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Sustainable Rooftop Technologies: A Structural Analysis of Buildings at WPI

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Sustainable Rooftop Technologies

A Major Qualifying Project submitted to the Faculty of Worcester Polytechnic Institute in partial fulfillment of the requirements for the Bachelor of Science degree

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ABSTRACT

This project evaluated the feasibility of the installation of sustainable rooftop technologies on selected buildings at Worcester Polytechnic Institute (WPI). This report includes the structural analysis and design of three sustainable rooftop technologies: solar panels, green roofs, and solar collectors. These technologies have the ability to save energy, while contributing to WPI's sustainability plan. Additionally, an economic analysis is prepared to show the simple payback periods of installing these sustainable rooftop technologies.

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CAPSTONE DESIGN STATEMENT

To fulfill the requirements of the Capstone Design, our team completed a Major Qualifying Project focused on the plan and design of sustainable rooftop technologies on existing buildings at Worcester Polytechnic Institute (WPI). Structural analyses of different buildings, as well as feasibility of construction and costs were addressed in this project. The Capstone Design constraints expected in this project include: economic, environmental, constructability, sustainability, ethical, and health and safety.

Design Problem

As Worcester Polytechnic Institute is committed to a sustainability plan of ecological stewardship, social justice, and economic security, every member of the WPI community should be engaged in this process. Our plan for sustainable rooftop technologies follows the same path of the already existing sustainability plan; it is our job to embrace this mission in the local community.

To approach the problem and support the WPI sustainability plan, our group designed solar panels, green roofs, and solar collectors, for a number of existing buildings on campus. Each proposed system generates a different optimal solution, which includes, but not limited to, energy efficiency, water storage, and building cool-off.

Economic

The plan of implementing sustainable rooftop technologies comes at a cost. For each alternative that was considered, there was a different design and associated cost. Our group provided costs for implementing each of these systems, which included the actual cost of the system, operational costs, and lifetime. Similarly, the simple payback period of the desired project was determined, and recommendations were provided based on this economic analysis.

Constructability

Constructability is one of the most important factors to consider for implementing these sustainable systems. Considerations regarding the type of building (academic/residential/recreational), type of roof (slope/flat), year built, and size of the building are all accounted under this criteria. Similarly, the following factors were analyzed and considered:

- Structural layout of the selected buildings.

- Zoning, permitting, and regulations.
- Construction schedule/time frame for each system.

Sustainability

Sustainability in this project consisted of economic, environmental and social aspects. The design and construction of sustainable rooftop technologies includes all of these aspects and brings them together. Solar panels, green roofs, and solar collectors alleviate environmental concerns by implementing new technology in existing buildings at WPI. Sustainable technologies reduce the consumption of energy, and they create more efficient buildings on campus. Implementing sustainable rooftop technologies on buildings at WPI can alleviate the urban heat island effect. This is accomplished by reducing energy usage and decreasing gas emissions with the use of natural sources of energy.

Environmental

Through the development of this project, another constraint similar to sustainability is environmental. Installing each sustainable rooftop technology requires construction on the WPI campus, which can negatively impact the environment. Noise and dust can emit into the air during the construction process of these rooftop technologies. Our group proposed installation processes, which will have the least amount of impact on noise and air pollution.

Health and Safety

It is of extreme importance to protect the public and the community of WPI of any possible risks. Health and safety of all the people involved in this project was considered, especially for potential users of the selected buildings. The design and construction of these systems are in accordance with the *International Building Code* and all safety factors.

Ethical

Ethical practices played an important role in this project. It was crucial to consider ethical codes for the design and construction of sustainable rooftop technologies. All the appropriate codes and regulations were considered in the implementation of these systems. Furthermore, the team completed confidentiality agreements for the information that was provided by WPI Facilities Department.

PROFESSIONAL LICENSURE

Civil engineering has been prevalent in human history since the beginnings of mankind. In addition to gathering food, society's main concern includes building a settlement, which requires civil engineering. Throughout time, civil engineering has advanced into a field, which now contains qualified individuals who have achieved a high level of education. Only a professional licensed civil engineer may prepare, sign, seal and submit engineering plans and drawings to a public authority for approval, or seal engineering work for public and private clients. The purpose of licensure is to protect the health and welfare of the public by regulating requirements to restrict engineering practice to qualified individuals. In order to become licensed, engineers must complete a number of requirements. First, one must complete a four or five-year college undergraduate degree. Following graduation, the individual must work under a professional engineer for at least four years, pass an intensive exam, and earn a license from their state's licensure board. Having a professional engineer's license means acceptance of both the technical and the ethical obligations of the engineering profession. Once a professional engineer is licensed, the individual is free to practice the discipline of civil engineering, and may stamp documents of any kind within their practice and expertise. Licensure is important since it is legally required to be a consulting engineer or a private practitioner. It can also raise prestige and accelerate career development.

The process of preparing a sustainable rooftop technology plan for WPI exposed our group to the concept of structural design and analysis, which is also required by professional licensed civil engineers. Our project explores alternative rooftop technologies that could possibly be employed by the WPI community. These alternative practices consist of installing solar panels, green roofs, and solar collectors to the roofs of chosen buildings at WPI. A structural analysis of the buildings was executed, as well as a proposed sustainable rooftop technology design. In order to install solar panels, green roofs, and solar collectors, one must make sure that the building can carry the loads imposed by these technologies. Additionally, our analysis included how efficient solar panels, green roofs, and solar collectors are.

Solar panels, green roofs, and solar collectors have the ability to deal with the negative impacts of the urban heat island effect by making the problem part of the solution. This project reflects the meaning of a professional licensed civil engineer. There are technical aspects to this project: designing the layout of solar panels, green roofs, and solar collectors, choosing a building and analyzing the structure’s support, and producing an economic evaluation. Finally, our project relates to the nature of a professional licensed engineer by promoting health and welfare in an ethical manner and making the WPI community more sustainable.

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CHAPTER 1: INTRODUCTION

This chapter contains an introduction to sustainable rooftop technologies, and their ability to mitigate global environmental problems. Additionally, this section lays out the goals and objectives for this project.

1.1 Problem Statement

Climate change, air pollution, and water pollution are a few of many environmental problems that the world is dealing with today. Specifically in urban areas, the heat island effect is another problem, which is increasing temperatures. The negative impacts from the heat island effect in urban cities include an increase in energy usage, increase in gas emissions, impaired water quality, and health risks. It is the responsibility of our generation to explore ways to preserve the environment for future generations. Implementing sustainable rooftop technologies is one practice, which can help reduce some of the environmental problems the world is dealing with today. Sustainable rooftop technologies include solar panels, solar collectors, green roofs, stormwater retention systems, and daylighting systems. All of these systems use the source of the problem, the sun, as a way to reduce environmental problems. Our objective is to explore three rooftop technologies, and investigate the structural impact these systems can have on buildings at Worcester Polytechnic Institute (WPI). The three technologies chosen were solar panels, green roofs, and solar collectors.

1.2 Goals and Objectives

The goal of this project is to provide recommendations and improvements for the installation of sustainable rooftop technologies on existing buildings at WPI. Additionally, the impact of these technologies on the net energy demands was investigated. The objectives for this project included:

1. Determine WPI's approach to sustainable practices, as well as its current sustainable building practices.
2. Identify candidate buildings at WPI for the installation of certain sustainable rooftop technologies.

3. Identify energy demand of each building to quantify the needed output for each sustainable rooftop technology.
4. Determine the design and construction process for each sustainable rooftop technology on the desired building for installation.
5. Identify structural design activities for the selected buildings, which include identifying structural reinforcements needed to withstand sustainable rooftop technologies.
6. Conduct an economic analysis to determine whether it is feasible to implement sustainable rooftop technologies at WPI.

CHAPTER 2: BACKGROUND

This chapter provides a brief introduction of the heat island effect, which is an environmental problem, which can be reduced in urban areas through sustainable rooftop technologies. Additionally, this section contains background information on various sustainable rooftop technologies: solar panels, solar collectors, and green roofs.

2.1 The Heat Island Effect

The heat island effect describes urban regions, which become hotter than its rural surroundings due to urban area development of buildings, roads, and other infrastructure, which replaces open land and vegetation. The annual mean temperature of a city with one million people or more can be 1.8°F warmer than its surroundings. However, the temperature difference can be as much as 22°F during the nighttime due to the buildup of heat on infrastructure from the sun during the day, which is slowly released throughout the night. Shaded or moist surfaces in rural areas remain close to air temperatures. Elevated temperatures in urban areas can negatively impact a community's environment and quality of life (United States Environmental Protection Agency, 2017).

2.1.1 Negative Impacts

Some of the negative impacts of the heat island effect include increased energy consumption, elevated emissions of air pollutants and greenhouse gases, compromised human health and comfort, and impaired water quality (United States Environmental Protection Agency, 2017):

1. ***Increased Energy Consumption:*** When the temperature rises in urban areas during the summertime, there is an increase of energy demand for cooling. Starting from 68-77°F, the electricity demand for cooling increases 1.5-2.0% for every 1°F increase in air temperatures (United States Environmental Protection Agency, 2017).
2. ***Elevated Emissions of Air Pollutants and Greenhouse Gases:*** The burning of fossil fuel increases air pollutants and greenhouse gas emissions. Fossil fuel power plants are used to supply electricity, which in turn emit sulfur dioxide, nitrogen oxides, particulate matter, carbon monoxide, mercury, and carbon dioxide. All of these pollutants are

harmful to human health and contribute to air quality problems including smog, fine particulate matter, acid rain, and global climate change.

3. ***Compromised Human Health and Comfort:*** High temperatures affect human health and contribute to discomfort, respiratory difficulties, heat cramps and exhaustion, non-fatal heat strokes, and heat-related mortality. The Centers for Disease Control and Prevention estimated from 1979-2003 that excessive heat exposure contributed to more than 8,000 premature deaths in the United States (United States Environmental Protection Agency, 2017).
4. ***Impaired Water Quality:*** High pavement and rooftop surface temperatures can heat stormwater runoff. Tests have shown that 100°F pavement can elevate initial rainwater temperature from 70°F to over 95°F (United States Environmental Protection Agency, 2017). This heated stormwater will eventually runoff into storm sewers and raise the water temperature of streams, rivers, ponds, and lakes. Rapid temperature changes in aquatic ecosystems can be fatal to aquatic life.

2.1.2 Strategies to Reduce Urban Heat Islands

There are various strategies, which help to reduce urban heat islands. One strategy is to increase tree and vegetation cover. This can provide shade and cooling to urban areas, as well as reduce stormwater runoff and protect against erosion. Another strategy is to implement more green roofs in urban areas. By growing a vegetative layer on a rooftop, the roof surface temperature will decrease and stormwater management will improve. Additionally, cool roofs are made of materials or coatings that reflect sunlight and heat away from a building. Cool roofs have the ability to reduce roof temperatures, increase the comfort of building occupants, and reduce energy demand. Vegetation cover, green roofs, and cool roofs are a few of many strategies that have the ability to reduce urban heat islands (United States Environmental Protection Agency, 2017).

2.2 Solar Panels

Solar energy is a renewable source of energy created from the sun. Solar energy produces energy through a process, which is sustainable, inexhaustible, non-polluting, noise-free, and does not emit greenhouse gases (Energy Matters, 2016). Solar panels in the United States should face south to absorb the most sunlight; however, solar panels do not need direct sunlight to produce electricity. Solar power has the capacity to provide energy for air conditioners, hot water heaters, cooking and electrical appliances, natural gas, electricity, or oil fuels (Solar Power Authority, 2017). Solar technologies can be expensive and require a lot of land area to collect the sun's energy at useful rates; however, solar electricity can pay for itself in the long term, usually five to ten years with tax incentives (Imboden, 2009). When solar panels are purchased, the federal solar tax credit allows the owner to deduct 30% of the cost of installing a solar energy system from the owner's federal taxes. Not only has the cost of solar panels dropped by 80% since 2008 due to its high demand, but maintenance is minimal and returns are high once solar panels have been installed (Solar Power Authority, 2017).

2.2.1 Solar Panel Properties

Solar panel systems (photovoltaic or PV system) are made up of semiconductor materials that convert sunlight into an electric current (Energy Matters, 2016). When sunlight hits the cells of the solar panels, electrons become loose from their atoms and flow through the cell generating electricity (Imboden, 2009). The semiconductor material is covered with an anti-reflective coating and made up of silicon wafers impregnated with impurities; these impurities have the ability to improve electrical properties. The solar cells are joined together by electrical contacts, and located between a superstrate layer on top and a back-sheet layer below (Energy Matters, 2016).

2.2.2 Solar Panel Process

The photovoltaic effect is the process by which light is converted to energy at the atomic level. The majority of energy the solar cells produce goes into a grid-connected inverter, which converts the electric charge from a direct current (DC) into an alternating current (AC). This allows the solar electricity current to flow to and from the grid connect inverter. The solar electricity can power the appliances in a building when needed, and the leftover solar electricity will flow to the grid-connected inverter where it is stored. If more energy is produced than used, then the owner is credited on their electricity bill, making this an incentive for building owners to implement renewable systems (Energy Matters, 2016).

2.2.3 Types of Solar Panel Systems

As the use of technology has increased over the years, different types of solar panels have been created. Of all these, approximately 90% of solar panels are made of silicon photovoltaic material (Battaglia, Cuevas & De Wolf, 2016). This section describes two different types of solar panel systems: crystalline silicon panels and thin-filmed panels.

Crystalline Silicon (Monocrystalline Silicon & Polycrystalline Silicon)

Crystalline silicon cells are the most common solar cells used in commercially available solar panels, consisting of more than 85% of the world's photovoltaic cell market sales (Battaglia, et. al., 2016). Crystalline silicon panels have two subtypes: Monocrystalline Silicon & Polycrystalline Silicon. The main difference between these types is the production technique. Each technique has its advantages and disadvantages. The cells have laboratory energy efficiencies of 25% for monocrystalline cells and over 20% for polycrystalline cells. However, industrially produced solar modules currently achieve efficiencies ranging from 18%–22% (Battaglia, et. al., 2016).

Monocrystalline solar panels have the highest efficiency rates since they are made out of the highest-grade silicon. Monocrystalline cells are produced from pseudo-square silicon wafers (substrates cut from boules grown by the Czochralski process), the float-zone technique, ribbon growth, or other emerging techniques. These other emerging techniques can have a specific reason for their utilization. For example, if produced using the ribbon growth technique, the production costs as well as the carbon footprint both decrease efficiency. These panels are also space-efficient. Since they yield the highest power outputs, they require less space compared to the other types. They also have a long life expectancy (25+ years) and tend to work better in low-light conditions. This type of panel is the most efficient and has a longer lifespan than other types of panels; however, it is the most expensive type of panel (Battaglia, et. al., 2016).

Polycrystalline silicon solar cells are a newer technology and vary in the manufacturing process. They are traditionally made from square silicon substrates cut from ingots cast in quartz crucibles. Polycrystalline cells are more cost effective to produce due to the fact that many cells can be created from a single block. However, every time silicon is cut, the edges become deformed, which results in a lower operating efficiency. Polycrystalline cells have become the dominant technology in the residential solar panels market because of their operating efficiencies, and the low-cost method by which they can be produced. In terms of efficiency, polycrystalline solar cells are now very close to monocrystalline cells (Battaglia, et. al., 2016).

Since crystalline cells were one of the first technologies, much of the production and manufacturing techniques have been refined to reach their maximum potential. Advantages of crystalline silicone cells include a high efficiency rate of about 12% to 24.2%, high stability, ease of fabrication, high reliability, and long lifespan. Other benefits include high resistance to heat and lower installation costs. Negatively, these panels are the most expensive, in terms of initial cost, and have a low absorption coefficient (Battaglia, et. al., 2016).

Thin-Film Panels

The differences between thin-film and crystalline silicon solar cells are the thin and flexible pairing of layers, and the photovoltaic material: either cadmium telluride or copper indium gallium dieseline instead of silicon. Thin-film solar panels are the least efficient type of solar panel. Depending on the technology, thin-film module prototypes have reached efficiencies between 7–13%, and production modules operate at about 9% (Battaglia, et. al., 2016).

Thin film panels are made by depositing photovoltaic substances (such as glass) into a solid surface. Multiple combinations of substances have successfully and commercially been used for the photovoltaic substance. Typical thin-film solar cells are one of four types, depending on the material used: amorphous silicon (a-Si) and thin-film silicon (TF-Si); cadmium telluride (CdTe); copper indium gallium dieseline (CIS or CIGS); and dye-sensitized solar cell (DSC) plus other organic materials (Battaglia, et. al., 2016).

Despite being the least efficient, thin-film panels have advantages that should be considered when planning for solar roofing. Thin-film material is 100 times thinner than traditional solar panels, provides flexibility, and is lightweight. Thin-film panels are created by combining consecutive thin layers of material together. The result is a single film that is capable of being distributed in rolls or sheets making it easier to handle. Thin-film panels are the lowest cost panels to produce because of their low material costs. However, thin-film panels require the most space for producing the same amount of power as other solar panels, making them less efficient. Additionally, the thin material’s durability begins to suffer over time, requiring frequent replacement (Battaglia, et. al., 2016).

2.2.4 Structural Considerations

Placing solar panels on the roof of a building adds various loads to the structure. To perform a structural analysis on the building involves to first define the loads, and then to determine how the loads affect the structure (Wrobel, 2017).

Solar panels add a dead load to the roof of a building. The dead load includes the self-weight of all the physical components of the solar panels. The dead load applied to the roof is a concentrated load located where the roof supports the panels, which is usually located at each corner of the panel (Wrobel, 2017). In geographic regions where snow loads are present on roofs, warm roofs are constructed which can help decrease the snow load. If solar panels are raised above the roof, then they do not receive the benefit of the warm roof to decrease the snow load, which results in an increase of the snow load as well (Wrobel, 2017). The design of snow loads for roofs that include solar panels shall be determined in accordance with ASCE 7-10. Wind loads are also considered as they have the ability to act in various directions, both upward and downward on solar panels. Wind loads also act on different locations of the solar panels depending on which direction the wind is blowing from (Wrobel, 2017). Some of the elements for which wind loads should be considered are: the ultimate design wind speed, risk category, wind exposure, internal pressure coefficient, component and cladding, and seismic concerns for non-structural attachments. Finally, seismic loads should be considered despite the geographic location of Worcester, MA, where earthquakes do not have a large effect on structures. Due to the complexity of wind loads and seismic loads acting on solar panels, these loads should not only be calculated in accordance with the ASCE 7-10, but also in accordance with solar panel related documents provided by the Structural Engineers Association of California. Finally, the size, quantity, and location of solar panels on the roof of a building should be considered. All of these factors will determine the effect of the loads, and the existing structures' capacity for the addition of solar panels.

2.2.5 Wind Design for Solar Panels

A document by the Structural Engineers Association of California titled, *Wind Design for Low-Profile Solar Photovoltaic Arrays on Flat Roofs*, provides information on the step-by-step process for calculating wind loads on solar panels. There are many factors to consider when analyzing the effect of wind loads on solar panels. This document provides information on the determination of wind loads for solar photovoltaic arrays, which is not explicitly covered by the methods contained in the ASCE 7-10 (Structural Engineers Association of California, 2012). Steps to determine wind loads on rooftop equipment and other structures are located in Table 29.1-1 in ASCE 7-10. However, in Step 7 of this table, the equation provided needs to be changed for the consideration of solar panels. The design wind pressure for rooftop solar arrays can be determined using the equation below (Structural Engineers Association of California, 2012).

$\mathbf{p = q_h * (GC_m)}$
p = wind pressure for rooftop solar arrays
$\mathbf{q_h}$ = velocity pressure evaluated at mean roof height of the building (lb./ft^2) $\mathbf{GC_m}$ = combined net pressure coefficient for solar panels (lb./ft^2)

Solar panels mounted on a roof are highly vulnerable to the speed and direction of the wind approaching the panel. There are three distinct regions or zones on a roof where the wind flow characteristics and resulting wind loading on solar panels are different: interior, edge, and corner zones. Wind loads on solar panels located in the corner zones of roofs are much greater than those in the middle of the roof. Higher tilt panels are particularly vulnerable to the vertical component of swirling winds in the corner vortices of the panels. Since solar panels in the northern hemisphere face south, the northeast and northwest corners of the panel create severe loading. The southeast and southwest corners of the panel still create loading, just not as strong as the other two corners (Structural Engineers Association of California, 2012).

Different restricting values for the size, height, spacing, and positioning of solar panels are presented in Table 1. These values will help when designing the roof layout and calculating wind load values. *Wind Design for Low-Profile Solar Photovoltaic Arrays on Flat Roofs* provides more detailed information and application for these values.

Table 1: Solar Panel Design Restrictions (Structural Engineers Association of California, 2012)

Characteristic	Quantity
Height of gap between panels and roof surface (h_1)	$\leq$ 2 ft
Maximum height above the roof surface (h_2) for panels	4 ft
Panel chord length (l_p)	$\leq$ 6 ft 8 in
Distance between solar panels and roof edge	$\leq 2*h_2$
Space between rows of solar panels	$\leq$ 2*panel characteristic height (h_c)
Panel tilt angle for typical installations	0-35 degrees

2.2.6 Seismic Requirements for Solar Panels

Similar to the previous section, a document by the Structural Engineers Association of California titled, *Structural Seismic Requirements and Commentary for Rooftop Solar Photovoltaic Array*, provides information on how to calculate and deal with seismic forces when designing solar panels. It is important to understand the effect of seismic forces on solar panels, and prepare for any type of loading. As described in the document, solar arrays can either be attached or unattached to the roof structure of a building (Structural Engineers Association of California, 2012). For our project, attached solar arrays are used, therefore the information obtained has different values and procedures than those for unattached solar arrays.

Solar panels and their structural support systems shall be designed to provide life-safety performance in the design basis earthquake ground motion. Life-safety performance means that solar panels are not expected to create a hazard to life. For example, as a result of breaking free from the roof, sliding off the roof's edge, exceeding the downward load-carrying capacity of the roof, or damaging skylights, electrical systems, or other rooftop features or equipment in a way that threatens life-safety. Solar array support systems that are attached to a roof structure shall be designed to resist the lateral seismic force (F_p) specified in Chapter 13 of ASCE 7-10. In the computation of F_p, an evaluation of flexibility and ductility capacity of the support structure is permitted to be used to establish seismic coefficients of component amplification factor (a_p) and component response factor (R_p). These values can be found in Table 13.5-1 of ASCE 7-10 (Structural Engineers Association of California, 2012).

2.3 Green Roofs and Stormwater Retention Systems

A green roof is a roof of a building that is covered with vegetation. There are two characterizations of green roofs: extensive green roofs and intensive green roofs. Intensive green roofs use planting mediums that have a greater depth than extensive green roofs; this requires more maintenance because of the larger plant varieties intensive planting mediums can support. An extensive green roof has vegetation ranging from sedums to small grasses, herbs, and flowering herbaceous plants. Extensive green roofs are ideal for efficient stormwater management and low maintenance needs. An intensive green roof has vegetation ranging from herbaceous plants to small trees. Intensive green roofs require professional maintenance and advanced green roof irrigation systems. Rooftop farms fall under the intensive green roof category. The growing medium for an extensive green roof is 6" or less, while the growing medium for an intensive green roof is greater than six inches (Jörg Breuning & Green Roof Service LLC, 2017). Green roofs have the ability to reduce urban heat islands and can also serve as a stormwater retention system.

2.3.1 The Urban Stormwater Problem

Urban areas generate more stormwater runoff than natural areas due to a greater percentage of impervious roof surfaces and paved surfaces that prevent water infiltration. The United States Environmental Protection Agency (USEPA) concluded that a typical city block generates more than five times as much runoff than a woodlot of the same area. Additionally, urban stormwater runoff carries pesticides, heavy metals, and contaminated nutrients, which have the ability to flow into various bodies of water. According to USEPA, "The most recent National Water Quality Inventory reports that runoff from urbanized areas is the leading source of water quality impairments to surveyed estuaries and the third-largest source of impairments to surveyed lakes (Andresen, Fernandez, Rowe, Rugh, VanWoert & Xiao, 2004)."

