabrication of the modular units. Utilizing modular construction inherently limits construction costs since the project timeline is significantly condensed. Off-site fabrication reduces on-site labor hours and, in turn, lessens costs spent on construction site laborers and safety measures. Thus, budgeted savings from the reduced costs can be reallocated to account for other costs throughout a project. Considerations must be taken when building in the northeastern United States and construction costs in New England are relatively expensive in comparison to the rest of the country. According to RSMeans, construction and materials in the Northeast are 18.3% more expensive compared to the national average. The individual costs and comparisons are listed below in *Figure 11*. The costs were determined using construction industry and material averages. (Biermeier, n.d.) (Johnstone, n.d.) (*Construction Materials Price Tracker*, n.d.) (ABC, n.d.)

DIVISION		MASSACHUSETTS		
		WORCESTER		
		MAT.	INST.	TOTAL
015433	**CONTRACTOR EQUIPMENT**		96.2	96.2
0241, 31 - 34	**SITE & INFRASTRUCTURE, DEMOLITION**	98.4	97.4	97.7
0310	Concrete Forming & Accessories	137.8	124.2	126.2
0320	Concrete Reinforcing	226.6	148.7	189.1
0330	Cast-in-Place Concrete	81.7	133.4	101.2
03	**CONCRETE**	108.7	131.2	118.8
04	**MASONRY**	107.9	134.8	124.3
05	**METALS**	180.7	129.3	164.7
06	**WOOD, PLASTICS & COMPOSITES**	139.9	124.2	131.7
07	**THERMAL & MOISTURE PROTECTION**	115.6	123.1	118.8
08	**OPENINGS**	100.1	132.8	108.0
0920	Plaster & Gypsum Board	135.9	124.6	128.5
0950, 0980	Ceilings & Acoustic Treatment	116.4	124.6	121.5
0960	Flooring	128.3	160.0	137.6
0970, 0990	Wall Finishes & Painting/Coating	93.4	138.9	120.7
09	**FINISHES**	116.3	132.9	125.3
COVERS	**DIVS. 10 - 14, 25, 28, 41, 43, 44, 46**	118.0	110.2	116.2
21, 22, 23	**FIRE SUPPRESSION, PLUMBING & HVAC**	97.7	105.2	100.7
26, 27, 3370	**ELECTRICAL, COMMUNICATIONS & UTIL.**	110.2	106.4	108.3
MF2018	**WEIGHTED AVERAGE**	117.8	119.0	118.3

DIVISION		MASSACHUSETTS		
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MF2018	**WEIGHTED AVERAGE**	117.8	119.0	118.3

Figure 11: RSMeans Building Costs.

Another aspect that must be taken into account relating to project management is the delivery and storage of modular units. One method to transport mods from a prefabrication site to the project site would be trucking the modular units individually as freight, However, the height and size could cause problems during transit. Highway overpasses, electrical lines, light poles, tight turns, and steep hills amongst other obstacles can pose risk to units of this size being shipped. Therefore, a precise yet efficient route must be created for the transportation of modularly constructed components. Another option is shipping the mods via train and, from there, transporting them via trucks to the site. If there are no obstructions via railroad, this method may be a more time and cost-efficient option.

Risk is also an extremely important factor to consider in project management. Calculating risk for each schedule item is vital to pinpoint any potential areas of concern so a plan can be devised to mitigate these potential problems or prevent them from happening at all. Risk is embedded in all aspects of a construction project, but the most common risks include a poorly defined scope, incomplete drawings, design errors, unknown site conditions, poorly written contracts, and unexpected increases in the cost of materials.

Sustainable Systems

Solar Power:

Solar power is the ability to harness energy from the electromagnetic radiation of the sun and two types of solar power technologies are currently capable of producing power with that radiation. The first of these technologies is known as a concentrating solar-thermal power (CSP) system. This system uses mirrors to reflect and concentrate the sunlight onto an array of receivers. These receivers collect that energy and convert it to heat, which in turn can be used to produce electricity or be stored. CSP systems are commonly found in power plants (Energy.gov, 2020). The second solar technology is called photovoltaics. Photovoltaics are most known for their usage in solar panels. Solar panels consist of a glass cover, frame, and photovoltaic (PV) cells. *Figure 12* below shows the process of photons from electromagnetic radiation colliding and ionizing the semiconductor material on the solar panel, resulting in outer electrons breaking their atomic bonds. The electrons are then forced by the structure of the semiconductor layer in a single direction creating a flow of electrons. This flow of electrons, or electricity, is what can then be used to power homes and even cities depending on the scale (SEIA, 2023).

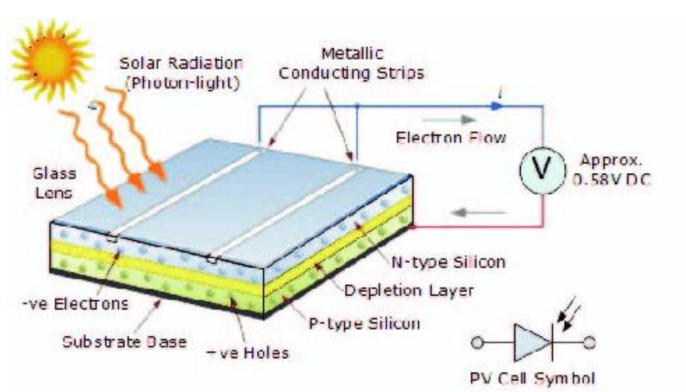

Figure 12: Diagram detailing a PV cell and the process of turning solar radiation into electricity (Himanshu et al., 2017).

Solar power systems can be designed in a variety of different ways. Residential applications will vary quite heavily from commercial systems. Most residential systems consist of a few flush-mounted PV panels whereas large plants or even cities will need vast arrays of ground-mounted panels to provide the necessary amounts of power. The most basic way to categorize solar power systems is how it is connected to the grid. Off-grid systems are isolated systems that are not connected to the power grid. They work by storing the energy during the day in a battery storage system. This method of solar design is extremely risky for larger systems as the facility is solely reliant on solar energy. As a result, they are typically more expensive as over-design is needed to ensure enough energy is produced to power all intended critical functions. Grid-tie solar, however, is connected to the grid and is the most common solution for residential and small commercial applications. Connection to the grid allows for greater flexibility as the building is less weather reliant. In periods of low sunlight, power can still be drawn from the grid unlike Off-Grid systems (ILUM Solar, 2023). Also, Massachusetts is one of the states that allow for net metering (Mass.gov, 2023). This occurs when the solar system produces more energy than the building needs. The excess energy is added to the power grid and credit is given by the utility company on the energy bill. Lastly hybrid solar systems combine the battery storage systems of off-grid systems and the connection to the grid of grid-tie systems. Hybrid systems offer the most flexibility as both stored energy and power from the grid can be used when energy production needs outweigh the power being produced (ILUM Solar, 2023).

Solar power over the last two decades has become a powerful energy option on the market. “According to Grand View Research Inc, the solar energy market is expected to grow by USD 176.2 billion by 2027, escalating at a CAGR of 4.3% over the forecast period” (AESolar, 2023). This is in large part due to advances in solar panel technology and the decrease in solar cell prices over the last decade. In the last decade alone PV systems have dropped in price by over 59% (SEIA, 2023).

It is important to note that since the global pandemic, prices of certain systems may be slightly elevated due to supply chain issues. However, experts predict that this will not affect the market share or the downward trend of prices in the coming decade (AESolar, 2023).

Hydronic Heating:

To provide a residence hall that students will actively want to live in, proper heat design is an important consideration especially given New England's harsh climate. As sustainability initiatives continue to grow, understanding building energy consumption and greenhouse gas emissions trends will become increasingly important. In total, heating, ventilation, and traditional HVAC air conditioning systems in residential and commercial buildings in the United States make up nearly 10% of all greenhouse gasses released as well as 25% of electricity usage (Aswani et al. 2012). The equipment necessary for operating the heating system such as heating fans and pump systems accounts for over half of the total energy consumed by the building and approximately 20% of the total energy consumed across every sector in the United States (Pérez-Lombard et al. 2008). As a result, energy-efficient solutions will be needed to reduce the carbon footprint of the building.

Traditional air conditioning systems take in external air and circulate that air through ducts and fans to heat spaces. However, other alternatives such as hydronic heating systems provide increased efficiency as well as better distributed heat (Falke, 2018). As can be seen from *Figure 13* below, at its core these systems need five main components to operate.

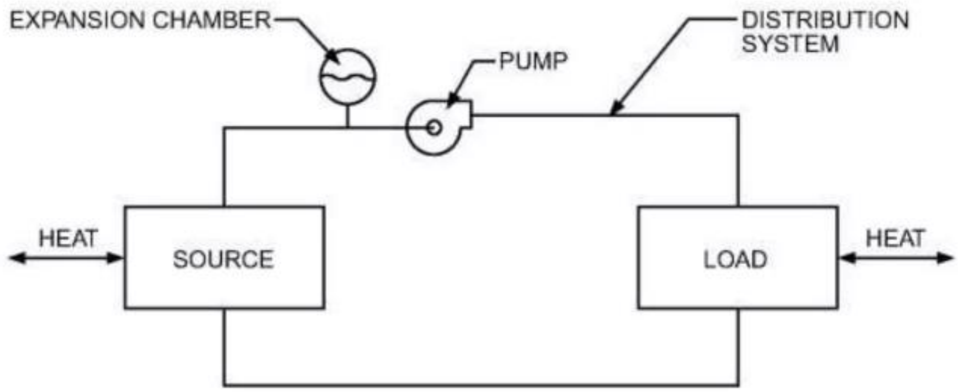

Figure 13: Diagram detailing the main components of a distributed heat system.

Hydronic systems work by heating spaces with a fluid medium as opposed to solely air. Water or other chemical fluid mediums are heated by a heat source typically in the form of a boiler. There are various types of boilers in terms of size and design to suit specific systems. The heated water is then distributed through the piping system using a heat pump to circulate the water throughout the building. The heat load typically consists of either radiant loops or baseboards.

Radiant loops (shown below in *Figure 14*) are installed underneath the flooring whereas baseboards are typically installed along the wall. Both radiate the heat from the piping system into the target space to provide heating.

Figure 14: Underfloor radiant loop installation (Energy.gov, 2023).

The installation of underfloor radiant looping can be categorized as either a wet or a dry installation. Wet installation refers to placing the radiant tubing directly into the concrete slab. Concrete is an excellent material for hydronic heating as the concrete provides a thermal mass to absorb and radiant heat evenly throughout the space. It also acts as protection forming a hard shell around the tubing. The thickness will affect how the system operates; however, underfloor tubing can be installed for both slab-on-grade foundations and thin slab applications of just 1-2" of concrete. Dry systems, or plate systems, are installed using prefabricated panels that have tracks built in for the tubing to be organized and secure. It is important to note that without the thermal properties of concrete to aid with the absorption and the release of heat, insulation or other heat reflectors may be needed.

Methodology

This section discusses the overall approach that was taken when it came to designing a new residence hall. The first considerations that are considered are the ideal construction methods themselves, and how they can be most effectively implemented for this type of project. Additionally, the maximization of site usage is discussed because of goals surrounding the promotion of positive mental health, both inside and outside of the building itself. To best convey how these ideas come together, two virtual models were developed using various software. The first was an architectural model that aided in the visualization of the design and dimensions of the building. Next, a structural model and dynamic analysis were completed to assist with calculations and the building material choice. To bring together the entire scope of the construction management plan, it is very important that the construction timeline and any potential risks were evaluated. Here, software was used to plan a construction schedule along with varying transportation methods for our construction management plan.

Objective #1: Consider Researched Design Options

In order to begin designing a new residence hall for upperclassmen students, our group evaluated and planned according to spatial availability at the site of the current Townhouses. Hence site-specific parameters and environmental factors must be taken into account in the design process. Moreover, it is important to consider the specific information regarding WPI's existing situation regarding residence halls and living spaces for upperclassmen.

For location options, it was found that WPI's current land plot for the Townhouse apartment-style residences is an ideal space to build a new residence hall. Two existing Townhouses must remain while the remaining would be demolished. It was also determined that retrofitting the current WPI townhouses was not a cost-efficient option, and a completely new building made the most sense. In the available space, one single building was designed to maximize the number of beds while also promoting social interaction in the building. After researching various methods of construction modular construction was decided to be implemented for the architectural and structural design while also utilizing traditional construction for more customizable sections of the building; particularly the foundation, bottom floor garage, and first floor. A traditionally constructed lobby space with a glass facade will be found on the first floor, complemented by a modularly constructed set of first-floor suites. Constructing the rest of the building through modular construction and prefabrication of identical mods allows for a faster project timeline since most of the essential pieces will be fabricated off-site while site work is being done. This also reduces the need for additional supervision and safety precautions on site when it is time for construction, thus construction costs are decreased significantly as well.

Another crucial aspect of this project is the benefits of amenities and extraneous services that will be provided in the residence hall. A parking lot for students residing in the new dormitory was determined to be a highly valued amenity, as it provides a large convenience for students who would like to bring a vehicle to campus. In an effort to maximize the space of the site, the building was designed to incorporate a parking garage on the ground floor with more parking spaces being included outside of the garage as well. Other services and spaces for prospective residents will be provided to promote community interaction and activity during free time outside of academic work. More services for WPI students will, in turn, allow residents to meet one another and take time away from the stressful engineering and science courses.

Objective #2: Design Building & Site

After the group compiled all data about WPI’s current situation and needs, the aspects of structural engineering and design must be considered. With plans to have parking available for residents, a traditional foundation would be best to include a parking garage, allow for customized social spaces in a first-floor lobby, and create a skeleton for our building’s upper floors. The reason why a frame was necessary for the upper floors is because modular construction practices would be utilized to build suites for residents. The method of modular construction was chosen because it inherently reduces construction time, which in turn reduces cost. To bring each prefabricated component together, welded connections were chosen for design. These mods would be prefabricated off-site via a manufacturer and shipped and stored at the site before assembly.

Structural Steel:

The structural steel was designed in Risa 3D where the size and shape of the steel columns and beams for the mods will be calculated. There will be 2 types of modular units in the structural steel framework of the building: the room mods and the hallway mods. The room mods will be used for residential areas of the building and 5 room mods make up a suite that will house 8 students. Hallway mods will be modular units that make up the hallways in between the suites. Each room and hallway modular unit will be designed by modeling a singular section of 4 of the mods stacked on top of each other with applied loads that are defined in ASCE 7. Once a design for the beams and columns is determined, a material take-off can be carried out and a set number of members we need for the project can be defined.

One crucial factor that must be considered is how the modular units will interlock with each other so that strength and stability are maintained. There are a lot of different ways to go about connecting the beams and columns of the modular units as well as connecting modular units to each other as mentioned in the research section above. These connections will be considered during the design process as once again they are crucial for the success of our structural steel and therefore the safety of future residents.

Once the steel model is defined and has a set number of members, the total weight of the structural steel can be calculated and the total weight for the entire building can be estimated.

Using this information and the geotechnical reports we can then determine what type of foundation is needed based on the bearing capacity of the soil in Worcester. The foundation can then be further designed from there on.

Vibration Analysis:

The Risa 3D structural steel model was used to conduct the dynamic analysis. The first step was to generate the mode frequencies in the X, Y, and Z directions. To do this the structure's weight must be simulated to get reasonable accuracy for the mode frequencies. Area loads were implemented per floor to represent the mass of the structure per floor. The load was then distributed amongst nodes located on every steel member in the Risa model. The split member feature was used to section each individual member into five separate members. As a result, there were hundreds of degrees of freedom present in the structure resulting in many mode frequencies that could be analyzed. Mode frequencies in the X, Y, and Z directions with the highest mass percentage participation correspond to the key frequencies to avoid. It is important to note that increasing the number of modes greatly increases the processing time of the dynamic analysis. As a result, analyzing fewer and more relevant modes will save time.

Once the mode frequencies have been calculated for the structure using the RISA dynamic analysis tool, the four site-specific parameters must be found. They include the final acceleration coefficient (S_{DS}), the spectral response acceleration parameter (S_{D1}), the spectral acceleration parameter (S_1), and the long-period transition period T_L. The ASCE 7 Hazard Tool was used to compile all the seismic factors needed to determine earthquake loads based on geographical location. All the factors have been determined for the proposed site (79 Park Ave, Worcester, MA 01605) as seen in *Figure 15* below:

Site Information	
Address:	*79 Park Avenue, Worcester, Massachusetts, 01605*
Elevation:	0 ft (NAVD 88)
Lat:	42.279041
Long:	-71.808037
Standard:	ASCE/SEI 7-22
Risk Category:	III
Soil Class:	C - Very Dense Soil and Soft Rock
Seismic Data	
S_S	0.22
S_1	0.052
S_{MS}	0.23
S_{M1}	0.066
S_{DS}	0.15
S_{D1}	0.044
T_L	6
PGA_M	0.12
V_{S30}	530
Seismic Design Category	A

Figure 15: This is the seismic data gained from the ASCE 7 Hazard Tool based on the location of the current WPI townhouses.

The structural characteristics are the seismic factors related to the steel structure. These factors include the building period (T), building period coefficient (C exponent (C building response coefficient (R), overstrength factor (Ωo), and deflection amplification factor, (Cd). The building period coefficient and exponent can be found in Table 12.8-2 of ASCE 7-16. The building response coefficient, overstrength factor, and deflection amplification factor can all be found in Table 12.2-1 of ASCE 7-16.

The site-specific seismic data in conjunction with the structural characteristics were then put into the seismic load generator tool in Risa to simulate seismic loads. Based on this data it will be possible to predict excitation behavior and help influence the structural design to avoid the phenomenon.

Objective #3: Designing Sustainable Energy Methods

Energy Consumption and Constructability:

College Residence halls of this magnitude require over a Megawatt hour per year. As a result, designing energy-efficient, cost-saving systems will be necessary to reduce the carbon footprint and overall cost of the building. Photovoltaic solar panel systems are a great way to produce green energy that can offset costs on WPI's utility bill.

The first step in designing the solar system for this planned residence hall was understanding the projected energy needs of the building. As aforementioned solar systems can vary heavily in panel design and size to meet the energy requirement needs of the building. It is very difficult to get an accurate estimate as a variety of factors such as square footage, climate, building capacity, type of building and HVAC can influence power consumption. *Figure 16* from the Energy Information Administration was used to estimate the facility's energy consumption.

Building Type	kWh/m^2 year
Dwellings	147
Retail	233
Offices	293
Schools	262
Hotels	316
Supermarkets	631
Restaurants	814
Hospitals	746

Figure 16: Average energy use intensity by building type in the United States.

Using the value of 262 kWh/m^2 per year for school buildings, it was calculated that the residence hall will be projected to use 1,248,672.6 kWh per year. With that estimate, the magnitude of the solar system design can be determined. Due to the location of the building, a ground-mounted system is not viable as there is not enough open space, and the permit process would be extensive given the proximity to Institute Park. As a result, a roof-mounted system is the most viable for this project. This comes with one major drawback which is the total surface area of the roof. There are few options for systems that are cost-effective and can provide enough power that the building can be solely reliant on solar energy. As a result, the solar system must be connected to the grid. A hybrid solar system would provide WPI with the largest amount of flexibility in terms of storing power in times of low use such as vacations and potential net metering which would most likely occur in the summer months.

Design Implications:

Planning the implementation of a solar-powered system introduces a variety of design challenges. The weight of the system introduces extra dead load to the roof. Typical systems add a load of 3-6 lb/sq-ft that needs to be accounted for. Also, solar systems that require penetrations in the roof increase the likelihood of future damage thus proper installation is very important to avoid future problems such as water damage. Additionally, the system must be capable of handling the lateral loads due to the wind forces. As a result, proper racking and panels must be rated appropriately to withstand the wind load. Additionally, solar panels make up a large portion of the roof's surface area to supply enough energy to power a significant portion of the building.

This detracts space from other systems that need to be placed on the roof such as air handling units.

Heating Design:

Based on the climate and the need for an efficient heating solution, a hydronic heating system was designed. As aforementioned, there are two primary heating delivery methods. Radiant loops are installed underneath the flooring whereas baseboards are typically installed along the wall. Both radiate the heat from the piping system into the space to provide heating. However, baseboards are far less efficient than underfloor radiant tubing (Warmup Inc., 2021). For this project, radiant loops will provide the best heat delivery solution. This is because the major drawback of radiant loops is the installation. Because the tubing sits beneath the flooring, the flooring design must be taken into account to accommodate the tubing. However, because the suites of the residence hall are to be built by modular construction, installation of the flooring and radiant loops becomes viable. A wet installation allows for optimal heating distribution and installation. The track and tubing will be laid out per modular before the six-inch floor slab is then poured. The concrete will provide an excellent thermal mass to absorb and radiate the heat evenly throughout the space in addition to protecting the radiant tubing.

Objective #4: Develop a Virtual Model of the Design

Architectural Design & Revit:

Based on initial drawings and sketches, a virtual design of the building was developed, including floor and site plans using Autodesk Revit. The floor plans for the entire building and individual modular units were developed through Revit. The design of the site components are also shown in exterior renderings of the model. In addition to this, a 3D architectural plan was also created and displayed through Revit. These plans incorporate structural aspects including the foundation's dimensions, as well as all the structural modular units described in objective two and the dimensions for structural elements for the first floor and parking garage. Visuals with final renderings of the model are provided in the results.

Risa 3D:

As mentioned above, the software that we have used to design the structural steel is Risa 3D, a very versatile structural design software. To begin the design, members, and nodes (where members intersect) are defined on a Cartesian system, and a rectangle modular unit was created. In our design of the floor plans, two different types of mods that have different dimensions and will need to support different loadings were created. The first type of mods are the residential mods that will make up and support the suites. These residential modular units are 12 ft wide by 40 ft long and 11 ft high. The second type of mods are the mods that the hallways between suites will be composed of. These mods are 24 feet long by 10 feet wide by 11 feet high and will have to be designed to support a larger live load than the residential mods. Once a singular mod is made, a member can be designated as a column, beam, horizontal brace, or vertical brace and the material, size, and type (such as WF or HSS) can be altered. As mentioned, the design of the mods features 4 vertical columns that are 11 feet each, eight 40-foot beams, eight 12-foot beams, and eight joists that are also 12 feet each. That design for a single modular unit is then arrayed on top of itself 3 more times to create 4 stories of a singular stack of modular units. Once this design of 4 modular units stacked on top of each other was finalized, each member was assigned to a "section set", a way to group members so that they all have the same properties (materials, shape, etc.). And when the loads are applied to the model, every member in the same section set is designed to be the same. The model includes 8 section sets and for the first three floors, which are identical, the modular units had four section sets: 40-foot beams, 12-foot beams, joists, and 11-foot columns as seen in *Figure 17*. The fourth-floor modular units have the same length beams and columns as the first three floors but will be designed to hold heavier loads as they will be supporting most of the solar panel weights, so there will be four more section sets as seen in *Figure 17*.

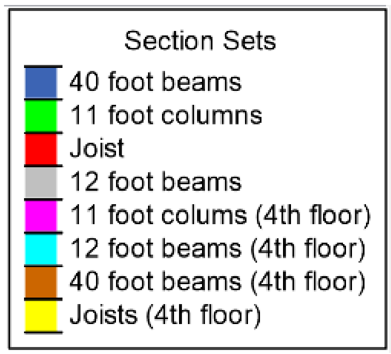

Figure 17: This is all the section sets that were used in the Risa model to create the residential mods. The hallway mods have a similar section set that is just uniform through all 4 floors rather than having different section sets for the fourth story.

Once these section sets were created the loadings in accordance with ASCE 7 (as seen in the background) were applied to the structure. Loads combinations were then generated with respect to dead, live, roof live, wind, snow, and earthquake loads, and the software was able to provide a suggested design for each section set. The suggested design that Risa provides is sufficient in strength and deflection. This whole process was repeated with the hallway units as the loadings slightly differ from the residential units.

All these numbers and results that were generated by Risa 3D were then verified as accurate with hand calculations. Three members of the residential units were chosen to be designed by hand, a 40-foot W-shape girder beam from the second story floor, a 40-foot W-shape roof girder beam from the fourth story, and finally a HSS column from the first story modular units.