2.3.2 Green Roof Stormwater Retention Success

Implementing green roofs in urban areas is a solution to reduce stormwater runoff. The Michigan State University Horticulture Teaching and Research Center conducted a 14-month study in which three simulated roof platforms were constructed. One of the roof platforms contained gravel, the other was vegetated, and the third was non-vegetated. Over a 14-month period, the vegetated roof had the greatest overall rainfall retention at 60.6%, while the non-vegetated roof had rainfall retention of 50.4%, and the gravel roof had rainfall retention of 27.2%. These percentages refer to the amount of rainfall that did not runoff the roof out of total amount of rainfall in the 14-month period. To conclude, vegetated roof platforms retain greater quantities of stormwater than conventional roofs. However, the study stated, "if the objective of a green roof is to maximize rainfall retention, then factors such as slope and media depth must be addressed (Andresen, et. al., 2004)."

2.3.3 Benefits of Green Roofs

Not only do green roofs control stormwater runoff, but their designs also have many other benefits (Andresen, et. al., 2004):

- Insulate buildings, which saves on energy consumption.

- Increase the lifespan of a typical roof by protecting the roof membrane from damaging ultraviolet rays, extreme temperatures, and rapid temperature fluctuations.
- Filter harmful air pollutants.
- Contribute to aesthetically pleasing environment to live and work by controlling the temperature of a building.
- Provide habitat for a variety of living organisms.
- Contribute to reducing the urban heat island effect.

2.3.4 Structural Considerations

Similar to solar panels, green roofs contribute dead loads, live loads, snow loads, rain loads, wind loads, and seismic loads to the roof of a structure. The most contributing factor to the loads on a green roof depends on the size and type of vegetation, which is used. An intensive green roof contributes more load than an extensive green roof due to the larger trees, plants, and sometimes water features that are being used. Additionally, the location of the stormwater storage has an impact on the structure of a building. Depending on the green roof, stormwater can be stored within the green roof itself, in a tank below the building, or drained towards the local watershed.

The structural considerations for green roof design are typically attributed to the different components (layers) of green roofs. A typical, modern, vegetated roof requires a minimum of eight layers: plant level (vegetation), substrate layer, insulation layer, filter fabric, drainage layer, protection fabric, roof barrier, and waterproof layer as shown in Figure 1 (Gartner, 2008). To conclude, the overall design and layers of a green roof determine the effect of the various loads on the structure of a building.

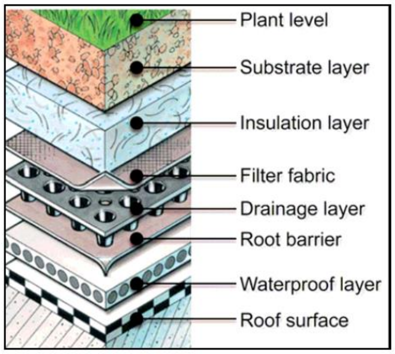

Figure 1: Layers of a Typical Modern Vegetated Roof (Gartner, 2008)

2.4 Solar Collectors

Solar collectors convert energy from the sun into usable heat in a solar water heating system. This energy can be used for hot water heating, pool heating, space heating, or even air conditioning (Apricus Solar Water Company, 2017).

2.4.1 Solar Collector Process

Solar collectors can be mounted on a roof, wall, or the ground. A circulation pump moves liquid through the collector, which then carries heat back to the solar storage tank. Throughout the day, water in the solar storage tank is heated up. When hot water is used, the solar preheated water is fed into the traditional water heater and supplied for its desired usage (Apricus Solar Water Company, 2017).

2.4.2 Types of Solar Collector Technologies

There are three main types of solar collector technologies: evacuated tube solar collectors, flat plate solar collectors, and thermodynamic panels. Each of these technologies has different advantages and can be used for different types of applications.

Evacuated tube solar collectors are the most popular and commonly used solar collector technology. They are light and compact, making them easy to install. The tubes have excellent insulation and are virtually unaffected by air temperature. Out of all the types of solar collector technologies, evacuated tube solar collectors are the most efficient with a rate of efficiency of 70% per cent (Apricus Solar Water Company, 2017). The technology lasts for over 20 years, and the tubes can be replaced individually if one becomes faulty, avoiding the need to replace the whole collector. In terms of material, the tubes are either made out of double glass or a glass-metal combination. Double glass tubes have a reliable vacuum, but reduce the amount of light that reaches the absorber inside. Additionally, they may experience more absorber corrosion due to moisture or condensation forming in the non-evacuated area of the tube. The glass-metal combination tubes allow more light to reach the absorber and reduce the chances of moisture corroding the absorber. In an evacuated tube solar collector, water is heated in the collector and is sent through pipes to the water storage tank, where it is then distributed throughout the building.

Flat plate solar collectors are another type of solar collector technology. This technology has a life expectancy of over 25 years. In an area that produces an average level of solar energy, the amount of energy a flat plate solar collector generates equates to around one square foot panel generating one gallon of one day's hot water. There are several different types of flat plate solar thermal technologies. The harp design is used in low-pressure thermos-syphon systems or pumped systems. The serpentine design uses a continuous S-shaped absorber and is used in compact hot water only systems, which do not utilize space heating. Flooded and boundary absorber systems use multiple layers of absorber sheet, where the heat is then collected in the boundary layer of the sheets. Polymer flat plate collectors are an alternative to metal plate collectors. Metal plates are more prone to freezing whereas the polymer plates themselves are freeze tolerant so they can dispense with antifreeze and use water as a heat transferring liquid. Polymer plates can be plumbed into an existing water tank, removing the need for a heat exchanger, which increases efficiency.

Thermodynamic panels are a new development in solar thermal technology. These panels are closely related to air source heat pumps in their design, but are deployed on the roof like regular solar collector panels, and do not have to be facing south. These panels can produce up to 100% of domestic heating needs. They also generate energy all year round since they do not rely on having optimal climate conditions to reach their maximum output potential. Thermodynamic panels act as a reverse freezer and do not use solar radiation to heat up heat transferring liquids. The panels have a refrigerant passing through them, which will absorb the heat. The heat that passes through the panel will then, in turn, become a gas. The gas is then compressed which raises its temperature, and it will then be passed on to a heat exchanging coil that is located within a hot water cylinder. The heated water in the cylinder is heated to 55°F and can then be distributed throughout the building.

2.4.3 Structural Considerations

Solar collectors impose similar loads to the roof structure as solar panels: dead loads, snow loads, wind loads, and seismic loads. Solar collectors add dead loads as a result from the weight of the collector, the mounting hardware, and the collector fluid. Typically, the collector has a dead load of approximately three to five pounds per square foot, but the exact weight considerations can be obtained from the manufacturer of the solar collectors (HTP, 2017).

In areas prone to heavy snowfall, such as Massachusetts, snow loads need to be considered in the design of the solar tubes. Ideally, solar collectors should be installed at an angle of 50º or greater to promote snow sliding off the tubes (HTP, 2017). Similarly, when installing solar tube collectors, wind and seismic resistance needs to be considered as well as the resultant stress on each of the attachment points. It is important to review the roof structure to ensure strength attachments of the solar collectors (HTP, 2017).

2.5 Types of Structural Reinforcements

Structural strengthening is used to reinforce structures due to deficiency, and to increase an existing element’s capacity to carry new loads, such as sustainable rooftop technologies. As with any structure or method of reinforcement, it is necessary to first identify and establish a good understanding of the existing conditions through a structural condition assessment. The most common techniques to reinforce structural elements are mentioned below and classified into two different categories: passive systems and active systems. When selecting the appropriate strengthening method, it is important to consider the following factors: magnitude of strength increase, size of building and structures, environmental conditions, accessibility, construction, and maintenance and life cycle costs (Shaw, n.d.).

2.5.1 Passive Systems

Passive systems do not introduce any forces to the structure; they contribute to the overall resistance of an element when it deforms. Section enlargement strategies are mostly used to improve strength, stiffness, and to reduce cracks. Some types of section enlargement strategies are: span shortening, externally bonded steel shapes, and epoxy injection (Shaw, n.d.).

Externally bonded fiber reinforced polymer (FRP) reinforcement is a method of reinforcement that involves adhering additional reinforcement to the exterior faces of an element. The success of this strengthening method depends on both the durability and lifespan of the reinforcement material, and the properties of the material used to attach the new reinforcement (usually epoxy material). This method, if adopted correctly and with the appropriate materials, is able to: reduce deflection, increase carrying capacity, increase flexural strength, and increase resistance to shear (Shaw, n.d.).

2.5.2 Active Systems

Active strengthening systems are identified as additional external forces to structural elements, which can increase strength and improve the service performance. Service performance reduces tensile stress and cracking (Alkhrdaji & Thomas, 2017).

A post-tensioning system is an external force method which implements a structural member using high strength cables, bars, and strands. This system usually connects the reinforcement to the existing member at anchor points (typically at the end of the member). The reinforcement is profiled along the span at different locations (Shoultes, 2017).

3.0 METHODOLOGY

This chapter provides an overview of how the project was completed. The chapter provides information on how the buildings were selected, as well as the design and structural considerations for each sustainable technology.

3.1 Identify Buildings for Consideration

The first step of this project involved identifying buildings at WPI for the application of sustainable rooftop technologies. Online research was conducted to create a list of requirements for buildings to have in support of sustainable rooftop technologies. Additionally, an initial list of all 29 buildings at WPI was created with pertinent information on each building. The two lists were compared to identify the buildings, which satisfied the criteria outlined in the list of requirements for supporting sustainable rooftop technologies. Out of the original 29 buildings, 11 buildings were identified for further analysis for solar panel, green roof, or solar collector installation. Eight of the 11 buildings had the ability to support all types of technologies, while the other three buildings had the ability to support only solar panels and solar collectors, since their roofs are sloped and have no flat section for green roofs.

A meeting with WPI Director of Facilities Operations, Bill Spratt, was used to narrow down the list of 11 buildings. After discussions about energy demand and the availability of design drawings, the list was narrowed down to three buildings: Gordon Library, Stoddard B, and the Gateway Parking Garage. Gordon Library was chosen for the installation of a green roof since the rubber rooftop is flat and was recently renovated. A recently renovated roof provides suitable conditions for the installation of a green roof without concern for failure or maintenance of an old roof. Stoddard B was chosen for the installation of solar collectors since it is a residential building and requires hot water supply for the hospitality of its students. Additionally, the building has separately metered energy consumption and water demand values, which allows for the determination of the number of solar collectors to meet the water demand of the entire building. The Gateway Parking Garage was chosen for the installation of solar panels since the electric bill is lower than other buildings, which allows a sufficient number of solar panels to produce energy for the entire parking garage. Like Stoddard B, the Gateway Parking Garage also has a separately metered energy consumption value, which allows for the determination for the number of solar panels to meet the energy demand for the entire parking garage.

3.2 Design and Analysis of Solar Panel Technology on Gateway Parking Garage

For the Gateway Parking Garage, a solar panel technology was chosen based on online research. First, different types of solar panels were researched, followed by research on different manufacturers of solar panels. A model was chosen based on sufficient energy production, allowing a minimal number of panels to produce energy for the entire garage. Additionally, low cost, low weight, and long lifespan were factors when choosing the solar panel manufacturer and model. Determining the cost of different models involved calling the manufacturer for quantitative information about the model.

3.2.1 Layout and Construction Process for Solar Panels on Gateway Parking Garage

Determining the layout of the solar panels involved calculating the number of solar panels needed to meet the energy demand value of the Gateway Parking Garage. The annual energy demand value of the Gateway Parking Garage was given by the WPI Facilities Department. By dividing the annual energy demand value of the garage by the annual energy production value of one solar panel, the number of panels to produce energy for the entire structure was calculated. A rectangular area was chosen for design based on available space on the top level of the Gateway Parking Garage. The solar panels were designed to be a minimum of 10 ft above the garage floor to allow for clearance of vehicles. Additionally, the panels were proposed to be inclined at 10° above the horizontal which is the minimum and recommended angle for the solar panel model, as well as facing south to absorb the maximum amount of sunlight. The construction process for the panels, including safety precautions, module mounting, mounting configurations, and maintenance and cleaning was found on the manufacturer’s website for the chosen solar panel model.

3.2.2 Structural Analyses and Design for Solar Panels on Gateway Parking Garage

After determining the layout and quantity of solar panels, a structural steel framework was designed to support all the solar panels. The initial design for the number of beams, girders, and columns was proposed based on the total solar panel area and existing conditions of the chosen installation area on the top level of the Gateway Parking Garage. Through an iterative process the initial design was changed due to various factors.

3.2.2.1 Solar Panel Load Calculations

The first step of the analysis involved considering all loads acting on the solar panels: dead load, live load, rain load, snow load, wind load, and seismic load. For solar panels, live load and rain load were considered negligible. Due to the 10° angle of the panels, all rain would runoff onto the parking garage floor and no ponding was expected. Live load was neglected since the solar panels are not designed for people to walk and operate on. Calculations for dead load, snow load, wind load, and seismic load are outlined in the sequence of tables below. ASCE 7-10 was used as a reference for these calculations, as well as solar photovoltaic array wind and seismic load documents from the Structural Engineers Association of California (Structural Engineers Association of California, 2012). The calculated design load values were input into the load combination equations outlined in the Step 5 table below. The governing load combination produced the largest load value, which would be used for application when designing the supporting steel framework. All load combination calculations were made in accordance with the Load and Resistance Factor Design (LRFD) method.

Step 1: Dead Load of Solar Panels	
Variable:	**Reference/*Equation*:**
Weight of Panel – lbs.	Obtained from Manufacturer's Website
Number of Panels	Previously Determined Based on Energy Values
Overall Weight of Panels – lbs.	*Weight of Panel * Number of Panels*
Area of Panels – ft^2	Determined Based on Dimensions and Number of Panels
Dead Load – psf	*Overall Weight of Panels/Area of Panels*

Step 2: Snow Load on Solar Panels	
Variable:	**Reference (ASCE 7-10)/*Equation*:**
Thermal Factor (C_t)	Table 7-3
Cold Roof Slope Factor (C_s)	Section 7.4.2 (Fig. 7-2)
i. Roof Slope	Slope of Solar Panels = 10°
Exposure Factor (C_e)	Table 7-2
	0.9
i. Terrain Category	Section 26.7
	Category B
Importance Factor (I_s)	Table 1.5-2
	1.0
i. Risk Category	Table 1.5-1
	Category II
Ground Snow Loads (ρ_g) - psf	Fig. 7-1
	50
Flat Roof Snow Load (ρ_f) - psf	Section 7.3
	$0.7 * C_e * C_t * I_s * \rho_g$
Sloped Roof Snow Load (ρ_s) - psf	Section 7.4
	$C_s * \rho_f$

Step 3a: Wind Load on Solar Panels	
Variable:	**Reference (ASCE 7-10)/*Equation*:**
Risk Category	Table 1.5-1
	Category II
Basic Wind Speed (V) - mph	Fig. 26.5-1A
	120
Wind Directionality Factor (K_d)	Table 26.6-1
	0.85
Exposure Category	Section 26.7
	Category B
Topographic Factor (K_{zt})	Section 26.8
	1.0
Gust Effect Factor (G)	Section 26.9
	0.85
Velocity Pressure Exposure Coefficient (K_z)	Table 29.3-1
	0.85
i. Height above ground level - ft	Height of Gateway Parking Garage
	60
Velocity Pressure (q_z) - psf	Section 29.3.2
	$0.00256 * K_z * K_{zt} * K_d * V^2$

Step 3b: Wind Load on Solar Panels	
Reference (Wind Design for Low-Profile Solar Photovoltaic Arrays on Flat Roofs):	
$A_{pv} \leq h$, therefore A_{pv} =	Lower Value of A_{pv} and h
i. Height of building (h) -ft	Height of Gateway Parking Garage
	60
ii. Width of building on longest side (WL) - ft	Width of Gateway Parking Garage
	268
iii. A_{pv} – ft	*0.5 * SQRT(h * WL)*
Normalized Wind Area (A_n)	$(1000/A_{pv}^2)$ * *Tributary Area of Beam*
i. Tributary area of beam – ft^2	Based on Design
ii. $A_{pv} \geq 15$ ft, therefore A_{pv} =	Greater Value of A_{pv} and 15 ft
Nominal Net Pressure ($(GC_m)_{nom}$)	Average of Two $(GC_m)_{nom}$ Values
i. Panel angle (ω) - °	Solar Panel Angle = 10°
ii. $(GC_m)_{nom}$ for $15° \leq \omega \leq 35°$	Fig. 29.9-1
	1.1
iii. $(GC_m)_{nom}$ for $0° \leq \omega \leq 5°$	Fig. 29.9-1
	0.75
Panel Chord Length Factor (γ_c)	$0.6+(0.06 * l_p)$
i. Chord length of solar panel (l_p) - ft	Width of Solar Panel
	3.275
$\gamma_p \leq 1.3$, therefore γ_p =	Lower Value of γ_p and 1.3
i. Mean parapet height above roof surface (h_{pt}) - ft	Average Height of Solar Panel Structure
	20.384
ii. For $h_{pt} > 4$ ft, Parapet Height Factor (γ_p)	$0.25 * h_{pt}$
Characteristic Height (h_c) – ft	$h_1+(l_p * SIN((\pi/180) * \omega))$
i. Solar panel height above roof at low edge (h_1) - ft	Minimum Height of Solar Panel Structure
	10
ii. $h_1 \leq 1$ ft, therefore h_1 =	Lower Value of h_1 and 1 ft
Array Edge Factor (E)	Fig. 29.9-1
	1.0
i. Horizontal distance from edge of panel to edge of roof (d_x) – ft	1
ii. d_x/h_c =	d_x/h_c
Net Pressure Coefficient (GC_m)	$\gamma_p * E * (GC_m)_{nom} * \gamma_c$
Design Wind Pressure (p) – psf	$q_z * GC_m$

Step 4a: Seismic Load for Solar Panels	
Variable:	**Reference (ASCE 7-10)/*Equation*:**
Risk-Targeted Maximum Considered Earthquake Spectral Response Accelerations (MCE_R) - %g	
i. S_s - %g	Fig. 22-1
	18
ii. S_1 - %g	Fig. 22-2
	7
Soil Classification	Section 20
	Site D
Site Coefficients	
i. F_a	Table 11.4-1
	1.6
ii. F_v	Table 11.4-2
	2.4
Spectral Response Acceleration Parameters	Section 11.4.3
i. S_{MS}	$F_a * S_s$
i. S_{M1}	$F_v * S_1$
Design Spectral Acceleration Parameters	Section 11.4.4
i. S_{DS}	$2/3 * S_{MS}$
ii. S_{D1}	$2/3 * S_{M1}$
Risk Category	Table 1.5-1
	II
Seismic Design Category (SDC)	Table 11.6-1
	B
Seismic Importance Factor (I_e)	Table 1.5-2
	1.0
Seismic Base Shear (V) - psf	Section 15.4.1.2
	$0.3 * S_{DS} * W * I_e$
i. Type of structure	Section 15.4.1.2
	Rigid Nonbuilding Structure
ii. Weight of structure (W) - psf	*2.07*

Step 4b: Seismic Load for Solar Panels	
Variable:	**Reference (ASCE 7-10)/*Equation:***
Fundamental Period (T) – s	Section 12.8.2.1
	$C_t * h_n^x$
i. Type of structural system	Table 12.8-2
	All Other Structural Systems
ii. C_t	Table 12.8-2
	0.02
iii. x	Table 12.8-2
	0.75
iv. Structural height (h_n) - ft	Average Height of Solar Panel Structure
	20.384
Vertical Distribution Factor (C_{vx})	Section 12.8.3
	$(W_x * h_x)^k/(W_i * h_i)^k$
i. k	Section 12.8.3
	1.0
ii. Weight of structure (W_x/W_i) - psf	*2.07*
iii. Structural height (h_x/h_i) - ft	*20.384*
Lateral Seismic Force (F_x) - psf	Section 12.8.3
	$C_{vx} * V$
Horizontal Seismic Load Effect (E_h) - psf	Section 12.4.2.1
	$P * Q_e$ ($Q_e=F_x$)
i. Redundancy Factor (ρ)	Section 12.3.4
	1.0
Vertical Seismic Load Effect (E_v) - psf	Section 12.4.2.2
	$0.2 * S_{DS} * D$

Step 5: LRFD Load Combinations per ASCE 7-10
1.4D
1.2D + 1.6L + 0.5(L_r/S/R)
1.2D + 1.6(L_r/S/R) + (L/0.5W)
1.2D + 1.0W + L + 0.5(L_r/S/R)
1.2D + E_v + 1.0E_h + L + 0.2S
0.9D + 1.0W
0.9D + 1.0E_h

3.2.2.2 Supporting Beam Calculations

The second step of the analysis involved sizing the steel beams supporting the solar panels. The steel beams were sized based on the governing load acting on the beams, as well as the size of the area (tributary area) each beam needs to support. The calculation process was completed twice: once to size the interior beams and once to size the exterior beams. Calculations were made to size structural steel members in accordance with the AISC Manual. The beams were sized based on strength requirements, which included choosing an initial beam size based on the required plastic section modulus, Z_x, and then updating the calculations to include the self-weight of the chosen beam size. This was an iterative process, and the tables below show the calculation process for choosing a beam size. In addition, flange local buckling and web local buckling were checked to ensure no buckling occurs within the chosen beam size.

Step 1: Initial Beam Size	
Variable:	**Reference/*Equation:***
Tributary Width of Beams - ft	Based on Design
w_u - k/ft	*Governing Load * Tributary Width * (k/1000 lb.)*
Length of Beam (L) - ft	Based on Design
Moment (M_u) – k*ft	$(w_u * L^2)/8$
Steel Yield Strength (F_y) - ksi	50 (A992 Steel)
Uncertainty Coefficient (Ø)	0.9
Plastic Section Modulus (Z_x) – in^3	$M_u/(Ø * F_y)$
Select Beam Size $Z_x \geq$ Calculated Z_x	AISC Table 3-2

Step 2: Check Weight of Selected Beam Size	
Variable:	**Reference/*Equation*:**
Selected Beam Weight - lb./ft	AISC Table 3-2
w_u - k/ft	*Step 1 w_u + 1.2 * Beam Weight * (k/1000 lb.)*
Moment (M_u) – k*ft	$(w_u * L^2)/8$
Plastic Section Modulus (Z_x) – in^3	$M_u/(Ø * F_y)$
Check if Calculated $Z_x \leq$ Selected Beam Z_x	AISC Table 3-2

Step 3: Flange Local Buckling	
Variable:	**Reference/*Equation*:**
$b_f/2t_f$	AISC Table 1-1
Young's Modulus (E) - ksi	29,000 (A992 Steel)
Steel Yield Strength (F_y) - ksi	50 (A992 Steel)
Limit Value	*0.38 * SQRT(E/F_y)*
$b_f/2t_f \leq$ Limit Value	

Step 4: Web Local Buckling	
Variable:	**Reference/*Equation*:**
h/t_w	AISC Table 1-1
Young's Modulus (E) - ksi	29,000 (A992 Steel)
Steel Yield Strength (F_y) - ksi	50 (A992 Steel)
Limit Value	*3.76 * SQRT(E/F_y)*
$h/t_w \leq$ Limit Value	

In addition to strength requirement, the steel beam sizes were selected based on serviceability. The selected beam size was checked for total service load and snow deflection. If the selected beam size did not pass these serviceability requirements, then a different beam size was chosen to satisfy serviceability. The deflection limits for serviceability were set based on the requirements in the International Building Code (IBC) which states: a roof beam supporting a plaster ceiling (similar to solar panels) must have a maximum total deflection = L/240, and a maximum snow load deflection = L/360 or 1" (International Building Code, 2014). The tables below show the calculation process for checking the serviceability of the beam size.

Step 5: Total Service Load	
Variable:	**Reference/*Equation*:**
Selected Beam Weight - lb./ft	AISC Table 3-2
w_T - lb./ft	*((DL + SL) * Tributary Width) + Weight of Beam*
Young's Modulus (E) - psi	29,000,000 (A992 Steel)
Moment of Inertia (I_x) – in^4	AISC Table 3-3
Total Deflection - in	*$(5 * w_T * L^4)/(384 * E * I_x)$*
Limit Value - in	*(L * 12 in/ft)/240*
Total Deflection $\leq$ Limit Value	

Step 6: Snow Deflection	
Variable:	**Reference/*Equation*:**
w_s - lb./ft	*SL*Tributary Width*
Young's Modulus (E) – psi	29,000,000 (A992 Steel)
Moment of Inertia (I_x) – in^4	AISC Table 3-3
Snow Deflection – in	$(5 * w_s * L^4)/(384 * E * I_x)$
Limit Value – in	*(L * 12 in/ft)/360 or 1 in*
Snow Deflection ≤ Limit Value	

3.2.2.3 Laterally Unsupported Beams

The next step involved checking the laterally unsupported distance of the beams to see if they needed additional support by adding more girders. This step was completed as an investigation for lateral-torsional buckling within the beam member. This process was completed for both the interior and exterior beam sizes, as well as the two different beam spans: 45.69 ft and 28.21 ft. After analysis, it was concluded that the original unbraced length for the beams that span 45.69 ft was too large and had to be decreased. This required changing the design by adding more girders to support the beams and reduce the unbraced length. The calculation process is outlined in the tables below.

Step 1: Unbraced Length Determination	
Variable:	**Reference:**
Plastic Length (L_p) - ft	AISC Table 3-2
Lateral-Torsional Buckling Moment Unbraced Length (L_r) - ft	AISC Table 3-2
Actual Unbraced Member Length (L_b) - ft	Distance Between Supporting Girders

Step 2: Calculation of Moment Capacity (M_n)	
Variable:	**Reference/*Equation:***
If $L_b \leq L_p < L_r$: Plastic Behavior (Zone 1)	
Moment Capacity (M_n) – k*ft	$F_y * Z_x$
i. Steel Yield Strength (F_y) - ksi	50 (A992 Steel)
ii. Plastic Section Modulus (Z_x) – in^3	AISC Table 3-2
If $L_p < L_b < L_r$: Inelastic Buckling (Zone 2)	
Moment Capacity (M_n) – k*ft	$M_p - (M_p - M_r) * ((L_b - L_p)/(L_r - L_p))$
i. Plastic Strength (M_p) – k*ft	$F_y * Z_x$
ii. Moment Capacity Between Inelastic and Elastic LTB (M_r) – k*ft	$0.7 * F_y * S_x$
iii. Elastic Section Modulus (S_x) – in^3	AISC Table 1-1
If $L_p < L_r \leq L_b$: Elastic Buckling (Zone 3)	
Moment Capacity (M_n) – k*ft	$((C_b*\pi^2*E)/(L_b/r_{ts})^2)*sqrt(1+(0.078*(J_c/(S_x*h_o))*(L_b/r_{ts})^2)*S_x$
i. r_{ts}, J_c, S_x, h_o	AISC Table 1-1
ii. C_b	1
iii. Young's Modulus (E) - ksi	29,000 (A992 Steel)

Step 3: Unbraced Length Check	
Variable:	**Reference/*Equation*:**
$ØM_n$ – k*ft	$0.9*M_n$
Previously Calculated Beam Moment (M_u) – k*ft	$(w_u*L^2)/8$
If $M_u \leq ØM_n$	Adequate Unbraced Length
If $M_u > ØM_n$	Decrease Unbraced Length

3.2.2.4 Supporting Girder Calculations

The calculation process for determining the girder sizes was the same as the process for determining the beam sizes. Strength and serviceability requirements were checked, and all calculations were made with the assistance of the AISC Manual. All girders were initially chosen to be the same size. Later in the design process, the software RISA was used to perform a structural analysis of the steel framework. A smaller moment value than originally calculated was acting on the girder, allowing for a smaller girder size to be chosen. However, one girder size remained the initial size due to its tributary width, which did not satisfy the snow deflection limit.

3.2.2.5 Laterally Unsupported Girders

The calculation process for checking the laterally unbraced length of the girders was the same as the process for checking the laterally unbraced length of the beams. This step was completed as an investigation for lateral-torsional buckling within the girder member. After analysis, it was concluded that the original unbraced length for the girders was too large and had to be decreased. This required changing the design by adding more beams to support the girders and reduce the unbraced length.

3.2.2.6 Supporting Column Calculations

The next step involved determining the supporting steel column sizes. This process was completed with the assistance of the AISC Manual. The size of the column depends on the column's length and the load acting on the column. After analysis, all of the supporting eight columns were sized to be the same. The calculation process for determining the column size is shown in the table below.

Column Size Determination	
Variable:	**Reference:**
Length of Column (L) - ft	Based on Design of Steel Structure
Available Strength of Axial Compression ($Ø_cP_n$) - k	AISC Table 4-1a
Load Acting on Column (P_u) - k	Calculated During Analysis
$Ø_cP_n \geq P_u$	Adequate Column Size

3.2.2.7 Second-Order Elastic Analysis

The next step involved using the structural analysis software, RISA, to determine member forces and lateral sway ΔH for the following LRFD load combination equation for gravity loads:

$$U = 1.2D + 1.6S + 0.5W$$

The horizontal seismic load was also accounted for as the lateral force acting on the steel frame. Dead, snow, and wind loads acting on each column were calculated, as well as the horizontal seismic load. In addition to the given load information, the size of all girders and columns previously calculated were inputted into the software. The design of the frame was checked for stability per Chapter C of *AISC Specification*.

After inputting the appropriate information, the output from the RISA structural analysis was used to perform an approximate second-order analysis to assess the adequacy of the selected column section for the combination of gravity and lateral loads. The approximate second-order analysis was based on Appendix 8 to *AISC Specification*. The calculation process and evaluation for performing an approximate second-order analysis to assess the adequacy of the column size is located in the tables below. This analysis resulted in the use of the interaction equation (AISC Equation H1-1) to check for combined bending and compression of the column member. From the RISA analysis, the moment obtained from the connection of the column and girder was smaller than the moment value used to design the original girder. Therefore, calculations were made to determine a new girder size smaller than the initial girder size.

Step 1: Column Load Effects from RISA Analysis	
Variable:	**Units:**
Factored Axial Force P_{nt} from No-Sway Analysis (Gravity Loads)	k
Factored Axial Force P_{lt} from Sway Analysis (Lateral Loads)	k
Factored Moment M_{nt} from No-Sway Analysis (Gravity Loads)	k*ft
Factored Moment M_{lt} from Sway Analysis (Lateral Loads)	k*ft

Step 2: Lateral Deflection from RISA Analysis	
Variable:	**Units:**
Total Story Shear ΣH	k
Lateral Deflection (drift) for Story ΔH	in

Step 3: Amplifier B_2	
Variable:	**Reference/*Equation*:**
Total Elastic Critical Buckling Load for the Story (P_{estory}) – k where R_m = 0.85 (conservative) L = frame height	$(R_m * \Sigma H * L)/\Delta H$
Total Vertical Load Supported by the Story (P_{story}) – k	Calculated from RISA
Amplifier $B_2 \geq 1$	$1/(1-(P_{story}/P_{estory}))$

Step 4: Amplifier B_1	
Variable:	**Reference/*Equation*:**
Smaller Factored Column End Moment due to Gravity Load (No Sway) Analysis: M_1	Units: k*ft
Larger Factored Column End Moment due to Gravity Load (No Sway) Analysis: M_2	Units: k*ft
Indicate: Single or Reverse Curvature	Single Curvature: + Reverse Curvature: **-**
Cm (+ for Single Curvature; **-** for Reverse Curvature)	$0.6 \pm 0.4(M_1/M_2)$
Required Second-Order Axial Strength (P_r) – k	$P_{nt} + (B_2 * P_{lt})$
Elastic Critical Buckling Load for Column (P_{el}) **-** $K_1 = 1.0$	$(\pi^2 * E * I)/(K_1 * L)^2$
Amplifier $B_1 \geq 1$ ($\alpha = 1.0$ for LRFD)	$C_m/(1- (\alpha * P_r)/P_{el})$

Step 5: Required Second-Order Strength Values	
Variable:	***Equation*:**
Required Second-Order Axial Strength (P_r) **-** k	$P_{nt} + B_2 * P_{lt}$
Required Second-Order Moment Capacity (M_r) – k*ft	$B_1 * M_{nt} + B_2 * M_{lt}$

Step 6: Effective Length Factor K for Moment Frame	
Variable:	**Reference/*Equation*:**
Rotational Resistance at the Top Joint (G_t)	$\sum(I_c/L_c)/\sum(I_g/L_g)$
Rotational Resistance at the Bottom Joint (G_b)	
Effective Length Factor (K_x)	AISC Fig. C-A.7.2. Alignment Chart Sidesway
Modified Effective Length Factor (K^*_x)	*Kx * SQRT(1 + ∑Pleaning/∑Pstability)*
∑Pleaning/∑Pstability = 3.5	

Step 7: Axial Capacity P_c	
Variable:	**Reference/*Equation*:**
Slenderness Ratio (x)	$(K^*_x * L)/r_x$
Slenderness Ratio (y) – $K_y = 1.0$	$(K_y * L)/r_y$
Limit Value	$4.71 * SQRT(E/F_y)$
Governing (K*L)/r ≤ Limit Value	Short to Intermediate Column
Governing (K*L)/r > Limit Value	Long Column
Available Axial Strength ($P_c = Ø_c P_n$)	AISC Table 4-1a
$P_r/P_c \geq 0.2$	AISC Equation H1-1a
$P_r/P_c < 0.2$	AISC Equation H1-1b

Step 8: Bending Moment Capacity & Interaction Equation	
Variable:	**Reference/*Equation*:**
Web Local Buckling	$h/t_w \leq 90.5$
Flange Local Buckling	$b_f/2t_f \leq 9.2$
Lateral Bracing (L_b) - ft	Column Length
Plastic Length (L_p) – ft	AISC Table 3-2
Lateral-Torsional Buckling Moment Unbraced Length (L_r) - ft	AISC Table 3-2
Nominal Flexural Strength (M_n) $L_b \leq L_p$ – k*ft	AISC Equation F2-1
Nominal Flexural Strength (M_n) $L_p \leq L_b \leq L_r$ – k*ft	AISC Equation F2-2
Nominal Flexural Strength (M_n) $L_b > L_r$ – k*ft	AISC Equation F2-3
Available Bending Capacity (M_{cx}) – k*ft	$Ø * M_n$
i. Uncertainty Constant (Ø)	0.9
AISC Equation H1-1a	$P_r/P_c + (8/9) * (M_{rx}/M_{cx})$
AISC Equation H1-1b	$P_r/2P_c + (M_{rx}/M_{cx})$
If AISC Equation H1-1 ≤ 1	Adequate Column Size

3.2.2.8 Baseplate Design

Baseplates were designed to connect each steel column to a 2 ft x 2 ft concrete column at a height of 3.67 ft. Out of the eight steel columns, three of them already have existing supporting concrete columns on the top level of the Gateway Parking Garage. The design proposal involves constructing five more of these concrete columns to provide support for each steel column. The baseplates were designed based on the load and moment acting on the concrete column. The dimensions and thickness of each A36 baseplate was determined. When determining the thickness of the baseplate, the largest load and moment values acting on the concrete columns from the RISA Analysis were chosen for analysis. This provided a minimum baseplate thickness, which would be suitable for each steel and concrete column connection. All calculations are located in the tables below.

Step 1: Footing Area and Minimum Baseplate Area	
Variable:	**Reference/*Equation*:**
Load Acting on Concrete Column (P_u) - k	*Previously Calculated P_u + Column Weight * Column Length*
Moment Acting on Concrete Column (M_u) – k*ft	RISA Analysis
Footing Area (A_2) – in^2	Area of Concrete Column
Minimum Baseplate Area (A_1 min) – in^2	$b_f * d$
i. Column Size b_f, d	AISC Table 1-1
$sqrt(A_2/A_1\ min) \leq 2$	

Step 2: Baseplate Dimensions	
Variable:	**Reference/*Equation*:**
Baseplate Area (A_1) – in^2	$P_u/(Ø_c * 0.85 * f'_c * SQRT(A_2/A_1))$
i. Concrete Strength (f'_c)	Based on Type of Concrete
ii. $Ø_c$	0.65
$A_1 \geq A_1$ min	
Δ - in	$(0.95d - 0.8b_f)/2$
Baseplate Dimension (N) - in	$SQRT(A_1) + \Delta$
Baseplate Dimension (B) - in	A_1/N
$Ø_cP_p$ - k	$Ø_c * 0.85 * f'_c * A_1 * SQRT(A_2/A_1)$
$Ø_cP_p \geq P_u$	

Step 3: Moment-Resisting Baseplate Thickness	
Variable:	**Reference/*Equation*:**
Eccentricity (e) - in	$(M_u * (12\ in/ft))/P_u$
i. Largest Moment from Risa Analysis (M_u) - k*ft	18.8
ii. Largest Axial Load from Risa Analysis (P_u) - k	49.36
Strength at Each Flange Edge of Baseplate (f) - ksi	$(-P_u/A) \pm ((P_u * e * c)/I)$
i. Baseplate Area (A) – in^2	B * N
ii. Variable c - in	$0.5 * N$
iii. Moment of Inertia (I) – in^4	$(1/12) * B * N^3$
Moment to Right at Center of Right Flange (M_u) - k*in	$(f_{CRF} * d * (d/2)) + ((f-f_{CRF}) * d * ((2/3) * d))$
i. Strength (f_{CRF}) - ksi	Strength at Center of Right Flange
ii. Distance (d) - in	Distance from Edge of Baseplate to Center of Right Flange
Minimum Thickness (t) - in	$sqrt((6 * M_u)/(Ø_b * F_y))$
i. Coefficient $Ø_b$	0.9
ii. Yield Strength of Baseplate (F_y) - ksi	36 (A36 Steel)
Average Baseplate Strength (f_p) - ksi	$(min\ f + max\ f)/2$
n - in	$(B - 0.8 * b_f)/2$
Bending Moment in Transverse Direction (M_u) - k*in	$f_p * n * (n/2)$
Bending Moment in Transverse Direction (M_u) < Moment to Right at Center of Right Flange (M_u)	
Choose Baseplate Thickness Greater than Calculated Minimum Thickness (t)	

3.2.2.9 Recalculation of Seismic Load

At this point in the process, the entire supporting steel structure has been designed and the seismic load was recalculated. According to ASCE 7-10, the superimposed weight of the designed structure must be less than 25% of the current structure weight. This check was done to assess the impact of the designed structure to the existing parking garage structure. The weight of the steel structure as well as the weight of the top floor of the Gateway Parking Garage were calculated to verify this weight requirement. Satisfaction of the weight requirement involved using new equations to calculate the new horizontal and vertical seismic loads. This calculation process is outlined in the tables below. These new seismic load values were plugged into the RISA analysis to check for adequacy of the column sizes. Additionally, the new seismic load values were used to check their effect on the original beam and girder design. After analysis, it was concluded that the updated seismic loads do not have a large impact on the steel framework design, and therefore does not need to be changed for the updated seismic changed for the updated seismic loads.

Step 1: Designed Structure Weight ≤ 25% of Current Structure	
Variable:	**Reference/*Equation*:**
Area of Top Floor of Garage – ft^3	*Length * Width * Floor Thickness*
Weight of Top Floor of Garage - k	*Weight of Concrete * Area*
Weight of Selected Beams – lb.	∑ *Weight of Beam * Length of Beam*
Weight of Selected Girders – lb.	∑ *Weight of Girder * Length of Girder*
Weight of Selected Columns – lb.	∑ *Weight of Column * Length of Column*
Combined Weight of Selected Members - k	*(Weight of Beams + Weight of Girders + Weight of Columns) * (k/1000 lb.)*
Combined Weight of Selected Members ≤ 0.25*Weight of Top Floor of Garage	ASCE 7-10 Section 15.3.1

Step 2: Horizontal Seismic Load (F_p) & Vertical Seismic Load (F_v)	
Variable:	**Reference/*Equation*:**
Horizontal Seismic Force (F_p) - psf	$((0.4 * a_p * S_{DS} * W_p)/(R_p/I_p)) * (1+(2 * (z/h)))$
i. Spectral Acceleration (S_{DS})	ASCE 7-10 Section 11.4.1
ii. Component Amplification Factor (a_p)	ASCE 7-10 Table 15.1
iii. Component Importance Factor (I_p)	ASCE 7-10 Section 13.1.3
iv. Component Operating Weight (W_p) - psf	*Combined Weight of Selected Members * (1000 lb./k) * (1/Solar Panel Area) + Solar Panel Dead Load*
v. Component Response Modification Factor (R_p)	ASCE 7-10 Table 13.5-1
vi. Height of Attachment Roof (z) - ft	Height of Gateway Parking Garage
vii. Average Roof Height of Structure with Respect to the Base (h) - ft	Average Height of Solar Panel Structure
Lower Limit - psf	$0.3 * S_{DS} * I_p * W_p$
Upper Limit - psf	$1.6 * S_{DS} * I_p * W_p$
Vertical Seismic Force (F_v) - psf	$0.2 * S_{DS} * W_p$

3.2.2.10 Reinforcement in 2 ft x 2 ft Concrete Columns

The final step involved designing reinforcement in the 2 ft x 2 ft concrete columns, which support the columns of the steel structure. The size and number of reinforcing steel bars depended on the interaction of axial force and bending moment acting on the concrete columns. Additionally, the size of the steel ties that wrap around the reinforcing steel bars was determined based on the geometry of the concrete column, as well as the diameter and spacing of the reinforcing steel bars. After analysis, it was determined that all eight concrete columns require the same type and size of reinforcement. The calculations are outlined in the tables below.

Step 1: Determination and Evaluation of Reinforcement Ratio p_g	
Variable:	**Reference/*Equation:***
Axial Force Acting on Concrete Column (P_u) - k	Risa Analysis
Moment Acting on Concrete Column (M_u) – k*ft	Risa Analysis
K_n Value	$P_u/(Ø * f'_c * A_g)$
R_n Value	$M_u/(Ø * f'_c * A_g * h)$
p_g Value	Concrete Column Strength Interaction Diagram
p_{min} Value	$(3 * SQRT(f'_c))/F_y$
i. Concrete Strength (f'_c) - psi	Depends on Type of Concrete
ii. Steel Yield Strength (F_y) - psi	Depends on Type of Reinforcing Steel
p_{max} Value	$0.85 * B_1 * (f'_c/F_y) * (\varepsilon_u/(\varepsilon_u + 0.004))$
i. B_1 Value	Depends on Type of Concrete
ii. Concrete Strain (ε_u)	Depends on Type of Concrete
$p_{min} < p_g < p_{max}$	

Step 2: Determination of Steel Reinforcement Bars and Steel Ties	
Variable:	**Reference/*Equation*:**
Area of Steel (A_s) – in^2	$p_g * A_g$
i. Gross Area of Concrete (A_g) – in^2	Area of Concrete Column
Diameter of Steel Reinforcement Bars (d_b) - in	$sqrt(A_s/((\pi/4) * N)$
i. Number of Steel Reinforcement Bars (N)	Based on Chosen Design
Diameter of Steel Ties (d_s) - in	$2 * Cover + 2 * d_s + ɣb + d_b = h$
i. Cover = Distance from Concrete Edge to Steel Tie - in	Based on Chosen Design (Typically ≥ 1.5 in)
ii. Distance Between Center of Steel Reinforcement Bars (ɣb) - in	*(ɣ = 0.60) * (b = Length of Concrete Column)*
iii. Width of Concrete Column (h) – in	Based on Concrete Column Width

3.3 Design and Analysis of Green Roof Technology on Gordon Library

The Gordon Library was selected to have a green roof technology. A research of the different types of green roofs was done together with the benefits of each technology. An extensive green roof system was chosen based on the structure of the building, the accessibility to the roof, and due to the system's low maintenance costs.

3.3.1 Layout and Construction Process for Green Roof on Gordon Library

To determine the layout of the roof garden on the Gordon Library, it was necessary to consider the layout of the roof and all the elements that comprise it. A green roof system is easily implemented on flat roofs that have plenty of open space and a sufficient area. Although much of the roof is open, a penthouse structure is located in the middle of the roof. The garden area chosen includes an area surrounding the penthouse, leaving a path for maintenance in the middle of the roof and leaving the edges of the roof open.

3.3.2 Structural Analyses and Design for Green Roofs

After determining the layout and the total area of the green roof, an analysis of the loads and capacity of the columns and slabs of the building was conducted. The analysis included the feasibility to impose an extra load on the roof of the building without causing any structural damage. This also involved investigating the impact to the building's seismic capacity.

3.3.2.1 Green Roof Load Calculations

Similar to the Solar Panels load calculations, Section 3.2.2, an analysis of the loads acting horizontally and vertically on the Gordon Library was conducted. The analysis considered dead load, live load, rain load, snow load, wind load, and seismic load. Roof live loads and rain loads were not neglected for this case because of their significant load value, and they were calculated with reference to ASCE 7-10 and the *International Building Code* (IBC). Calculations for all these loads are shown in the tables below. Values for all equations and factors are also shown in the tables. ASCE 7-10 and IBC were used as a reference for these calculations, as well as the *Massachusetts Building Code*. The governing load combination produced by the loads acting on the system was used to determine if the strength capacity of the columns and the two-way slab was sufficient. All load combination calculations were made in accordance with the Load and Resistance Factor Design (LRFD) method.