For the design of the beams, the dead, live, roof live, snow, and wind loads for the second story were all gathered, and the governing load combination was determined. The moment was then calculated based on the governing load combination and the length of the beam. The moment was then set equal to a known safety factor, the yield stress of the steel, and the plastic section modulus of a beam. The needed plastic section modulus was then calculated, and the size beam that is just greater than the needed section modulus is identified as the beam size most sufficient to support the loadings on the beam. Once a beam size is determined the beam was checked so that it can be determined the beam can hold its self-weight along with the vertical and horizontal loadings. Serviceability for deflection was next checked for the specifically determined beam size, once for the live load only and again for the dead loads and live load combined. The beams also had flexure requirements as found in the Chapter F Specifications of the 15th edition AISC steel manual. The beams were first checked for yielding where the moment calculated earlier had to be lower than the nominal moment M_n, which is the yield stress of the steel F_y multiplied by the plastic section modulus Z_x. Once the web and flange of the W-shaped beam were determined to be slender, the beam simply needed to be checked for shear and lateral torsional buckling. Using Chapter F of the specifications of the AISC Steel manual the determined beam size was deemed sufficient for lateral torsional buckling. Lastly, shear was checked in accordance with Chapter G of the manual.

For the design of the columns, only flexural buckling was checked for. Chapter E of the AISC manual states that non-slender elements must just be designed for flexural buckling and our sections were determined to be non-slender via the slenderness ratio. The loading determined via the software was compared to the flexural buckling stress F_{cr}, multiplied by the section area A_g.

Objective #5: Estimate Timeline, Budget and Create Other CPM Plans

To plan the project schedule, scheduling software must be utilized. First, a rough draft schedule was created in Microsoft Excel in order to better visualize all the events within the project. All event items and their corresponding durations were added in Microsoft Excel. Primavera P6 software was used to complete the project's full timeline and schedule. P6 is an advanced software that is commercially used and trusted by many companies for projects of all sizes.

A digitized budget was created using Microsoft Excel as well. The budget consists of various sections for all the aspects of the project and there is also a budget for mod pricing that conveys the total cost of all the mods as well as one individual mod. Likewise, the traditional construction costs and shipping fees were budgeted apart from the costs of the modular components.

Another area of focus for the construction project management aspects of this project is the transportation plan to get all materials needed on site. and considerations were made for the shipping of modular components of the building. One potential option was shipping the units by train to the Union Station (Worcester, MA) freight yard and then transporting them by truck to the site (79 Park Ave, Worcester). The second option was to truck the units the entire distance, from the mod manufacturer to the site. The main difference between these two options is that the railroad method involves fewer obstacles (bridges, overpasses, electrical wires, etc.), however, the train introduces an added step where the units would have to be handled and dealt with twice, once at the freight yard and then once again at the site. A route from Union Station to the townhouses that includes no obstructions was planned and permitted for real construction. The route was mapped out via Google maps.

Another significant aspect of the construction project management part was risk assessment and management. In order to determine the risk of the project, risk was determined for each schedule item through the use of a risk matrix. The matrix displays the probability of a problem occurring and the severity of the said problem to determine risk. The probability of a problem occurring is located on the x-axis with rankings 1 through 5, with 1 being no chance of occurring and 5 being highly likely of occurring. The severity of risk (this problem going wrong) is located on the y-axis also, with 1 through 5 rankings, 1 being not severe and 5 being extremely severe. Each schedule item received a probability and severity ranking. The schedule items with the highest combined rankings were determined to be the riskiest. A corresponding course of action was chosen to better avoid these potential risks.

Deliverables

Structural Design

The main design component that is crucial for all buildings is the design of structural aspects. This section will discuss all structural steel design results and as mentioned above in the methodology section, the structural steel was designed and modeled via Risa. There are a total of two Risa 3D models, one for the design of the residential mods and one for the hallway mods as these hallway mods slightly differ in dimensions and must hold more load. In the design of the modular units, the first-floor units will be constructed as typical modular units, with columns, floor beams, and ceiling beams. The second, third, and fourth stories will just be constructed with columns and ceiling beams and use the ceiling beams of the previous floor as a floor as seen in *Figure 18.*

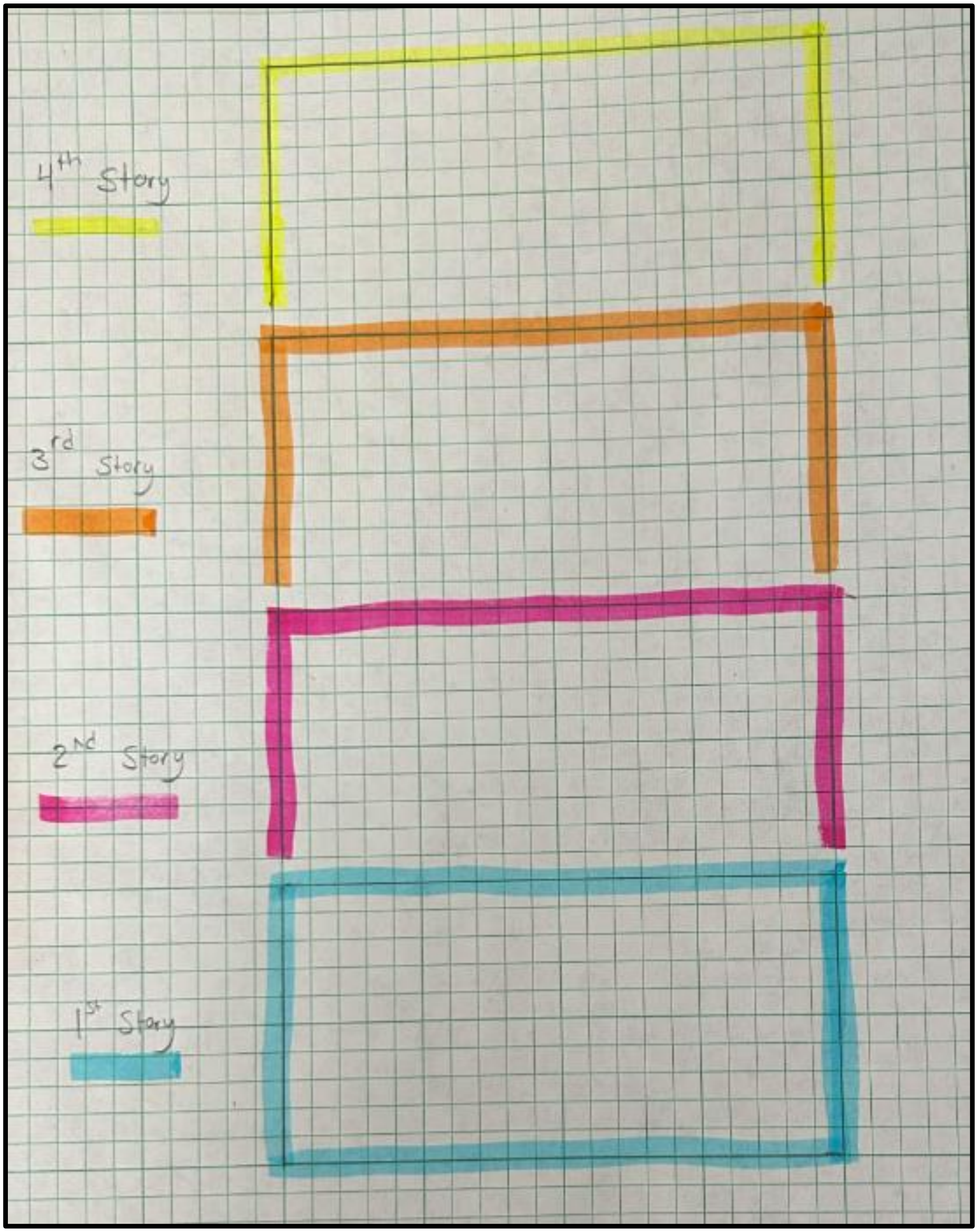

Figure 18: This figure shows each story of modular units from a side view. The second, third, and fourth-story mods do not have floor beams.

Structural Steel Model (residential modular units):

The structural steel for the modular units was designed to the following loadings in accordance with ASCE 7 (*Minimum Design Loads for Buildings and Other Structures*) and the AISC (*American Institute of Steel Construction*) steel manual. The dead loads are the only loads that are not defined in ASCE 7 and were calculated manually by adding up the weight of all the stationary gravity loads. For the dead loads of the steel design, 75 psf (pounds per square foot) was allotted for the weight of the concrete flooring and 4 psf for the weight of the corrugated metal decking used to hold the concrete flooring. The first 3 floors were designed to hold 79 psf or .079 ksf (kips per square foot) dead loads and the fourth floor was designed to hold an 85 psf or .085 ksf dead load to consider the weight of the solar panels that are located on the roof. The minimum required live loads, as specified in table 4-1 of ASCE 7 for buildings with residential purposes is 40 psf, thus what the structural steel will be designed to withhold. For roof live loads, the minimum required load is 20 psf for ordinary flat roofs (as found on our residence hall) according to table 4-1 of ASCE 7. Similarly, the snow loads were determined using factors and a formula found in ASCE 7, and in chapter 7, the thermal factor C_t, the exposure factor C_e, and the importance factor I_s can be found. C_e and C_t are defined in tables 7-2 and 7-3 and the importance factors are defined in table 1.5-1. And then using the roof snow load formula shown in the background, the snow load was calculated to be 43.4 psf as seen in the Appendix.

The lateral loads were also taken into account when designing the structural steel, with the wind loads being calculated in Risa based on topographic and elevation factors found in ASCE 7 section 26.8 (p. 25). These factors account for wind gusts when the structure is located on top of a hill, which is the case for this residence hall. There happens to be a hill on site with a slope of .5 feet of rise per 2 feet of run, and based on this, topographical factors K_1 K_2 and K_3 were determined. The elevation factor K_e was calculated from the formula mentioned in the background but simply just rounded to 1, as it was very close to 1. Given these factors, the wind loads were calculated in Risa to be 27.16 psf, 28.84 psf, 29.9 psf, and 15.51 psf for the first, second, third, and fourth story respectively (*Figure 19*. The earthquake loads were calculated using factors S_{DS}, S_{D1} as defined in section 11.4.4 of ASCE 7, factor S_1 as defined in section 11.4.1 of ASCE 7, and factor T_L which is found in section 11.4, and the loads are shown in *Table 1*. Like the wind loads, the earthquake loads can be broken down story by story and have magnitudes of 4.78 psf, 11.64 psf, 17.46 psf, and 24.4 psf for the first, second, third, and fourth stories respectively (F*igure 19*).

The actual model comprises sixteen 11-foot HSS (Hollow Structural Sections), ten 40-foot-wide flange beams, thirty-five 12-foot-wide flange joists, and ten 12-foot-wide flange beams, all of which are all A992 steel. As seen in *Figure 21*, the model is one section of mods stacked on top of each other, with the first three stories being made from identical members (same section sets) and the fourth-floor mods changed slightly to account for the additional weight of the solar panels. In the model (shown in *Figure 21*) there are four HSS14x10x4 columns that will act as the columns for the fourth story mods and 12 HSS14x10x8 columns, the columns for the first, second, and third-floor mods. Each story's 40-foot beams and 12-foot end beams are W10x49 shapes as well as the joists for the fourth floor, totaling 27 W10x49 members for the model. Thus leaving just 28 W6x8.5-shaped joists for the ground, first, second, and third floors as seen in *Figure 1*. There are 80 mods per floor, every beam/column quantity was multiplied by 80 to determine the total number of each steel member for the entire building resulting in a total of 320 11-foot HSS14x10x4 columns, 960 11-foot HSS14x10x8 columns, 800 40-foot W10x49 beams, 800 12-foot W10x49 beams, 560 12-foot W10x49 joists and 2,240 12-foot W6x8.5 joists. In total the steel members come out to be 89,280 feet in length and weigh in at 3,478,560 pounds as seen in *Figure 20*.

	Residential Modular Units					
	Dead Loads (psf)	Live Loads (psf)	Roof Live Loads (psf)	Snow Loads (psf)	Earthquake Loads (psf)	Wind Loads (psf)
1st story	79	40	N/A	N/A	4.78	27.16
2nd story	79	40	N/A	N/A	11.64	28.44
3rd story	79	40	N/A	N/A	17.46	29.91
4th story	85	N/A	20	43.3	24.4	15.51

Figure 19: A spreadsheet with all loadings that residential mods will undergo and was designed to uphold.

	Structural Steel (Residential Units)				
	Mods	W10x49 (12-foot beams)	W10x49 (40-foot beams)	W6x8.5 (12-foot Joists)	HSS14x10x8 (11-foot columns)
1st Story	80	320	320	1120	320
2nd Story	80	160	160	560	320
3rd Story	80	160	160	560	320
		W10x49 (12-foot beams)	W10x49 (40-foot beams)	W10x49 (12-foot Joists)	HSS14x10x4 (11-foot columns)
4th Story	80	160	160	560	320

Figure 20: A table with a breakdown of all the steel used to create the residential mods.

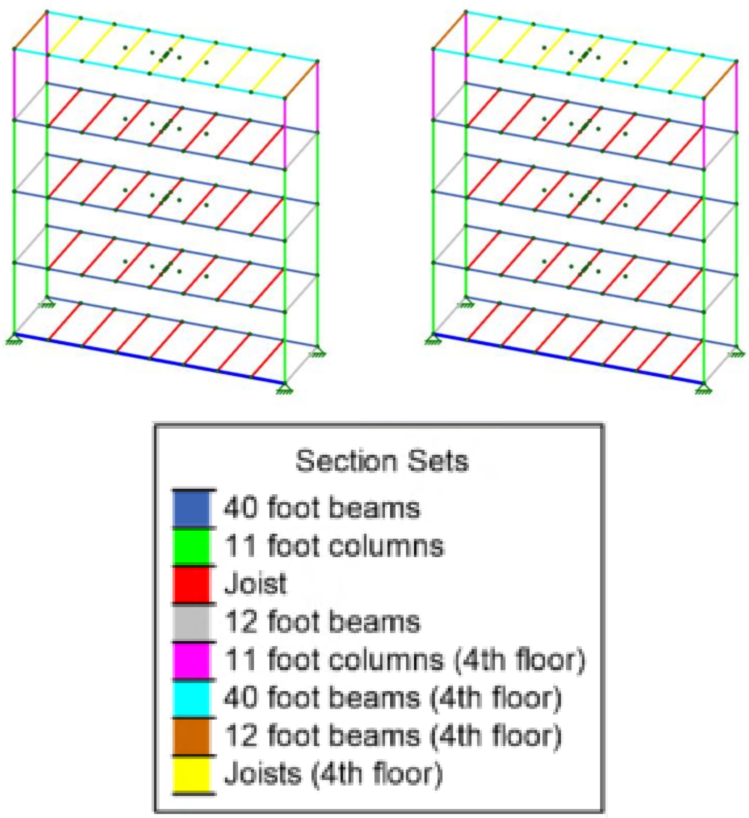

Figure 21: This figure shows the residential model that was created in RISA, with a list of the color-coded section set each member is defined under. Additional viewpoints and angles of the residential mod units can be found in the appendix.

There are 80 mods per floor arranged in an L-shape that make up the structural steel for the residential suites where students will leave. There is a gap at the bend of the L that will be traditionally constructed for all 4 floors as seen in *Figure 2*. The suites are made from 5 mods, most of which stem along the longer part of the L and the first three floors are identical, with the same layout and member sizes. The fourth story layout will be the same layout with slightly different sized beams. The hallway mods will be wedged in between the residential mods and create a hollow tube inside the L (as seen in *Figure 22)* and there will be 18 hallway mods in total per floor, and 72 total hallway mods for all four stories.

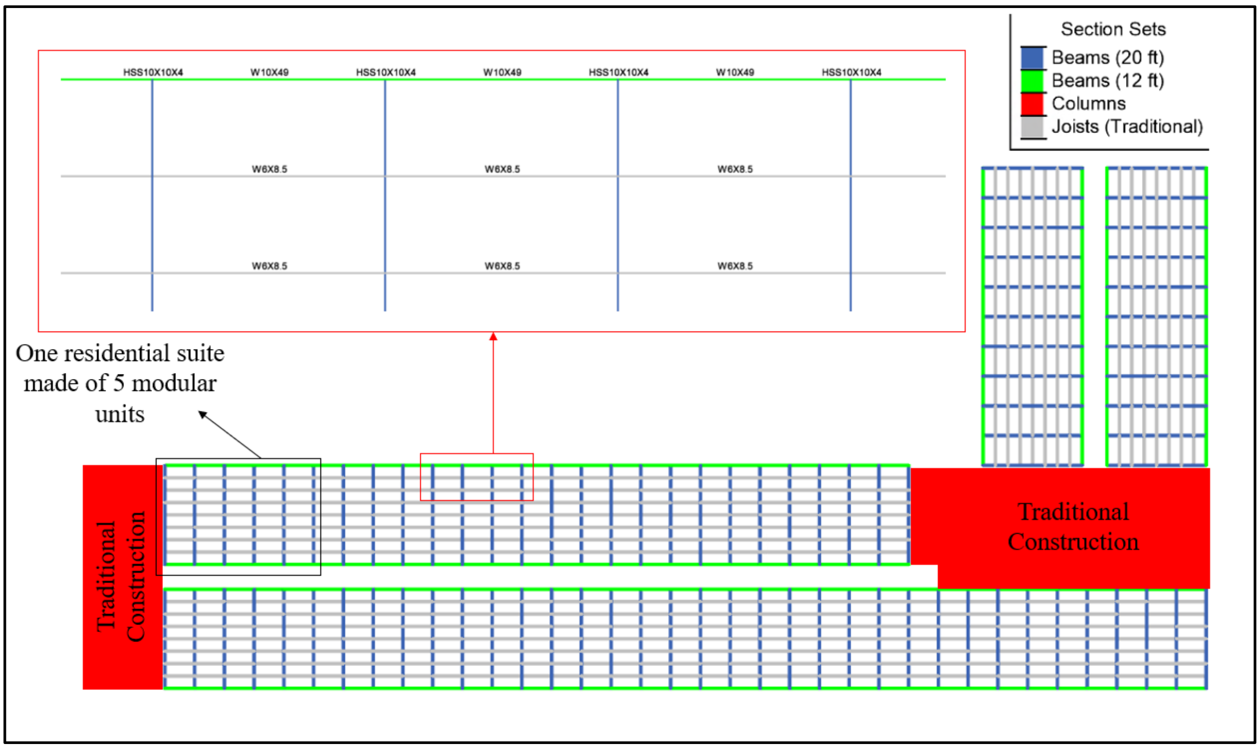

Figure 22: This shows the layout for the residential mods first 3 floors of structural steel. The areas in red will be constructed traditionally and the hallway mods are located on the white stripes in the middle of the long and short end of the L.

Structural Steel (hallway modular units):

In the design process, a second prefabricated unit that will serve as the hallways/corridors was designed as the dimensions are slightly different between the hallway mods and residential mods. The loadings for the hallway units are very similar to the loadings for the residential units except the live loads were increased due to higher foot traffic in hallways in comparison to student residences. Minimum live loads for corridors are specified as 100 psf in table 4-1 of ASCE 7. All the other loads are identical to the residential modular units loads as seen in *Table 23.*

The model seen below in *Figure 25* is very similar to the residential mods and in turn was designed almost the same way, with a few minor changes. In the floor plans, the hallways are wedged in between and perpendicular to the residential units. Thus leading the dimensions of the hallway mods to be 10 feet wide by 24feet long (the length of two residential mods) and 11 feet tall. Since these modular units are a bit smaller in size than the residential mods, all four stories of mods are identical, unlike the residential mods that have altered fourth-floor mods and only have 3 joists per mod. The model as shown below contains 16 HSS14x10x8 columns (gray in *Figure 25*), strategically designed to match the columns of the residential mods, making intermodular connections stronger and easier to implement. Between the four stories, there are also 10 W14x68 24-foot beams (blue in *Figure 25*), 10 W14x68 10-foot beams (red in *Figure 1*), and 15 W14x68 10-foot joists (green in *Figure 25*). There will be a total of 18 hallway mods per floor or 72 hallway mods in total for all 4 floors. A total of 108 W14x68 10-foot beams, 108 W14x68 24-foot beams, 162 W14x68 10-foot joists, and 288 HSS14x14x8 11-foot columns will be used to create the hallway mods (*Figure 24*).

	Hallway Modular Units					
	Dead Loads (psf)	Live Loads (psf)	Roof Live Loads (psf)	Snow Loads (psf)	Earthquake Loads (psf)	Wind Loads (psf)
1st story	79	100	N/A	N/A	4.78	27.16
2nd story	79	100	N/A	N/A	11.64	28.44
3rd story	79	100	N/A	N/A	17.46	29.91
4th story	85	N/A	20	43.3	24.4	15.51

Figure 23: A spreadsheet with all loadings that our hallway mods will undergo and was designed to uphold.

	Structural Steel (Hallway Units)				
	Mods	W14x68 (10-foot beams)	W14x68 (24-foot beams)	W14x68 (10-foot Joists)	HSS14x10x8 (11-foot columns)
1st Story	18	72	72	108	72
2nd Story	18	36	36	54	72
3rd Story	18	36	36	54	72
4th Story	18	36	36	54	72

Figure 24: A table with a breakdown of all the steel used to create the hallway mods

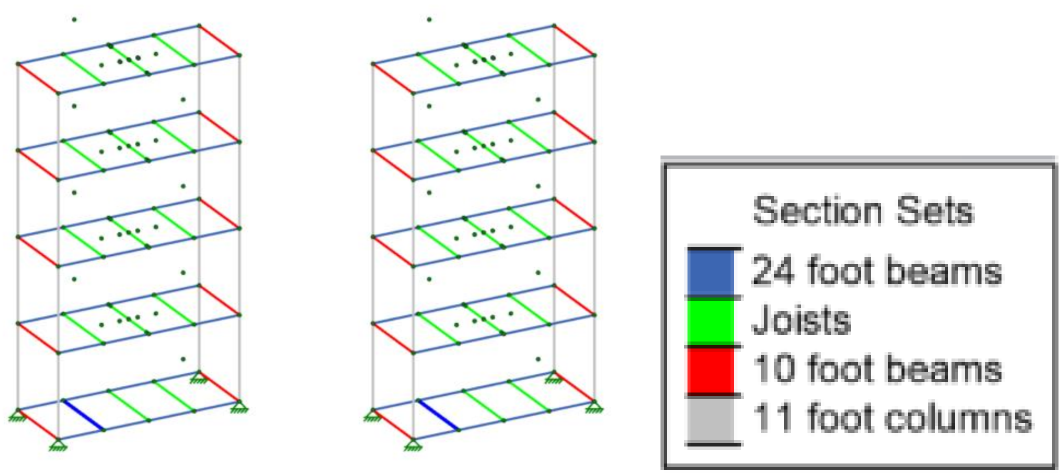

Figure 25: This figure is the model for the hallway mods (on the left) and the corresponding section sets for the members (on the right). Additional viewpoints and angles can be found in the appendix.

Vibration Analysis:

In order to analyze the mode frequencies of the structure it is important to include many degrees of freedom in the steel design. For every degree of freedom, there is a corresponding mode frequency. An efficient method of introducing these degrees of freedom into the model is adding nodes. The split member feature was used to section each individual member into five separate members. This meant that over a thousand mode frequencies in the X, Y, and Z directions could be generated. *Figure 26* below shows the model with the added nodes.

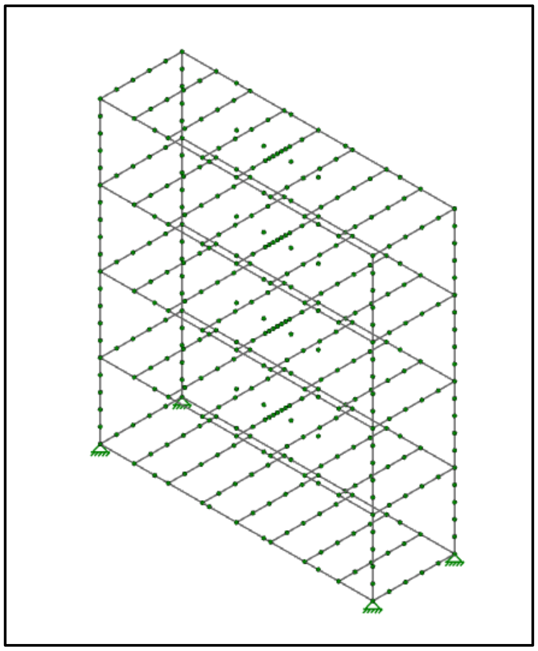

Figure 26: This is the Risa model with each individual member split by four nodes to generate better dynamic analysis results.

Once a sufficient number of modes could be generated due to the increased nodes, the dynamic analysis could begin. Using the Risa3D Dynamic Analysis tool, a list of mode frequencies was generated. In *Figure 27* below, the chart includes the first 15 modes, frequencies, corresponding period, and mass concentration per mode.