Step 1: Dead Load of Building	
Variable:	**Reference/*Equation*:**
Weight of Green Roof– psf.	Obtained from System Selected (Extensive Green Roof)
Overall Weight of Building – lbs.	Determined from Structural and Architectural Drawings of Building (Excel Spreadsheet created to determine weight of each floor)
Weight of Building - psf.	*Overall Weight of Building per floor/ area of floor*
Mechanical/Electrical/Plumbing (MEP) –psf.	Determined from Research and Assumptions
Dead Load - psf	*Sum of all dead loads in pounds per square feet.*

Step 2: Live Load on Gordon Library	
Variable:	**Reference (ASCE 7-10) /*Equation*:**
Estimated area of occupancy per floor based on usage	Table 4-1
Live Load – psf.	*Live load Occupancy Diagram for Building*[1]

[1] Live loads will vary for each floor based on occupancy areas (See Appendix C.1 and C.5 for a detailed representation of live loads in Gordon Library)

Step 3: Snow Load on Roof	
Variable:	**Reference (ASCE 7-10)/*Equation*:**
Thermal Factor (C_t)	Table 7-3 = 1.0
Cold Roof Slope Factor (C_s)	Section 7.4.2 (Fig. 7-2)
Exposure Factor (C_e)	Table 7-2 = 0.9
i. Terrain Category	Section 26.7 = B
Importance Factor (I_s)	Table 1.5-2 = 1.10
i. Risk Category	Table 1.5-1 = III
Ground Snow Loads (ρ_g) - psf	Fig. 7-1 = 50
Flat Roof Snow Load (ρ_f) - psf	Section 7.3
	$\rho_f = 0.7*Ce*Ct*Is*\rho_g$ ρ_f = 34.65 psf

Step 4: Rain Load on Roof	
Variable:	**Reference (FM Global Data Sheets) /*Equation*:**
Minimum rain load - psf.	DS 1-54/ Section 2.5.2.8 =32 psf

Step 5: Wind Load Acting Horizontally on Building	
Variable:	**Reference (ASCE 7-10)/*Equation*:**
Risk Category	Table 1.5-1 = III
Basic Wind Speed (V) - mph	Fig. 26.5-1A/780 CMR 1609 = 134mph
Wind Directionality Factor (K_d)	Table 26.6-1 = 0.85
Exposure Category	Section 26.7 = B
Topographic Factor (K_{zt})	Section 26.8 = 1.0
Gust Effect Factor (G)	Section 26.9 = 0.85
Velocity Pressure Exposure Coefficient (K_z)[2]	Table 29.3-1
i. Height above ground level - ft	Height of Gordon Library from Ground Level = 59.5 ft.
Velocity Pressure (q_z) - psf[3]	Section 29.3.2
	$0.00256*K_z*K_{zt}*K_d*V^2 = 33.13$
Main Wind Frame Resistance System	
Internal Pressure Coefficient (GC_{pi}) [4]	Table 26.11-1 = ± 0.18
External Pressure Coefficient (C_p)	Figure 27.4-1
i. Windward Wall	Cp = 0.8
ii. Leeward Wall (North-South) [5]	Cp = -0.33
iii. Leeward Wall (East-West)	Cp = -0.5
Wind Pressure on Parapets	Section 27.4.5
i. Combined Net Pressure Coefficient (GC_{pn})	+1.5 for windward parapet -1.0 for leeward parapet
ii. Wind Pressure at Parapet	Equation 27.4-4 $p_p = q_p(GC_{pn})$
Design Wind Pressure (p)	Equation 27.4-1 $p = qGC_p - qi(GC_{pi})$
Components and Cladding (C&C)	
External Pressure Coefficient (GCp)	Figure 30.4-1 & Figure 30.4-2 *ASCE 7-10*
i. Zone 4[6]	Figure 30.4-1 *ASCE 7-10*
ii. Zone 5	
iii. Zone 1	Figure 30.4-2 *ASCE 7-10*
iv. Zone 2	
v. Zone 3	
vi. 10 Percent of Least Horizontal Dimension (a)	a = 9.87 ft

[2] Values for K_z vary along the height of the building, see ASCE 7-10, Table 29.3-1 for values at z height.
[3] This velocity pressure value is considered at the top of the parapet of the building. Values at each story level will vary.
[4] Negative values indicate pressure acting away from the building
[5] Values may be linearly interpolated from ASCE 7-10, Figure 27.4-1
[6] Each zone will have a positive and negative value to consider for (GC_p)

The tables presented below (Table 2 and 3) were created to demonstrate the typical values for each zone of the building as mentioned in Chapter 30 of the ASCE 7-10. These negative and positive values for GCp and GCpi were taken from the different tables in the chapter. In addition, these values were used with the wind force at the leeward side of the building to obtain the maximum force that cladding and components of the building can withstand,

Table 2: GCp Values from ASCE 7-10 for Each Zone in Gordon Library

GCp Table (Figures ASCE 7-10)										
AREA (SF)	ZONE 1		ZONE 2		ZONE 3		ZONE 4		ZONE 5	
	+	-	+	-	+	-	+	-	+	-
≤ 10 ft^2	0.3	-1	0.3	-1.8	0.3	-2.8	1	-1.1	1	-1.4
≥ 500 ft^2 walls & ≥ 100 ft^2 roof	0.2	-0.9	0.2	-1.1	0.2	-1.1	0.7	-0.8	0.7	-0.8

Table 3: GCpi Values from ASCE 7-10 for Each Zone in Gordon Library

GCp +/- GCpi Table										
AREA (SF)	ZONE 1		ZONE 2		ZONE 3		ZONE 4		ZONE 5	
	+	-	+	-	+	-	+	-	+	-
≤ 10 ft^2	0.48	-1.18	0.48	-1.98	0.48	-2.98	1.18	-1.28	1.18	-1.58
≥ 500 ft^2 walls & ≥ 100 ft^2 roof	0.38	-1.08	0.38	-1.28	0.38	-1.28	0.88	-0.98	0.88	-0.98

Figure 2 illustrates the wind forces acting on a building with a flat roof, similar to the Gordon Library. For simplicity of calculations for the Main Wind Force Resisting System (MWFRS), it can be assumed that the interior wind forces cancel each other as they have the same value going in opposite directions. This is the influence of factor GCpi in the design of wind pressure equation shown in the table above.

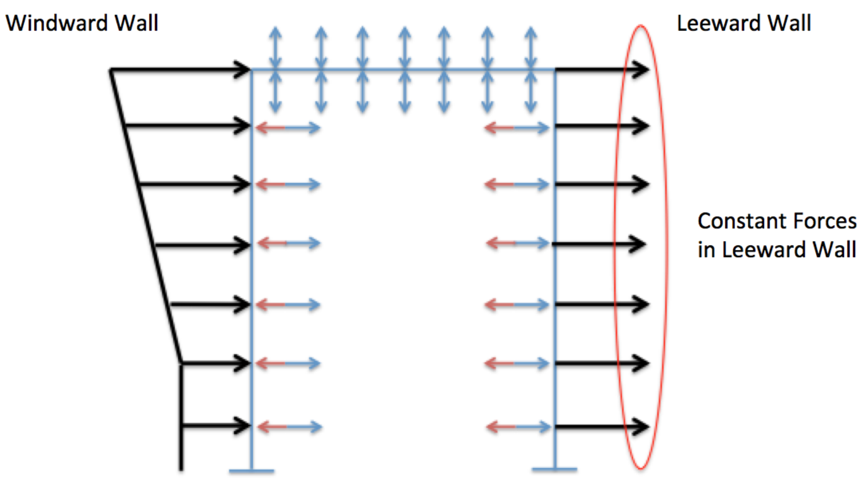

Figure 2: Wind Pressure Diagram for Flat Roof Building

Similarly, Figure 2 above shows the direction of the wind forces and their distribution based on the wall being analyzed. The windward wall, as shown in the figure, has a varying wind force along the height of the building until it reaches a constant wind force at elevations less than 15 feet. The leeward wall has a constant, outward wind force acting along its height. In addition, the weight of the building is different that the actual dead load because the total weight was considered for seismic load purposes as an "effective seismic weight" as illustrated in Step 6 below.

Step 6: Seismic Load Acting Horizontally on Building	
Variable:	**Reference (ASCE 7-10)/*Equation*:**
Risk-Targeted Maximum Considered Earthquake Spectral Response Accelerations (MCE_R) - %g	
i. S_s - %g	Fig. 22-1/ 780 CMR Massachusetts 1609 = 18
ii. S_1 - %g	Fig. 22-2/ 780 CMR Massachusetts 1609 = 7
Soil Classification	Section 20
Site Coefficients	
i. F_a	Table 11.4-1 = 1.6
ii. F_v	Table 11.4-2 = 2.4
Spectral Response Acceleration Parameters	Section 11.4.3
i. S_{MS}	F_a*S_s = 0.29
i. S_{M1}	F_v*S_1 = 0.17
Design Spectral Acceleration Parameters	Section 11.4.4
i. S_{DS}	$2/3*S_{MS}$ = 0.192
ii. S_{D1}	$2/3*S_{M1}$ = 0.112
Risk Category	Table 1.5-1 = III

Seismic Design Category (SDC)	Table 11.6-1 = B
Seismic Importance Factor (I_e)	Table 1.5-2 = 1.25
Effective Weight of Structure (W)[7]	Section 12.7.2
Response Modification Coefficient (R)	Table 12.2-1 = 3
Seismic Base Shear (V) - psf	Section 15.4.1.2
	$Cs*W$
i. Type of structure	Section 15.4.1.2
ii. Seismic Response Coefficient (Cs)	$S_{DS}/(R*I_e)$ = 0.08
Fundamental Period (T) - seconds	Section 12.8.2.1
	$Ct*h_n^x$ =0.63
i. Type of structural system	Table 12.8-2
ii. C_t	Table 12.8-2 = 0.016
iii. x	Table 12.8-2 = 0.9
iv. Structural height (h_n) - ft	Average Height of Building (For Gordon Library mean height is the same as height above ground) = 59.5
Vertical Distribution Factor (C_{vx})	Section 12.8.3
	$(W_x*h_x)^k/(W_i*h_i)^k$
i. k	Section 12.8.3 = 2
Lateral Seismic Story Force (F_x)[8] – psf	Section 12.8.3/Table 4 and 5
	$C_{vx}*V$
Shear Force for each story (V_x)	ΣFi
Horizontal Seismic Load Effect (E_h) - psf	Section 12.4.2.1
	$p*Q_e$
i. Redundancy Factor (ρ)	Section 12.3.4 = 1.0
Vertical Seismic Load Effect (E_v) - psf	Section 12.4.2.2
	$0.2*S_{DS}*D$

Table 4: Values and Base Shear (V) in Gordon Library

Floor	Cvx	Fx (kips)	Vx (kips)
3rd - Roof	0.081	88.99	88.99
2nd - 3rd	0.209	229.40	318.39
1st - 2nd	0.383	420.62	739.01
Ground - 1st	0.327	360.53	1099.54

[7] Effective seismic weight is evaluated for all permanent elements above the level of the slab-on-grade, which turn out to be the ground floor of the Gordon Library. Since the roof snow load is greater than 30 psf, 20 percent of snow load is included as part of the effective seismic weight of the building.
[8] Fx is a story force. It is applied discretely at each story level.

Table 5: Values and Base Shear (V) with Green Roof

Floor	Cvx	Fx (kips)	Vx
3rd - Roof	0.074	92.11	92.11
2nd - 3rd	0.191	237.43	329.54
1st - 2nd	0.349	435.35	764.90
Ground - 1st	0.385	479.25	1244.15

From comparison of Table 4 and Table 5, the base shear values and forces along the building's height differ when a green roof technology is installed on the building. The forces and base shear have a higher value with a green roof because the effective seismic weight of the building increases when implementing the extra weight of the green roof.

Step 7: LRFD Load Combinations per ASCE 7-10
1.4D
1.2D + 1.6L + 0.5S
1.2D + 1.6S + L
1.2D + 1.6S + 0.5W
1.2D + 1.0W + L + 0.5(L_r/S/R)
1.2D + E_v + 1.0E_h + L + 0.2S
0.9D + 1.0W
0.9D + 1.0E_h

3.3.2.2 Factored Design Load of Columns in Gordon Library

The second step of the structural analysis consisted of calculating the factored design load acting on each column of the building. For simplicity purposes, three sections were considered for this analysis. The sections were selected so the calculations could be applied to the rest of the building due to symmetry. Figure 3, represents the sections that were chosen for the building.

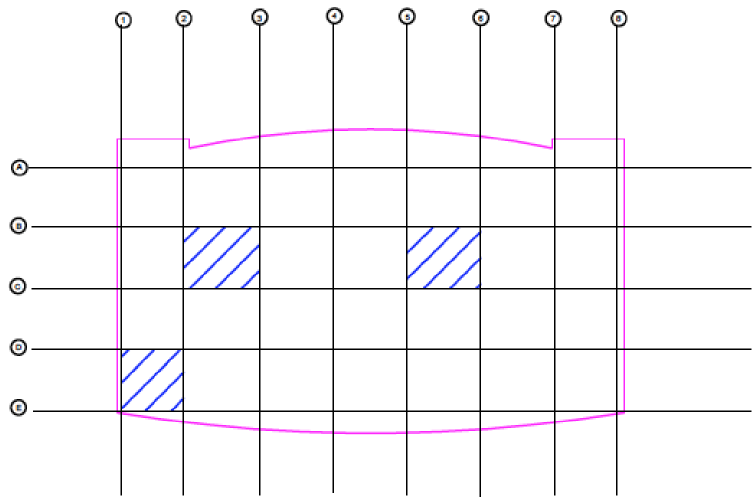

Figure 3: Plan View of Overall Building Sections Analyzed

In addition, the sections included the most critical columns due to the loads acting on the building. A typical section span included four columns arranged as shown in Figure 4.

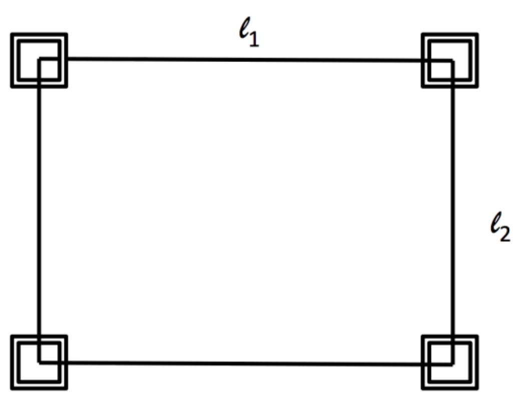

Figure 4: Typical Column Section Gordon Library

The values for ℓ_1 and ℓ_2 varied according to the section being analyzed. These values were either 21'or 25' for ℓ_1 and 20 feet for ℓ_2.

The calculation process for the factored design load (Pu) included all the variables and inputs shown in Table 6. Each column in the building had different factored design load for each floor. The calculation process for (Pu) started by analyzing the first column section in the roof, consequently, the same column section for the third, second and first floor. This analysis had to take into consideration all the loads for each floor and a sum of the loads above each floor. This means that the factored design load for the column section in the first floor had a higher value than the same section in the roof.

Table 6: Factored Design Load (Pu) Inputs

Variables	Description
Tributary Area (ft^2)	Area that a specific column supports
Tributary Area Drop Panel (ft^2)	Area of the drop panel or solid head of column
Dead Load (kips)	Dead load in pounds per square foot for each floor exclusion drop panels
Dead Load Drop Panel (kips)	Based on the dimensions of the drop panel
Dead Load Total (kips)	Sum of the two dead loads above
Live load (kips)	Live load that is acting on the tributary area of the column being analyzed
Snow Load (kips)	A constant load based on the tributary area of the column

3.3.2.3 Axial Load Capacity Calculations

The next step involved calculating the design axial load capacity ΦPn of each column in the building. This was done to determine if the calculated Pu values from the previous step in each column satisfy the condition of:

$$\Phi Pn > Pu$$

The axial load capacity was calculated according to the following formula:

$$\Phi Pn = 0.85\emptyset(0.85f'c\ (Ag - Ast) + Astfy)$$

This is the formula for a reinforced concrete circular column with spiral, where $\emptyset = 0.70$ or 0.75 according to the type of column being analyzed. For other cases where the column has ties, $\emptyset = 0.65$. This calculation included the variables shown in Table 7 below.

Table 7: Data Required to Obtain Factored Design Load

Variables	Symbol
Axial Capacity	Pn
Gross Column Area	Ag
Area of steel bars	Ast
Steel Strength (psi)	fy
Concrete Strength (psi)	f'c
Reduction Factor	ϕ

Analyzing the axial capacity of the columns was the first check to determine if the existing building could support the new superimposed load of the green roof. However, it was necessary to include the analysis of the combined axial and flexural effects in each column of the building to have a complete check.

3.3.2.4 Interaction Diagram Columns (Pn-Mn)

Investigation of combined axial and flexural effects consisted of constructing an interaction diagram for each critical column of the Gordon Library building. The interaction diagram was created as a comprehensive check to determine if the columns of the building could support the superimposed loads and the resulting factored design axial force (Pu) and moment (Mu). Tables 8 and 9 below, present the variables and formulas needed to construct an interaction diagram for one particular column. The columns of the building have a mix of rectangular and circular sections, which means that the shape of the column is rectangular but its reinforcement is circular. For calculation purposes, the column was considered as a circular column. Specific input values were updated based on the column being analyzed. Reinforcement and the dimension of the column were the two variables that typically changed within each floor of the building.

Table 8: Input and Design Summary for Interaction Diagram

Input Data & Design Summary			
Variable	**Symbol**	**Description & Formula**	**Units**
Concrete Strength	f_c'	Based on Structural Drawings =4	*ksi*
Rebar Yield stress	f_y	Based on Structural Drawings =60	*ksi*
Section Size	Ag	Area of Concrete Colum (*b*h)*	*in^2*
Modulus of Elasticity Steel	Es	29,000	*ksi*
Strain Concrete	εc	Max Strain Value = 0.003	-
Diameter of Column	D	Based on Structural Drawings	*in*
Column Vertical Reinforcement	Size	Dowel Size and Quantity	*#*
Spiral Reinforcement	Size	Rebar size for spiral	*#*
Factored Axial load	P_u	Based on Design	*kips*
Factored Magnified Moment	M_u	Based on Design	*ft-kips*
Factored Shear Load	V_u	Based on Design	*kips*

Table 9: Design Summary Formulas and Variables Interaction Diagram

Formula	Symbol	Description
(fy/Es)	εy	Strain Steel
(h)-(cover)-(d. spiral bar) - (d. vertical bar)/2	dt	Distance Vertical bar to edge of concrete
[0.003/(0.003+εy)]*dt	Xb	Location of PNA
β1*Xb	ab	Depth of Whitney Stress Block
arcos[((h/2)-ab)/(h/2)]	α	Compression Block Prop.
((h^2)/2)*[(αrad/2)-(0.25sin2α)]	A	Area of Compression Block
[((h^3)/4) * ((sinα)^3)/3]/A	X	Centroid of Compression Block
0.85*f'c*A	Cc	Compressive Force in Compression Block
#of bars* As*fy	T1	Area of Tension Steel
#of bars* As*Es*εs3	T2	Area of Tension Steel
#of bars*As*(fy-0.85*f'c)	Cs1	Area of Compression Steel
#of bars*As*(Es*εs2-0.85*f'c)	Cs2	Area of Compression Steel

3.3.2.5 Two-Way Dome Slab

After examining the factored design loads and the combined axial and moment capacity for the columns in the building, an analysis of the two-way dome (or waffle) slab was conducted. The process for this calculation was based on the load factors acting on the entire slab of the building. Each floor of the Gordon Library was analyzed to determine the factored design load (Wu) in pounds/feet based on the load combinations. The process is similar to the method used for calculating the factored design load (Pu) for columns. Similar to Table 7 for the factored design load (Pu), the calculation process included all the loads acting on the slab, but rather than using the tributary area, it used the tributary width of the member being analyzed. The typical building sections for analysis of the waffle slab are the same as those for the columns, Figure 4 above. All manual calculations only considered the gravity loads acting on the building.

3.3.2.6 Two-Way Dome Slab Capacity

The final step of the structural analysis of the Gordon Library consisted of calculating the moments and shear strength of the slab. The moments (Mu) and the shear (Vu) at different points within the slab were compared with the actual concrete capacities $\emptyset Mn$ and $\emptyset Vc$ based on the design of the structure. These capacities had to satisfy the following equations:

$$\emptyset Mn > Mu$$

$$\emptyset Vc > Vu$$

In order to determine the two-way dome slab capacity a series of steps were completed. These included determining the drop panel size on the columns, moments at end span and middle columns, moments in the column and middle strip, and the shear strength capacity and load.

3.3.2.7 Determining Drop Panel

In the construction process based on the Concrete Reinforcing Steel Institute (CRSI), the solid heads over the columns are treated as they were drop panels in a conventional flat slab. The use of top and bottom steel bars accounted for the negative and positive moments acting on the slab, and its reinforcement was based on the superimposed loads acting on the slab in each floor. As there were no structural drawings that account for the dimensions of the drop panel for the Gordon Library, the solid heads were calculated using the following specifications.

The solid head shall extend in each direction from the centerline of the column a distance not less than 1/6 the span length center to center, in accordance with the following equation from ACI 318-14.

$$Min. Solid\ Head = \frac{1}{6} l_1 + \frac{1}{6} l_2$$

3.3.2.8 Two-Way Dome Slab Capacity (End Span)

The second step to calculate the capacity of the two-way dome slab consisted of calculating the moments in the column and slab for an end span. The values for the moments for an end span and interior span changed for any building. Similarly, for the Gordon Library the section that consisted of an end span also had different lengths in comparison to a section in the interior of the building. For this reason it was necessary to calculate the moments for an end span and interior span with the change of span length.

Table 10: Moment Distribution within End Span of Column and Slab

Variable	Formula	Description
Total Static Moment (Mo)	$\frac{WuL^2}{8}$	L= length of clear span outside of column supports Wu= total factored load in k/ft including drop panel
Exterior Column (Negative Factored Moment) ACI 13.6.4.2		
Moment (Mu_{EXT})	0.26Mo	Column Strip resists 100% of Mu
Bottom (Positive Factored Moment) ACI 13.6.4.4		
Moment (Mu_{BOT})	0.52Mo	
Column Strip (Mu)	0.60Mu	Column strip resists 60% of Mu
Middle Strip (Mu)	0.40Mu	Middle Strip resists 40% of Mu
Interior Column (Negative Factored Moment) ACI 13.6.4.1		
Moment (Mu_{INT})	0.70Mo	
Column Strip (Mu)	0.75Mu	Column strip resists 75% of Mu
Middle Strip (Mu)	0.25Mu	Middle Strip resists 25% of Mu

Figure 5 below illustrate the moment distribution along the slab of the building. This figure helps illustrate how each moment differs for end span and interior span and for column strips and middle strips. This figure was used together with Table 10 above to determine the respective moment (Mu) values according its location.

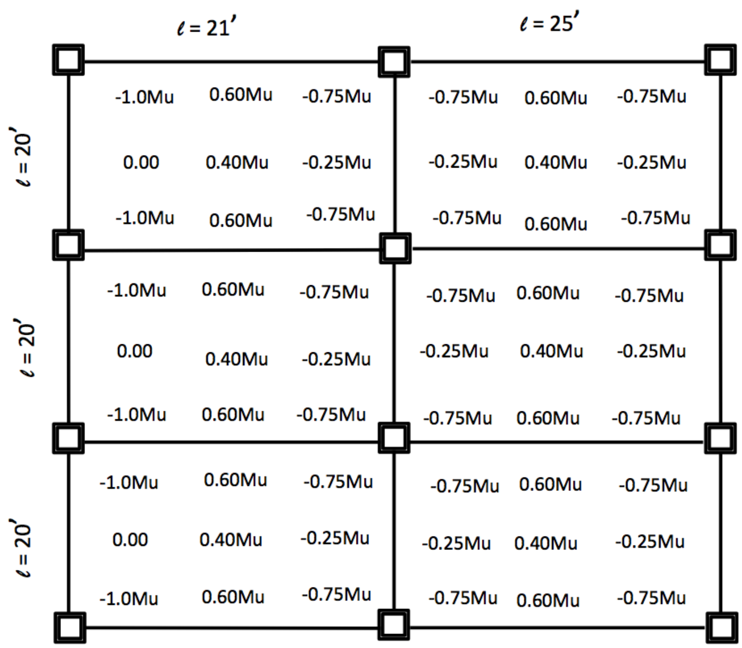

Figure 5: Moment Distribution on Waffle Slab

In the case of determining flexural reinforcement for the slab an additional step was needed. However, as the structural drawings of the Gordon Library already provided details about the reinforcement, this step was not done.

3.3.2.9 Shear Constants & Shear Calculation (End Span)

In order to calculate the factored shear (Vu) of the slab it was necessary to calculate the shear at the exterior column with its appropriate critical section. Figure 6 below illustrates an end span column in the first floor of the Gordon Library and the shear constants needed to obtain the factored shear. The dotted line in Figure 6 represents the critical section of the column to be analyzed.

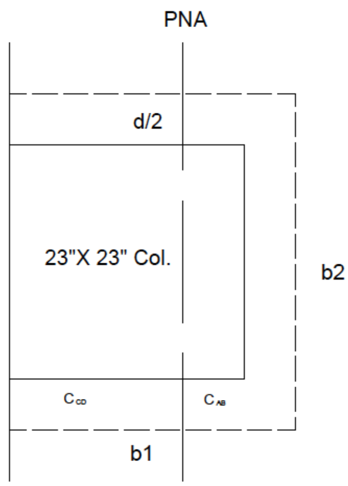

Figure 6: Plan View End Span Column

The following equations were used to calculate the properties of the figure illustrated above.

$$b1 = column\ size + d/2$$

$$b2 = column\ size + d$$

$$b_o = 2 * b1 + b2$$

$$A_c = b_o * d$$

$$Jc = \frac{b_1 d^3}{6} + \frac{2d[(C_{AB})^3 + (C_{CD})^3]}{3} + b_2 d (C_{AB})^2$$

These properties of the column were determined manually, however they can also be obtained from Table 11-5 “Peripheral Shear Constants at Columns” from the CRSI. The table from the CRSI provides enough information regarding the shear constants for a corner, edge and interior column with respective column dimensions and slab/drop panel. See Chapter 6, for an overview of Table 11-5 presented by the CRSI.

Calculating the factored shear (Vu) consisted of solving the equations tabulated in Table 11. Ac and C_{AB} values are the same as previously calculated.

Table 11: Factored Shear (vu) Calculations

Description/Variable	**Formula**	
Shear (Vu)	$\frac{wuL}{2} - \frac{Mu_{int} - Mu_{ext}}{L}$	
Factored Shear (vu)	$\frac{Vu}{Ac} + \frac{\gamma_v MuC_{AB}}{Jc}$	ACI R11.11.7.2
γ_v	(1-γ_f)	ACI Eq. 11-37
γ_f	$\frac{1}{(1+\left(\frac{2}{3}\right)\sqrt{b_1/b_2}}$	ACI Eq. 13-1
Mu	0.30Mo	ACI 13.6.3.6
Shear Check at Exterior Column		
Shear Capacity (vc)	$(\frac{\alpha_s\, d}{b_o} + 2)\sqrt{f'c}$	ACI Eq. 11-32
α_s	30 for edge columns	
Design Moment Strength (ΦMn)		
Area of Steel (As)	# of bars in column strip*Area of bars	
a	$\frac{A_s f_y}{0.85 f'cb}$	
Effective width (b)	Half the width of the panel	
ΦMn	$\phi As f_y(d - \frac{a}{2})$	
Reduction Factor (ϕ)	0.9	
Moment Check at Exterior Column		
ΦMn > 0.26γ_fMo		
$\rho = \frac{As}{bd}$		
$\rho_{max} = 0.011$	ACI 13.5.3.3	
$\rho_{max} > \rho$		

3.3.2.10 Two-Way Dome Slab Capacity (Interior Span)

Table 12: Moment Distribution with Interior Span

Variable	Formula	Description
Total Static Moment (Mo)	$\frac{WuL^2}{8}$	Mo is the same as end span
Panel Moments		
Bottom (Positive Factored Moment) ACI 13.6.4.4		
Moment (Mu_{BOT})	0.35Mo	
Column Strip (Mu)	0.60Mu	Column strip resists 60% of Mu
Middle Strip (Mu)	0.40Mu	Middle Strip resists 40% of Mu
Top (Negative Factored Moment) ACI 13.6.4.1		
Moment (Mu_{TOP})	0.65Mo	
Column Strip (Mu)	0.75Mu	Column strip resists 75% of Mu
Middle Strip (Mu)	0.25Mu	Middle Strip resists 25% of Mu

3.3.2.11 Shear Constants & Shear Calculation (Interior Span)

A similar approach was taken to determine the moments (Mu) and the shear (Vu) in the interior span. Some factors varied in value due to the increase in moment and changes in the properties of the column being analyzed. Table 12 was used to calculate the factors, and the same equations under Figure 6 were used to calculate the properties of the column. However, the neutral axis of the column changed, as shown in Figure 7, therefore the equations were altered due to this change.

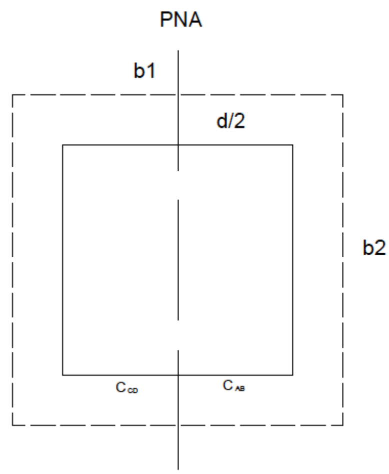

Figure 7: Plan View Interior Span Column

Investigation of the existing two-way dome slab included the analysis of the columns and slab of the first floor of the Gordon Library. This process helped as a guide to determine the capacity of all the columns and slabs in the building, especially the ones in the first floor. In this particular case, only two section of the first floor were analyzed to show its procedure. The section consisted of two edge columns and two interior columns that could fit by symmetry the rest of the floor. As the roof slab has a similar arrange as the slab of the first floor, the slab was not checked to see if it could support the loads of the green roof. It was assumed that the check of the first floor slab would be the most critical of the building. It is important to note that some sections might change depending on the structure of the building.

3.4 Design and Analysis of Solar Evacuated Tubes on Stoddard B

For Stoddard B, a solar evacuated tubes model was chosen based on online research. First, different types of solar panels were researched, followed by research on different manufacturers of this particular system. A model was chosen based on the energy production and comparison to the energy consumption of the building. Other factors considered were cost, number of collectors needed, and weight. The information and cost of the considered models were accessible online.

3.4.1 Layout and Construction Process for Solar Collectors on Stoddard B

Determining the layout of the system involved calculating the number of solar collectors needed to meet the energy demand value of Stoddard B. The annual energy demand value of the building was given by the WPI Facilities Department. The number of panels was calculated by dividing the annual energy demand value of the building by the annual energy production value of one solar panel. The two biggest sides of the building were chosen to place the system based on their individual flat roof and ample space. This system is framed with its mounting system and built into the roof.

The solar collectors need an angle of about 40° above the horizontal, the typical angle range for this collector is between 20° and 80°, and need to face south to absorb the maximum amount of sunlight. The process for constructing this system which include safety precautions, module mounting, mounting configurations, and maintenance and cleaning was obtained from the manufacturer's website for the chosen solar panel model.

3.4.2 Structural Analyses and Design for Solar Collectors on Stoddard B

After determining the layout and quantity of solar collectors, the building was submitted to a structure analysis to investigate its adequacy to support the added weight. The analysis consisted of designing the minimum member's size and reinforcement to support the added load caused by the solar system. If any of the actual members or reinforcement were smaller than the proposed design, then the structure cannot support the new load. Through trial and error, a final design for each of the members would be constructed.

3.4.2.1 Solar Collectors Load Calculations

The first step of the analysis involved considering all loads acting on the solar collectors: dead load, live load, rain load, snow load, wind load, and seismic load. For the collectors, live load and rain load were considered negligible. Due to the angle of the collectors, all rain not absorbed by the collectors would runoff onto the roof and drain so no ponding was expected. Live load was neglected since the collectors are not designed for people to walk and operate on. Calculations for dead load, snow load, wind load, and seismic load are outlined in the sequence of tables below; the methods for these calculations are very similar to the solar panels (Section 3.2.2) since they are both photovoltaic systems. ASCE 7-10 was used as a reference for these calculations, as well as solar photovoltaic array wind and seismic load documents from the Structural Engineers Association of California (Structural Engineers Association of California, 2012). The calculated design load values were input into the load combination equations outlined in Step 5 below. The governing load combination produced the largest load value that would be used for application when designing the structure's members. All load combination calculations were made in accordance with the Load and Resistance Factor Design (LRFD) method.

Step 1: Dead Load of Solar Collectors	
Variable:	**Reference/*Equation*:**
Weight of collectors + Water capacity – lbs.	Obtained from Manufacturer's Website
Number of collectors	Previously Determined Based on Energy Values
Overall Weight of collectors – lbs.	*Weight of collectors * Number of collectors*
Area of collectors – ft^2	Determined Based on Dimensions and Number of collectors
Dead Load - psf	*Overall Weight of collectors/Area of collectors*

Step 2: Snow Load on Solar Collectors	
Variable:	**Reference (ASCE 7-10)/*Equation*:**
Thermal Factor (C_t)	Table 7-3
Exposure Factor (C_e)	Table 7-2
	0.9
i. Terrain Category	Section 26.7
	Category B
Importance Factor (I_s)	Table 1.5-2
	1.10
i. Risk Category	Table 1.5-1
	Category II
Ground Snow Loads (ρ_g) - psf	Fig. 7-1
	50
Flat Roof Snow Load (ρ_f) - psf	Section 7.3
	$0.7 * C_e * C_t * I_s * \rho_g$

Step 3a: Wind Load on Solar Collectors	
Variable:	**Reference (ASCE 7-10)/*Equation*:**
Risk Category	Table 1.5-1
	Category III
Basic Wind Speed (V) – mph	Fig. 26.5-1B
	135
Wind Directionality Factor (K_d)	Table 26.6-1
	0.85
Exposure Category	Section 26.7
	Category B
Topographic Factor (K_{zt})	Section 26.8
	1.0
Gust Effect Factor (G)	Section 26.9
	0.85
Velocity Pressure Exposure Coefficient (K_z)	Table 29.3-1
	0.668
i. Height above ground level - ft	Height of Stoddard B
	26
Velocity Pressure (q_z) - psf	Section 29.3.2
	$0.00256 * K_z * K_{zt} * K_d * V^2$

Step 3b: Wind Load on Solar Collectors	
Reference (Wind Design for Low-Profile Solar Photovoltaic Arrays on Flat Roofs):	
$A_{pv} \leq h$, therefore A_{pv} =	Lower Value of A_{pv} and h
i. Height of building (h) -ft	Height of Stoddard B
	26
ii. Width of building on longest side (WL) - ft	Width of Stoddard B, building with collectors
	48
iii. A_{pv} - ft	*0.5 * SQRT(h * WL)*
Normalized Wind Area (A_n)	$(1000/A_{pv}^2)$ * *Roof area*
i. Tributary area of beam – ft^2	Based on Design
ii. $A_{pv} \geq 15$ ft, therefore A_{pv} =	Greater Value of A_{pv} and 15 ft
Nominal Net Pressure ($(GC_m)_{nom}$)	$(GC_m)_{nom}$ Values
i. Panel angle (ω) - °	Solar Panel Angle = 40°
ii. $(GC_m)_{nom}$ for $15° \leq \omega \leq 35°$	Fig. 29.9-1
	0.3
Panel Chord Length Factor (γ_c)	$0.6+(0.06 * l_p)$
i. Chord length of soalr collectors (l_p) - ft	Width of Solar collectors
	5.79
Characteristic Height (h_c) - ft	$h_1+(l_p * SIN * \omega)$
i. Solar panel height above roof at low edge (h_1) - ft	Minimum Height of Solar Collector
	6.3
ii. $h_1 \leq 1$ ft, therefore h_1 =	Lower Value of h_1 and 1 ft
Array Edge Factor (E)	Fig. 29.9-1
	1.0
i. Horizontal distance from edge of collector to edge of roof (d_x) - ft	3
ii. d_x/h_c =	d_x/h_c
Parapet Height Factor (γ_p) *=1.0* if h_{pt} is less than 4 ft	h_{pt}=0.25(solar collector height above roof)
	1.0
Net Pressure Coefficient (GC_m)	$\gamma_p * E * (GC_m)_{nom} * \gamma_c$
Design Wind Pressure (p) - psf	$q_z * GC_m$

Step 4a: Seismic Load for Solar Collectors	
Variable:	**Reference (ASCE 7-10)/*Equation*:**
Risk-Targeted Maximum Considered Earthquake Spectral Response Accelerations (MCE_R) - %g	
i. S_s - %g	Fig. 22-1
	18
ii. S_1 - %g	Fig. 22-2
	7
Soil Classification	Section 20
	Site D
Site Coefficients	
i. F_a	Table 11.4-1
	1.6
ii. F_v	Table 11.4-2
	2.4
Spectral Response Acceleration Parameters	Section 11.4.3
i. S_{MS}	$F_a * S_s$
i. S_{M1}	$F_v * S_1$
Design Spectral Acceleration Parameters	Section 11.4.4
i. S_{DS}	$2/3 * S_{MS}$
ii. S_{D1}	$2/3 * S_{M1}$
Risk Category	Table 1.5-1
	III
Seismic Design Category (SDC)	Table 11.6-1
	B
Seismic Importance Factor (I_e)	Table 1.5-2
	1.25
Seismic Base Shear (V) - psf	Section 12.81
	$W*C_s$
i. Type of structure	Section 15.4.1.2
	Rigid Nonbuilding Structure
ii. Weight of structure (W) - psf	*12.4*
Response Modification Factor (R)	Table 12.2-1
	3.0
Seismic Response Coefficient (C_s)	Section 12.8.1.1
	$S_{DS}/(R/ I_e)$

Step 4b: Seismic Load for Solar Collector	
C	**Reference (ASCE 7-10)/*Equation:***
Fundamental Period (T) – s	Section 12.8.2.1
	*0.1 * Stories above base*
i. Type of structural system	Table 12.8-2
	All Other Structural Systems
Vertical Distribution Factor (C_{vx})	Section 12.8.3
	$(W_x * h_x)^k/(W_i * h_i)^k$
i. k	Section 12.8.3
	1.0
ii. Weight of structure (W_x/W_i) - psf	*12.4*
iii. Structural height (h_x/h_i) - ft	*26*
Lateral Seismic Force (F_x) - psf	Section 12.8.3
	$C_{vx} * V$
Horizontal Seismic Load Effect (E_h) - psf	Section 12.4.2.1
	$P * Q_e$ $(Q_e=F_x)$
i. Redundancy Factor (ρ)	Section 12.3.4
	1.0
Vertical Seismic Load Effect (E_v) - psf	Section 12.4.2.2
	$0.2 * S_{DS} * D$

Step 5: LRFD Load Combinations per ASCE 7-10
1.4D
1.2D + 1.6L + 0.5(L_r/S/R)
1.2D + 1.6(L_r/S/R) + (L/0.5W)
1.2D + 1.0W + L + 0.5(L_r/S/R)
1.2D + E_v + 1.0E_h + L + 0.2S
0.9D + 1.0W
0.9D + 1.0E_h

3.4.2.2 Slab Calculations and Design

The second step of the analysis involved designing the minimum slab requirements for the new imposed loads plus any loads acting on top of the slab as dead loads. For this procedure, the slab was assumed to be a continuous one-way slab with interior supports. The design of this member included the minimum thickness of the slab as well as its minimum required reinforcement. This step was completed twice, once for the roof slab and another for the first-floor slab. The remaining slabs are assumed to be the same as the first floor since they have smaller loads acting on them making the design of the first-floor slab adequate for their loads. Calculations were made to design the slabs with the *Reinforced Concrete*[9] book that is in accordance with the ACI 318-11 code. Using the ACI code, the self-weight of the slab can be calculated and then it is designed by adding this new weight to all the loads acting on top of the member; the self-weight includes a metal deck, which is a permanent formwork, as well as MEP which weight were estimated after research. The end result of the design consists of the thickness, reinforcement size and spacing, and the maximum allowed moment. The tables below show the calculation process for choosing a thickness and rebar number & spacing. Finally, the moment capacity ΦM_n is calculated and compared to the design moment acting on the slab.

Step 1: Slab Thickness, Constants, and Actual Moment	
Variable:	**Reference/*Equation:***
Spacing between supports (l) - ft	Assumed
Steel Yield Strength (F_y) - ksi	60 (A432 Steel)
Concrete Yield Strengh (F'c) -ksi	3
Thickness (h) - inches	*l/24*
Self Weight (S_w) psf	h(150 psf)
w_u - psf	*Governing Load including self weight and any weight on system*
Design Moment (M_u) – k*ft	$(w_u * L^2)/9$
Steel Ratio design (ρ_{des})	$0.85\beta_1(F'c/ F_y)(\varepsilon_u/\varepsilon_u+0.005)$
Max Steel Ratio(ρ_{max})	$0.75[0.85\beta_1(F'c/ F_y)(87/87+ F_y)]$
β_1	0.85
Minimum steel ratio (ρ_{min})	0.0018

[9] MacGregor, James, and James Wight. *Reinforced Concrete: Mechanics and Design*. Fourth ed., 2005.

Step 2a: Trial Design	
Variable:	**Reference/*Equation:***
Uncertainty Coefficient Φ	0.9
Cover- inches	*0.75*
Unit width (b)- inches	12
Depth (d_{design})- inches	*h-cover-0.25*
Whitney's stress block (a assumed)- inches	1
Area of steel (A_s) per unit width@ max moment	$M_u / \Phi F_y(d_{design}-a/2)$

Once step two is done, the reinforcement can be chosen from table A-9 [10] (Reinforced Concrete, 2005). With the new area of steel, step two is repeated utilizing this new information to get the actual design specification. Following step two, the actual steel ratio is calculated and checked to be sure it is within parameters. In the final step, the shear and moment capacities are calculated and compared to the design load values V_u and M_u.

Step 2b: Final Design	
Variable:	**Reference/*Equation:***
Actual Depth (d)-inches	*h-cover-half bar diameter*
Design Steel Ratio (ρ)	A_s/bd
Unit Width (b)- inches	12
Actual Whitney's stress block (a)- inches	$A_s F_y./0.85*F'c*b$

Step 3: Shear & Moment	
Variable:	**Reference/*Equation:***
Shear Capacity (ΦV) -kips	Φ in shear=0.75 Φ*(SQRT(F'c))*b*d
Design Shear (V_u) -kips	1.15(W_u*l)(1/2)+dW_u
Moment Capacity (ΦM_n) –kip*ft	$\Phi A_s*F_y*(d-a/2)$

[10] Areas of Bars in a Section 1ft Wide, Annex 9

3.4.2.3 Beam Calculations and Design

The third step of the analysis is designing the member beneath the slab; in this case the beams. Much like the slab, the proposed design is the minimum requirements for the beam to support the new loads created by the solar collectors. The calculations for this design were conducted following the steps in the book (Reinforced Concrete, 2005). The beams were estimated as best as possible since no dimensions were provided in the drawings. The design of these members resulted in the required reinforcement and the allowed moment. This procedure was done once for the 1st floor and all other beams are assumed to be the same. The first floor's loading exceeds that of the roof with the solar collectors, making this design adequate for all other floors. For this design, an initial assumption is made that the steel stress is equal to the yield stress. If this assumption is correct, the steel ratio is less than the balanced steel ratio having no need to check it. The tables below show the calculation steps in order to choose reinforcement and calculate the allowed moment.

Step 1: Known Values and Constants	
Variable:	**Reference/*Equation:***
Length (l) – ft	Assumed
Steel Yield Strength (F_y) - ksi	60 (A432 Steel)
Concrete Strength (F'c) -ksi	3
Area (b x h) –in^2	*(8 x 10)=80*
Self-Weight (S_w) psf	Area(l)(150 psf)
Factored Load (W_u)- psf	*Governing Load including self-weight and any acting on system*
Allowed area of steel (A_s)- in^2	*Same as the slab= 0.7*
β_1	0.85
Modulus of elasticity of steel (E_s) -psi	29,000,000

Following step one, a bar size was chosen for reinforcement. Given the area of the bar, different values can be calculated in order to make sure that the initial assumption is correct. The assumption is acceptable if the net tensile strain in the reinforcement is larger than its yield strain. If the net tensile strain is equal or larger than 0.005, then the beam is tension controlled and the uncertainty coefficient is equal to 0.9.

Step 2: Assumption Check	
Variable:	**Reference/*Equation:***
Design Depth (d)-inches	*h-cover-half bar diameter*
Design Whitney's Stress Block (a)- inches	A_s*Fy/0.85*F'c*b
C	a/ β_1
Net tensile strain (ε_t)	0.003[(d-c)/c]
Yield strain in tension (ε_y)	F_y/E_s
Uncertainty Coefficient Φ	0.9

Step 3: Moment Check	
Variable:	**Reference/*Equation:***
Design Moment (M_u) –kip*ft	(W_u*l^2)/8
Moment Capacity (ΦM_n) –kip*ft	ΦA_s*F_y*(d-a/2)

3.4.2.4 Column Calculations and Design

The fourth and final step for the structural analysis was to design the minimum size and reinforcement for the columns. Similar to the slab and beams, this process was conducted following the column chapter in the book (Reinforced Concrete, 2005). The cross sections of the columns were measured using a measuring tape and the height was given in the drawings. Different columns in the first floor were measured to be more accurate (all measured columns had the same area). With these known values and the imposed load, an adequate design can be proposed. The analysis starts by calculating the imposed load acting on the columns, like all the other members. For this design, the member is assumed to be governed by axial forces since the lateral force resisting system was assumed to be shear walls. The axial load depends on the tributary area of each column resulting in three types, each with a different tributary area. The column that was analyzed for design purposes is the column with the biggest tributary area, given that this one will have the largest axial load. All other columns are assumed to have the same reinforcement. The design of the column was completed following the steps below.

Step 1: Known Values and Constants	
Variable:	**Reference/*Equation:***
Height of columns (H) - ft	8.67
Steel Yield Strength (F_y) - ksi	60 (A432 Steel)
Concrete Strength (F'c) -ksi	3
Area of entire column (b x h) (A_g) –in^2	*(12 x 12)=144*
Self-Weight (S_w) psf	Area(l)(150 psf)
Factored Load (W_u)- psf	*Governing Load including self-weight and any load acting on system*
Largest tributary area –ft^2	*15.67 x 15.67*
Uncertainty Coefficient Φ	0.65
Tie size	Bar # 3

Step 2: Point Load and Steel Area	
Variable:	**Reference/*Equation:***
Design Axial Load (P_u)- kips	W_u *Tributary Area
Axial Load Capacity (ΦP_n)	$0.80\Phi(A_sF_y+0.85*F'c*(A_g-A_s)$
Area of Steel (A_s) –in^2	$[P_u/(0.80\Phi*(F_y-0.85*F'c))]–[(0.85*F'c*A_g)/(F_y-0.85*F'c)]$
Steel Ratio (ρ)	A_s/A_g
Allowed Steel Ratio (ρ)	1-2%

Step 3: Tie Spacing	
Variable:	**Reference/*Equation:***
Spacing of ties is equal to smallest number of the following equations	16*bar diameter
	48*tie diameter
	Smallest column dimension

3.5 Economic Analysis

This section contains information on the economic analysis to determine whether it is feasible to implement the chosen sustainable rooftop technologies. The simple payback period will be evaluated by calculating the total installation cost, as well as the net annual energy savings of the sustainable rooftop technology. This evaluation will result in a recommendation to WPI on if they should invest in the proposed designs on the three structures. When determining the total installation cost of the technology, the unit cost values for labor, material, and equipment were considered using the *Building Construction Costs* source created by R.S. Means Company. This section outlines the calculation process to perform the economic analysis for each sustainable rooftop technology.

3.5.1 Economic Analysis of Solar Panels on the Gateway Parking Garage

When determining the overall installation cost of the proposed solar panel design elevated above the Gateway Parking Garage, many factors were accounted for. These factors included total cost of the steel framework, added 2 ft x 2 ft concrete columns, reinforcement within the concrete columns, and solar panels. The total cost for each factor was added together to produce the overall cost of the proposed solar panel design. This value was compared to the annual energy demand cost of the Gateway Parking Garage to determine how many years it would take to pay off the solar panel design and begin making a profit. These were compared since the chosen number of solar panels can produce the annual energy demand of the Gateway Parking Garage.

3.5.1.1 Total Cost of Steel Framework

The steel framework total cost was determined by first calculating the total weight of the steel members. The overall weight of the miscellaneous items in the framework, which includes steel, plates, studs, and connections was estimated by taking 10% of the total steel member weight (R.S. Means Company, 2017). This calculation process is shown in the tables below.

Step 1: Total Weight of Steel Members	
Variable:	**Reference/*Equation:***
Member Weight – lb./ft	Based on Chosen Member Size
Member Length – ft	Based on Structural Layout
Member Quantity	Based on Structural Layout
*∑Member Weight*Member Length*Member Quantity*(tons/2000 lb.) - tons*	

Step 2: Total Weight of Miscellaneous Items	
Variable:	**Reference/*Equation:***
Overall Weight of Miscellaneous Items - tons	RS Means Building Construction Costs
i. Steel	*10%*Total Weight of Steel Members*
ii. Plates	
iii. Studs	
iv. Connections	

Once the weight of the steel framework was determined, the total cost was calculated using the construction cost data and equation shown in the table below. The costs include unit cost values for labor, materials, and equipment. Labor rate accounts for the workers constructing and installing the steel framework, material rate accounts for the steel members and miscellaneous items, and equipment rate accounts for the tools used to construct the steel framework (R.S. Means Company, 2017). These rates are represented in $/ton.