Frequencies

	Mode	Frequency (Hz)	Period (Sec)	SX Participation	SY Participation	SZ Participation
1	1	0.724	1.382	67.428		
2	2	1.044	0.957			68.196
3	3	1.693	0.591			
4	4	2.678	0.373	7.554		
5	5	2.79	0.358		4.316	
6	6	2.798	0.357			
7	7	2.858	0.35		1.433	
8	8	2.901	0.345			
9	9	2.989	0.335		1.352	
10	10	3.003	0.333			0.037
11	11	3.042	0.329		61.547	
12	12	3.045	0.328		0.667	
13	13	3.047	0.328			0.063
14	14	3.256	0.307			7.545
15	15	4.793	0.209			
16	16 (residual)	NA	NA	25.018		
17	17 (residual)	NA	NA		30.681	
18	18 (residual)	NA	NA			24.147
19	Totals:			100	100	100

Figure 27: These are the first 15 mode frequencies of the steel model.

The first mode, often known as the natural frequency, in the X-direction is 0.724 Hz with a period of 1.382 seconds. 67.43% of the mass of the structure contributes to this mode frequency. The second mode is the most significant for the Z-direction. The frequency is 1.044 Hz with a period of 0.957 seconds. 68.20% of the mass of the structure contributes to this mode frequency. In a typical dynamic analysis, a seismic load is added forcing a frequency on the structure. These frequencies are then compared to the natural frequencies of the structure to predict excitation behavior. Due to an unexpected error in the program, an equivalent static analysis was conducted to predict the structure's behavior during seismic loading.

The equations for calculating the base shear on the structure are shown in *Figure 28* below. The most important parameters for calculating the base shear are the seismic weight (W) and the period of the natural frequency (T). The seismic weight consists of the mass of the structure. The period of the first mode was used for this analysis. The seismic weight is 154.56 kips and the period is 1.382 seconds. (The full calculations can be found in Appendix Q).

$$V = \frac{S_{D1}W}{T(R/I)} \qquad (2.10a)$$

but not to exceed

$$V_{\max} = \frac{S_{DS}W}{R/I} \qquad (2.10b)$$

and not less than

$$V_{\min} = 0.044 S_{DS} I W \qquad (2.10c)$$

Figure 28: Equations to solve for base shear.

The base shear calculated was 0.796 kips. The maximum base shear was 3.62 kips and the minimum base shear was 1.275 kips. Because the original base shear value falls below the minimum base shear, the minimum value was used for this analysis. Next, a spreadsheet was made to organize the induced seismic load by floor (*Figure 29*). The aim was to create a force distribution per floor to simulate the seismic loads from an earthquake.

K is a factor found by using:

$$k = 1 + ((T-.5)/2)$$

The fourth column is simply the weight of the floor times the height of the floor to the power of k. This creates a trend of larger values as the building height increases. The fifth column is the 4th column value at a particular floor divided by the sum of the fourth column. This creates coefficients that when multiplied by the base shear of 1.275 kips give the force distribution of the seismic load per floor.

Floor	Weight	Floor Height	Weight x height^k	(Weight x height^k)/Σ(weight x height^k)	Force Applied
Roof	40,800	44	9525229.973	0.480216078	0.6122755
Fourth	37,920	33	5848532.613	0.29485476	0.375939819
Third	37,920	22	3260614.171	0.164384414	0.209590128
Second	37,920	11	1200923.07	0.060544739	0.077194543

Figure 29: Force distribution of the seismic load per floor.

The model below shows the isolated seismic loading per floor (*Figure 30*). The 2nd floor experiences 0.077 kips, the third-floor experiences 0.21 kips, the fourth floor experiences 0.376 kips and the roof will experience 0.612 kips.

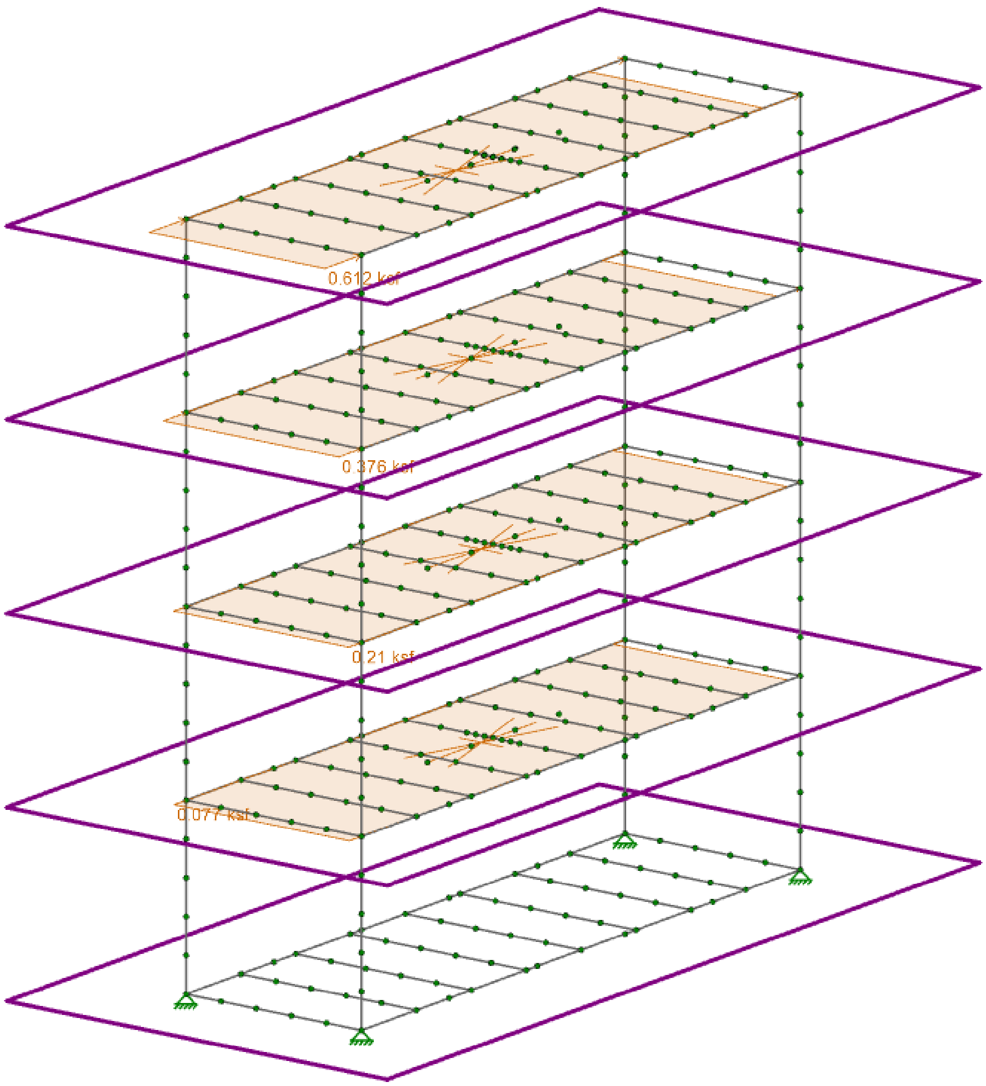

Figure 30: Distributed seismic loads per floor added to the steel Risa model.

Following the distributed seismic loads, a code check (*Figure 31*) was conducted to confirm that all 391 steel members within the structure would hold under these forces. The code check comes in the form of a chart that includes every member and its structural properties. By right clicking a member, a detailed report can be generated showing the member passing the required strength tests. Also, if a member failed under the seismic load, it would be highlighted in red. For this code check the table was sorted by Unity Check (UC) max to min. If the UC of a member was over 1, then it failed. The highest UC exhibited was member 391 (M391) with a value of 0.777 confirming that all members passed.

Member AISC 15th (360-16): ASD Steel Code Checks (By Combination)

Hot Rolled Steel | Cold Formed Steel | Wood | Aluminum | Stainless | Concrete

	LC	Member	Shape	UC Max	Loc[ft]	Shear UC	Loc[ft]	Dir	Pnc/om [k]	Pnt/om [k]	Mnyy/om [k-ft]	Mnzz/om [k-ft]	Cb	Eqn
1	1	M391	HSS14X10X4	0.777	2.2	0.094	2.2	y	250.393	323.353	73.737	104.314	1.139	H1-1b
2	1	M387	HSS14X10X4	0.766	2.2	0.093	2.2	y	250.393	323.353	73.737	104.314	1.139	H1-1b
3	1	M379	HSS14X10X4	0.711	2.2	0.09	2.2	y	250.393	323.353	73.737	104.314	1.147	H1-1b
4	1	M383	HSS14X10X4	0.701	2.2	0.089	2.2	y	250.393	323.353	73.737	104.314	1.147	H1-1b
5	1	M390	HSS14X10X4	0.546	2.2	0.095	2.2	y	250.393	323.353	73.737	104.314	1.214	H1-1b
6	1	M299	W10X49	0.543	8	0.148	7.25	y	388.595	431.138	70.609	150.699	1.614	H1-1b
7	1	M74	W10X49	0.543	0	0.148	0.75	y	388.595	431.138	70.609	150.699	1.614	H1-1b
8	1	M386	HSS14X10X4	0.537	2.2	0.093	2.2	y	250.393	323.353	73.737	104.314	1.215	H1-1b

Figure 31: Code check for steel Risa model.

Inter Module Connections:

As discussed above in the background section, Fitting to Fitting inter-module connections have been deemed to be the most effective and will be used to connect the modular units of the residence hall. The columns of all four floors will either be HSS14x10x4 or HSS14x10x8 (with the majority being HSS14x10x8) which means they are the same dimensions (14x10) with differing thicknesses which will be easy to connect to fittings. Also, as discussed earlier, FTF connections perform better on average than BTB, CTC, and LD overall. In a test where IMC connections were tested and ranked based on structural metrics, construction metrics, and manufacturing metrics, FTF connections ranked the highest, with FTF connections achieving the 5 highest scores as seen in Appendix J. The connections that will be used to connect the mods are FTF connections for this reason and can be seen in *Figure 32* below. This connection also has an ultimate load capacity of just over 1200 kN (*Figure 33)* or about 270 kips which is greater than the loadings the building will be put under.

Figure 32: A physical model of the FTF connection that will be used to connect the modular units. There are 3 long stayed bolts on each column that connect the gusset plate to the top and bottom columns (Deng et al., 2022).

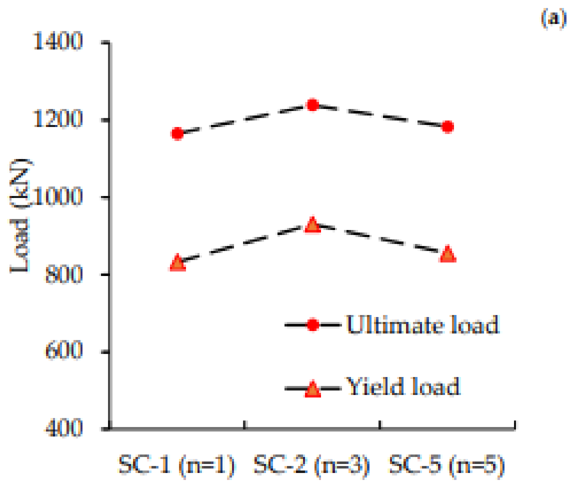

Figure 33: This graph shows the Ultimate and Yield loads for the different FTF connections, with the middle (SC-2 (n=3)) being the connection that will be used for the residence hall IMC's (Deng et al., 2022).

Reinforced Concrete - Shear Wall Design:

Shear walls are a type of structural wall that are designed to resist lateral loads. These lateral loads on a building typically come from wind and seismic activity (MacGregor and Wight, 2005). The shear walls of this residence hall are configured into both elevator shafts rather than a planar wall orientation. This is important because the elevator shafts are designed to resist potential lateral forces from all directions in the building. To design the shear walls, ACI 318-14 was used as reference. ACI 318-14 is the standard for building code requirement when designing structural reinforced concrete (*318 Building Code Topic*, 2022). When calculating the dimensions of the wall, the first step was to sum the live and dead loads from the steel structural model. These loads were turned into resultant forces in the lateral direction. An example of resultant lateral forces can be seen in *Figure 34*.

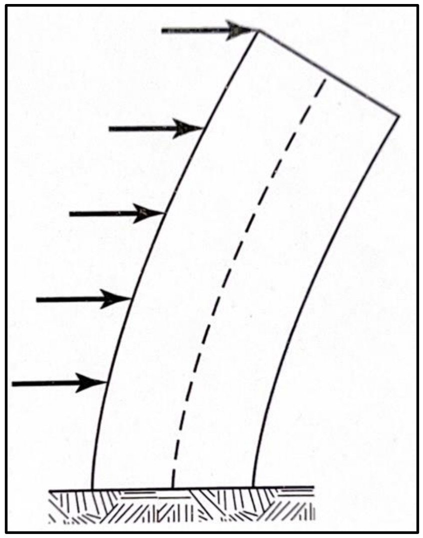

Figure 34: Shear wall with lateral forces applied. (MacGregor and Wight, 2005).

After determining the resultant forces of each floor, the strength of the concrete itself was selected. For the compressive strength ("f'c" in Appendix C) 4000 psi was chosen and for the yield strength ("Fy" in Appendix C), 60 ksi. The resulting concrete mix is relatively strong. However, in order to maximize the allowable loading on the wall, adding steel reinforcement was required. When calculating the minimum amount of steel reinforcement required ACI 318-14(2.2) is followed: $\rho = As/(bd)$.

Steel reinforcement of shear walls is required in both horizontal and vertical directions. After inputting the desired wall dimensions, the minimum steel area required in the horizontal direction must be greater than 0.3 in/ft, so No. 5 steel bars satisfy this with an area of 0.31 in^2 with 6" spacing. In the vertical direction the minimum steel area required was 0.15in^2/ft, so No. 4 bar with an area of 0.2 in^2/ft and 6" spacing satisfied the requirement as well (*Steel Rebar Sizes - Steel Rebar Stock | Harris Supply Solutions*, 2021). To better understand the placement and spacing of the bars, *Figure 35* can be used as reference.

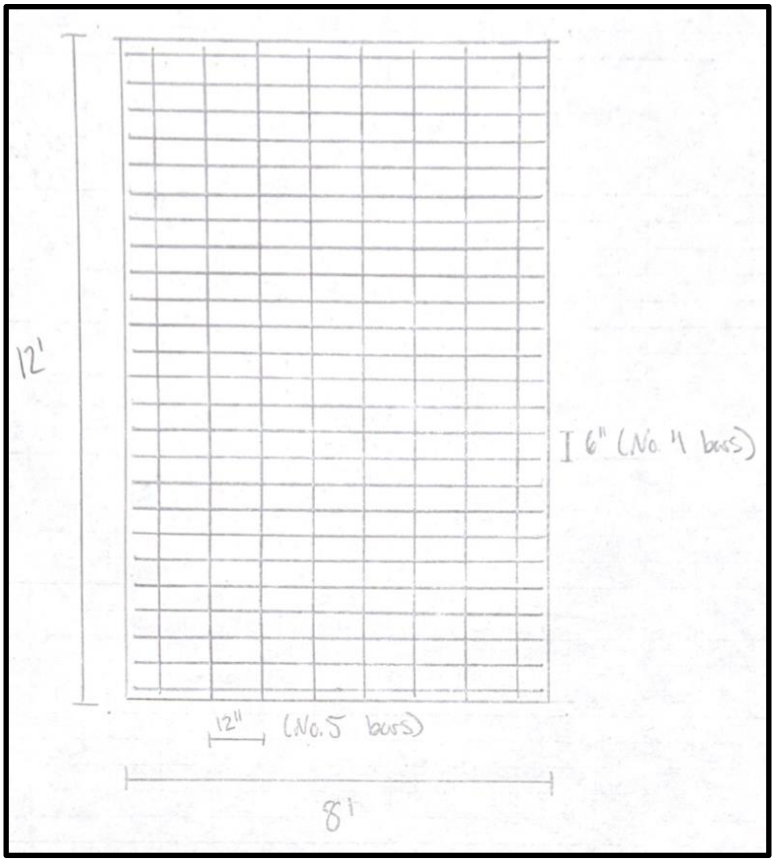

Figure 35: 8' Shear wall segment's rebar spacing with dimensions.

Reinforced Concrete - Foundation Design:

Foundations are one of the most important aspects of building design since the entire building rests upon the foundation. The two main components of a foundation are the footing and the retaining wall. Our residence hall will feature a footing around the entire building perimeter and a retaining wall around much of the building that is below the soil line. There are several options of footing shapes. However, a continuous spread wall footing was chosen (*Figure 36*).

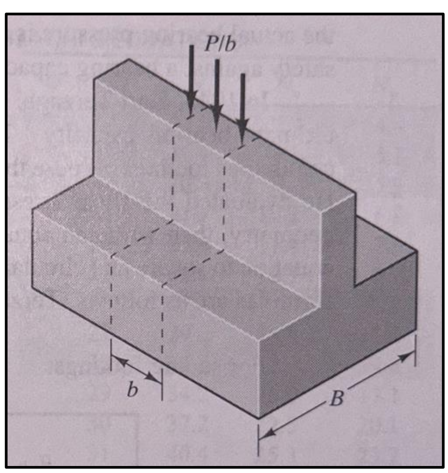

Figure 36: Continuous spread wall footing shape (MacGregor and Wight, 2005).

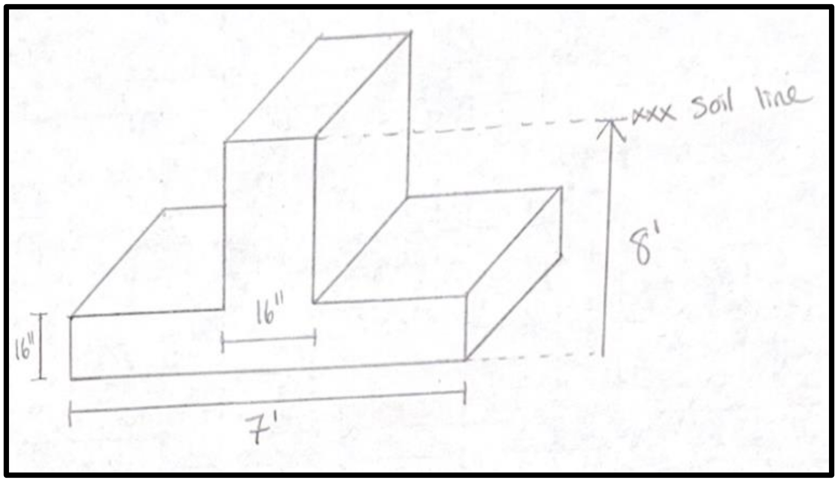

Figure 37: Residence Hall Footing Design with Dimensions.

Based on the design criteria of prefabrication as well as building loadings, the foundation was designed as seen in *Figure 37*. In addition to the concrete, steel reinforcement is also added to the footing. Every ten inches, No. 5 (0.31mm) bars will be spaced out in the horizontal direction for reinforcement. To get these calculations, an excel spreadsheet was created (appendix D), which allowed for moderation of the design. The calculations are based on an assumed soil bearing capacity of 4 ksi and an assumed soil density of 150 pcf. The bearing capacity was selected because it was typical soil load. The soil density was chosen because it is a denser option, which leads to a more conservative calculation.

Reinforced Concrete - Floor Slab:

To further connect our modular units together we are joining them together with reinforced concrete floor slabs. Each floor will be pre-fabricated and shipped to the project site with the rest of its respective modular unit. This means our design will feature cha one way slab with 12-foot space, as seen in figure 38. When designing the floor slab, itself, a 6 in thickness was chosen along with a compressive strength ("f'c" in Appendix P) 4000 psi. Steel rebar reinforcement design indicates that using No. 4 bars at 12-inch spacing would be sufficient for this floor slab. The bar will be placed 5 inches deep in the concrete, leaving 1 inch of cover as seen in figure __5__. These specifications were chosen to create a durable floor that could not only support dead and live loads, but also help increase the stability of the modular units.

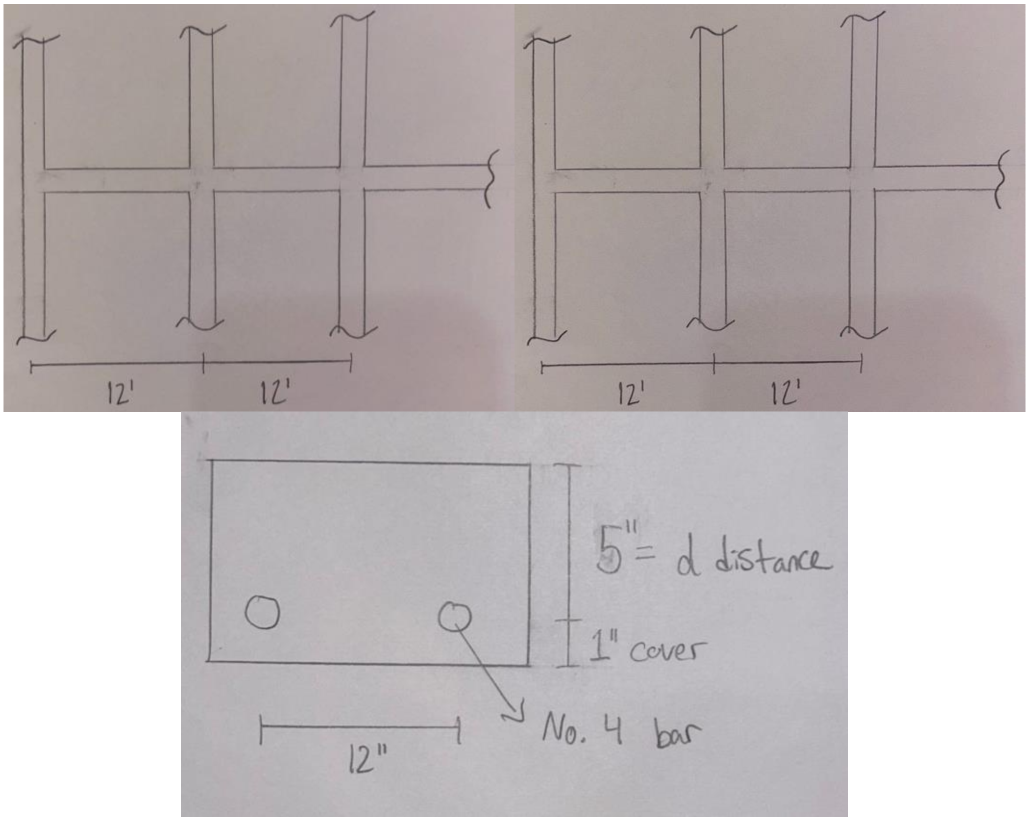

Figure 38: Placement of one-way slab design(left) and cross section of reinforced slab design with dimensions(right)

Between each floor slab that is placed in the project there will be a closure pour connection, as seen in figure 39. A closure pour was chosen to do this because it allows the slabs to be shipped individually and connected on site. Also, the same rebar size being used in the floors can be given a slightly longer development length and used for the connections. This makes a closure pour connection simple and quick for a prefabricated building.

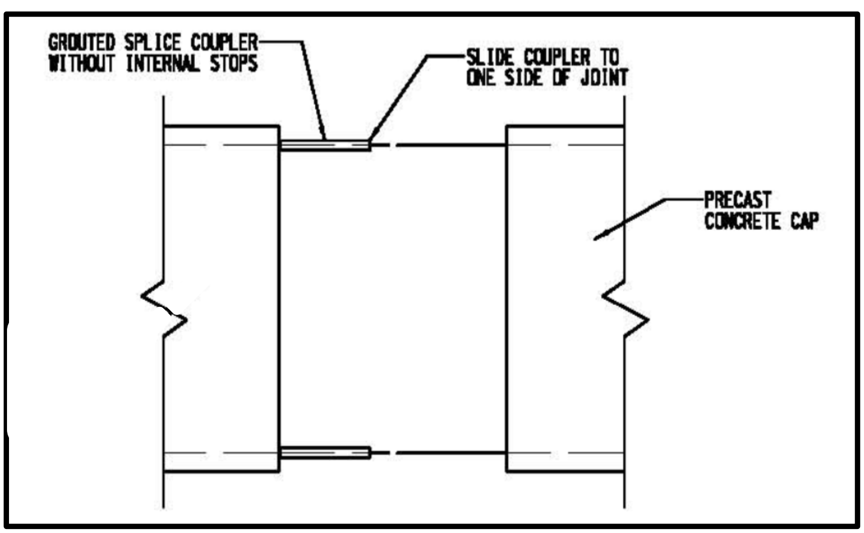

Figure 39: Diagram of closure pour between two concrete floor slabs. (*Prefabricated Bridge Elements and Systems - ABC - Accelerated - Technologies and Innovations - Construction - Federal Highway Administration*, n.d.)