Step 3: Total Cost of Steel Framework	
Variable:	**Reference/*Equation:***
Labor Unit Cost - $/ton	RS Means Building Construction Costs
i. Steel Members	400
ii. Misc. Steel/Plates/Studs/Connections	400
Material Unit Cost - $/ton	RS Means Building Construction Costs
i. Steel Members	3,000
ii. Misc. Steel/Plates/Studs/Connections	3,400
Equipment Unit Cost - $/ton	RS Means Building Construction Costs
i. Steel Members	200
ii. Misc. Steel/Plates/Studs/Connections	200
(Total Steel Member Weight + Total Miscellaneous Weight)[Labor Unit Cost + Material Unit Cost + Equipment Unit Cost] - $*	

3.5.1.2 Total Cost of Added 2 ft x 2 ft Concrete Columns

As a portion of the solar panel design, five 2 ft x 2 ft concrete columns were proposed to support the columns of the steel framework. The total construction cost of the added concrete columns was calculated using the unit costs and equation shown in the table below. The cost elements accounted for are the same as for the steel framework (labor, material, and equipment);

however, each has different values and are represented for 24” x 24” cast-in-place concrete columns (R.S. Means Company, 2017).

Total Cost of Added 24” x 24” Concrete Columns	
Variable:	**Reference/*Equation*:**
Number of Added Concrete Columns	5
Labor Unit Cost - $	RS Means Building Construction Costs
	400
Material Unit Cost - $	RS Means Building Construction Costs
	241
Equipment Unit Cost - $	RS Means Building Construction Costs
	32
Number of Added Concrete Columns[Labor Unit Cost + Material Unit Cost + Equipment Unit Cost] - $*	

3.5.1.3 Total Cost of Reinforcement Within Concrete Columns

Proposed reinforcement within the concrete columns included 6 #9 steel bars. This was proposed within all eight concrete columns supporting the steel framework columns, including the three concrete columns that already exist. Since no structural drawings were provided for the Gateway Parking Garage, it was assumed that 6 #9 steel rebar does not exist within the three existing concrete columns. Therefore, the total cost for the reinforcement included all eight concrete columns. The first step required determining the material cost, labor cost, and equipment cost of the #9 steel rebar, which is outlined in the Step 1 table below (R.S. Means Company, 2017). The second step required calculating the total cost of the steel rebar, which is outlined in the Step 2 table below.

Step 1: Material Cost, Labor Cost, and Equipment Cost of #9 Steel Rebar	
Variable:	**Reference/*Equation*:**
Material Unit Cost - $	RS Means Building Construction Costs
	64.50
Labor Unit Cost - $	RS Means Building Construction Costs
	60.50
Equipment Unit Cost - $	RS Means Building Construction Costs
	15.35

Step 2: Total Cost of Steel Rebar	
Variable:	**Reference/*Equation*:**
Number of Concrete Columns	8
Number of #9 Steel Rebar per Column	6
*Number of Concrete Columns*Number of #9 Steel Rebar per Column*[Material Unit Cost + Labor Unit Cost + Equipment Unit Cost] - $*	

3.5.1.4 Total Cost of Solar Panels

The total cost of solar panels was based on the chosen SPR-P17-350-COM model from the manufacturer SunPower. The total cost was calculated in the table below based on the unit cost of the technology and unit installation cost for the Northeast region of the United States provided by SunPower (SunPower Corporation, 2017).

Total Solar Panel Cost	
Variable:	**Reference/*Equation*:**
Number of Solar Panels	272
Cost of Solar Panels - $	*Unit Technology Cost*Number of Solar Panels*
i. Unit Cost of Technology ($/panel)	SunPower Corporation
	635.50
Unit Installation Cost ($)	*Panel Energy Production*Unit Installation Cost*Number of Solar Panels*
i. Panel Energy Production (watt/panel)	SunPower Corporation
	350
ii. Unit Installation Cost ($/watt)	SunPower Corporation
	4.00
Cost of Solar Panels + Unit Installation Cost - $	

3.5.1.5 Simple Payback Period of the Proposed Solar Panel Design

The construction costs of the steel framework, added concrete columns, steel rebar, and solar panels were summed together to produce the overall cost of the proposed solar panel design. The next step involved determining the net annual energy savings by installing solar panels on the Gateway Parking Garage. This was calculated in the table below using the annual energy demand and energy operational cost values obtained from the WPI Facilities Department, as well as the total annual solar panel energy production value. For this design, the total annual solar panel energy production is greater than or equal to the annual energy demand of the

Gateway Parking Garage, therefore, the total annual solar panel energy production is multiplied by the energy operational cost.

Step 1: Net Annual Energy Savings	
Variable:	**Reference/*Equation:***
Annual Energy Demand - kWh	WPI Facilities Department
	137,207
Energy Operational Cost - $/kWh	WPI Facilities Department
	0.14
Total Annual Panel Energy Production - kWh	*Number of Solar Panels*Annual Panel Energy Production*
i. Number of Solar Panels	272
ii. Annual Panel Energy Production – kWh/panel	511
*Energy Operational Cost*Total Annual Panel Energy Production - $*	

The next step involved determining the number of years to pay off the installation of the proposed solar panel design. The annual operational cost and lifespan of the solar panels, provided by SunPower, were multiplied together and added to the overall installation cost of the proposed solar panel design. This was calculated to give the total installation cost of the solar panel design over a 25-year span (the lifespan of the solar panels). Dividing this value by the net annual energy savings of the Gateway Garage would produce the simple payback period of the solar panel system. This calculation process is outlined in the table below.

Step 2: Simple Payback Period of the Proposed Solar Panel Design	
Variable:	**Reference/*Equation:***
Annual Operational Cost of Solar Panels - $	SunPower Corporation
	2,593
Lifespan of Solar Panels - years	SunPower Corporation
	25
*(Overall Cost of Solar Panel System + Annual Operational Cost*Lifespan)/(Net Annual Energy Savings) - years*	

3.5.2 Economic Analysis of Green Roof on Gordon Library

The costs of installation and materials for the green roof had a more simplistic approach in comparison to the solar panels on Gateway Garage. The main factors that were included in this economic analysis were:

- Total cost of green roof per square foot (includes labor and material)
- Annual maintenance of green roof per square foot
- Energy reduction savings per square foot for implementing a green roof technology on Gordon Library (annually)
- Stormwater net savings per square foot (annually)

The total cost of all these factors was added together to determine the overall cost of the proposed green roof on the Gordon Library and the possible savings on an annual basis. In addition, the added cost of implementing a roof garden was compared to the costs of installing a conventional roof.

3.5.2.1 Cost of Green Roof Technology

The cost of the green roof on consisted of calculating the area of the technology on the roof of the Gordon Library. Not all the roof was used for this technology, therefore the cost of installation and maintenance of the green roof depended solely on its total area. The calculation for this cost consisted of multiplying the gross area of the green roof times the cost per square foot. For the calculation of the annual maintenance of the green roof, a similar approach was taken. It consisted of multiplying the annual maintenance cost per square foot times the gross area of the green roof.

The results of these calculations are shown in Chapter 8, Economic Analysis of Sustainable Roofing Technologies.

3.5.2.2 Energy Savings of Green Roof Installation on Gordon Library

Part of the economic analysis of the green roofs consisted on calculating the total savings of installing this technology on the building. The savings included the overall energy reduction due to the benefits of roof gardens and stormwater savings. These costs were estimated using a national average consumption for college buildings, as specific information regarding energy consumption of the Gordon Library was not obtained.

3.5.3 Economic Analysis of Solar Collectors on Stoddard B

When determining the overall installation cost of the proposed solar collector design on Stoddard B, a simple approach was employed. The result of this analysis is the number of years the solar collectors will take to pay itself off. This was completed by estimating the fixed cost of the proposed systems as well as the amount of money it will save WPI per year in the future.

3.5.3.1 Fixed Cost

The first step in the analysis was to determine the fixed cost, which are costs that do not change with an increase or decrease in the amount of goods or services provided. In this case, the fixed cost referred to the price of technology and its installation cost. The table below provides the steps to determine this.

Step 1: Fixed Cost	
Variable:	**Reference/*Equation*:**
Cost of Solar Panels - $	*Unit Technology Cost*Number of Solar Panels*
Installation Cost ($)	*Collector installation cost per hour*(time required for installation) + Tank installation cost*
Cost of Solar Panels + Installation cost -$	

3.5.3.2 Annual Savings

To determine the annual savings, a number of variables that change with time were considered. These variables are the current spending of WPI in water heating, the annual savings provided by the collectors, and any maintenance fee. The WPI facilities departments and the technology's webpage provided this information. The table below shows the procedure to calculate these variables.

Step 2: Annual Savings	
Variable:	**Reference/*Equation:***
Annual Total Savings of collectors	Number of Collectors*Savings per panel
Annual Maintenance Cost- $	Obtained from Website
Annual Savings – Maintenance Cost -$	

3.5.3.3 Payback Period

The calculation of the amount of years the technology will take to pay itself off is fairly simple. The annual savings from the technology is divided from the cost of technology plus its installation, or fixed cost, as seen in the equation below.

$$Payback\ Period = \frac{Fixed\ Cost}{Annual\ Savings}$$

CHAPTER 4: IDENTIFY BUILDINGS FOR CONSIDERATION

Through online research, a list of requirements for buildings to have for supporting sustainable rooftop technologies was created. This list was categorized into the following sections: age of building, exposure to sun, slope of roof, and existing sustainable rooftop technology. A description of each category and its corresponding requirement for sustainable rooftop installation is located in Table 13.

Additionally, an initial list of all 29 buildings at WPI was created. The list contains the following information related to each building: type of building, year constructed, number of stories, trees or buildings blocking south side of roof, type of roof, and existing sustainable rooftop technology. The number of stories does not include the basement because the basement does not affect the elevation of the building above ground. Solar panels and solar collectors must be angled facing south, therefore sloped roofs facing south or flat roofs are sufficient for the installation of these technologies. Green roofs can only be constructed on flat roofs. A list of the buildings at WPI and their respective information is located in Table 14.

By comparing Table 13 and Table 14, 11 buildings were identified, out of the initial 29 buildings, for further analysis for solar panel, green roof, or solar collector installation. The 11 buildings are identified and highlighted in Table 14. Eight out of the 11 buildings have the ability to support solar panels, green roofs, and solar collectors. Three out of the 11 buildings have the ability to support only solar panels and solar collectors, since their roofs are sloped and have no flat section for green roofs. The next step involved bringing the list of 11 buildings to WPI Facilities Department for further examination and analysis.

Table 13: List of Requirements for Buildings to have for Supporting Sustainable Rooftop Technologies

Category	Description	Requirement
1) Age of Building	Depending on the material, roofs typically last anywhere from 20-50 years before maintenance needs to occur.	For maintenance purposes, the building must have been constructed within the last 50 years (1967).
2) Exposure to Sun	In order for solar panels, green roofs, and solar collectors to produce the most energy, they need to have the greatest exposure to the sunlight. Green roofs also need exposure to rainfall.	Physical Observation: make sure there are no surrounding trees or buildings which block the roof's exposure to the sunlight (south side of roof).
		The building must be greater than two stories tall. This does not include if there is a basement.
3) Slope of Roof	According to the geographic location of Worcester, MA, solar panels and solar collectors have the greatest exposure to sunlight when they are faced south. Green roofs can only be placed on a flat roof; solar panels and solar collectors can be placed on a sloped or flat roof.	The roof of the building must be flat.
		If the roof of the building is sloped, there must be a sloped portion facing south.
4) Existing Sustainable Roofing Technology	Sustainable roofing technology includes a roof which contains any type of solar panel, green roof, or solar collector system.	The building must not already have an existing sustainable roofing technology.

Table 14: Initial List of Buildings at WPI

Building	Type of Building	Year Constructed	Number of Stories (Not Including Basement)	Trees or Buildings Blocking South Side of Roof	Type of Roof	Sustainable/ Type
Gateway Park I	Academic	2007	4	No	Flat	No
Gateway Park II	Academic	2013	4	No	Flat	No
Alden Hall	Academic	1940	Varies (>2)	No	Slope - Not Facing South	No
Atwater Kent	Academic	1907	3	No	Slope/Flat	No
Bartlett Center	Administration	2006	3	No	Slope - Facing South	No
Boynton Hall	Administration	1868	3	No	Slope - Facing South	No
Daniels Hall	Residential	1963	4	No	Flat	No
East Hall	Residential	2008	5	No	Flat	Green Roof
Ellsworth Apartments	Residential	1972	2	No	Slope - Facing South	No
Faraday Hall	Residential	2013	4	No	Flat	No
Founders Hall	Residential	1984	4	No	Slope - Facing South	No
Fuller Laboratories	Academic	1990	4	No	Flat	No
Gateway Parking Garage	Parking	2010	5	No	Flat	No
Gordon Library	Academic	1967	4	No	Flat	No
Goddard Hall	Academic	1965	4	No	Flat	No
Harrington	Athletic	1968	3	No	Slope - Facing South	No
Higgins Laboratories	Academic	1941	3	No	Slope/Flat	No
Higgins House	Administration	1923	2	No	Slope - Facing South	No
Institute Hall	Residential	1989	3	No	Flat	No
Kaven Hall	Academic	1954	2	No	Slope - Facing South	No
Morgan Hall	Residential	1958	4	No	Flat	No
Olin Hall	Academic	1958	2	No	Slope/Flat	No
Project Center	Academic	1902	2	Yes (Building)	Slope - Not Facing South	No
Rubin Campus Center	Academic	2001	2	Yes (Building)	Flat	No
Salisbury Laboratories	Academic	1889	4	No	Flat	No
Sanford Riley Hall	Residential	1926	4	No	Slope - Facing South	No
Sports & Recreational Center	Recreational	2012	4	No	Flat	Solar Collectors
Stoddard Complex	Residential	1969	3	No	Flat	No
Stratton Hall	Academic	1894	4	No	Flat	No
Washburn Shops	Academic	1868	3	No	Slope/Flat	No

Key:
Meets Requirement - Building Chosen for Solar Panels, Green Roof, and Solar Collectors
Meets Requirement - Building Chosen for Solar Panels and Solar Collectors Only
Meets Requirement - Building Not Chosen
Does Not Meet Requirement

A meeting was scheduled with the Director of Facilities Operations, Bill Spratt, on Thursday, October 26th, 2017. The beginning of the meeting involved describing the objectives and goals of the project. The list of 11 buildings chosen for further consideration was then given to Mr. Spratt. After discussion about energy consumption and available design drawings, the list was narrowed down to three buildings: Gordon Library, Stoddard B, and the Gateway Parking Garage. Additionally, it was discovered that Washburn Shops is the powerhouse which produces and distributes energy to all of the buildings on the WPI campus. Because of this, there is an overall energy consumption value for the campus, and only certain buildings have a separately metered energy consumption value.

Gordon Library was chosen for the installation of a green roof since the rubber rooftop is flat and was recently renovated. Stoddard B was chosen for the installation of solar collectors on its flat stone rooftop. Stoddard B provides an application for the installation of solar collectors since it is a residential building and requires hot water supply for the hospitality of its students. Additionally, this building has a separately metered energy consumption value. The Gateway Parking Garage was chosen by Mr. Spratt for the installation of solar panels. It was chosen since the electric bill is lower than other buildings, which allows a sufficient number of solar panels to produce energy for the entire parking garage. For this application, the solar panels would be elevated above the top level of the parking garage, slanted at an angle facing south. Since the Gateway Parking Garage is not on the main campus, there is a separately metered energy consumption value for this structure.

To conclude, Mr. Spratt informed us that roofs require maintenance every 25-30 years, and he said that the cost of energy consumption is $0.14 per kW. This cost value will be helpful for performing an economic evaluation of each building; calculating the current cost of energy for the building and determining how much money the sustainable rooftop technology would save over time. Finally, Mr. Spratt said he would be able to provide us with design drawings for each of the considered buildings.

CHAPTER 5: SOLAR PANEL INSTALLATION ON GATEWAY PARKING GARAGE

This chapter contains information on the specific type of solar panel technology chosen for the Gateway Parking Garage. The technology was chosen based on ease of installation, energy production, and cost. Additionally, this chapter contains information about the rooftop layout and construction process for the installation of solar panels. Pertinent information includes the number of solar panels, dimensions of the technology, as well as the specific location on the roof where the technology should be installed. Finally, this chapter contains associated structural analyses and design information for the steel frame supporting the solar panels above the Gateway Parking Garage.

5.1 Selected Solar Panel Technology on Gateway Parking Garage

For the application of solar panels on the Gateway Parking Garage, polycrystalline silicon solar panels were chosen because they are a more economic option than monocrystalline solar panels for larger scaled applications (Battalia, et. al., 2016). SunPower is a manufacturer of solar panel technologies which has been leading global solar innovation since 1985. After researching their products, the SPR-P17-350-COM model was chosen for the application of polycrystalline silicon solar panels. This model minimizes white space between solar cells, eliminates reflective metal lines on the cells, and lowers electrical resistance between cells which increases efficiency (SunPower Corporation, 2017). Additionally, each panel produces a large amount of power, approximately 350 W, which is beneficial for the application on the Gateway Parking Garage (SunPower Corporation, 2017). Our plan was to design a solar panel system which was elevated above the top level of the parking garage. This involved designing a framework of steel columns and beams to support the solar panels. Based on a previous SunPower solar panel application on a parking garage, the panels will be installed directly next to each other, angled facing south, on top of the designed steel structure. Table 15 below contains information on the type, size, weight, energy production, lifespan, and costs of the chosen SunPower polycrystalline silicon solar panel.

Table 15: Type of Solar Panel Technology Information (SunPower Corporation, 2017)

Building	Gateway Parking Garage
Sustainable Rooftop Technology	Polycrystalline Silicon Solar Panel
Manufacturer	SunPower
Model	SPR-P17-350-COM
Panel Dimensions	81.4" x 39.3" x 1.8"
Panel Gross Area	22.25 ft^2
Panel Weight	51 lbs.
Panel Energy Production	350 W
Estimated Panel Lifespan	25 years
Unit Cost of Technology[11]	$635.50/panel
Unit Installation Cost	$4.00/Watt
Annual Unit Operational/Maintenance Cost	$9.53/panel

5.2 Layout and Construction Process for Solar Panels on Gateway Parking Garage

This section contains information on the layout and construction process for the installation of solar panels on the Gateway Parking Garage.

5.2.1 Layout on Gateway Parking Garage

To determine the layout of the solar panels, the number of solar panels to produce the energy consumption demand of the Gateway Parking Garage was calculated. Given by WPI Facilities Department, the annual energy consumption of Gateway Parking Garage is 137,207 kWh. The chosen SunPower polycrystalline silicon solar panel produces 511 kWh of energy per year. By dividing the annual energy consumption demand of Gateway Parking Garage by the annual energy production capacity of one solar panel, it was determined that at least 269 panels would be needed to meet the energy consumption demand of the Gateway Parking Garage. Table 16 contains information on energy, cost, number of panels, and total area of panels. To produce a distributed rectangular area, 272 solar panels were proposed for design. Excess energy produced can be distributed to other surrounding buildings, or sold to a local electrical company. All panels are angled at 10° above the horizontal and facing south. The angle of 10° above the

[11] Unit Cost of Technology does not include installation cost; Unit Cost of Technology only accounts for the panel.

horizontal was chosen since this value is the minimum and recommended angle for the chosen SunPower solar panel model. The panels will be elevated on top of a framework of steel columns, beams, and girders. Figure 8 below displays an overhead visual of the top level of the Gateway Parking Garage with the proposed solar panel location, and Figure 9 displays an overhead visual of the solar panel layout.

Table 16: Installation of Solar Panels on Gateway Parking Garage Information

GATEWAY PARKING GARAGE	Current	Installation of SunPower Polycrystalline Silicon Solar Panel
Annual Energy Demand/Production	137,207 kWh	511 kWh/panel
Annual Cost of Energy Paid/Saved	$19,209	$71.54/panel
Number of Panels	272 panels	
Total Area of Panels	6,052 ft^2	

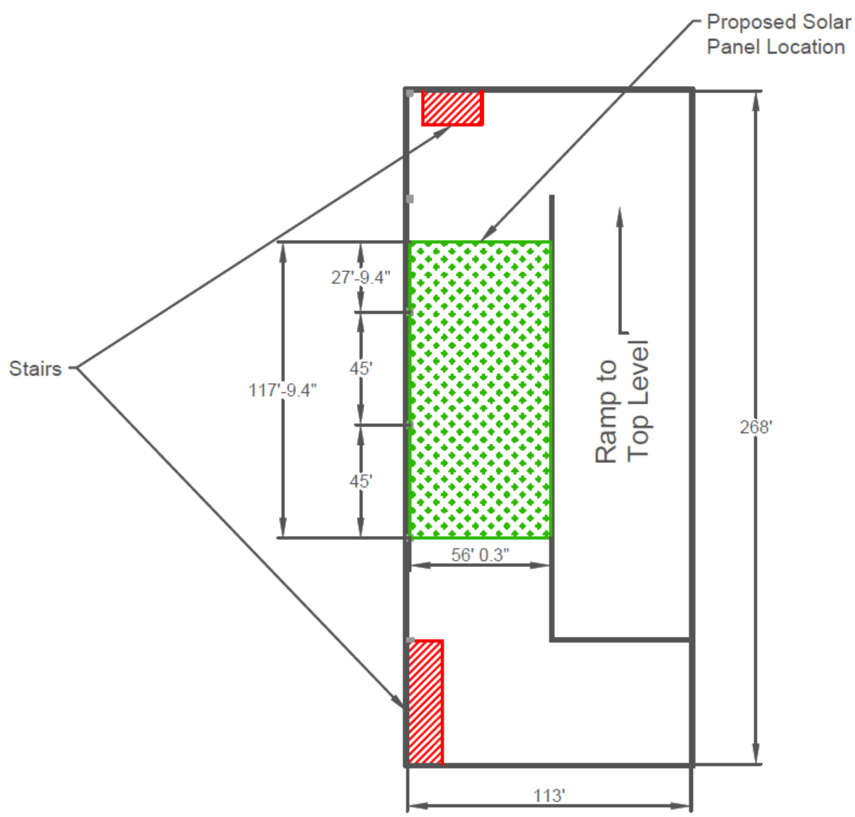

Figure 8: Plan View of Top Level of Gateway Parking Garage with Proposed Solar Panel Location

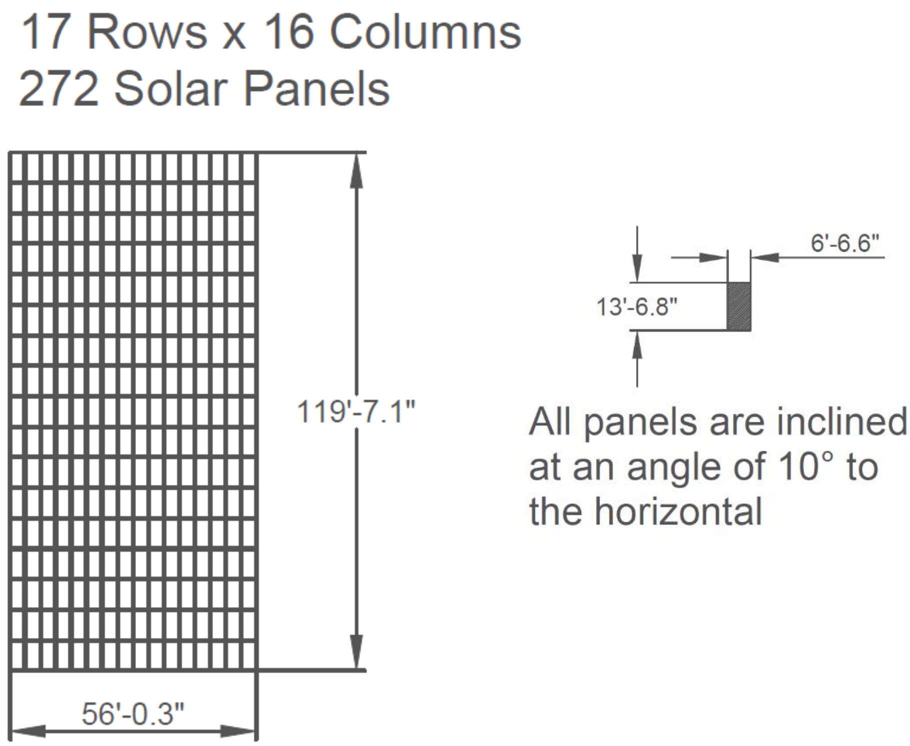

Figure 9: Plan View of Solar Panel Layout

5.2.2 Construction Process for SunPower Polycrystalline Silicon Solar Panels

Table 17 contains information on the construction process for SunPower Polycrystalline Silicon Solar Panels. This information from the manufacturer includes safety precautions, module mounting, mounting configurations, and maintenance and cleaning. Figure 10 below displays the orientation, dimensions, and mounting location of the selected SunPower solar panel model. Figure 11 below shows the clamp force location and how the clamp force must be applied. These figures aided when designing the layout of the solar panel configuration by providing dimensions and the orientation of the solar panel model.