Architectural Design

Using Autodesk Revit, the following building model (*Figure 40*) was designed in accordance with the structural design of both the modular designed, 'L-shaped' portion of the building as well as the traditionally constructed first floor lobby and parking garage underneath. Using the appropriate dimensions given through design calculations, the Revit model accurately displays the size and scale of the building. The exterior siding of the building is made from aluminum composite panels and windows are distributed identically for each of the five types of prefabricated mods that make up one residential unit or suite.

Figure 40: Rendered 3D exterior view of the building.

The topography of the site was incorporated to best represent the land of the WPI Townhouse site where the building would be built. This includes an approximately eight foot slope decline along the length of the building, landing on either side of the building where the wall openings of the parking lot begin. In Revit, a building pad was created into the incline of the hill. As demonstrated in *Figure 36*, the northern and western sides of the site are lined with trees to serve as a boundary for the property line.

The parking garage is included underneath the first-floor lobby on the inside of the building's 'L-shape' (*Figure 41*). Access to the garage is through the site parking lot that is on the exterior of the garage. The site lot (*Figure 40*) includes 97 parking spaces while the garage includes 139, coming to a total of 236. This component of the building is very important for student life and experience since the building is farther from campus than other upperclassmen residence halls, and although the walk from the WPI townhouses is still relatively short, the parking lot adds a large sense of convenience. Retaining walls are placed against the southern side of the building that touches the soil and structural concrete columns support the first-floor lobby above.

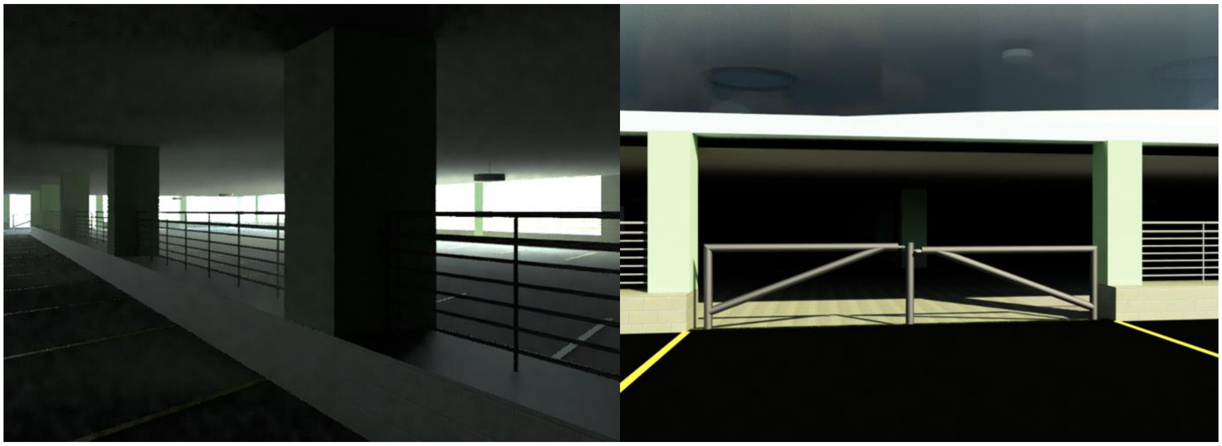

Figure 41: Parking garage underneath the first-floor lobby.

On the east side of the building is the entrance to the lobby with access provided by the ground level staircase and ADA accessible ramp (*Americans with Disabilities Act Ramp Slope - HandiRamp*, n.d.). There is also ground access through the parking garage entrance in which entrants may utilize the egress stairs or elevator in the east-most stairway to access the rest of the building. Both stairwells (*Figure 42*) and elevator shafts on either end of the building provide access to the roof as well. The elevator door openings have also been designed to be utilized as service elevators to transport bulky objects.

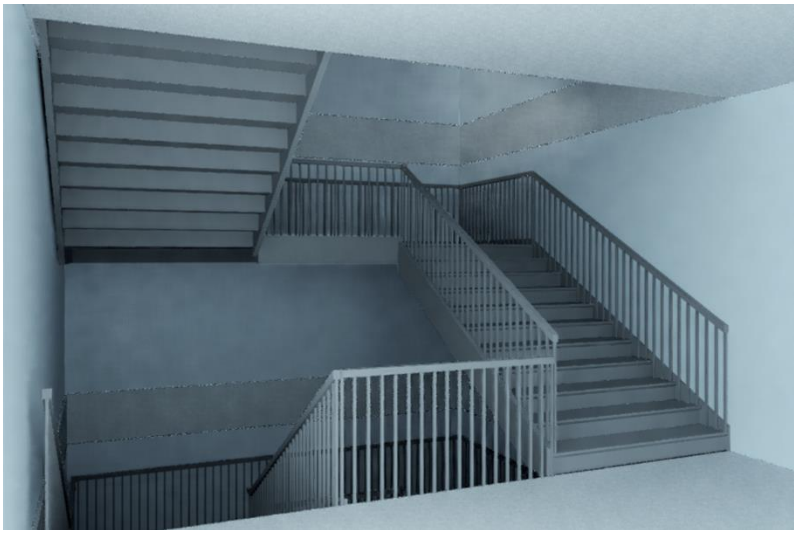

Figure 42: Egress staircase within one of the building's stairwells.

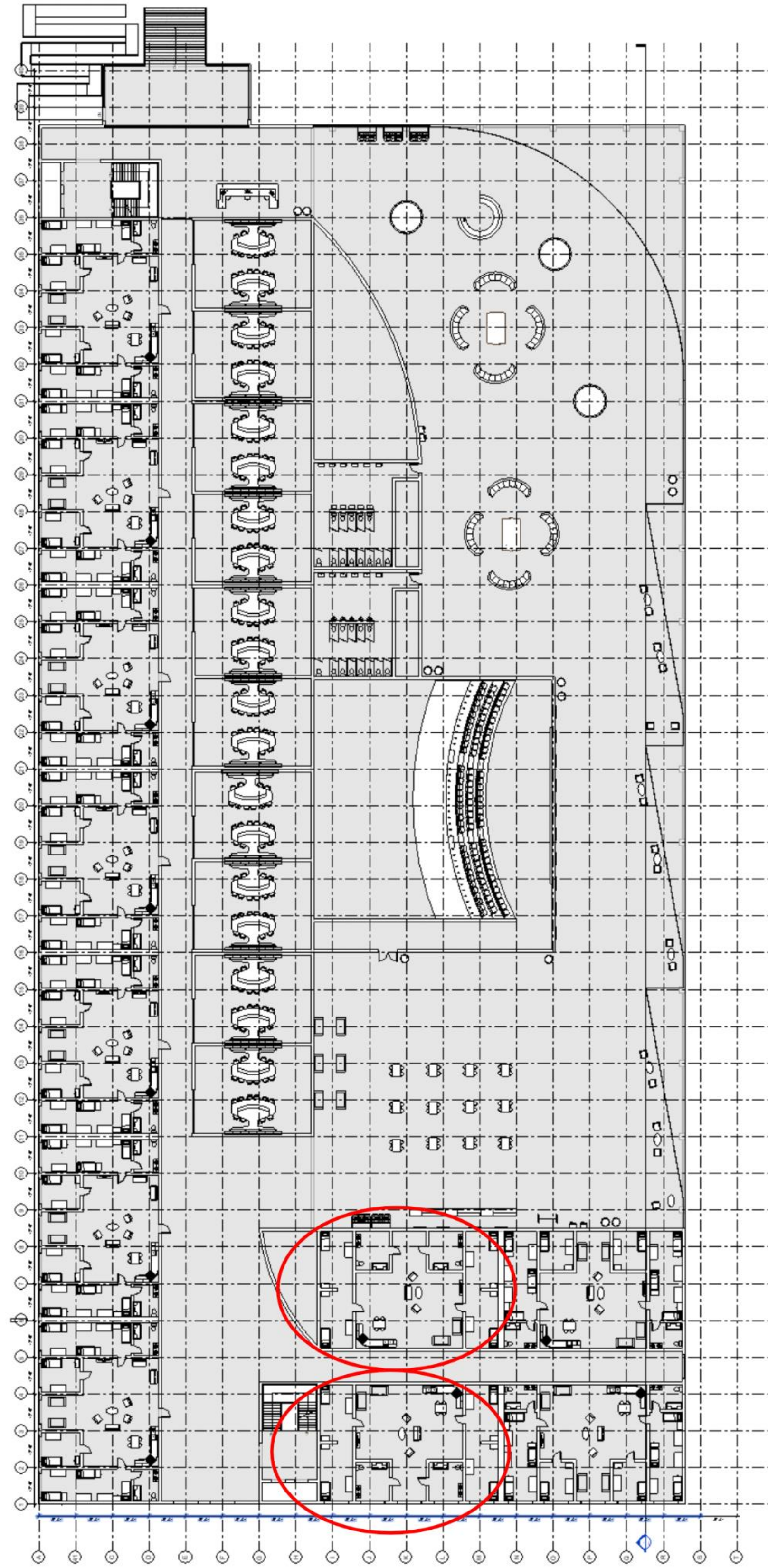

Figure 43: Level 1 Floor Plans, including lobby, theater, Techsuites, & ADA units circle in red.

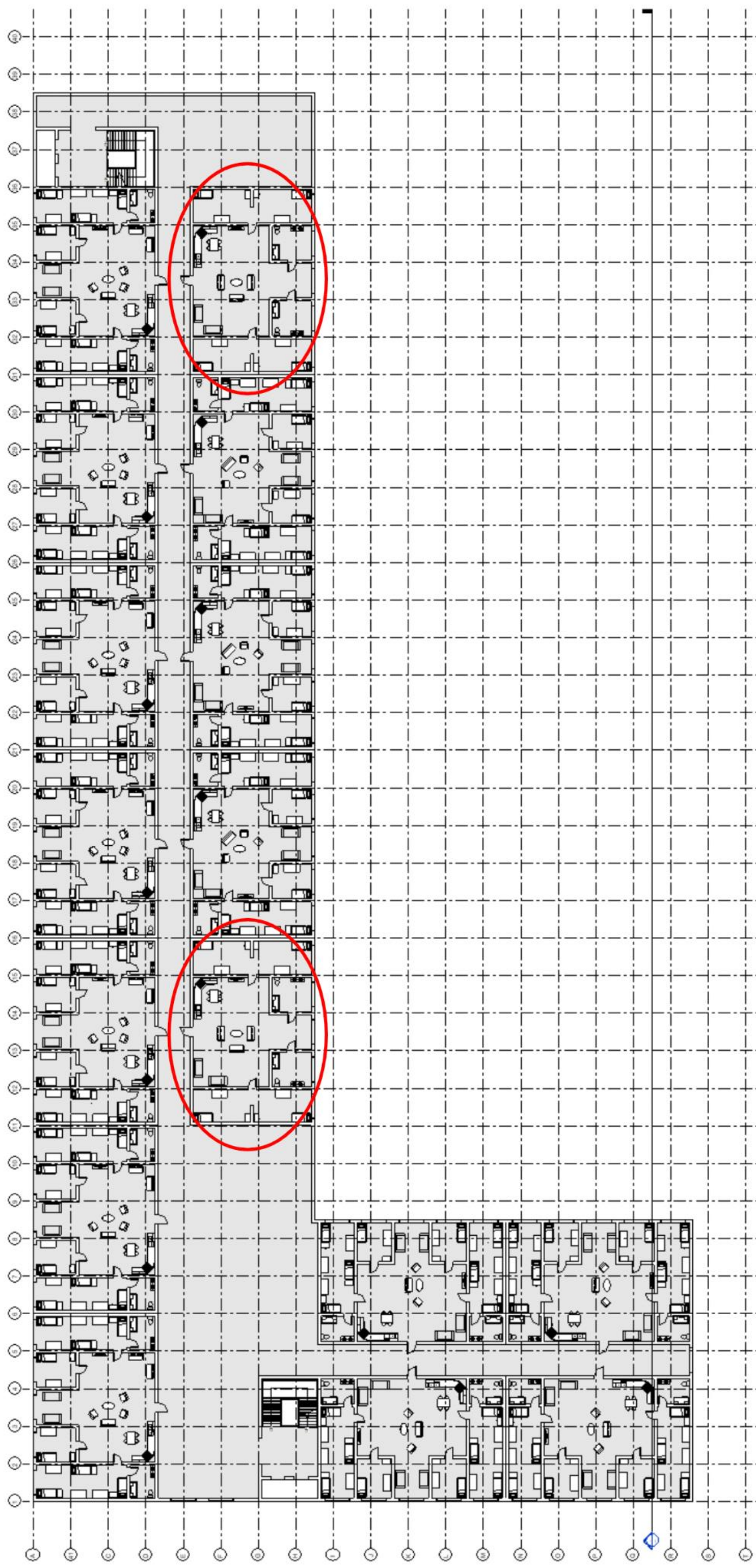

Figure 44: Floor plans for levels 2-4. ADA units are circled in red.

The floor plans used to design each floor's model were produced accordingly. As demonstrated in *Figures 43* & *44,* ADA units for each floor have been circled. The modular unit design and layout for floors two, three, and four are identical (*Figure 40*).

The design of non-ADA units on the first floor consists of the same layout. However, the mods on the long, inner-side of the 'L-shape' have been designed to include study spaces, otherwise known as "Techsuites", for students to have twenty-four-hour access to do work and concentrate in an isolated, quiet space (*Figure 45*).

Figure 45: First floor Techsuite.

The rest of the first floor includes the large lobby space as depicted in *Figure 46.* This area was designed with heavy consideration towards the promotion of mental health and socialization. Firstly, the concept of natural light and openness was decided as a key component of this lobby space. The large glass facade and the detailed, jagged design allow for plenty of natural light and social interaction (*Figure 46*). This in turn allows for a sense of community and connectedness to the outside natural landscape and street activity on Park Avenue located on the western side of the building. Residents may also be able to have a view of rooftops on WPI's campus, which is south of the lobby's glass facade. Despite being an off-campus residence hall, students in this building will not feel isolated or distant from the campus with the ability to observe much of the nearby area as well as the school itself.

Figure 46: First floor lobby space with seating and exterior view through glass facade.

The seating alongside the glass facade allows for students to have another place to interact, study, or relax. The large circular couches provide more of a lounge-style seating area for the same purposes (*Figure 46*). Also visible in this rendering is the circular, column-like fish tanks that extend from the floor to the ceiling of the lobby. Fish tanks incorporate wildlife and water, both which have been described to improve mental well-being. In other views of the lobby, the light blue color of the interior walls and the green, plant-walls can be seen as well and were included to greaten the sense of nature within the building (*Figure 47*). Light blue has been selected as the wall color for the lobby due to its proven calming effect. Supplementing the rounded walls, open/airy space, and ample natural light, this lobby is complete with many considerations for mental health and sociability (Channon, 2019).

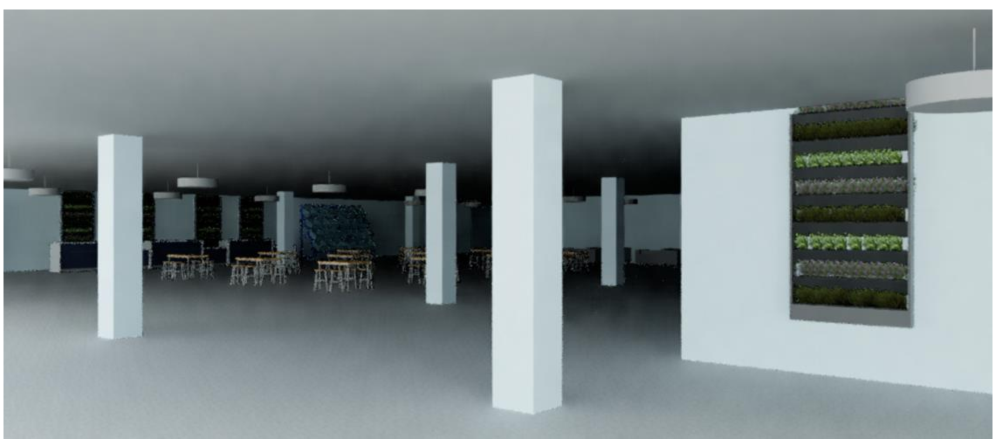

Figure 47: Light blue walls and plant walls in the first-floor lobby are visible in this rendering. Also shown is the food convenience store and seating area for students.

Figure 48: Theater seating in the lobby theater on the first floor.

Further amenities that are included in the first-floor lobby are the two convenience-style food counters where occupants of the building may purchase snacks or drinks. Included with this is twelve tables and seating for students to eat, socialize, or work on school assignments (*Figure 47*). Another major amenity included in the lobby is the theater (*Figure 48*), which is yet another place for residents to congregate and be among one another. The theater is included with ample seating as well as a large projector screen. This space can be utilized for scheduled movie-nights (facilitated through the residence hall staff) but also has the potential to be reserved by students during available times to present on the large screen.

The prefabricated mods all have an identical floor layout except for the two ADA units on each floor, which have a lesser number of beds and larger bathroom space. To accommodate handicap accessibility, ADA units provide much more open space than the traditional rooms, specifically in the bedrooms and bathrooms. Wheelchair accessibility is a major factor that went into the layout design of the ADA units as well, allowing for optimal mobility in and out of rooms and increased space in the bathroom and shower.

Like the interior walls of the lobby, the interior walls of the units have a calming, light blue color. Priorly mentioned, each unit consists of five prefabricated mods put together to create a suite that will house 8 students. In non-ADA units, the two mods on either end of the unit account for a three-person bedroom (*Figure 49*) with a bathroom on the hallway side of the mod. There are also two separate single bedrooms adjacent to the end-mods on the exterior side of the mod. Each bed also comes with a desk space for work and an armoire for storage of clothes and other belongings. In the central space of each unit is a living room area, complete with couches, lounge chairs, a coffee table, and HD television. Equipped with a stove, oven, sink, refrigerator, dishwasher, and cabinets for storage, the suites are perfect for those who like to independently cook (*Figure 49*). ADA units have four beds in total, rather than eight, and the bathroom is in the bedroom space adjacent to the end-piece modules due to spatial and access considerations.

Figure 49: (Left) Three-person bedroom with beds, desks, and armoires. (Right) Kitchen space as well as living room seating and hallway door for unit.

Sustainable System Design

Solar Power:

The total surface area of the proposed roof design is 4,765.93 m^2. Looking at current leading PV panels on the market, 400W panels are one of the more powerful and efficient modules available. A typical commercial solar panel is 2.024m x 1.024m x .04m (2.073m^2). A 400W panel covering 2.073m^2 will produce 192.99 Watts/m^2. It is also important to note that the solar panel array will likely not be the only machinery on the roof. Air handling units or other systems may be needed for the residence hall. Therefore, the long leg of the roof's surface was dedicated to the solar array. The total area of this section is 3,010.06m^2. To comply with building code, solar panels must be at least 12 inches from the edge of the roof. In addition, it is good practice to leave space for the conduit to allow for easier maintenance in the future. As a result, 109.7m x 24.5m or 2695 m^2 was dedicated to the solar array. At a maximum of 192.99 Watts/m^2 being produced, there is a max of 538784.4 kWh that can be harnessed per hour for that surface area. However, spacing between rows of solar panels are necessary to account for the potential tilting of the panels and the potential thermal expansion of the aluminum frames that hold the PV panels in place. As a result, there will be 5” spacing between the rows of solar panels and 1” inter-row panel spacing (Diehl, 2021). Taking these measurements into account, an array of 51 by 22 solar panels for a total of 1,122 solar panels can be placed on the roof for a potential total of 448.8 kWh produced each hour. *Figure 50* below shows the average kWh per day and per month using a solar calculator for the Worcester area.

Month	Sunlight Hours	KWh per day	KWh per month
January	2.06	924.528	28660.368
February	2.88	1292.544	36191.232
March	4.03	1808.664	56068.584

April	5.21	2338.248	70147.44
May	5.85	2625.48	81389.88
June	6.1	2737.68	82130.4
July	6.12	2746.656	85146.336
August	5.69	2553.672	79163.832
September	4.56	2046.528	61395.84
October	2.93	1314.984	40764.504
November	2.16	969.408	29082.24
December	1.55	695.64	21564.84

KWh per Year:	671705.496

Figure 50: Maximum projected energy production based on average sunlight hours in the Worcester Area.

The total floorspace of the residence hall is 19063.7 m^2. The facility's energy consumption is estimated to be 4,994,689.4 KWh per year. This is based on the 262 kWh/m^2 per year estimate in *Figure 16*. The solar power system can produce a maximum power output of 671,705.5 KWh per year. This means that the solar system at max power can produce 13.4% of the projected energy consumption.

The solar power system will be roof mounted using aluminum racks. The racks will allow for the panels to be mounted either flush, or tilted up to an angle of 15 degrees. The ability to tilt the PV panels allows a greater portion of the surface area of each panel to face the sun based on the height of the building and time of day. Controllers installed with the system can automate the tilting of these panels for optimal performance. In addition to controllers, string inverters and DC power optimizers will be installed to prevent widespread outages caused by shade and to ensure maximum power draw and therefore maximum power production.

The cost for the system is an important factor to consider when trying to determine the viability of the system. According to energy rates in Massachusetts, one kilowatt hour costs $0.23 (EnergySage, 2023). The cost of PV panels can be estimated by a commercial market rate in Massachusetts of $1.50 per watt. (Hyder, 2022). Additionally, the framing can be estimated at $0.10 per watt (Lozanova, 2021).

Cost Estimation = $673,200 (panels) + $44,880 (framing) = $718,080.

The average efficiency of a solar panel system over the first five years of its lifespan is 97.125%. This means the system will produce on average 652393.96 kWh per year. Based on the cost of electricity and the estimated energy consumption of the residence hall, the projected cost of power without the solar panel system is $1,148,778.57 per year. The projected cost of power with the solar panel system is $994,286.30 per year. The solar panel system is estimated to save

WPI $150,050.61 per year. Over the first five years of its lifespan, the system will save $750,253.06 from the utility bill effectively paying for itself.

Loading Considerations

Next it is important to analyze the structural design of the solar array. The weight of each panel in the system is approximately 20.3kg. With a total of 1122 PV panels, the total weight of the system is 22776.6 kg or 50213.81 lbs. The dead load of just the PV panels adds 1.006 lb/ft across the roof without accounting for the framing. In addition, the connections between the roof and framing result in concentrated loads along those areas. As a result, a conservative estimate of 6 psf can be used to account for the solar array and any additional roof equipment. A roof steel beam design in Appendix B shows the calculations for the added 6 psf of distributed dead load.

Commercial solar panels can be rated for wind loads up to 100-115 psf. To calculate the wind load on the PV panels, equation 29.4-5 in *Figure 51* was used. (The full calculations are located in Appendix F).

$$p = q_h K_d (GC_{rn}) \ (\text{lb/ft}^2) \qquad (29.4\text{-}5)$$

Figure 51: Equation 29.4-5, ASCE 7 to find Pressure.

The three parameters needed to find the pressure due to the wind load are the Velocity Pressure (qh), Wind Directionality Factor (Kd) and the Nominal Net Pressure Coefficient (GCrn). *Figure 52* below shows the equation to find the velocity pressure.

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Figure 52: Equation 26.10-1, ASCE 7 to find Velocity Pressure.

This equation includes four additional parameters: Velocity Pressure Exposure Coefficient (Kz), Topographical Coefficient (Kzt), Ground Elevation Factor (Ke) and the Wind Speed (V).

Figure 53 below shows the table for finding the Velocity Pressure Coefficient. The construction site will experience Exposure B conditions and the mean roof height is 65 ft. From the table that corresponds to a value of 0.845.

Table 26.10-1. Velocity Pressure Exposure Coefficients, K_h and K_z.				
Height above Ground Level, z or h		Exposure		
ft	m	B	C	D
0–15	0–4.6	0.57 (0.70)*	0.85	1.03
20	6.1	0.62 (0.70)*	0.90	1.08
25	7.6	0.66 (0.70)*	0.94	1.12
30	9.1	0.70	0.98	1.16
40	12.2	0.74	1.04	1.22
50	15.2	0.79	1.09	1.27
60	18.3	0.83	1.13	1.31
70	21.3	0.86	1.17	1.34
80	24.4	0.90	1.21	1.38
90	27.4	0.92	1.24	1.40
100	30.5	0.95	1.26	1.43
120	36.6	1.00	1.31	1.48
140	42.7	1.04	1.34	1.52
160	48.8	1.08	1.39	1.55
180	54.9	1.11	1.41	1.58
200	61.0	1.14	1.44	1.61
250	76.2	1.21	1.51	1.68
300	91.4	1.27	1.57	1.73
350	106.7	1.33	1.62	1.78
400	121.9	1.38	1.66	1.82
450	137.2	1.42	1.70	1.86
500	152.4	1.46	1.74	1.89

Figure 53: Table 26.10-1 to find Velocity Pressure Exposure Coefficient.

Next the Topographical Factor was determined using the equation below (Figure 54) and table in *Figure 2*.

$$\boxed{K_{zt} = (1 + K_1 K_2 K_3)^2}$$

Figure 54: Equation to find the topographical factor.

K1 is found by taking the height of the hill adjacent to the site over the length of the hill. The hill on this site most closely resembles a 2D escarpment. The height of the hill is 15 ft and its length is 40 ft. The ratio of the height over length equals .375. From the table this gives a value for K1 of .32. For K2 the distance of the building from the hill is needed. The long portion of the building with the solar array is approximately 150 ft from the crest of the hill. x/L gives a ratio of 3.75. From the table that gives a value for K2 of .0675. For K3 the height of the building is needed. The ratio of the height of the building by the length of the hill gives a ratio of 1.625. From the table that gives a value for K3 of .005. When plugged into the equation the Topographical Factor came out to almost exactly 1. This is because the building is far enough away from any large topographical features that would potentially increase the wind loads.

For the Ground Elevation Factor, only the height above sea level is needed. The site is 499 ft above sea level. Using the table from *Figure 2,* the Ground Elevation Factor is .9821 after interpolation.

The design wind speed is 120mph. Taking the values found in the previous part, the velocity pressure qh = 30.6 psf.

Next, using the Wind Directionality Factor table below (*Figure 55*) found in chapter 26 of ASCE 7, we can see from the table below that rooftop equipment has a Kd value of .85.

Table 26.6-1. Wind Directionality Factor, K_d.

Structure Type	Directionality Factor K_d
Buildings	
Main wind force resisting system	0.85
Components and cladding	0.85
Arched roofs	0.85
Circular domes	1.0*
Chimneys, tanks, and similar structures	
Square	0.90
Hexagonal	0.95
Octagonal	1.0*
Round	1.0*
Solid freestanding walls, roof top equipment, and solid freestanding and attached signs	0.85
Open signs and single-plane open frames	0.85
Trussed towers	
Triangular, square, or rectangular	0.85
All other cross sections	0.95

*Directionality factor $K_d = 0.95$ shall be permitted for round or octagonal structures with nonaxisymmetric structural systems.

Figure 55: Table to find Wind Directionality Factor.

The last part of the equation is finding the Net Pressure Coefficient (GCrn). That is done by using the following equation and table from chapter 29 in ASCE 7 (*Figure 56 & 57*).

$$(GC_{rn}) = (\gamma_p)(\gamma_c)(\gamma_E)(GC_{rn})_{\mathrm{nom}}$$

Figure 56: Equation to solve for the Net Pressure Coefficient

Yp was equal to 0.9 + the ratio between the height of the parapet and the roof height = 0.977. Yc was equal to 0.6 + .06 * the chord length of one solar panel. The chord length of one panel is 6.64 ft. Yc came out to 0.9984. Y_E refers to an uplift load to account for the solar panels that are most exposed to the lateral forces of the wind. ASCE classifies exposed panels as any panel within 1.5 chord lengths from the edge of the array. For exposed panels use $Y_E = 1.5$. Otherwise use 1 for the non-exposed panels. Lastly, the Nominal Net Pressure Coefficient is found by taking the nominal area of one panel (22.31 ft^2) and using that value to determine the Nominal Net Pressure Coefficient based on the 0-15 degree tilt angle graph. From the graph, $(GCrn)_{nom}$ = 0.96. Solving for the exposed panels, the Net Pressure Coefficient is 1.405. For the non-exposed panels, the Net Pressure Coefficient = 0.936.

$\gamma_p = \min(1.2, 0.9 + h_{pt}/h)$;
$\gamma_c = \max(0.6 + 0.06L_p, 0.8)$; and
γ_E = 1.5 for uplift loads on panels that are exposed and within a distance $1.5(L_p)$ from the end of a row at an exposed edge of the array; $\gamma_E = 1.0$ elsewhere for uplift loads and for all downward loads, as illustrated in Figure 29.4-7; a panel is defined as exposed if d_1 to the roof edge $> 0.5h$ and one of the following applies:

1. d_1 to the adjacent array $> \max(4h_2$, 4 ft (1.2 m), or
2. d_2 to the next adjacent panel $> \max(4h_2$, 4 ft (1.2 m).

$(GC_{rn})_{nom}$ is the nominal net pressure coefficient for rooftop solar panels as determined from Figure 29.4-7.

When $\omega \le 2°$, $h_2 \le 0.83$ ft (0.25 m), a minimum gap of 0.25 in. (6.4 mm) is provided between all panels, and the spacing of gaps between panels does not exceed 6.7 ft (2.04 m), the procedure of Section 29.4.4 shall be permitted.

The roof shall be designed for both of the following:

Figure 57: Table for finding parameters to solve for the Net Pressure Coefficient.

The pressure on the exposed panels is a maximum of 36.52 psf which is well below the typical panel rating of 100-115 psf. The pressure on the non-exposed panels came out to a maximum of 24.26 psf. The full wind load calculations can be found in Appendix F.

Typically, PV panels come with warranties of 20-25 years, but advances in solar efficiency during the life span of the PV panels mean that the system can be run well past 25 years if well maintained. For example, leading panels offer annual efficiency degradation rates as low as .35% a year and efficiency rating of 89-90% after 25 years. So, the proposed 448.8 kWh system proposed will be able to power the residence hall for decades to come.

Hydronic Heating Design:

In order to understand the magnitude of the system to size the necessary boilers, a comprehensive estimate of the energy for heating is required. A good estimation is that approximately 30 BTU's will be required for every square foot of space (PlumbersStock, 2022). This will give a conservative estimate as well as ensuring system stability. The hydronic heating system will be installed in the modular portion of the building which has a combined square footage of 153,600 ft^2. For 30 BTU's per square foot, a 4.6 kBTU boiler system is required. In reliable heating design, multiple boilers are common to increase the efficiency of the overall system (D'Antonio, 2006). As a result, three boilers with a rating of at least 1.6 kBTU will be required to provide sufficient heating.

Expansion tanks are auxiliary water tanks that handle the thermal expansion of the water as it heats up to avoid excess pressure. The size of the expansion tanks is based on the temperature of the water being heated, the pressure the tank undergoes, and the volume of water the boiler can contain. The system per International Plumbing Code (IPC) should not exceed 80 psi so that value is used as a maximum working pressure. The initial water temp is 90 degrees Fahrenheit and will be heated up to 140 degrees Fahrenheit. The boilers each have a volume of 1,000L for a total of 3,000L. As a result, three 45 gallon expansion tanks will be needed to dissipate the water pressure of the system. Condensate neutralizers will be installed in order to filter the acidic condensate produced by the gas-powered boilers. Backflow preventers will also be added to avoid water from flowing in the wrong direction.

For the pump design, an estimate of the gallons per minute (GPM) is needed. The equation used to find the GPM is:

$$GPM = BTU/h / (t \times 500)$$

The BTU/h is 4.6 kBTU and the typical temperature differential (t) for hydronic systems is 20 degrees. This gives a GPM of 460. It is important to note that this is a conservative estimate and would highly likely be less after a full professional design. In this project, the pump is going to need to be able to distribute 460 GPM. The flow rate in typical hydronic systems with PEX tubing is 8 ft/s. To calculate the necessary pipe diameter use the equation:

$$\text{Pipe Diameter} = \text{sqrt}((.4088 / \text{flow rate}) * GPM).$$

This comes out to 4.85" or 5" diameter piping. As can be seen from *Figure58* below, there are 1750 RPM pumps capable of distributing water at 460 GPM in the pipe diameter range at minimal pressure head.

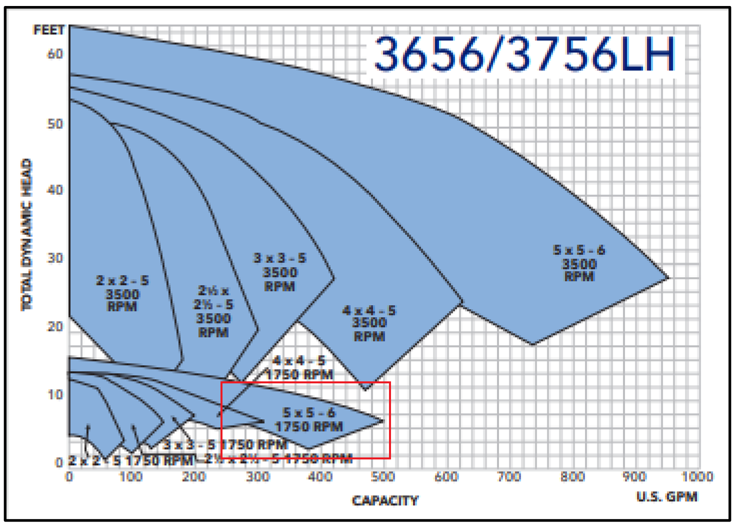

Figure 58: The chart above compares different models of pumps and their GPM performance against the total dynamic pressure head.

The final main design component is the underfloor radiant loops. Typically, 300 ft is the largest conventional length to avoid problems with pressure head. The spacing between the loops should not exceed 12 inches to ensure sufficient and comfortable heating. At 12 inch spacing the ratio between the loops and floorspace is 1:1. Each suite in the residence hall has a floorspace of 2,400 square feet meaning 8 loops are required to fully heat the space.

The cost estimation for the hydronic system equipment is given in *Figure 59* below. It is important to note that these costs are based on only the necessary components for this system design. Controllers, different types of manifolds or other equipment can be retrofitted in the future if the need arises.

Equipment	Cost
Condensing Boiler	$100,000
Expansion Tank	$6,000
PEX Piping + Radiant Tubing	$168,960
Condensate Neutralizer	$690
Backflow Preventer	$450
Centrifugal Pump	$2,900
Thermostats	$6,000
Total	$250,000

Figure 59: Cost estimate for the hydronic system equipment.

Construction Project Management Plan

Schedule:

A detailed construction schedule with 32 schedule items was created. Many steps that are typically included towards the end of the schedule were eliminated because of the prefabrication process. For this building, most construction labor that would be required for floors two through four have also been completed in the prefabrication phase. The first half of the schedule involves traditional construction, which is very similar in duration to other similar sized projects, and the latter half of the overall schedule consists of crane set-up and assembling the mods. This process of mod installation is exceptionally short, accounting for just over two months of the schedule. The projected timeline is similar to the Hilton Palacio del Rio, a modularly built hotel in San Antonio, Texas. The duration of the project was fifty-five days to place seventeen stories of mods (SBG San Antonio Staff Reports, 2020).

When assessing the critical path of the construction schedule, the traditional construction of the first floor and then the mod placement are both included. The critical path is all the activities that have 0 lag time which means that the blue activities from *Figure 60* must start and finish on the exact dates that they are listed. The time constraint for the critical path and the whole project is just 201 days. All the green activities have a lag which means they do not need to be started or finished on the dates listed on the schedule. Lag is the amount of time an activity can be started or finished after its original scheduling and allows a project to be more flexible when addressing unexpected issues. All the lag times for each activity are listed in Appendix N. Overall, there is not much lag in the schedule since it is simplified and therefore relatively short in length. It was also important to ensure that there is not much lag so the modules can be installed as soon as possible in sequence. However, there is lag for the finishing and aesthetic types of activities. Activities that involve how the building looks can be worked on simultaneously to structural components, but the structural components are longer as the critical path has multiple structural activities.

This can also be compared to other construction projects that use traditional construction. Traditional schedules are more linear meaning they do not have two major milestones starting at the same time as this project. There is no prefabrication, so other traditional schedules look like the first half of this schedule for the entire schedule.

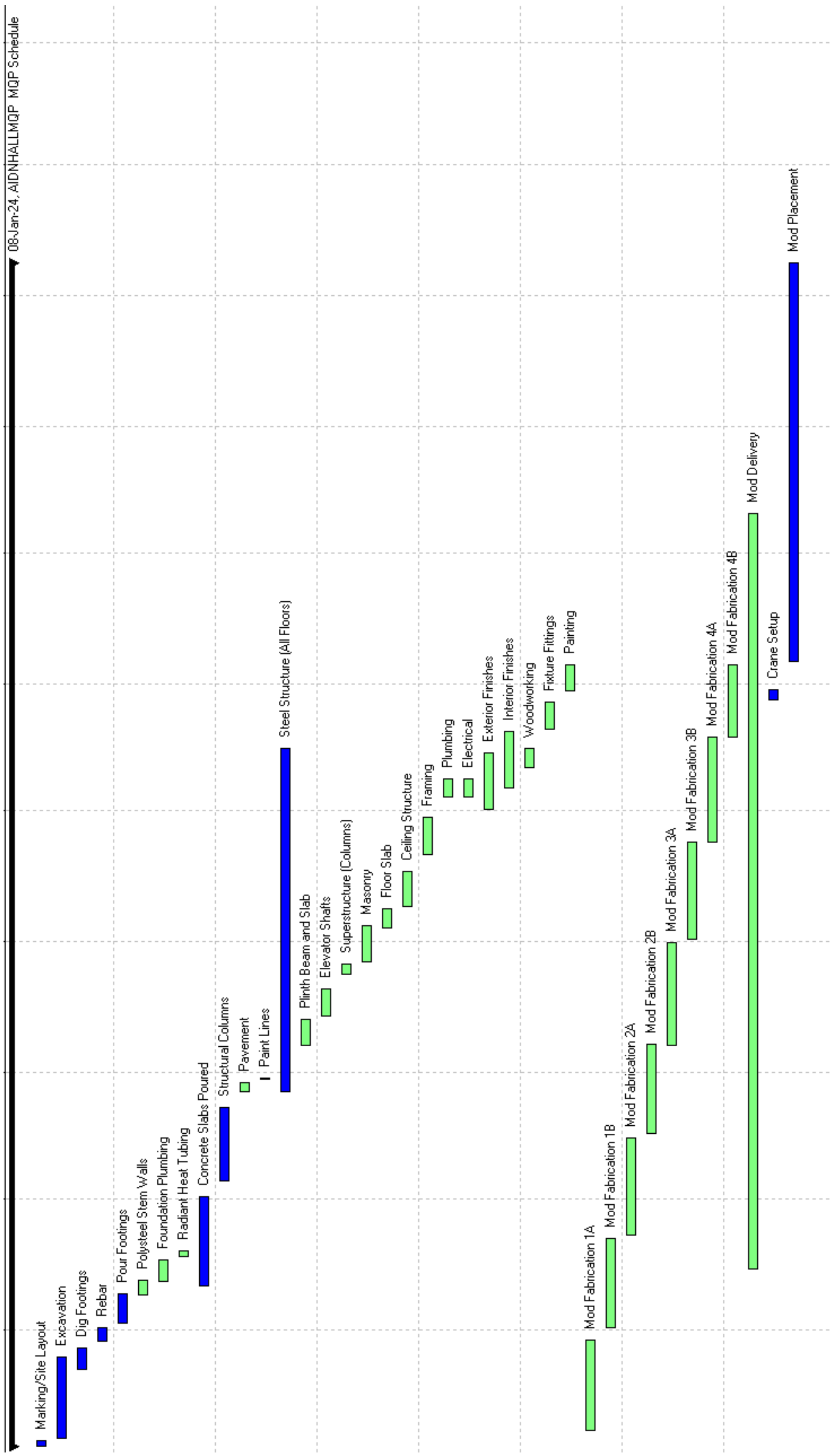

Figure 60: Project Schedule in Primavera P6, Critical Path Blue.

Budget and Project Costs:

The total project budget is $44,382,194 which equates to $322 per square foot. Even with the high Massachusetts building costs the project cost is still relatively low when compared to the national average. The budget is separated into 5 parts, and these can be found in Appendix E.

Another part of this deliverable was comparing traditional construction project budgets to modular construction to demonstrate that projects using modular construction will have less total project costs than traditionally constructed projects. To prove this claim, metrics from buildings with similar functionalities but different construction methods were gathered, and their costs were calculated, adjusted and compared. These calculations can be found in Appendix M. These metrics consist of 3 buildings built with modular construction, a hotel and 2 apartment buildings and a traditionally built residence hall. They contain size, location, year, and cost and the following cost per square foot prices have already been adjusted for those factors. The average cost per square foot of the modular buildings was $418.29. (Finley, 2017)(Sundt, 2021)(Ansari, 2020)(Howard Stein Hudson, 2018). The cost per square foot of the traditionally constructed residence hall was $460.27. These metrics show that the modular buildings are $42 or 10% less expensive than the traditional building. Our budget and costs can also be compared to these metrics, more specifically the residence hall. Both buildings have the same functionality and are both new, having been built or planned to be built within the past 3 years. The residence hall from this project costs $322 per square foot as stated above the residence hall built traditionally priced at $460.27 per square foot. That is an approximately $138 per square foot or a 30% difference. After analyzing and comparing general metrics to each other, and then comparing those metrics to the metrics from this project, it is appropriate to claim that modular construction is a less expensive project construction method than traditional construction.

There are also metrics of other residence halls that were used. When this project is compared to metrics from the other new residence halls that use traditional construction methods, via Hoar Construction, the numbers are favorable. When adjusted to size, location, and inflation, the budget for this project is cheaper than the budget for University of Alabama’s England Hall, Michigan State University’s target value residence hall model and a new Auburn University residence hall. This shows that our budget is cheaper and thus our construction method is more efficient than traditional construction proving modular construction leads to less total project costs.

Risk:

A matrix was created in order to assess this risk during the construction process of the project. Both the X and the Y-axes have ranks 1 through 5. The X-axis ranks impact, and the y-axis ranks probability. With this format, the most probable and impactful risks will be located in the top right portion of the figure and the least likely and least impactful risks are in the bottom left area. To further illustrate this concept, the graphic was color-coded on a green to red scale to show the riskiness of all the schedule items. The color code shifts from green to red as it goes from the bottom left to the top right. The items including a deeper green color are less risky than others and vice versa, the redder the square, the more risk. This was done so that the graphic was easy to decipher, and true risks can be identified. This matrix is found in Appendix I and was translated into a bar chart showing the risks from highest to lowest which is shown in *Figure 6*1.

Event	Risk
Excavation	20
Crane Setup	15
Floor Slab	15
Mod Delivery	12
Concrete Slabs	12
Structural Columns	10
Structural Steel	10
Plinth Beam and Slab	10
Rough-In Plumbing	9
Dig Footing	8
Pour Footing	8
Stem Wall	8
Foundation Plumbing	8
Water Proofing	8
Elevator Shafts	6
Framing	6
Mod Fabrication	5
Parking Garage Floor	5
Mod Assembly	5
Ceiling Structure	4
Painting	4
Electrical	4
Exterior Finishes	4
Pavment	3
Fixtrue Fittings	3
Rebar	2
Woodworking	2
Masonry	2
Interior Finishes	2
Paint Parking Lines	1

Figure 61: Risk Calculation Bar Chart.

When assessing the graphic, it can be determined that the highest risks include Mod Delivery, Concrete Slabs Poured, Excavation/Demolition, Structural Columns and Steel, Plinth Beam and Slab, Crane Setup, and Floor Slab. The lowest risk of the construction process includes Painting Parking Lot Lines, Rebar, Woodworking, Masonry, and Interior Finishes. When looking at the items in the graphic, it is apparent that items involving the structural integrity of the building pose the biggest risk. In turn, this means that during the construction of the project, these items would require a significant amount of attention and planning. Since the critical path has multiple structural activities, these activities have 0 lag which means they must be done as efficiently as planned placing risk on the project timeline. Some mitigation techniques for these risks involve reviewing all structural plans in extreme detail, hiring reliable structural contractors and allocating extra time in the schedule in the items so everything can be completed without any rush or flaws. The event with the most risk is the demolition of the existing townhouses and excavation of the site. This is due to the fact that the current WPI Townhouses were built in 1950, meaning there is a high probability for asbestos in the current structures. A major risk is presented particularly for demolition workers, as asbestos is a toxic mineral that can be extremely toxic when inhaled. The most feasible method of mitigation is to safely remove all materials in front of the existing building that might possibly contain asbestos prior to whole building demolition. By using this method, the toxic asbestos can be contained in a controllable manner.

Another important general mitigation technique is insurance, or the subcontractors that WPI hires for construction should have insurance to mitigate the possibility of an unexpected large financial burden. When looking at the least risky items, it was noticed that most of these items are related to the aesthetics and non-structural aspects of the design. These items are less risky because they are simply cosmetic, and any issues can be easily resolved by replacement or repair. Because these items do not have much risk, they do not require much mitigation. With that being said, the aesthetics of a building are nonetheless important because it is more visible to the consumer.

Additional risk that must be considered is the project's delivery system. Through research of possible methods, it was decided that construction manager at-risk (CMR) is the most appropriate option for WPI to adopt for this project, as WPI is known to use this method in the past. When using CMR, a construction manager coordinates all the subcontractors, which then simplifies the role for the owner; WPI in this case. The guaranteed maximum price of the project is outlined at the beginning of the project via contracts for the owner, designer, and construction manager and all costs that the guaranteed maximum price being covered by the construction manager. This makes for a more predictable budget and incurs less risk for the owner because a spike in costs or unexpected expenses will not affect money spent on WPI's end. CMR is also generally appropriate for large private projects since the construction manager can separate the scope of the project into smaller portions that are bid on separately.

Transportation:

When considering the transportation of the prefabricated mods to the job site, the most efficient method determined was to ship the modular units into Worcester's Union Station via train and then transition the mods onto trucks to ship to the Townhouse location (71 Park Ave, Worcester). This form of shipping is also known as multi-modal transit and is much cheaper than conventional trucking. Similarly, this method can cut shipping costs by more than half. The cost per net ton of multi-modal transit is $105.01 whereas the cost per net ton of trucking is $214.96 (SOURCE). The railcar to truck load ratio is 1:4, meaning a single shipment on rail is 4 times larger than a truck load, making rail shipping more efficient and ideal for our large shipments. The second part of the multi-modal shipping is the delivery of the mods from the freight yard to the job site on trucks.

Different potential routes have been determined after exploring the roads and taking into consideration overpasses and bridges in the area. One route in particular is a total of 3.86 miles and the main road that is involved is Belmont Street. There is one bridge on Belmont Street that would require a permit. But such a permit is attainable and normal when shipping large and heavy loads. There is another point of concern that pinpointed on the left turn, coming from the direction of Shrewsbury Street onto Belmont Street. This left turn is approximately a 180 degree turn and presents potential traffic and navigation issues. All other potential routes had multiple points that proved to be too concerning such as overhead passes, tunnels and insufficient bridges.

It was decided that the first route (*Figure 62)* was the best shipping option from Union Station. The left turn onto Belmont was taken into consideration, and although it is a point of concern, any issues with this route are highly unlikely, as there are no serious immobility issues such as insufficient bridges or short overpasses.

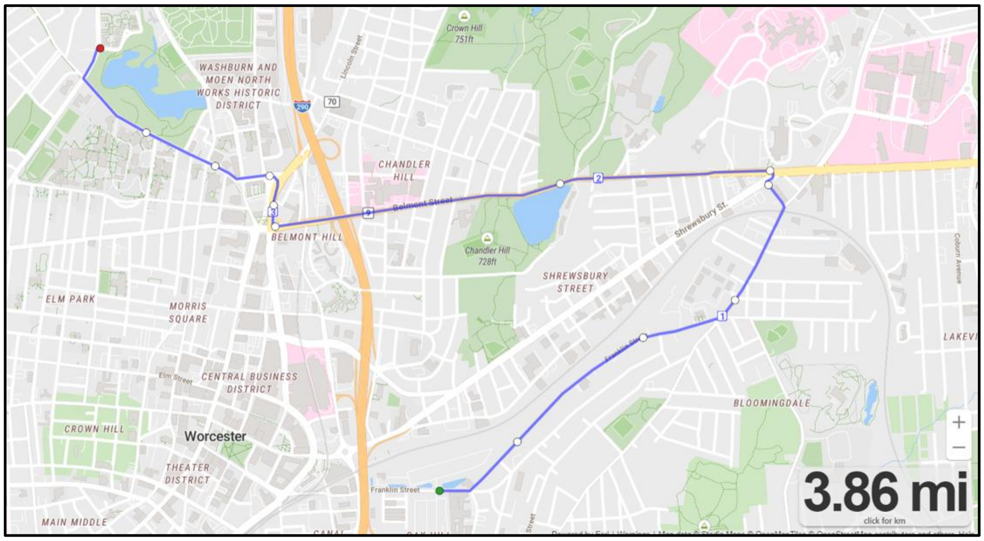

Figure 62: Shipping Route from Union Station.