Table 17: Construction Process for SunPower Polycrystalline Silicon Solar Panels (SunPower Corporation, 2016)

Safety Precautions	Module Mounting	Mounting Configurations	Maintenance and Cleaning
1) Installations should be performed in compliance with the National Electrical Code (NEC) and local codes	1) For 96 cell solar panel model on Gateway Parking Garage: a. Silver frame type b. Pressure clamps	1) Panels must be installed in landscape orientation at a minimum angle of 10° above the horizontal	1) Trained SunPower support personnel should inspect all modules annually
		2) Minimum of 4" of clearance between the module frames and the structure	
2) Installation of panels should be performed by qualified personnel	2) Load ratings: a. Wind load = 2400 Pa b. Snow load = 5400 Pa c. Cyclonic wind load = 7500 Pa	3) Required clearance between installed modules is a minimum of 1/4" distance	2) Periodic cleaning of module glass results in improved performance levels
	3) Mounting location should be 398 mm from the edge of panel (Figure 10)	4) Clamp force location is located in Figure 11	

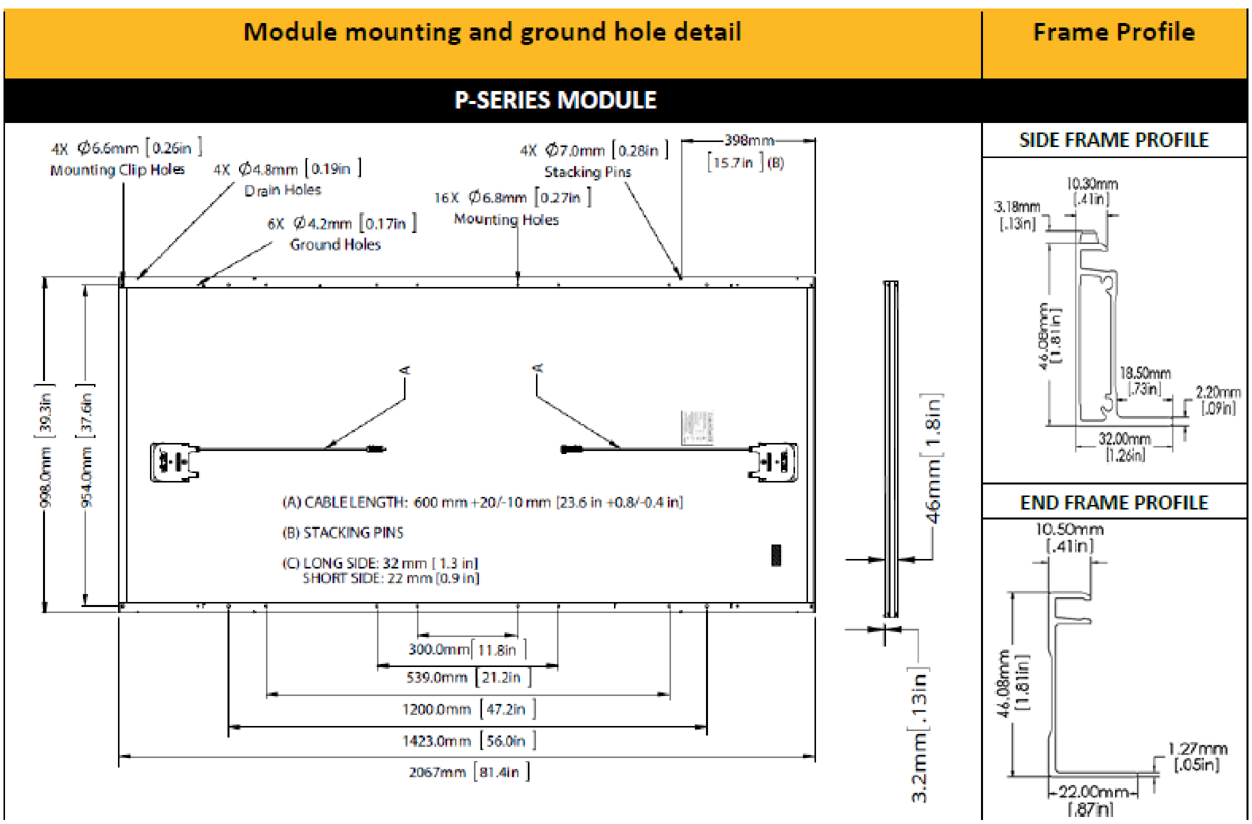

Figure 10: Selected SunPower Module Design (SunPower Corporation, 2016)

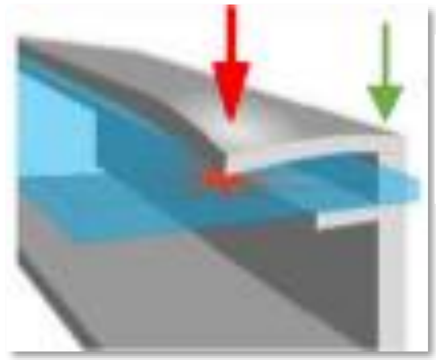

Figure 11: Clamp Force Locations (SunPower Corporation, 2016)

5.3 Structural Analyses and Design for Solar Panels on Gateway Parking Garage

After determining the layout and quantity of solar panels, a structural steel framework was designed to support all 272 solar panels. A plan view of the original framework design is shown below in Figure 12. For this original proposal, Table 18 contains information on the number of steel members to support the panels. After continuing the structural analysis, this initial design was changed due to various factors. The original proposed design is also shown on Page 1 of Appendix B.1.

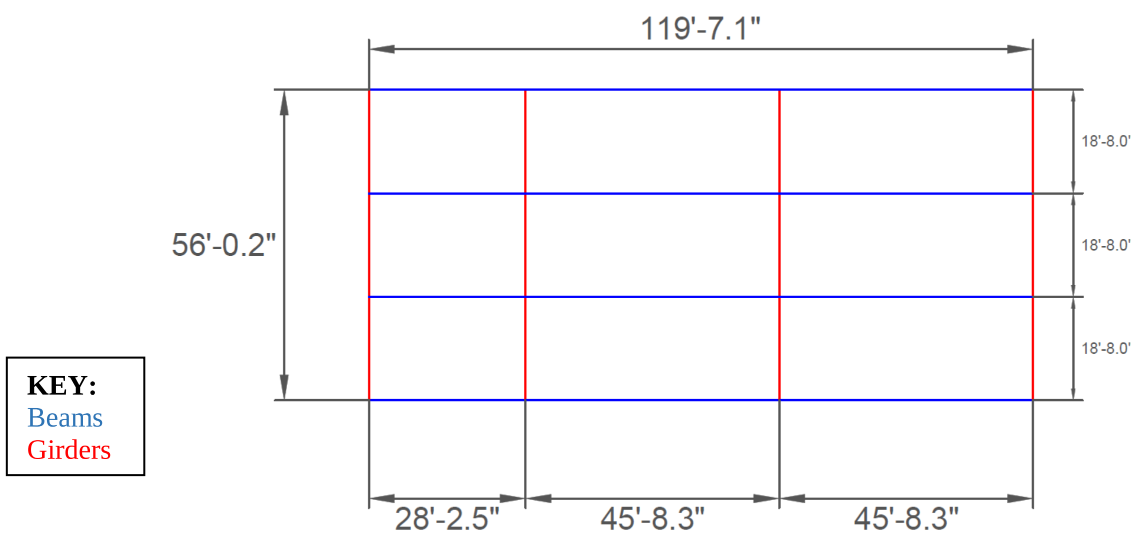

Figure 12: Plan View of Original Design Proposal at 10° Angle

Table 18: Number of Steel Members for Original Proposal Design

Member Type	Quantity
Beam	12
Girder	4
Column	8

5.3.1 Solar Panel Load Calculations

The first step in our analysis involved considering all loads acting on the solar panels: dead load, live load, rain load, snow load, wind load, and seismic load. For solar panels, live load and rain load were considered negligible. Due to the 10° angle of the panels, all rain would runoff onto the parking garage floor and no ponding was expected. Live load was neglected since the solar panels were not designed for people to walk and operate on. Dead load was calculated by using the weight, area, and number of panels. Finally, snow load, wind load, and seismic load were calculated in accordance with the ASCE 7-10. In addition to the ASCE 7-10, wind and seismic load were calculated in accordance with documents from the Structural Engineers Association of California, which provided information specifically to solar photovoltaic arrays (Structural Engineers Association of California, 2012). All calculated design loads are shown below in Table 19, and all calculations can be found in Appendix B.1. Provided in the ASCE 7-10, there are seven load combinations which were considered when determining the governing load acting on the panels. These combinations accounted for both gravity and lateral loads. All load combination calculations were made in accordance with the Load and Resistance Factor Design (LRFD) method. The calculated load combinations are displayed in Table 20.

Table 19: Calculated Design Loads Acting on Solar Panels

Loads	Value[12] (psf)
Dead (D)	2.07
Snow (S)	28.98
Wind (W)	25.62
Seismic Horizontal (E_h)	0.12
Seismic Vertical (E_v)	0.079
Roof Live (L_r)	0
Live (L)	0
Rain (R)	0

Table 20: Calculated LRFD Load Combinations per ASCE 7-10

Load Combinations		Value[12] (psf)
Gravity Loads	1.4D	2.9
	1.2D + 1.6L + 0.5(L/S/R)	16.97
	1.2D + 1.6(L_r/S/R) + (L/0.5W)	61.61
	1.2D + 1.0W + L + 0.5(L_r/S/R)	42.48
	1.2D + E_v + L + 0.2S	8.36
Lateral Loads	0.9D + 1.0W	1.86
	0.9D + 1.0E_h	1.98

5.3.2 Supporting Beam Calculations

Different beam sizes were calculated for the exterior and interior beams due to different supporting tributary widths. Steel beam sizes were determined by checking for strength and serviceability requirements. Serviceability included both total deflection and snow deflection. The total deflection limit and snow deflection limit are based on the International Building Code (IBC) which states: a roof beam supporting a plaster ceiling (similar to solar panels) must have a maximum total deflection = L/240, and a maximum snow load deflection = L/360 or 1” (International Building Code, 2014). All calculations were made with the assistance of the AISC Manual. Table 21 shows the process for determining the final member sizes, and Appendix B.2 shows all supporting calculations for both the exterior and interior beam sizes.

[12] Pounds per square foot (psf) refers to the total area of the solar panels at the 10° angle above the horizontal

Table 21: Beam Member Sizes

Beam Size			Span Length (ft)	Required Plastic Section Modulus Z_x (in^3)	Available Plastic Section Modulus Z_x (in^3)	Total Deflection = L/240 (in)	Deflection Limit (in)	Snow Defection (in)	Snow Deflection Limit = L/360 or 1" (in)	Commentary
Exterior Beam	1.	W14 x 26	45.69	42.4	40.2	-	-	-	-	Failed to Support Self-Weight (Strength)
	2.	W14 x 30		44.7	47.3	3.72	2.28	-	-	Failed Deflection Performance (Serviceability)
	3.	***W24 x 55***		***46.8***	***134***	***0.86***	***2.28***	***0.68***	***1***	***Satisfied Strength and Serviceability***
Interior Beam	1.	W21 x 44	45.69	83.7	95.4	2.5	2.28	-	-	Failed Deflection Performance (Serviceability)
	2.	***W24 x 68***		***89.4***	***177***	***1.2***	***2.28***	***1***	***1***	***Satisfied Strength and Serviceability***

5.3.3 Laterally Unsupported Beams

Figure 13 is an updated plan view of the original design; it has been updated to include member sizes. As displayed, the beams have a laterally unsupported distance of 45.69 ft or 28.21 ft, which also represents the spacing of the supporting girders. With the concern for lateral-torsional buckling, the next step involved checking to see if the selected beam sizes could withstand this laterally unbraced length L_b without failing. It was determined that the unbraced length L_b = 45.69 ft was too large for the design loads, and therefore needed to be decreased. Additional lateral support to the beams was proposed by adding girders to reduce the unbraced length L_b. Table 22 shows the process for determining appropriate unbraced length L_b values for the W24 x 55 and W24 x 68 beams. Figure 14 shows the new design of the steel members. All supporting calculations are shown in Appendix B.3.

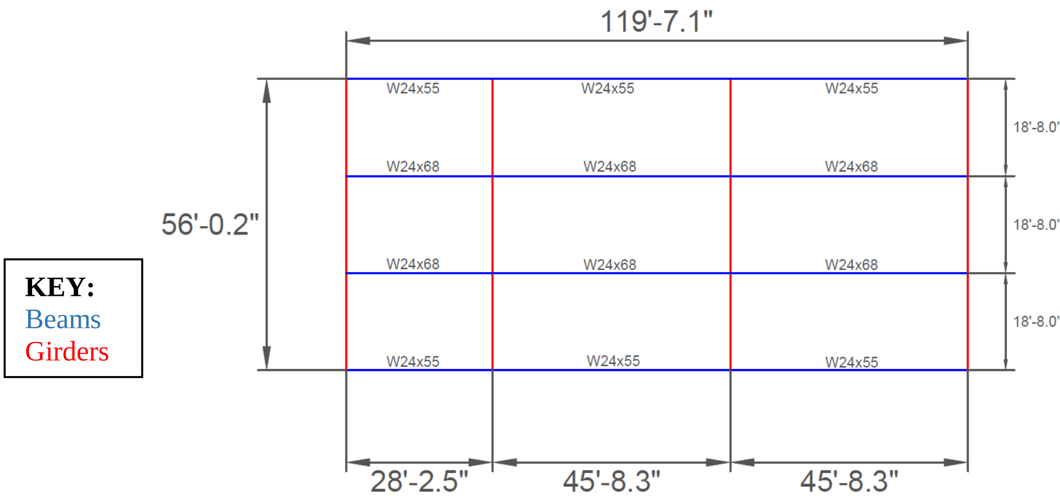

Figure 13: Plan View of Original Design with Beam Sizes

Table 22: Determination of Revised Beam Layout Based on Design for Lateral-Torsional Buckling

Member Size	Length of Member (ft)	Unbraced Length (ft)	Zone	Beam Moment M_u (k-ft)	Moment Capacity $ØM_n$ (k-ft)	Commentary
W24 x 55 (Exterior Beams)	45.69	45.69	Elastic Buckling (Zone 3)	175.437	51.38	Exceeds Moment Capacity
		15.23	Elastic Buckling (Zone 3)		256	***Unbraced Length Satisfies Moment Capacity***
	28.21	***28.21***	Elastic Buckling (Zone 3)	66.85	96.93	***Unbraced Length Satisfies Moment Capacity***
W24 x 68 (Interior Beams)	45.69	45.69	Elastic Buckling (Zone 3)	335.217	106.16	Exceeds Moment Capacity
		15.23	Inelastic Buckling (Zone 2)		482.1	***Unbraced Length Satisfies Moment Capacity***
	28.21	***28.21***	Elastic Buckling (Zone 3)	127.83	209.7	***Unbraced Length Satisfies Moment Capacity***

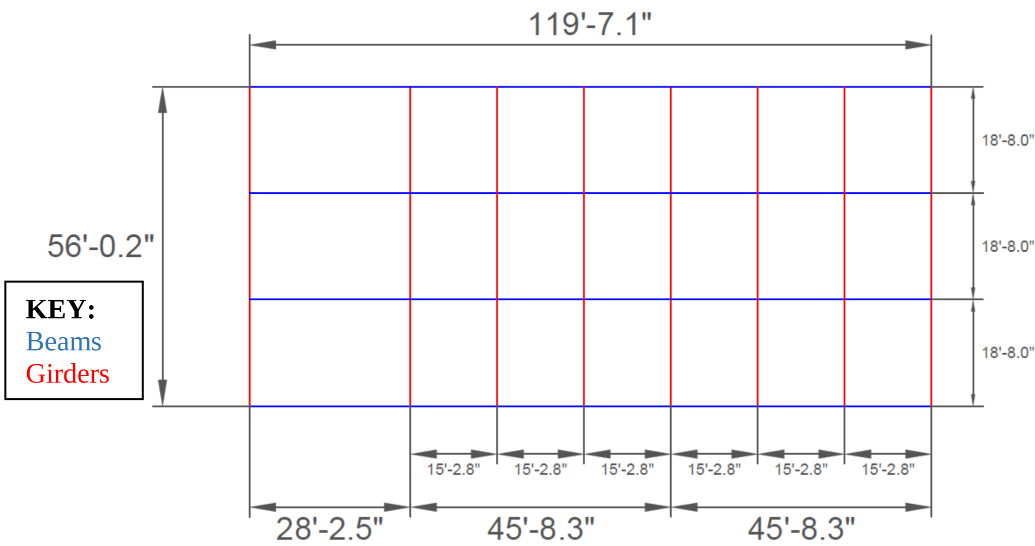

Figure 14: Plan View of Revised Design to Address Lateral-Torsional Buckling in Beams

5.3.4 Supporting Girder Calculations

Similar to the beam calculations, a girder size was calculated for all girders within the frame. Steel girder sizes were determined by checking for strength and serviceability requirements. Serviceability included both total deflection and snow deflection, with limits the same as the beam analysis. All calculations were made with the assistance of the AISC Manual.

Later in the design process, the software Risa was used to perform a structural analysis of the steel framework. It was determined that a smaller moment value than originally calculated was acting on the girder, allowing for a smaller girder size to be chosen. However, one girder remained the initial size of W30 x 108 since its tributary width did not satisfy the snow deflection limit. Displayed in Appendix B.8, a new lighter girder size was calculated, despite the one girder with the larger tributary width. Table 23 shows the process for determining the final member size, and Appendix B.4 shows all supporting calculations for the initial supporting steel girder size.

Table 23: Girder Member Sizes

Girder Size		Span Length (ft)	Required Plastic Section Modulus Z_x (in^3)	Available Plastic Section Modulus Z_x (in^3)	Total Deflection (in)	Deflection Limit = L/240 (in)	Snow Deflection (in)	Snow Deflection Limit = L/360 or 1" (in)	Commentary
1.	W21 x 68	56.02	154.7	160	7.79	2.8	-	-	Failed Deflection Performance (Serviceability)
2.	W30 x 108	56.02	159.7	346	2.66	2.8	Tributary Width ≤ 15.23 ft: 0.75	1	Smaller Size can be Selected Based on Smaller Moment from Risa Analysis (Appendix B.8)
							Tributary Width = 21.72 ft: 1	1	
3.	***W30 x 90***	***56.02***	***141.2***	***283***	***2.68***	***2.8***	Tributary Width ≤ 15.23 ft: ***0.93***	***1***	***Satisfied Strength and Serviceability***
							Tributary Width = 21.72 ft: 1.3	1	Failed Snow Deflection: Remains ***W30 x 108***

5.3.5 Laterally Unsupported Girders

The current design is shown below in Figure 15, which remains the same as the previous design shown in Figure 14, however, now has labeled girder sizes. As displayed, the girders have a laterally unsupported length L_b of 18.67 ft, which also represents the spacing of the beams. With concern for lateral-torsional buckling, the next step involved checking to see if the selected girder size could sustain this unbraced length without failing. It was determined that the unbraced length L_b = 18.67 ft was too large for the design loads, and therefore needed to be decreased. Table 24 shows the process for determining appropriate unbraced length L_b for the initial W30 x 108 girders. Figure 16 shows the new design of the steel members. All supporting calculations are shown in Appendix B.5.

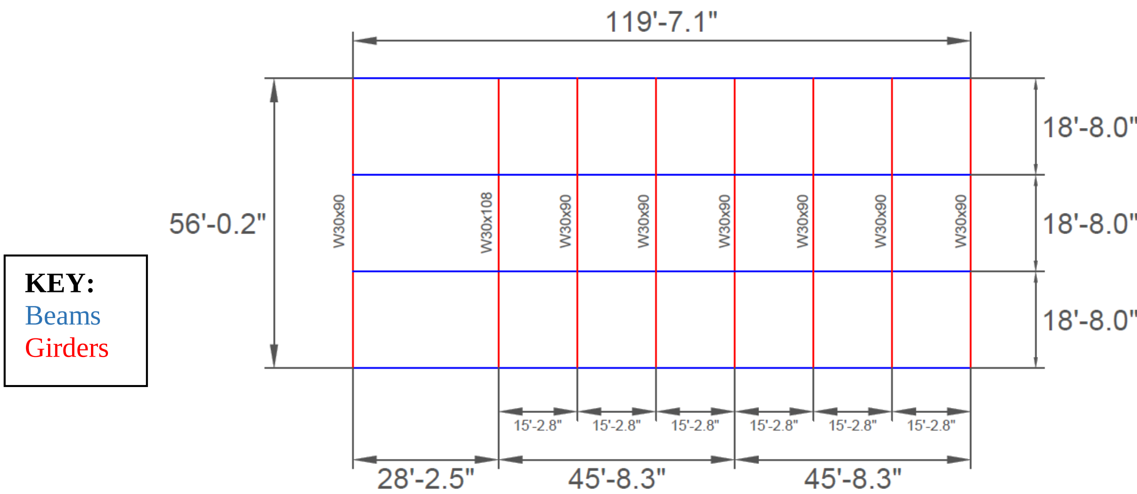

Figure 15: Plan View of Current Design with Girder Sizes

Table 24: Determination of Revised Girder Layout Based on Design for Lateral-Torsional Buckling

Member Size	Length of Member (ft)	Unbraced Length (ft)	Zone	Beam Moment M_u (k-ft)	Moment Capacity $ØM_n$ (k-ft)	Commentary
W30 x 108	56.0167	18.67	Inelastic Buckling	598.98	391	Exceeds Moment Capacity
		9.335	Inelastic Buckling		1236	***Unbraced Length Satisfies Moment Capacity***

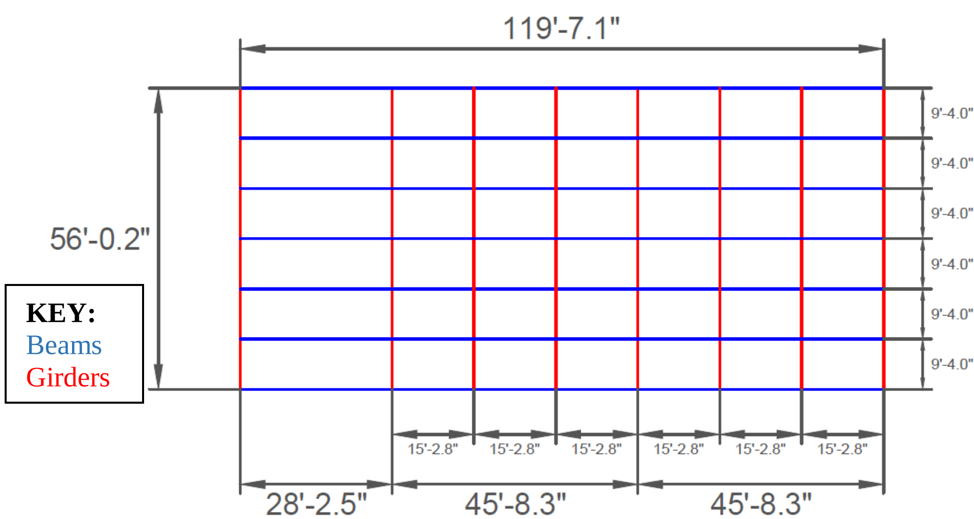

Figure 16: Plan View of Final Beam and Girder Layout

5.3.6 Supporting Column Calculations

Figure 17 displays a plan view of the locations and labels of each column supporting the beam and girder frame. Located on the west side (outer edge) of the parking garage are three existing 2 ft x 2 ft concrete columns which are at a height of 3.67 ft above the parking garage floor. These existing columns are located 45 ft apart, and have proposed steel columns placed on top of them to support the solar panel steel frame. Originally, the steel column sizes were designed for the existing floor conditions of the parking garage. Using the AISC manual, the designed steel column's axial strength was checked to satisfy the axial strength capacity of the member size and length. The designed steel column's axial strength was the same for all column members since each column supports the same tributary width, which is equal to half of the girder length. Likewise, all girders were designed to support the area of loads imposed by the beams, and the beams were designed to support the area of loads from the solar panels. To conclude, the column's axial strength and design is based on the imposing girder supported by the column. The initial steel column sizes chosen are displayed in Table 25.