Lazy-S Curve:

Another result of modular construction is that project costs are much more upfront when compared to the costs of a standard project that uses traditional construction. In a traditional construction project, most of the higher-valued work is completed in the middle of the schedule. Since modular prefabrication is used, a larger percent of the budget is being allocated early-on in the project. To elaborate, the demolition, excavation and installation of storm water drainage account for $761,309.56. For these scheduled events, contingencies have been budgeted for, assuming potential problems may be found with the soil with marshland nearby. To demonstrate this idea, an S-Curve was created to illustrate the cash flow over the course of the project's lifetime. The chart displays the value of work completed in dollars versus an increment of time or, in this case, weeks. The Lazy S-Curve for this project is different from most projects. Because the costs are more upfront the graph juts upwards from the beginning and does not have a flatline until the end of the project. The flatline at the end of the project is longer than a traditional project. When looking at our graph in *Figure 63* compared to an average S-Curve in *Figure 64*, it can be seen the majority of the project money spent in the project is spent at the beginning and middle of the project with very little money being spent at the end. In the average S-Curve graph the line is more evenly disturbed which means the costs are spent more in the middle of the project timeline and have a small flatline of spending at the beginning and end of the project.

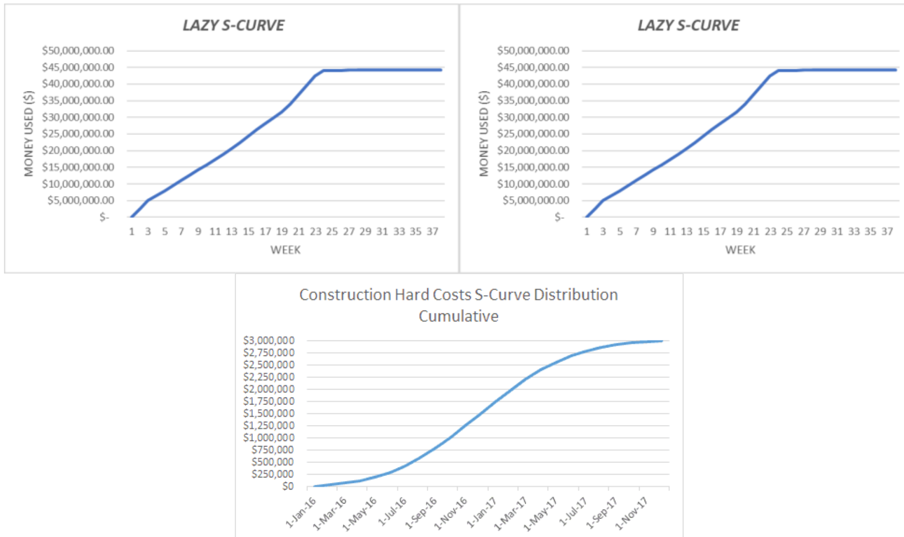

Figure 63 & 64: Lazy S-Curve (Left) and Traditional Lazy S-Curve (Burton, 2023)(Right).

Break-even Analysis:

To determine the breakeven point for the project, cost of living for students in the residence hall must be determined. The residence hall in this project is most comparable to WPI's East Hall when it comes to modernity and scale. On average, it costs $10,790.33 per student to live in East Hall. Since this project's residence hall offers more amenities, is more recently built, and overall is a more premium product it is reasonable to set the room and board price slightly higher at $12,500.00 per student. This price is also fair when compared to one of the nicest on campus living situations in the country. Ranked as number 2 best college dorms in the United States on Niche, Washington University in St Louis charges on average $13000 per year for housing. To add on, many other New England colleges charge northward of $14000 for room expenses (*America's Most Expensive College Dorms*, 2016).

The break-even point was calculated by adding all the costs of the building construction, the energy used per year, labor hours per year for building workers, and contingency maintenance costs. Next, the amount of yearly revenue WPI would receive from the residence hall was estimated, assuming each student pays $12,500.00 along with the allotted $57,960 in tuition. Assuming WPI allocates a small percentage (12.5%) of the students' tuition and all of their rent money to the refunding of the building costs it was determined the breakeven point will occur in the 6th year of operation.

Results

When analyzing the first thesis multiple results have been discovered from our deliverables. Our budget has proven that modular construction is cheaper than traditional construction. 85% of the square footage of our building consists of the modular units with only 76% of our budget being allocated to these modular units. This 9% difference, along with all the metrics discussed previously clearly show that modular construction methods require less construction costs when compared to traditional construction. It is also clear that this project timeline is much shorter than the average project timeline. It takes fifteen months to build the average apartment building with similar functionality, and eighteen months to build Eats Hall at WPI. With the schedule being only seven months, it is clearly much quicker to use a modular method.

An important result that was derived from the deliverable section is that the residence hall presented can be built cheaper than traditional construction methods, while maintaining strong design and implementing sustainable energy systems. A dynamic analysis was conducted to find the natural frequency of the structure. This analysis was able to show that under seismic load all members of the steel model would survive under load. Additionally, the solar power system will be able to run at an average efficiency rate of 94.475% over the course of its twenty-five-year life span. It will produce over 13% of the building's total estimated energy consumption and based on Worcester County energy rates will save WPI an estimated $145,957 per year, meaning the system could theoretically pay for itself in just under five years. Lastly, the hydronic heating system provides an energy efficient means for the students to comfortably enjoy their heated rooms in the wintertime.

Another result derived from analyzing the deliverables is that the project will stimulate the local economy. This project will use a local network and local supplies meaning the project will purchase materials and utilize subcontractors from Worcester County and support these businesses in the area. This project will generate hundreds of construction-related jobs to community members, providing them with wages while also keeping the income in the community. The income from the project is tax locally so that means revenue stays in the community and supports local schools, parks and other public local amenities. This project also means that our 470 new residents will be living locally for a large portion of the year. These residents will purchase items from local stores, go to local restaurants and support local businesses creating an increased community cash flow.

After extensive research surrounding mental health at WPI and college campuses in general, compiling data from WPI students, an analysis of interviews with WPI residential and wellness staff, specific design choices have been implemented to improve mental health and sociability within our residence hall. The walls of the suites and lobby will be a light blue to induce a calming effect and a sense of relaxation to help reduce stress. Curved walls will also be implemented in the lobby to increase a feeling of openness and improve the overall sociability, and therefore wellness for residents. Coupled with this effect, green walls and other nature related aspects including fish tanks will be incorporated throughout the building to create a higher connectivity to nature and in turn improve mental health. In addition, the lobby has been designed with a large glass facade and each suite with five large windows that reach the living room/kitchen area and each room to make sure there is ample natural light throughout the building. Open common spaces are also incorporated into the design in the lobby and via one common area per floor, promoting sociability and interaction amongst residents, in turn hopefully creating a “social hub” at WPI that the university currently lacks.

Conclusions

The first thesis, which states the method of modular construction leads to reduced construction prices, a shorter schedule, and a quicker return on investment is proved true through the research, analysis, and design in this report. Based on results we were able to definitively conclude that modular construction requires less construction costs than traditional construction, has a shorter project timeline than traditional construction and has a better return on investment than traditional construction.

The second thesis involved the consideration of mental health in the design of the residence hall. Generally, the added housing options for upper class students can alleviate unnecessary stress in finding a place to live beyond one's first year at WPI. Besides this, the design incorporation of spacious areas, color choices, natural lighting, plants and animals, as well as other amenities in this residence hall would encourage students to interact and engage with their community and surrounding environment. Additionally, the large, open spaces in this building provide a unique alternative to other WPI residence halls, which are not as spacious. Students may also utilize common spaces such as the first-floor tech suites and seating areas for academic purposes, all while in the presence of their fellow classmates. Overall, this proposed residence hall provides WPI students with a sense of community, connectedness, relaxation, and most importantly, a sense of being at-home.

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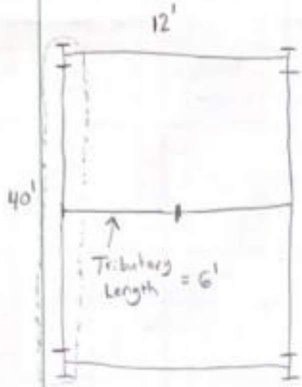
Appendix A: Beam Design Calculations

Steel Beam 1



Dead Load (D) = 79 psf

Live Load (L) = 40 psf

Roof Live Load (L_R) = 20psf

Snow Load (S) = 43.4 psf

Wind Load (W) = 27.16 psf

Governing Load Combination:

$1.2D + 1W + 1L + .5S$

$= 1.2(79\,psf \times 6') + (27.16\,psf \times 6') + (40\,psf \times 6') + .5(43.4\,psf \times 6')$

$= 1101.96\ lb/ft = 1.10196\ k/ft$

$$\text{Moment} = \frac{wL^2}{12} = \frac{(1.10196\,\frac{k}{ft})(40\,ft)^2}{12} = 135.94\ k\text{-}ft$$

$M_u = \phi F_y Z_x \rightarrow 135.94\ k\text{-}ft\ (12\ in) = .9\,(50\ ksi)(Z_x)$

$1631.36\ k\text{-}in = 45\ ksi\ (Z_x)$

$Z_x = 36.25\ in^3$ (table 1-1)

For W 10 × 49 $\longrightarrow Z_x = 60.4\ in^3 > 36.25\ in^3$ so $\boxed{O.K.}$

Self Weight

$1.2(.049\,\frac{k}{ft}) + 1.10196\,(\frac{k}{ft}) = 1.69\,\frac{k}{ft}$ (New Governing Load)

$$\text{Moment} = \frac{(1.16\,\frac{k}{ft})(40\,ft)^2}{12} = 154.67\ k\text{-}ft$$

$M_u = \phi F_y Z_x \longrightarrow 154.67\ k\text{-}ft\ (12\ in) = .9\,(50\ ksi)\ Z_x$

$1856\ k\text{-}in = 45\ ksi\ (Z_x)$

$Z_x = 41.24\ in^3 < 60.4\ in^3$

so W10 × 49 O.K. for self weight

Steel Beam II

Serviceability

$$\Delta DL + LL = \frac{5wL^4}{384EI} \rightarrow \frac{5(.079\,\tfrac{k}{ft} + .04\,\tfrac{k}{ft})\left(\tfrac{1ft}{12in}\right)\left(40ft \times \tfrac{12in}{1ft}\right)^4}{384(29000\,ksi)(272\,in^4)} = .61''$$

E = 29000 ksi
I → table 1-1

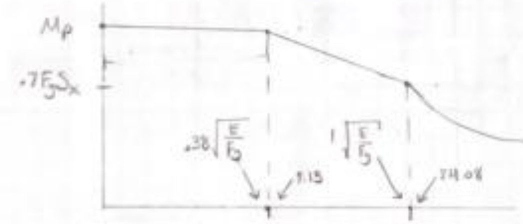
$$\frac{L}{240} = \frac{480''}{240} = 2'' \longrightarrow .61'' < 2'' \quad \text{so } \boxed{OK}$$

$$\Delta LL = \frac{5wL^4}{384EI} \rightarrow \frac{5(.04\,\tfrac{k}{ft})\left(\tfrac{1ft}{12ft}\right)\left(40ft \times \tfrac{12in}{1ft}\right)^4}{384(29000\,ksi)(272\,in^4)} = .29''$$

$$\frac{L}{360} = \frac{480''}{360} = 1.33'' \longrightarrow .29'' < 1.33'' \quad \text{so } \boxed{OK}$$

Flexure

Compactness Test → Flange

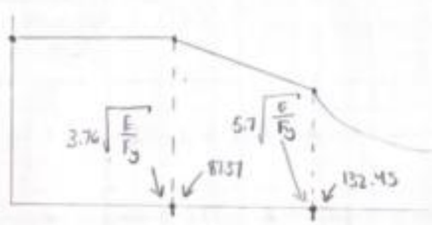


$\frac{b_f}{2t_f}$ → table 1-1

$$\frac{b_f}{2t_f} = 8.93 < 9.15$$

so COMPACT

Compactness Test → Web



$\frac{h}{t_w}$ → table 1-1

$$\frac{h}{t_w} = 23.1 < 87.37$$

so COMPACT

Steel Beam III

Flexural Yielding

$M_n = M_p = F_y Z_x = (50\ ksi)(60.4\ in^3) = 3020\ k\text{-}in = 251.67\ k\text{-}ft$

$M_u < \phi M_n \rightarrow 135.94\ k\text{-}ft < (.9)(251.67\ k\text{-}ft)$

$135.94\ k\text{-}ft < 226.5\ k\text{-}ft$ so $\boxed{OK}$

Lateral Torsional Buckling

Table 3-2 $\rightarrow L_p = 8.97', L_r = 31.6', L_b = 40'$

Since $L_b > L_r$ we use equation (F2-3) $M_n = F_{cr} S_x$

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2} \sqrt{1 + .078 \frac{J_c}{S_x h_o} \left(\frac{L_b}{r_{ts}}\right)^2} \quad (F2-4)$$

$$C_b = \frac{12.5 M_{max}}{2.5 M_{max} + 3M_A + 4M_B + 3M_C} \quad (F1-1)$$

* Based on Risa generated moment diagram *

$$= \frac{12.5(152.8)}{2.5(152.8) + 3(47.88) + 2(49.82) + 3(19.87)} = 2.7$$

$S_x = 54.6\ in^3, r_{ts} = 2.84\ in, J_c = 1.39\ in^4, h_o = 9.44\ in$ (Table 1-1)

$$F_{cr} = \frac{2.7(\pi^2)(29000\ ksi)}{\left(\frac{480\ in}{2.84\ in}\right)^2} \sqrt{1 + .078 \frac{(1.39)\ in^4}{(54.6\ in^3)(9.44\ in)} \left(\frac{480\ in}{2.84\ in}\right)^2} = 71.62\ ksi$$

$M_n = F_{cr} S_x = (54.6\ in^3)(71.62\ ksi) = 3910.49\ k\text{-}in = 325.87\ k\text{-}ft$

$M_u < \phi M_n \rightarrow 135.94\ k\text{-}ft < (.9)\ 325.87\ k\text{-}ft$

$135.94\ k\text{-}ft < 293.29\ k\text{-}ft$ so $\boxed{OK}$

Steel Beam III

Flexural Yielding

$M_n = M_p = F_y Z_x = (50\ ksi)(60.4\ in^3) = 3020\ k\text{-}in = 251.67\ k\text{-}ft$

$M_u < \phi M_n \rightarrow 135.94\ k\text{-}ft < (.9)(251.67\ k\text{-}ft)$

$135.94\ k\text{-}ft < 226.5\ k\text{-}ft$ so $\boxed{OK}$

Lateral Torsional Buckling

Table 3-2 $\rightarrow L_p = 8.97',\ L_r = 31.6',\ L_b = 40'$

Since $L_b > L_r$ we use equation (F2-3) $M_n = F_{cr} S_x$

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2} \sqrt{1 + .078 \frac{J_c}{S_x h_o} \left(\frac{L_b}{r_{ts}}\right)^2} \qquad (F2\text{-}4)$$

$$C_b = \frac{12.5 M_{max}}{2.5 M_{max} + 3M_A + 4M_B + 3M_C} \qquad (F1\text{-}1)$$

* Based on Risa generated moment diagram *

$$= \frac{12.5(152.8)}{2.5(152.8) + 3(47.88) + 2(49.82) + 3(17.87)} = 2.7$$

$S_x = 54.6\ in^3,\ r_{ts} = 2.84\ in,\ J_c = 1.39\ in^4,\ h_o = 9.44\ in$ (Table 1-1)

$$F_{cr} = \frac{2.7(\pi^2)(29000\ ksi)}{\left(\frac{480\ in}{2.84\ in}\right)^2} \sqrt{1 + .078 \frac{(1.39)\ in^4}{(54.6\ in^3)(9.44\ in)} \left(\frac{480\ in}{2.84\ in}\right)^2} = 71.62\ ksi$$

$M_n = F_{cr} S_x = (54.6\ in^3)(71.62\ ksi) = 3910.49\ k\text{-}in = 325.87\ k\text{-}ft$

$M_u < \phi M_n \rightarrow 135.94\ k\text{-}ft < (.9)\ 325.87\ k\text{-}ft$

$135.94\ k\text{-}ft < 293.29\ k\text{-}ft$ so $\boxed{OK}$

Steel Beam IV

Shear

$V_n = .6 F_y A_w C_{v1}$ (G2-1) $F_y = 50$ ksi, $A_w = 14.4 \text{ in}^2$

Since $\frac{h}{t_w} = 2.81 < 2.24\sqrt{\frac{E}{F_y}} = 2.24\sqrt{\frac{29000 \text{ ksi}}{50 \text{ ksi}}} = 53.94$, $C_{v1} = 1$

$V_n = .6(50 \text{ ksi})(14.4 \text{ in}^2) = 432$ K

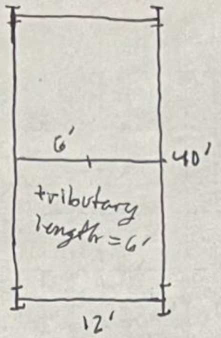
$V_u = \dfrac{(1.10196)\frac{k}{ft}(40 \text{ ft})}{2} = 20.39$ K

$\phi V_n > V_u \longrightarrow .9(432k) > 20.39$ K

$388.8 > 20.39$ K so $\boxed{OK}$

Appendix B: Roof Beam Design Calculations

Steel Roof Beam I



Dead Load (D) = 85 psf
Roof Live Load (L_R) = 20 psf
Wind Load (W) = 27.16 spf
Live Load (L) = 40 psf
Snow Load (S) = 43.4 psf

Governing Load Combination

$1.2D + W + L + .5S$

$1.2(85 \times 6) + (27.16 \times 6) + (40 \times 6) + .5(43.4 \times 6)$

$W_u = 1.145 \text{ k/ft}$

Moment: $\frac{wL^2}{12} = \frac{(1.145 \frac{k}{ft})(40ft)^2}{12} = 152.69 \text{ k-ft}$

$M_y = \phi F_y Z_x$

$152.69 \text{ k-ft} (12in) = .9(50ksi)(Z_x)$

$1832.256 \text{ k-in} = 45 ksi (Z_x)$

$Z_x = 40.717 in^3$ (Table 1-1)

For W10x49:

$Z_x = 60.4 in^3 > 40.717 in^3$ ✓

self weight

$1.2(.049 \frac{k}{ft}) + 1.145 \frac{k}{ft} = 1.204 \frac{k}{ft}$ governing load

moment $= \frac{(1.204 \frac{k}{ft})(40ft)^2}{12} = 160.528 \text{ k-ft}$

$M_u = \phi F_y Z_x$

$160.528 \text{ k-ft} (12in) = .9(50ksi)(Z_x)$

$1926.336 \text{ k-in} = 45 ksi (Z_x)$

$Z_x = 42.81 in^3$

$42.81 in^3 < 60.4 in^3$ ✓ ok for self weight

Roof Steel Beam II

Serviceability

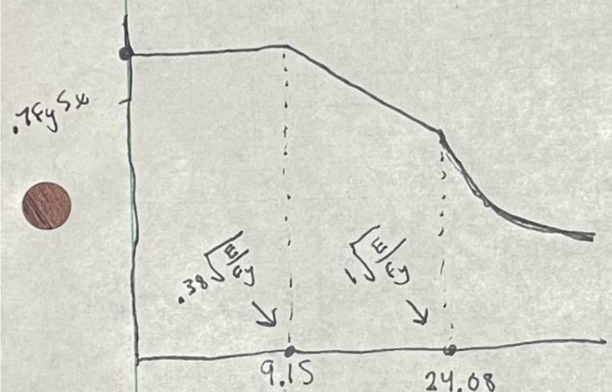
$$\Delta_{DL+LL} = \frac{5wL^4}{384EI} = \frac{5(.085+.04\left(\frac{k}{ft}\right))\left(\frac{1ft}{12in}\right)\left(40ft \times \frac{12in}{1ft}\right)^4}{384(29000ksi)(272in^4)} = .913''$$

$$\frac{L}{240} = \frac{480''}{240} = 2'' \qquad .913'' < 2'' \ \boxed{\checkmark}$$

$$\Delta_{LL} = \frac{5(.04 k/ft)\left(\frac{1ft}{12in}\right)\left(40ft \times \frac{12in}{1ft}\right)^4}{384(29000ksi)(272in^4)} = .29''$$

$$\frac{L}{360} = \frac{480'}{360} = 1.33'' \qquad .29'' < 1.33'' \ \boxed{\checkmark}$$

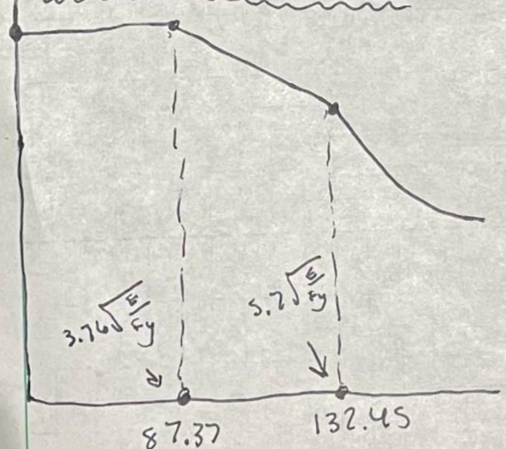
Flexure: compactness Test for Flange



$$\frac{b_f}{2t_f} = 8.93 < 9.15 \text{ (table 1-1)}$$

$\therefore$ compact

compact Test for web



$$\frac{h}{t_w} = 23.1 < 87.37 \text{ (table 1-1)}$$

$\therefore$ compact

Roof Steel Beam II

Serviceability

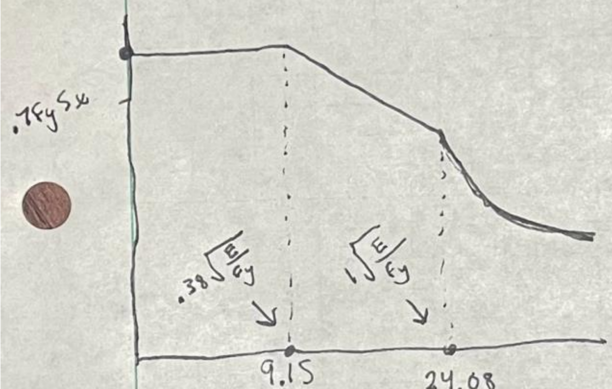
$$\Delta_{DL+LL} = \frac{5wL^4}{384EI} = \frac{5(.085+.04(\frac{k}{ft}))(\frac{1ft}{12in})(40ft \times \frac{12in}{1ft})^4}{384(29000ksi)(272in^4)} = .913''$$

$$\frac{L}{240} = \frac{480''}{240} = 2'' \qquad .913'' < 2'' \ \boxed{\checkmark}$$

$$\Delta_{LL} = \frac{5(.04k/ft)(\frac{1ft}{12in})(40ft \times \frac{12in}{1ft})^4}{384(29000ksi)(272in^4)} = .29''$$

$$\frac{L}{360} = \frac{480'}{360} = 1.33'' \qquad .29'' < 1.33'' \ \boxed{\checkmark}$$

Flexure: compactness Test for Flange



$$\frac{b_f}{2t_f} = 8.93 < 9.15 \text{ (table 1-1)}$$

$\therefore$ compact

compact Test for web

$$\frac{h}{t_w} = 23.1 < 87.37 \text{ (table 1-1)}$$

$\therefore$ compact

Roof Steel Beam III

Flexural Yielding:

$M_n = M_p = F_y Z_x = (50ksi)(60.4in^3) = 3020k\text{-}in$ or $251.67k\text{-}ft$

$M_u < \phi M_n$ $\quad$ $160.528k\text{-}ft < .9(251.67k\text{-}ft)$

$160.528k\text{-}ft < 226.5$ $\boxed{\checkmark}$

Lateral Torsional Buckling:

$L_p = 8.97ft \quad L_r = 31.6ft \quad L_b = 40ft$ (Table 3-2)

$L_b > L_r \therefore$ use (equ. F2-3) $M_n \cdot F_{cr} S_x$

$$F_{cr} = \frac{C_b \pi^2 E}{\left(L_b/r_{ts}\right)^2}\sqrt{1 + .078\frac{J_c}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2} \quad (F2\text{-}4)$$

$$C_b = \frac{12.5 M_{max}}{2.5 M_{max} + 3M_A + 4M_B + 3M_C} \quad (F1\text{-}1)$$

(Analysis based on Risa generated moment diagram)

$$= \frac{12.5(152.8)}{2.5(152.8) + 3(47.88) + 4(48.82) + 3(19.87)}$$

$cb = 2.7$

$S_x = 54.6in^3 \quad r_{ts} = 2.84in \quad J_c = 1.39in^4 \quad h_o = 9.44in$ (Table 1-1)

$$F_{cr} = \frac{2.7\pi^2(29000ksi)}{\left(\frac{480in}{2.84in}\right)^2}\sqrt{1 + .078\frac{1.39in^4}{54.6in^3(9.44in)}\left(\frac{480in}{2.84in}\right)^2} = 71.62ksi$$

$M_n = F_{cr} S_x = 54.6in^3(71.62ksi) = 3910.49k\text{-}in$ or $325.87k\text{-}ft$

$M_u < \phi M_n$ $\quad$ $160.528k\text{-}ft < .9(325.87k\text{-}ft)$

$160.528k\text{-}ft < 293.29k\text{-}ft$ $\boxed{\checkmark}$

Shear:

$V_n = .6F_y A_w C_{v1}$ (G2-1) $\quad \frac{h}{t_w} = 2.31 < 2.24\sqrt{\frac{E}{F_y}} \rightarrow 2.31 < 2.24\sqrt{\frac{29000ksi}{50ksi}}$

$2.31 < 53.95 \quad \therefore C_v = 1$

$F_y = 50ksi$

$A_w = 14.4in^2$ $\quad$ $V_n = .6(50ksi)(14.4in^2) = 432k$

$V_u = \frac{wL}{2} = \frac{1.145(40ft)}{2} = 22.9k$

$V_u < \phi V_n$ $\quad$ $\phi V_n = .9(432k) = 388.8k$

$22.9k < 388.8k$

$\boxed{\checkmark}$

Appendix C: Column Design Calculations

Column Design page 1

HSS 14x10x8 or HSS 14x10x$\frac{1}{2}$

*Flexural buckling → required to be satisfied as limit state for both slender & non-slender elements.

(E3-1): $\phi_c P_n = F_{cr} A_g \phi$

(E3-2): $\frac{kL}{r} \leq 4.71\sqrt{\frac{E}{F_y}}$ $F_{cr} = [0.658^{F_y/F_e}]F_y$

(E3-3): $\frac{kL}{r} \geq 4.71\sqrt{\frac{E}{F_y}}$ $F_{cr} = [0.877] F_e$

- k = effective length factor ⇒ Table C-A-7.1 ⇒ k = 0.65
- L = unbraced length = 11' column
- r = radius of gyration ⇒ table 1-11 = 5.23

$$\frac{kL}{r} = \frac{(0.65)(11' \times 12''/_{1'})}{5.23} = 16.4 < 200 \checkmark$$

$$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29000 \text{ ksi}}{50 \text{ ksi}}} = \boxed{113.43 \geq 16.4}$$

↳ use (E3-2)

(E3-2): $F_{cr} = (0.658^{F_y/F_e})F_y$

$$↳ F_e = \frac{(\pi^2)E}{(\frac{kL}{r})^2} = \frac{(\pi^2)(29000 \text{ ksi})}{(16.4)^2} = 1064.16 \text{ ksi} = F_e$$

$$F_{cr} = [0.658^{F_y/F_e}]F_y = [0.658^{\frac{50 \text{ ksi}}{1064.16 \text{ ksi}}}]50 \text{ ksi}$$

⇒ $F_{cr} = 49.03$ ksi

(E3-1): $\phi_c P_n = F_{cr} A_g \phi = (49.03 \text{ ksi})(20.9)(0.9) = 922.25^k$

Continued...

$P_n = 107.5^k$ (RISA model provided), $\phi_c P_n = 922.25^k$

$107.5^k < 922.25^k$ ok ✓

Slenderness

$b/t = 18.5$

$\lambda_r = 1.4\sqrt{E/F_y} = 1.4\sqrt{\frac{29000\text{ ksi}}{50\text{ ksi}}} = 33.71$

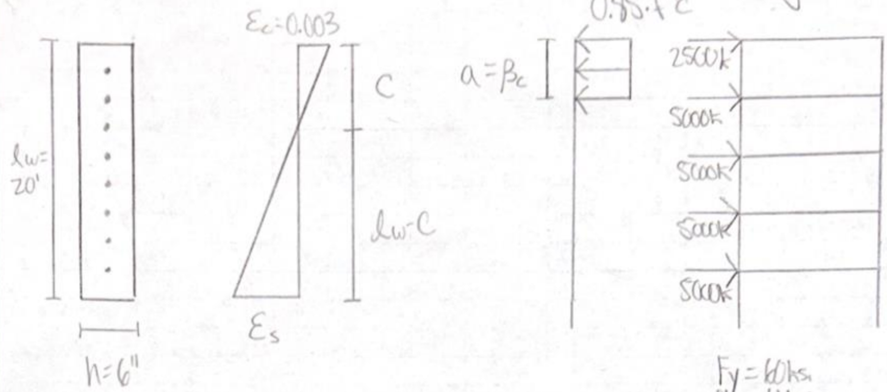
$33.71 > 18.5$

→ Therefore, not slender & only need to account for flexural buckling.

⇒ HSS 14×10×1/2 is okay.

Appendix D: Shear Wall Calculations

20' Shear Wall | pg 1



F_y = 60ksi
f'_c = 4ksi

$$V_u = \underline{2500k} + 4(5000k) = 22{,}500\text{ k}$$

$$V_{N_{req}} = \frac{V_u}{\phi} = \frac{22{,}500\text{k}}{0.85} = 26{,}471\text{ kip}$$

$$V_c = 3.3\sqrt{f'_c}\cdot hd + \frac{N_ud}{4\ell_w} \qquad d = (0.8)\ell_w = (0.8)20 = 16$$

$$V_c = \left(3.3\sqrt{4000}\,(6)(16)20(12) + \frac{(2500)(16)(12)\,1000}{4(20)(12)}\right)\frac{1}{1000}$$

$$V_c = (4{,}808{,}685.90 + 500{,}000)\ \frac{1}{1000}$$

$$V_c = 5{,}308.68\text{ kips}$$

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

$$5{,}308.68\text{k} \geq 2\sqrt{4000}\,(6/12)(0.8\cdot 20)$$

$$5{,}308.68\text{ k} \geq 1011.92\text{ k}\ \checkmark$$

Reinforcement:

Vertical: $\rho_{min} = 0.0025$

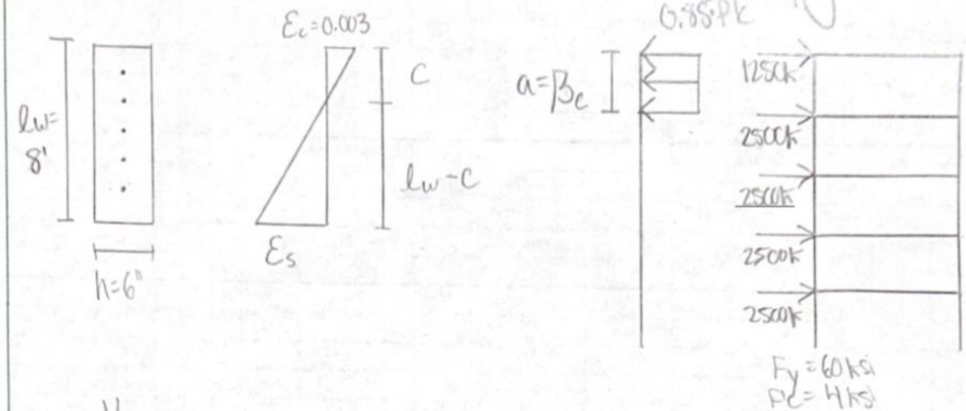
$A_v = 0.0025(10)(6) = 0.15\ in^2/ft$

use No. 4 bars @ 6" spacing

horizontal:

$A_t = 0.0025(10)(12) = 0.3\ in^2/ft$

Use No. 5 bars @ 12" spacing



$$V_u = \underline{1250k} + 4(2500k) = \underline{11,250\ k}$$

$$V_{N\,req} = \frac{V_u}{\varnothing} = \frac{11,250k}{0.85} = 13,235.29 \text{ kips}$$

$$V_c = 3.3\sqrt{f'c}\cdot h \cdot d + \frac{N_u d}{4 l_w} \qquad d = (0.8) l_w = (0.8)(8) = 6.4$$

$$V_c = \left(3.3\sqrt{4000}\,(6)(6.4)(20)(12) + \frac{(1250)(16)(12)\,1000}{4(20)(12)}\right)\frac{1}{1000}$$

$$V_c = (1,923,474.36 + 250,000)\frac{1}{1000}$$

$$V_c = 2,173.47 \text{ kips}$$

$$V_c \geq 2\sqrt{f'c}\, b_w d$$

$$2173.47k \geq 2\sqrt{4000}\,(6/12)(0.8\cdot 8)$$

$$2173.47k \geq 404.77\ k \ \checkmark$$

Reinforcement:

vertical:

$A_V = 0.0025(10)(6) = 0.15 in^2/ft$

Use No.4 bars @ 6" spacing

horizontal:

$A_t = 0.0025(10)(12) = 0.3 in^2/ft$

Use No. 5 bars @ 12" spacing

Appendix E: Foundation Calculations

Spread Footing:

f'c	4	ksi			**Factored Loads (Qu)**		
Fy	60	ksi		Qu=	3.633	k/ft^2	
Soil Bearing capacity	4	k/ft^2					
Soil Density	150	pcf		Mu=	12.12		
Dead Load	12.652	k/ft					
Live Load	6.404	k/ft		Vu=	6.660		
Soil line height	8	ft					
a	16	in		Vc=	(f'c)^0.5*b*d	d=0.7589	
					ØVc=Vu		
Effective Allowable Bearing (Qe)				d=(Vu/ØVc)	12.6	in	
Qself	0.15	k/ft^2					
Qsoil	1.05			h=	d + clear cover		
Qe	2.8	k/ft^2		h=	15.6	in	
				Round to nearest 0.5 in			
Breq length from Areq				h=	16	in	
6.805714286	ft			d=	13	in	
Round to nearest 0.5ft							
b=	7	ft					

New Vu Calculation					
Vu=	5.449	k/ft	<	6.660	k/ft
		OK		OK	
Minimum Steel 1 ft Width					
As, min	0.3456	in^2			
Use No. 5 bars @ 10 in spacing					
Shear Check					
ØVc=	7.201	k/ft	>	5.449	k/ft
		OK		OK	

Spread Footing | pg 1

$f'c = 4$ ksi
$F_y = 60$ ksi
Soil bearing capacity: $4 k/ft^2$ (assumed)

$DL = 12.652 k/ft$
$LL = 6.404 k/ft$

Soil density = 150 pcf (assumed)

Effective Allowable Bearing = q_e

$q_e = q_a - q_{soil} - q_{self}$

$q_{self} = (1ft)(0.15 k/ft^3) = 0.15 k/ft^2$

$q_{soil} = (8ft - 1ft)(.15 k/ft^3) = 1.05 k/ft^2$

$q_e = 4 k/ft^2 - 0.15 k/ft - 1.05 k/ft^2 = 2.8 k/ft^2 = q_e$

Find breq from Areq

$Areq = \frac{DL + LL}{q_e} = \frac{12.652 + 6.404}{2.8} = 6.80$

$\boxed{b = 7ft}$

Factored Loads = q_u

$q_u = \frac{1.2(DL) + 1.6(LL)}{b} = \frac{1.2(12.652) + 1.6(6.404)}{7}$

$q_u = 3.633 k/ft^2$

$M_u = \frac{1}{8} q_u (b-a)^2 \Rightarrow \frac{1}{8}(3.633)(7ft - \frac{16in}{12in})^2$

$M_u = 12.12$

$d = h - 3 = 12 - 3 = 9$

$V_u = q_u \left(\frac{b-a}{2} - d \right)$

$V_u = 3.633 k/ft^2 \left[\frac{7ft - \frac{16in}{12in}}{2} - \frac{9in}{12in} \right] = 6.66 k/ft^2$

$V_c = \sqrt{f'c}\, bd = \frac{\sqrt{4000}}{1000} \cdot 12 \cdot d \Rightarrow 0.7589d \ ^k/ft$

$\emptyset V_c = V_u \Rightarrow d = \frac{V_u}{\emptyset V_c} = \frac{6.66}{(0.75)(0.7589d)} \approx 11.70''$

$h = d + \text{clear cover} (3'')$

$h = 11.7 + 3 \approx \boxed{16'' = h}$ $\quad d = 16 - 3 = 13'' = d$

New Vu Calculation

$V_u = 3.633\left[\left(\frac{7 - \frac{16}{12}}{2}\right) - \frac{13}{12}\right] = 5.449 \ ^k/ft < 6.66 \ ^k/ft$ ✓

Calculate As for 1ft width

$A_{s,min} = 0.0018 A_g \qquad A_g = h \cdot b$

$A_{s,min} = 0.0018(16)(12) = .3456 \ in^2/ft = A_{s,min}$

$\frac{.3456}{12in} = \frac{.31}{X in}$

$X = 10.76 in$ $\Rightarrow$ Use No. 5 bars @ 10" spacing

Check Shear

$\emptyset V_c = (0.75)(8)(0.0018)^{1/3} \frac{\sqrt{4000}}{1000}(12)(13) = 7.201 \ ^k/ft$

$7.201 \ ^k/ft > 5.449 \ ^k/ft$ ✓

$V_c = \sqrt{f'c}\, bd = \frac{\sqrt{4000}}{1000} \cdot 12 \cdot d \Rightarrow 0.7589d \ ^k/_{ft}$

$\emptyset V_c = V_u \Rightarrow d = \frac{V_u}{\emptyset V_c} = \frac{6.66}{(0.75)(0.7589d)} \doteq 11.70''$

$h = d +$ clear cover (3")

$h = 11.7 + 3 \doteq \boxed{16'' = h}$ $\quad d = 16 - 3 = 13'' = d$

New Vu Calculation

$V_u = 3.633\left[\left(\frac{7-\frac{16}{12}}{2}\right) - \frac{13}{12}\right] = 5.449 \ ^k/_{ft} < 6.66 \ ^k/_{ft}$ ✓

Calculate As for 1ft width

$A_{s,min} = 0.0018 A_g \qquad A_g = h \cdot b$

$A_{s,min} = 0.0018(16)(12) = .3456 \ in^2/ft = A_{s,min}$

$\frac{.3456}{12in} = \frac{.31}{X in}$

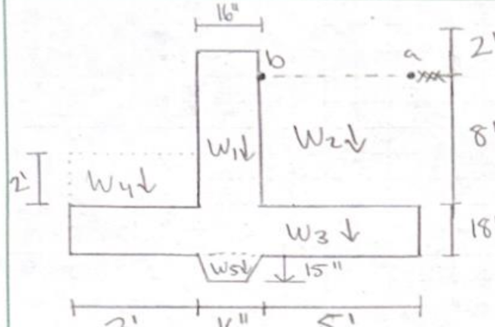
$X = 10.76 in$ $\Rightarrow$ Use No. 5 bars @ 10" spacing

Check Shear

$\emptyset V_c = (0.75)(8)(0.0018)^{1/3}\frac{\sqrt{4000}}{1000}(12)(13) = 7.201 \ ^k/_{ft}$

$7.201 \ ^k/_{ft} > 5.449 \ ^k/_{ft}$ ✓

Retaining Wall | pg 3



$F_y = 60$ ksi
$f'_c = 4$ ksi
$\gamma = 150$ pcf

$P = \frac{1}{2} k_a h \gamma h (h + 2h') = \frac{1}{2}(0.33)(150\ pcf)(10')(10' + 2(2)) = 3456\ lb$

$\frac{h^2 + 3hh'}{3(h + 2h')} = y = \frac{10^2 + 3(10)(2)}{3(10 + 2(2))} = \frac{160}{42} = 3.809\ ft$

$M_u = 1.6(3456\ lb)(3.809\ ft) = 21{,}062\ ft \cdot lb$

Now $\rho_{max} = 0.0197 \Rightarrow 0.4\rho_{max} = 0.009$

$M_u = \phi \rho f_y b d^2 (1 - 0.59 \rho \frac{f_y}{f'_c})$

$21{,}062 = (0.75)(.009)(60)(12")\ d^2 (1 - 0.59(.009)\frac{60}{4}) \Rightarrow d = 10$

Check Shear

$V_u = 1.6(3456) = 5530\ lb$

$\phi(1.0)\sqrt{f'_c}\ b_w \cdot d \Rightarrow 0.75(1.0)\sqrt{4000}\ (12)(10) = 5692\ lb$

Stability Check

Componet Weight	W (lb)	X (ft)	Mn (ft·lb)
$W_1 = 16/12 \cdot 10 \cdot 150$	2000	3.67	7,340
$W_2 = 5 \cdot 10 \cdot 150$	7500	6.87	51,225
$W_3 = 18/12 \cdot 8.33 \cdot 150$	1875	4.16	7,800
$W_4 = 2 \cdot 3 \cdot 150$	900	1.50	1,350
$W_5 = 16/12 \cdot 15/12 \cdot 150$	+ 250	3.67	+ 918
Total	12,525		68,633

Distance of Resultant from edge

$$a = \frac{68{,}633 - 21{,}062}{12{,}525} = 3.80\text{ft}$$

inside middle 1/3 ✓

$$\text{SF Overturning Moment} = \frac{68633}{21062} = \underline{3.25} \geq 1.5 \checkmark$$

Bearing Pressure Largest @ "b"

$$W = 12{,}525 + 400\,\text{lb/ft}^2 \cdot 4.33\text{ft} = 14{,}257\text{ lb}$$

$$M_r = 68633 + 400\,\text{lb/ft}^2 \cdot 4.33\text{ft} \cdot 5.75\text{ft} = \underline{78{,}592\text{ lb}}$$

$$a = \frac{78{,}592 - 21{,}062}{14{,}257} = \underline{4.03\text{ft}}$$

inside middle 1/3 ✓

Minimum Steel

$\rho = .05 \Rightarrow \quad A_s = .005(12)(10) = 0.6\text{in}^2/\text{ft}$

Use No. 7 bars @ 11" spacing

Shear Capacity

$\rho = \frac{A_s}{bd} = \frac{0.6 \cdot \frac{12}{11}}{12 \cdot 10} = 0.00545$

$V_c = 8\rho^{1/3}\sqrt{f'c} = 8(.00545)^{1/3}\sqrt{4000} = 88.7 \text{ psi}$

$\lambda_s = \sqrt{\frac{2}{1+d/10}} = \sqrt{\frac{2}{1+10/10}} = 1.0$

$\phi V_c = \phi \lambda \lambda_s V_c bd = 0.75(1.0)(1.0)(88.7)(12)(10)$

$\Rightarrow 7983 \text{ lb} > V_u = 1.6(3456) = 5530 \text{ lb}$

$7983 \text{ lb} > 5530 \text{ lb}$ ✓

Wind Load Calculations

$p = q_h K_d (GC_{rn})$

q_h = Velocity pressure
K_d = wind directionality Factor
GC_{rn} = Net Pressure Coefficient
p = pressure

$q_h = .00256 K_z K_{zt} K_e V^2$

K_z = Velocity Pressure Exposure Coefficient
K_{zt} = Topographical Factor
K_e = Elevation Factor
V = basic wind speed

$K_z = .845$ (From Table 26.10-1)

$K_{zt} = (1 + K_1 K_2 K_3)^2$

$K_1 = .32$ $K_2 = .0675$ $K_3 = .005$ (From Table 26.8-1)

$K_{zt} = (1 + (.32)(.0675)(.005))^2 = 1.0002 \approx 1$

$K_e = e^{-.0000362(h)}$ $h = 499$ (height above sea level)

$K_e = e^{-.0000362(499)}$

$K_e = .9821$ (Confirmed by Table 26.9-1)

$q_h = .00256(.845)(1.0002)(.9821)(120)^2$

$q_h = 30.6 \text{ lb/ft}^2$

↳ $V = 120$ mph for basic wind speed

$K_d = .85$ (From Table 26.6-1)

Appendix F: Solar Panel Wind Load Calculations

Wind Load Calculations

$p = q_h k_d (GC_{rn})$

q_h = Velocity pressure
k_d = wind directionality Factor
GC_{rn} = Net Pressure Coefficient
p = pressure

$q_h = .00256 k_z k_{zT} k_e V^2$

k_z = Velocity Pressure Exposure Coefficient
k_{zT} = Topographical Factor
k_e = Elevation Factor
V = basic wind speed

$k_z = .845$ (From Table 26.10-1)

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

$k_1 = .32$ $\quad k_2 = .0675$ $\quad k_3 = .005$ (From Table 26.8-1)

$k_{zT} = (1 + (.32)(.0675)(.005))^2 = 1.0002 \approx 1$

$k_e = e^{-.0000362(h)}$ $\quad h = 499$ (height above sea level)

$k_e = e^{-.0000362(499)}$

$k_e = .9821$ (Confirmed by Table 26.9-1)

$q_h = .00256(.845)(1.0002)(.9821)(120)^2$

$q_h = 30.6 \text{ lb/ft}^2$

$\hookrightarrow V = 120$ mph for basic wind speed

$k_d = .85$ (From Table 26.6-1)

$GC_{rn} = \gamma_p \gamma_c \gamma_E (GC_{rn})_{nom}$ $(GC_{rn})_{nom}$ = Nominal Net Pressure Coefficient

$\gamma_p = \min(1.2, .9 + h_{pt}/h)$ h_{pt} = height of parapet
h = height of roof

$\gamma_p = .9 + 5/65 = .9769$

$\gamma_c = \max(.6 + .06L_p, .8)$ L_p = Chord Length of panel

$\gamma_c = .6 + .06(6.64042) = .9984$

γ_E = 1.5 for panels $1.5L_p$ from the edge of the array
1 for central panels

$(GC_{rn})_{nom} = .96$ (From chart on figure 29.4-7)

$GC_{rn} = (.9769)(.9984)(1.5)(.96) = 1.40455$
$GC_{rn} = (.9769)(.9984)(1)(.96) = .93637$

---

p (exposed) $= (30.6)(.85)(1.40455) = 36.532$ psf

p (non-exposed) $= (30.6)(.85)(.93637) = 24.355$ psf

$36.532 < 113$ ✓

Appendix G: Budget

Activity	Price	
FOUNDATION		
Marking	$	3,000.00
Excavation	$	711,309.56
Footings	$	125,396.00
Rebar	$	5,602.80
Pour Footings	$	64,829.22
Stem Walls	$	125,396.00
Foundation Plumbing	$	137,800.00
Radiant Heat tubing	$	35,000.00
Concrete Slabs Poured	$	64,829.22
PARKING GARAGE		
Structural Collumns	$	414,528.00
Pavement	$	143,262.00
Painting Lines	$	15,120.00
FIRST FLOOR		
Steel Structure	$	2,605,593.86
Plinth Beam and Slab	$	150,265.86
Elevator Shafts	$	190,265.86
Superstructure (Columns)	$	250,265.86
Masonry	$	100,265.86
Floor Slab	$	140,985.86
Ceiling Structure	$	172,800.00
Framing	$	71,865.86
Plumbing	$	179,865.86
Electrical	$	208,665.86
Exterior Finishes	$	969,411.72
Interior Finishes	$	2,630,505.86
Woodworking	$	74,889.86
Fixture Fittings	$	93,465.86
Waterproofing	$	200,265.86
Painting	$	110,745.86
PREFABRICATION		
Mod Fabrication Floor 1A	$	4,224,499.40
Mod Fabrication Floor 1B	$	4,224,499.40
Mod Floor Fabrication 2A	$	4,224,499.40
Mod Floor Fabrication 2B	$	4,224,499.40
Mod Floor Fabrication 3A	$	4,224,499.40
Mod Floor Fabrication 3B	$	4,224,499.40
Mod Floor Fabrication 4A	$	4,224,499.40
Mod Floor Fabrication 4B	$	4,224,499.40
Mod Delivery	$	480,000.00
Crane Setup	$	10,000.00
Mod Assmebly	$	100,000.00
Total	$	44,382,193.81

MODULE BUDGET	
ITEM	**COST**
12 Ft Joist Beams	$ 145.83
11 Ft Columns	$ 98.18
20 ft Beams	$ 90.00
12 Ft Beams	$ 37.50
Toilets	$ 100.00
Showers	$ 1,400.00
Stove	$ 100.00
Windows (8x8)	$ 100.00
Windows (4x8)	$ 100.00
Dishwasher	$ 50.00
Lighting	$ 100.00
Drywall	$ 200.00
Electrical	$ 1,200.00
Plumbing	$ 1,000.00
Chairs	$ 50.00
Couches	$ 100.00
Flooring	$ 100.00
Ceiling	$ 70.00
Paint	$ 144.00
Counter Top	$ 50.00
Tables	$ 30.00
Beds	$ 280.00
Desks	$ 80.00
Dresser	$ 80.00
Sinks	$ 75.00
Refridgerator	$ 150.00
Manufacturing	$ 36,000.00
Per Mod	$ 41,930.52
Total	$ 33,795,995.21

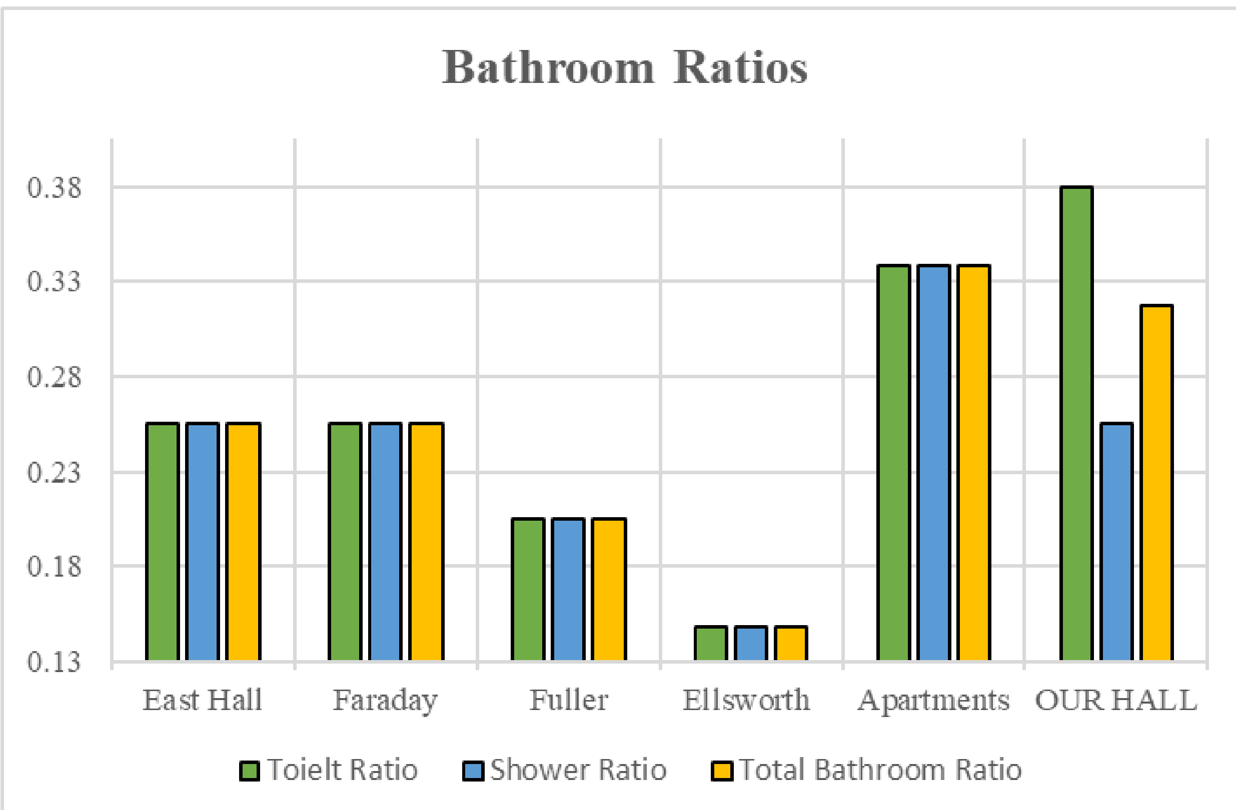
PARKING GARAGE/ FRIST FLOOR BUDGET		
ITEM	**COST SQ FT**	**COST**
Pavement	$ 3.00	$ 143,262.00
Elevators		$ 140,000.00
Interior Finsihes	$ 0.70	$ 30,240.00
Exterior Finishes	$ 5.00	$ 54,800.00
Woodworking	$ 0.57	$ 24,624.00
Plumbing	$ 3.00	$ 129,600.00
Electrical	$ 2.00	$ 86,400.00
Painting	$ 1.75	$ 75,600.00
Amenities		$ 1,250,000.00
Furniture		$ 250,000.00
Fixture Fittings	$ 1.00	$ 43,200.00
Floor Slabs	$ 2.10	$ 90,720.00
Celing Structure	$ 4.00	$ 172,800.00
Framing	$ 5.00	$ 21,600.00
Glass Façade	$ 25.00	$ 96,000.00
HVAC		$ 72,000.00
Labor (30%)		$ 804,253.80
Total		$ 3,485,099.80

SITEWORK BUDGET		
ITEM	**COST SQ FT**	**TOTAL COST**
Shipping		$ 480,000.00
Crane Operation		$ 110,000.00
Excavation	$ 2.18	$ 211,309.56
Storm Water Drain		$ 50,000.00
Demolition	$ 10.00	$ 500,000.00
Total		$ 1,351,309.56

FOUNDATION BUDGET		
ITEM	**COST SQ FT**	**COST**
Concrete	$ 10.00	$ 250,792.00
Rebar	$ 1.40	$ 5,602.80
Marking		$ 3,000.00
Plumbling	$ 4.00	$ 172,800.00
Labor(30%)		$ 129,658.44
Total		$ 561,853.24

BUILDING BUDGET		
ITEM	**COST SQ FT**	**TOTAL COST**
12 Ft Joist Beams	$ 16.67	$ 714,000.00
11 Ft Columns	$ 16.36	$ 440,640.00
20 ft Beams	$ 18.00	$ 734,400.00
12 Ft Beams	$ 15.00	$ 183,600.00
Solar Panels		$ 718,080.00
Heat System	$ 12.00	$ 1,200,000.00
Labor (30%)		$ 1,197,216.00
Total		$ 5,187,936.00

Appendix H: Bathroom Ratio



Appendix J: Hoar Metrics and Comparison Calculations

AU New Residence Hall				UA England Hall			
	GSF	Beds	SF/Bed		GSF	Beds	SF/Bed
Ground Floor	28,487	4	0	Ground Floor	42,111	0	0
Floor 2	25,266	93	271.68	Floor 2	31,220	124	251.77
Floor 3	25,266	93	271.68	Floor 3	31,220	124	251.77
Floor 4	25,266	93	271.68	Floor 4	31,220	124	251.77
Floor 5	21,366	88	242.80	Floor 5	31,220	124	251.77
	125,651	371	338.68		166,989	496	336.67
Adjusted Cost Date	4Q2023			Adjusted Cost Date	4Q2023		
Construction Cost	$		62,494,292	Construction Cost	$		67,619,266
Total Project Cost	$		74,993,150	Total Project Cost	$		77,515,154
Construction Cost/SF	$		497	Construction Cost/SF	$		405
Total Cost/SF	$		597	Total Cost/SF	$		464
Construction Cost/Bed	$		168,448	Construction Cost/Bed	$		136,329
Total Cost/Bed	$		202,138	Total Cost/Bed	$		156,281

MSU Preliminary Project Cost Evaluation				MSU Target Value Model			
	GSF	Beds	SF/Bed		GSF	Beds	SF/Bed
Ground Floor	35,400	0	0	Ground Floor	26,550	0	0
Floor 2	30,400	103	295.15	Floor 2	25,232	90	280.36
Floor 3	30,400	103	295.15	Floor 3	25,232	90	280.36
Floor 4	30,400	103	295.15	Floor 4	25,232	90	280.36
Floor 5	30,400	103	295.15	Floor 5	25,232	90	280.36
	157,000	412	381.07		127,478	360	354.11
Adjusted Cost Date	4Q2023			Adjusted Cost Date	4Q2023		
Construction Cost	$		64,350,000	Construction Cost	$		52,249,741
Total Project Cost	$		74,002,500	Total Project Cost	$		60,087,202
Construction Cost/SF	$		410	Construction Cost/SF	$		410
Total Cost/SF	$		471	Total Cost/SF	$		471
Construction Cost/Bed	$		156,189	Construction Cost/Bed	$		145,138
Total Cost/Bed	$		179,618	Total Cost/Bed	$		166,909

OUR HALL SQ FT	PERCENTAGE OF	OUR HALL COST	Size Comparative AU Cost
97020	77%	$44,382,193.81	$ 57,744,725.81
OUR HALL SQ FT	PERCENTAGE OF	Our Hall Cost	Size Comparasion UA England Hall Cost
97020	58%	$44,382,193.81	$ 45,036,021.82
OUR HALL SQ FT	PERCENTAGE OF	Our Hall Cost	Size Comparison MSU Cost
97020	76%	$44,382,193.81	$ 45,730,716.88

AU New Residence Hall				UA England Hall			
	GSF	Beds	SF/Bed		GSF	Beds	SF/Bed
Ground Floor	28,487	4	0	Ground Floor	42,111	0	0
Floor 2	25,266	93	271.68	Floor 2	31,220	124	251.77
Floor 3	25,266	93	271.68	Floor 3	31,220	124	251.77
Floor 4	25,266	93	271.68	Floor 4	31,220	124	251.77
Floor 5	21,366	88	242.80	Floor 5	31,220	124	251.77
	125,651	371	338.68		166,989	496	336.67
Adjusted Cost Date	4Q2023			Adjusted Cost Date	4Q2023		
Construction Cost	$		62,494,292	Construction Cost	$		67,619,266
Total Project Cost	$		74,993,150	Total Project Cost	$		77,515,154
Construction Cost/SF	$		497	Construction Cost/SF	$		405
Total Cost/SF	$		597	Total Cost/SF	$		464
Construction Cost/Bed	$		168,448	Construction Cost/Bed	$		136,329
Total Cost/Bed	$		202,138	Total Cost/Bed	$		156,281

MSU Preliminary Project Cost Evaluation				MSU Target Value Model			
	GSF	Beds	SF/Bed		GSF	Beds	SF/Bed
Ground Floor	35,400	0	0	Ground Floor	26,550	0	0
Floor 2	30,400	103	295.15	Floor 2	25,232	90	280.36
Floor 3	30,400	103	295.15	Floor 3	25,232	90	280.36
Floor 4	30,400	103	295.15	Floor 4	25,232	90	280.36
Floor 5	30,400	103	295.15	Floor 5	25,232	90	280.36
	157,000	412	381.07		127,478	360	354.11
Adjusted Cost Date	4Q2023			Adjusted Cost Date	4Q2023		
Construction Cost	$		64,350,000	Construction Cost	$		52,249,741
Total Project Cost	$		74,002,500	Total Project Cost	$		60,087,202
Construction Cost/SF	$		410	Construction Cost/SF	$		410
Total Cost/SF	$		471	Total Cost/SF	$		471
Construction Cost/Bed	$		156,189	Construction Cost/Bed	$		145,138
Total Cost/Bed	$		179,618	Total Cost/Bed	$		166,909

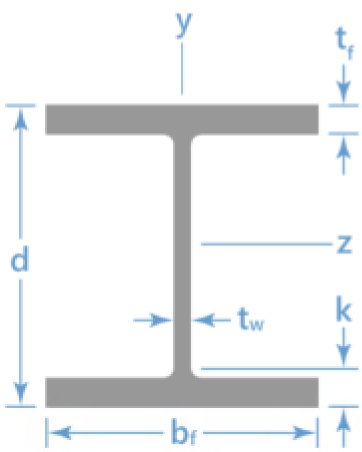
OUR HALL SQ FT	PERCENTAGE OF	OUR HALL COST	Size Comparative AU Cost
97020	77%	$44,382,193.81	$ 57,744,725.81
OUR HALL SQ FT	PERCENTAGE OF	Our Hall Cost	Size Comparasion UA England Hall Cost
97020	58%	$44,382,193.81	$ 45,036,021.82
OUR HALL SQ FT	PERCENTAGE OF	Our Hall Cost	Size Comparison MSU Cost
97020	76%	$44,382,193.81	$ 45,730,716.88

Appendix K: Risk Matrix Table

		Impact				
Probability		1 (Insignificant)	2 (Minor)	3 (Signficant)	4 (Major)	5 (Severe)
	5 (Almost Certain)					
	4 (Likely)	-Painting		-Mod Delivery		-Excavation
	3 (Significant)	-Fixture Fittings		Rough-In Plumbing	Concrete Slabs Poured	-Crane Set Up, Floor Slab
	2 (Unlikely)	-Rebar, Woodworking	-Electrical, Exterior Finishes	-Elevator Shafts, Framing	Dig Footings, Pour Footings, Stem Wall, Foundation Plumbing, Water Proofing	-Structural Columns, Structural Steel, Plinth Beam and Slab
	1 (Rare)	Paint Parking Lines	-Masonry, Interior finishes	-Pavement	Ceiling Structure	- Mod Fabrication, Parking Garage Floor, Mod Assembly

Appendix L: Rankings of IMC Connections (Corfar & Tsavdaridis,

Appendix M: Steel Beam and Column Section Sizes and Properties



Shape Name: W10X49

Geometric Properties

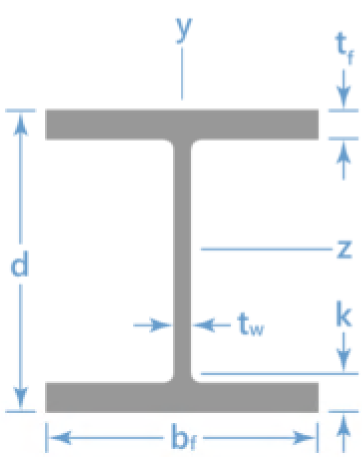
d	10	in	t_f	0.56	in
b_f	10	in	t_w	0.34	in

Section Properties

I_{yy}	93.4	in^4	C_w	2070	in^6
I_{zz}	272	in^4	W_{no}	23.6	in^2
Area	14.4	in^2	S_w	33	in^4
Z_{yy}	28.3	in^3	r_T	2.74	in
Z_{zz}	60.4	in^3	J	1.39	in^4

Connection Detailing Properties

k_{det}	1.25	in	k_{des}	1.06	in



Shape Name: W6X8.5

Geometric Properties

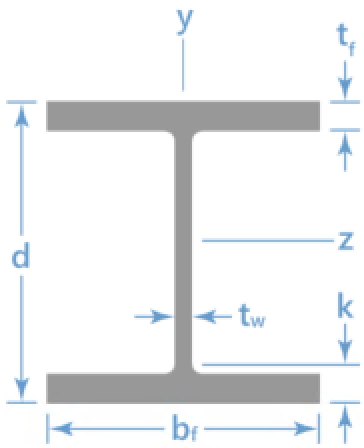
d	5.83	in	t_f	0.195	in
b_f	3.94	in	t_w	0.17	in

Section Properties

I_{yy}	1.99	in^4	C_w	15.8	in^6
I_{zz}	14.9	in^4	W_{no}	5.55	in^2
Area	2.52	in^2	S_w	1.06	in^4
Z_{yy}	1.56	in^3	r_T	1.038	in
Z_{zz}	5.73	in^3	J	0.033	in^4

Connection Detailing Properties

k_{det}	0.688	in	k_{des}	0.445	in



Shape Name: W14X68

Geometric Properties

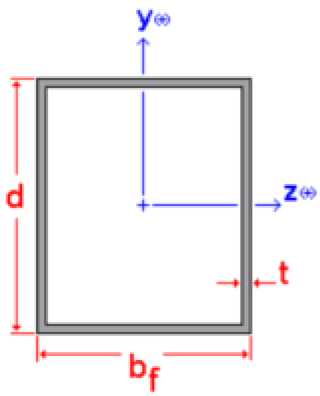
d	14	in	t_f	0.72	in
b_f	10	in	t_w	0.415	in

Section Properties

I_{yy}	121	in^4	C_w	5380	in^6
I_{zz}	722	in^4	W_{no}	33.2	in^2
Area	20	in^2	S_w	59.8	in^4
Z_{yy}	36.9	in^3	r_T	2.71	in
Z_{zz}	115	in^3	J	3.01	in^4

Connection Detailing Properties

k_{det}	1.563	in	k_{des}	1.31	in



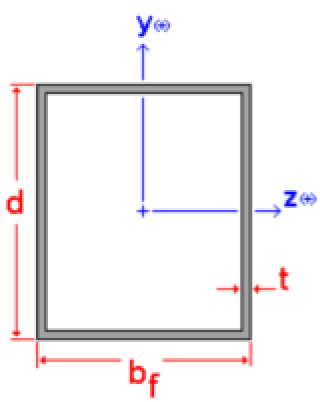
Shape Name: HSS14X10X8

Geometric Properties

d	14	in	t	0.465	in
b_f	10	in			

Section Properties

I_{yy}	341	in ^{4}	Z_{yy}	78.5	in ^{3}
I_{zz}	573	in ^{4}	Z_{zz}	98.8	in ^{3}
Area	20.9	in ^{2}	J	685	in ^{4}



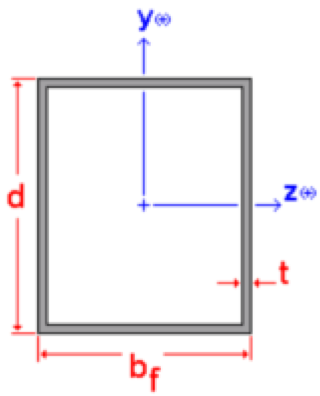
Shape Name: HSS14X10X8

Geometric Properties

d	14	in	t	0.465	in
b_f	10	in			

Section Properties

I_{yy}	341	in ^{4}	Z_{yy}	78.5	in ^{3}
I_{zz}	573	in ^{4}	Z_{zz}	98.8	in ^{3}
Area	20.9	in ^{2}	J	685	in ^{4}



Shape Name: HSS14X10X4

Geometric Properties

d	14	in	t	0.233	in
b_f	10	in			

Section Properties

I_{yy}	186	in ^{4}	Z_{yy}	41.8	in ^{3}
I_{zz}	310	in ^{4}	Z_{zz}	52.4	in ^{3}
Area	10.8	in ^{2}	J	362	in ^{4}

Appendix N: Modular vs. Traditional Comparison Metrics

	MODULAR			*TRADITIONAL*
	Star Apartments	**28 Austin St Aparments**	**NuLu Hotel**	**Cal State University Sacramento Hornet Commons**
Location, Year	Los Angeles, CA, 2014	Newton, MA, 2018	Louisville, Kentucky, 2018	Sacramento, CA, 2020
Total Sq Footage	95000	82000	98000	365000
Beds	N/A	N/A	N/A	1100
Cost	$ 40,000,000.00	$ 24,700,000.00	$ 43,000,000.00	$ 168,000,000.00
Cost/Sq Ft	$ 421.05	$ 301.22	$ 438.78	$ 460.27
Inflation Adjusted Cost/Sq Ft	$ 487.49	$ 317.06	$ 450.32	$ 460.27

Appendix O: Lag Times Chart

Activity	LAG (DAYS)
FOUNDATION	
Marking	0
Excavation	0
Footings	0
Rebar	0
Pour Footings	0
Stem Walls	13
Foundation Plumbing	13
Radiant Heat tubing	13
Concrete Slabs Poured	0
PARKING GARAGE	
Structural Collumns	0
Pavement	138
Painting Lines	137
FIRST FLOOR	
Steel Structure	0
Plinth Beam and Slab	127
Elevator Shafts	122
Superstructure (Columns)	118
Masonry	111
Floor Slab	109
Ceiling Structure	102
Framing	94
Plumbing	86
Electrical	86
Exterior Finishes	82
Interior Finishes	79
Woodworking	81
Fixture Fittings	74
Painting	67
PREFABRICATION	
Mod Fabrication Floor 1A	181
Mod Fabrication Floor 1B	165
Mod Floor Fabrication 2A	148
Mod Floor Fabrication 2B	131
Mod Floor Fabrication 3A	115
Mod Floor Fabrication 3B	97
Mod Floor Fabrication 4A	81
Mod Floor Fabrication 4B	68
Mod Delivery	42
Crane Setup	0
Mod Assmebly	0

Appendix P: Floor Slab Design

6" Floor Slab | pg 1

$f'c = 4\,ksi$

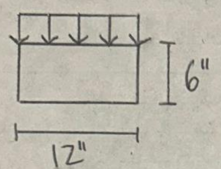
$fy = 60\,ksi$

$W_{LL} = 40\,psf$

$W_{DL} = (6")(12\,^{in}/_{ft})(150\,^{lb}/_{ft^3}) = 75\,^{lb}/_{ft^2}$

$W_u = 1.2\,W_{DL} + 1.6\,W_{LL}$

$W_u = 1.2(75\,^{lb}/_{ft^2}) + 1.6(40\,^{lb}/_{ft^2}) = 154\,^{lb}/_{ft^2}$



$\Rightarrow$ $W_u = (154\,^{lb}/_{ft^2})(1\,ft)$

$W_u = 0.154\,^{k}/_{ft}$

Finding M_u:

interior support $\Rightarrow$ $M_u = 1/9\, W_u (L)^2$

$M_u = 1/9\,(0.154\,^{k}/_{ft})(12)^2$

$M_u = 2.464\,ft\text{-}k$

mid span $\Rightarrow$ $M_u = 1/14\, W_u (L)^2$

$M_u = 1/14\,(0.154\,^{k}/_{ft})(12)^2$

$M_u = 1.584\,k\text{-}ft$

exterior $\Rightarrow$ $M_u = 1/24\,(W_u)(L)^2$

$M_u = 1/24\,(0.154)(12)^2$

$M_u = 0.924\,k\text{-}ft$

$\phi = 0.9$

$$\rho_{max} = 0.85\beta_1 \frac{f'c}{fy} \cdot \frac{\varepsilon_u}{\varepsilon_u + .005}$$

$$\rho_{max} = 0.85(0.85)\left(\frac{4}{60}\right)\left(\frac{.003}{.003+.005}\right) = 0.01806 = \rho_{max}$$

6" Floor Slab

Finding d @ Mu:

$$M_u = \phi \rho f_y b d^2 \left(1 - 0.59 \rho \frac{f_y}{f'c}\right)$$

$$d^2 \geq \frac{(2.464\ k\text{-}ft)(12\ in/ft)}{(0.9)(0.018)(60\ ksi)(12\ in)\left[1 - 0.59(0.018)(60\ ksi/4\ ksi)\right]}$$

$d^2 \geq 22.89\ in^2$

$d \geq 4.78\ in = d_{req}$

$5" \geq 4.78"$ Ok ✓

Minimum Steel:

$\rho = \frac{A_s}{b \cdot d}$ ← steel reinforcement ratio

minimum steel area:

$A_{s,min} = 0.0018(h \cdot s)$

h = slab thickness
S = slab rebar spacing

$A_{s,min} = 0.0018(6 \cdot 12)$

$A_{s,min} = 0.1296\ in^2$

No. 3 bar = $0.11\ in^2$
No. 4 bar = $0.2\ in^2$

use No.4 bars @ 12" spacing

Appendix Q: Seismic Base Shear Calculations

$I = 1.25 \quad R = 8 \quad S_{D1} = .044 \quad S_{DS} = .15$

$T_x = 1.382\,s$

$W = (.079k \times 3)(480\,ft^2) + (.085k)(480\,ft^2) = 154.56\,k$

$$V_x = \frac{S_{D1} W}{T\left(\frac{R}{I}\right)} = \frac{.044(154.56)}{1.382\left(\frac{8}{1.25}\right)} = .769k$$

$$V_{x\,max} = \frac{S_{DS} W}{\frac{R}{I}} = \frac{.15(154.56)}{\left(\frac{8}{1.25}\right)} = 3.62k$$

$$V_{x\,min} = .044\,S_{DS} I W = .044(.15)(1.25)(154.56)$$

$$= 1.275k$$

$\therefore$ use $V_{x\,min} = 1.275k$

for base shear