After consideration, the recommendation is to place a 2 ft x 2 ft concrete column with a height of 3.67 ft under each of the five remaining steel columns (C1, C2, C3, C4, C8), which were currently designed to extend to the garage floor. This would allow each steel column across from each other to be identical, having the same length. The new selected steel column sizes are displayed in Table 26. Figure 18 displays an elevation view of the columns placed on top of the 2 ft x 2 ft concrete columns. All column calculations are shown in Appendix B.6.

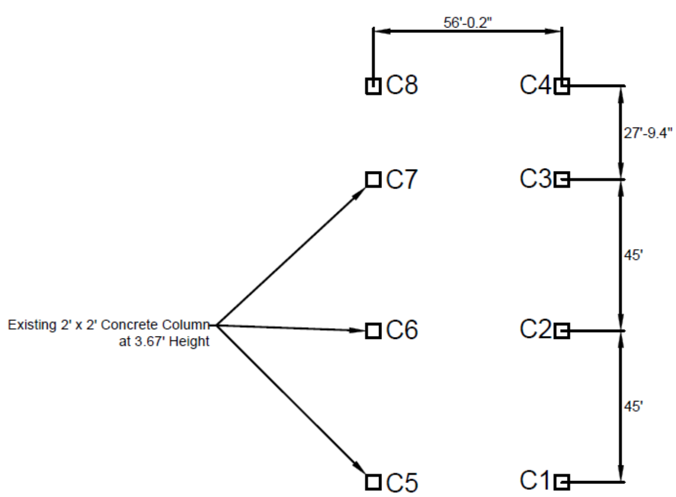

Figure 17: Plan View of Columns and Spacing

Table 25: Initial Chosen Column Sizes

Column Number	Member Size	Member Length (ft)	Column Axial Strength P_u (k)	Axial Strength Capacity $Ø_cP_n$ (k)
1	W8 x 31	10	48.35	317
2	W8 x 31	17.93	48.35	178
3	W8 x 31	25.87	48.35	86.5
4	W8 x 31	30.77	48.35	61
5	W8 x 31	6.33	48.35	362
6	W8 x 31	14.26	48.35	230
7	W8 x 31	22.2	48.35	111
8	W8 x 31	30.77	48.35	61

Table 26: Final Chosen Column Sizes

Column Number	Member Size	Member Length (ft)	Column Axial Strength P_u (k)	Axial Strength Capacity $Ø_cP_n$ (k)
1	W8 x 31	6.33	48.35	362
2	W8 x 31	14.26	48.35	230
3	W8 x 31	22.20	48.35	111
4	W8 x 31	27.10	48.35	74.5
5	W8 x 31	6.33	48.35	362
6	W8 x 31	14.26	48.35	230
7	W8 x 31	22.20	48.35	111
8	W8 x 31	27.10	48.35	74.5

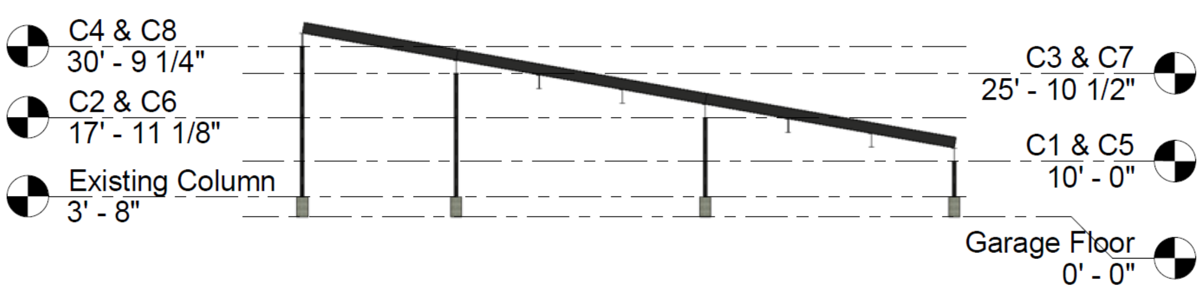

Figure 18: Elevation View of Columns Placed on 2 ft x 2 ft Concrete Columns

5.3.7 Second-Order Elastic Analysis

An approximate, second-order elastic analysis was performed to ensure that the columns were sufficiently designed to satisfy the stability requirements of Chapter 3 of the *AISC Specification* for Structural Steel Design. The first step of this process involved inputting both gravity and lateral loads, acting on the designed rigid frame, into the structural analysis software, Risa. Moment values due to gravity and lateral loads were obtained, from Risa, for the top and bottom of the column. The next step involved inputting these moment values into a created Excel sheet to go through the approximate second-order elastic analysis calculation outlined in Appendix 8 of the *AISC Specification*. The outcome of this analysis involves obtaining a value from the AISC Chapter H interaction equation. If the outcome value is less than or equal to one, then the column is adequate. If the outcome value is greater than one, then a new column or girder size must be chosen, and the analysis must be repeated. For this analysis, all of the selected column sizes satisfied the interaction equation and were considered adequate. The input for the Risa analysis and the outline for the second-order elastic analysis are displayed in Appendix B.7. Table 27 shows various results from the second-order elastic analysis for each column.

From the Risa analysis, the moment obtained from the connection of the column and girder was smaller than the moment value used to design the original girder. Therefore, calculations were made to determine a new girder size smaller than the original girder size. This girder size, W30 x 90, is located above in Table 23. The calculation process for the new girder size is located in Appendix B.8.

Table 27: Results from Approximate Second-Order Elastic Analysis and Interaction of Flexure and Compression

Column Number	Member Size	Multiplier B_1	Multiplier B_2	P_r (k)	M_{rx} (k-ft)	P_c (k)	M_{cx} (k-ft)	Interaction Value	Interaction Limit
1 & 5	W8 x 31	1	1	13.17	33.39	370	114	0.31	1
2 & 6	W8 x 31	1	1.03	26.39	41.11	243.32	97.2	0.48	1
3 & 7	W8 x 31	1	1.12	37.73	44.31	117	78.36	0.83	1
4 & 8	W8 x 31	1	1.11	24.53	25.66	80.5	64.69	0.66	1

5.3.8 Baseplate Design

Baseplates were designed to connect the steel columns to the supporting 2 ft x 2 ft concrete columns. The length, width, and thickness of each baseplate were determined using the load and moment acting on the concrete column from the steel column. The moment was found at the bottom of the steel column using the Risa analysis. To determine the moment-resisting thickness of the baseplate, the largest moment and load values out of all the columns were used to determine the largest minimum thickness required for the baseplates. After analysis, it was concluded that all baseplates will have the same length, width, and thickness. The results of the baseplate design are shown in Table 28. Figure 19 displays the design of the baseplate connecting the steel column from the rigid frame to the 2 ft x 2 ft concrete column. All supporting calculations are shown in Appendix B.9.

Table 28: Baseplate Design Results

Column Number	Baseplate Material	Width-B (in)	Length-N (in)	Thickness t_{req} (in)
1	A36 Steel	9	9	0.5
2	A36 Steel	9	9	0.5
3	A36 Steel	9	9	0.5
4	A36 Steel	9	9	0.5
5	A36 Steel	9	9	0.5
6	A36 Steel	9	9	0.5
7	A36 Steel	9	9	0.5
8	A36 Steel	9	9	0.5

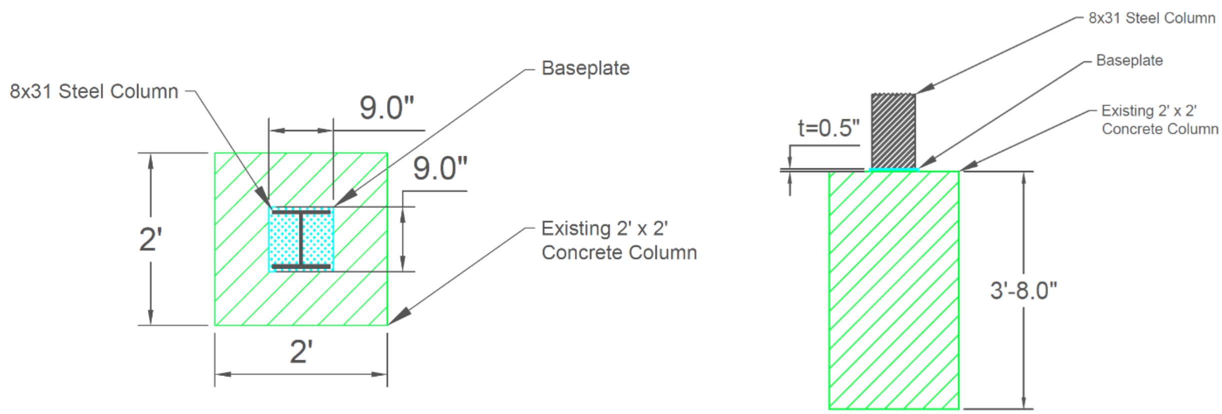

Figure 19: Overhead and Side Elevation of Baseplate Design

5.3.9 Recalculation of Seismic Load

Now that the steel frame has been designed to support the solar panels, the seismic load was recalculated. From the ASCE 7-10, it is required that the designed structure weight is 25% less than the current structure weight. This check was done to assess the impact of the designed structure to the existing parking garage structure. Based on this, new horizontal and vertical seismic loads were calculated. The recalculation of the new horizontal seismic load equals 1.28 psf, compared to the original 0.12 psf. The recalculation of the new vertical seismic load equals 0.64 psf, compared to the original 0.079 psf. These new seismic load values were plugged into the Risa analysis to check for the adequacy of the column sizes. Additionally, the new seismic load values were used to check their effect on the original beam and girder design. After analysis, it was concluded that the updated seismic loads do not have a large impact on the steel framework design, and therefore does not need to be changed for the updated seismic loads. Calculations for the new seismic load are contained in Appendix B.10.

5.3.10 Reinforcement in 2 ft x 2 ft Concrete Columns

The last step involved designing the reinforcement needed to be placed inside of the 2 ft x 2 ft concrete columns supporting the steel frame. This involved determining the number and type of steel reinforcing rebar, as well as the thickness of the ties wrapped around the steel rebar. The design of the reinforcement is based on axial force and bending moment acting on the concrete columns from the steel frame. Due to the small axial force and bending moment values, the smallest reinforcement ratio value, ρ_g equal to 0.01, was chosen for each concrete column based on the concrete column strength interaction diagram. The minimum and maximum reinforcement ratio values, ρ_{min} and ρ_{max}, were calculated the same for each concrete column based on their concrete properties. After ensuring that the reinforcement ratio ρ_g satisfied the calculated minimum and maximum values, ρ_g was used to determine the area of the reinforcing steel rebar. Table 29 displays the results from the concrete reinforcement calculations. Figure 20 shows a cross section of the 2 ft x 2 ft concrete column with the designed reinforcement. All supporting calculations are located in Appendix B.11.

Table 29: Concrete Reinforcement Results

Required Steel Area (in^2)	5.76
Reinforcing Steel Rebar Area (in^2)	5.96
Calculated Minimum Reinforcement Ratio ρ_{min}	0.003
Reinforcement Ratio ρ_g	0.01
Calculated Maximum Reinforcement Ratio ρ_{max}	0.021
Bar Diameter (in)	9/8
Number of Bars	6
Tie Diameter (in)	2
6 #9's with 2" ties in existing 2' x 2' concrete	

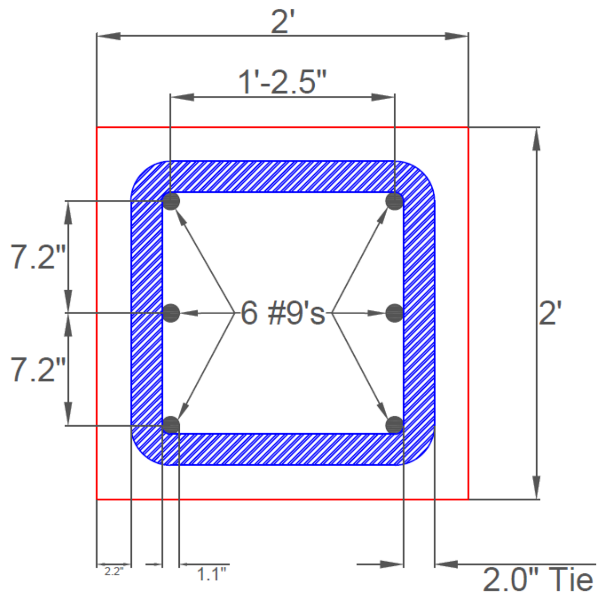

Figure 20: Designed Reinforcement in 2 ft x 2 ft Concrete Column

5.3.11 Supporting Steel Frame Final Results

To conclude, a steel frame containing beams, girders, and columns was designed to support solar panels above the top level of the Gateway Parking Garage. It was recommended that the columns of the steel frame be placed on top of 2 ft x 2 ft concrete columns at a height of 3.67 ft. Three out of eight of these concrete columns already exist, however, reinforcing steel was designed for each concrete column. Baseplates were designed to connect the steel columns to the concrete columns. Table 30 displays the size, length, and number of steel members needed for the steel frame. A 3-D perspective of the overall steel design is shown below in Figure 21.

Table 30: Steel Frame Member Properties

Type of Member	Member Size	Member Length (ft)	Quantity
Exterior Beams	W24 x 55	45.69	4
		28.21	2
Interior Beams	W24 x 68	45.69	10
		28.21	5
Girders	W30 x 90	56.02	7
	W30 x 108		1
Columns	W8 x 31	6.33	2
		14.26	2
		22.20	2
		27.10	2

Figure 21: Overall Steel Design 3-D Perspective

CHAPTER 6: GREEN ROOF INSTALLATION ON GORDON LIBRARY

This chapter contains information on the specific type of green roof chosen for the Gordon Library. The type of green roof was chosen based on ease of installation, energy reduction, and cost. Additionally, this chapter contains information about the rooftop layout and construction process for the installation of a green roof. Pertinent information includes the dimensions of the green roof, as well as the specific location on the roof for its installation. Finally, this chapter contains associated structural analyses and design information.

6.1 Selected Green Roof Technology on Gordon Library

For the application of a green roof on Gordon Library, an extensive green roof was chosen. An extensive green roof was selected, over an intensive green roof, since extensive green roofs are less expensive, have a lower overall weight, and require less maintenance. Since there is no public access to the Gordon Library rooftop, a green roof with sidewalks, benches, and tables for human interaction was not designed. Instead, the technology was designed over an area with the purpose of reducing the building’s energy consumption. In addition, the low cost of an extensive green roof can make this system cost-effective, feasible for construction, and easy to implement on the sustainable plan of WPI. The green roof will have small pathways along its sides only for maintenance use. Roof and gutter checks for extensive green roofs are required twice a year. Additionally, extensive green roofs require weeding three times a year and the application of fertilizer once a year (Green Roof Guide, n.d.). Table 31 contains information on the type, weight, energy reduction, lifespan, and costs of an extensive green roof.

Table 31: Information about Green Roof Installation[13]

Building	Gordon Library
Sustainable Rooftop Technology	Extensive Green Roof
Dimensions	80' x 167'-85'8" x 30'8"
Gross Area	10,732.89 ft^2
Weight	20-35 psf (4-6" soil depth)
Energy Reduction	12% overall energy reduction
Lifespan	50 years
Cost of Technology & Installation	$15 psf
Annual Operation & Maintenance Cost[14]	$0.27 psf

This table presents an overall description of the selected technology for the Gordon Library. All the information that is presented in the table was specifically selected to suit the dimensions of the roof and energy consumptions of the building. Likewise, it is shown that the lifespan of the roof extends up to 50 years, reducing maintenance, repairs and restoration costs of the roof.

6.2 Layout and Construction Process for Green Roof on Gordon Library

This section contains information on the layout and construction process for the installation of a green roof on Gordon Library.

6.2.1 Layout on Gordon Library

Based on rooftop drawings of Gordon Library provided by WPI Facilities Department, a roof garden design, which will reduce the overall energy of the building by approximately 12% was proposed (Urban Design Tools, 2017). The proposed design produces an overall area of 10,733 square feet with a six-inch soil depth. The extensive green roof that was chosen will contain vegetation including sedum, herbs, perennials, and shrubs. Additionally, the roof garden

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[13] (Urban Design Tools, 2017)

[14] http://www.epdmroofs.org/attachments/sproul-et-al_economic-comparison-of-white-green-and-black-flat-roofs-in-the-us.pdf

will contain drainage plates to collect all water absorbed by the vegetation. Water is retained within pockets on the upper sides of the drainage plates. Excess water will spill over the edges of the plates and funnel towards the existing drainage system on the roof. Figure 22 illustrates the proposed layout of the green roof on Gordon Library and the appropriate path for maintenance. It is clear that most of the roof is composed of the green roof technology. This is because the benefit of a roof garden is noticeable with a larger area. The proposed design is quite simple due to the flat surface of the roof. This is beneficial for construction and design purposes.

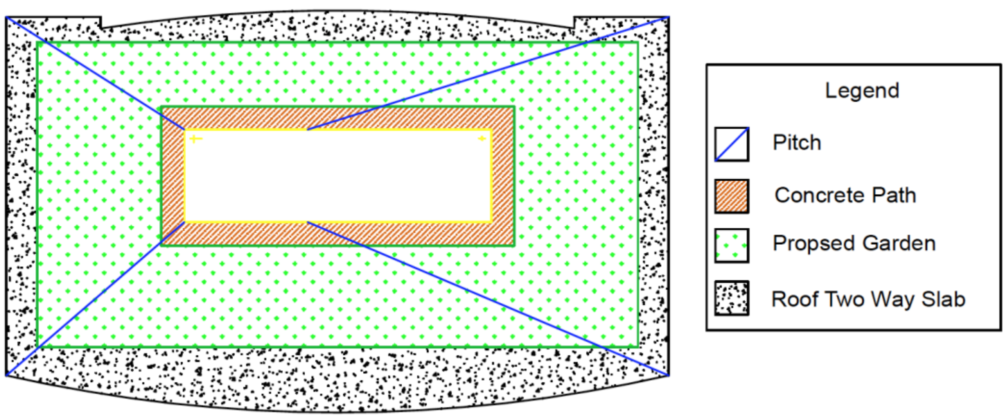


Figure 22: Green Roof Design and Layout on Gordon Library

6.2.2 Construction Process for Extensive Green Roofs

The steps for the installation process of an extensive green roof are displayed in Table 32. A visual of the different layers of an extensive green roof is provided in Figure 23.

Table 32: Steps for Installation of Extensive Green Roof[15]

Step 1	Install a waterproof membrane that possesses monolithic properties. It could be made of plastic or rubber, and it needs to fit on top of a traditional roof decking.
Step 2	Place one sheet of plastic with a maximum thickness of 6 millimeters over the already installed waterproof membrane.
Step 3	Install one or more sheets of foam insulation with a ¾" thickness over the plastic sheet. This layer provides proper contact with the damp soil.
Step 4	If the space directly below the green roof does not have proper conditioning, some protection needs to be provided to the waterproof membrane. The protection can be made up of fan –board-type insulation or can be a layer of building felt.
Step 5	Add one drainage mat with capillary spaces at the top portion of the insulation, after the protective layer. To prevent soul from clogging over the mat, place the mat in a manner that the felt side faces upward.
Step 6	Install framing around the perimeter of the green roof. This can be done with wood, mesh gutter-type guards, or some other type of edging material that can hold soil with more strength to keep it in the right place. Intermediate angle-type support, over vertical edging, might be required to support or improve sturdiness.
Step 7	The horizontal leg in the support system can be slipped under a drainage mat that is weighted with a specific amount of topping soil so that overturning can be avoided.
Step 8	Once the structure is ready, add soil to the sections.
Step 9	Once soil is added, set plants in specific locations.
Step 10	Water the area to allow for proper settling of plants.

[15] (My Rooff, 2017)

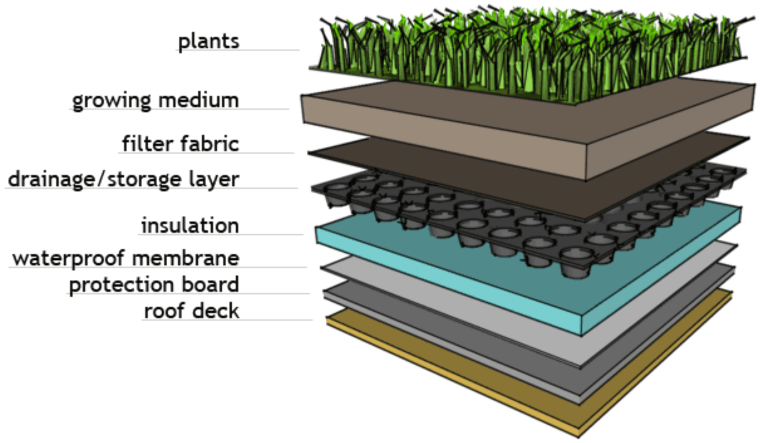


Figure 23: Layers of an Extensive Green Roof (My Rooff, 2017)

6.3 Structural Analyses and Design for Green Roof on Gordon Library

After obtaining a proper design and layout for the roof garden on Gordon Library, a structural anlysis was made to determine all the loads acting on the existing building (vertically and horizontally), the capacity of the existing roof to support the loads, and the economic cost of implementing a new system into the building. The structural analyses evaluated the existing roof of the building, and its capability to sustain a green roof based on the design provided.

6.3.1 Vertical and Horizontal Load Calculations

One of the first stages to complete the structural analysis of the building was to obtain all the loads that are acting on the building. This include, Live, Dead, Rain, Snow, Wind and Seismic loads. For this case, the analysis of the loads was done in two different ways. One, for the existing building as it is, and the second one for the green roof layout. The purpose of this is to show how some of the loads differ when they have an extra applied oad on the roof, in this case the roof garden. Each load that is acting on the building had its own characteristcs in accordance with ASCE 7-10, FM-Global and the *International Building Code* (IBC). It is important to state that the analysis for the gravity loads acting on the building is different than the one for lateral loads. Calculating the load combinations manually can be very complex when the combination requires both vertical and lateral loads. For that reason, the use of the software packages RISA 2D and RISA 3D was necessary in the analysis. This software allows the user to compute all the loads acting on the building while using as many load combinations as needed.

6.3.1.1 Live Loads

Live loads of the 1st, 2nd, 3rd and roof floor were based on occupancy and architectural designs of the building. *The American Society of Civil Engineers* 7-10 (ASCE 7-10) Chapter 4, was used to get this occupancy loads. Live loads in each floor are going to differ, as each floor has a different area for specific live loads. Some assumptions were made to select this area, as there are no specific details in the drawings that determine where each live load will be acting in each floor. The live loads that were considered in this building for calculation are shown in Table 33 below.

Table 33: Live Loads in Accordance with ASCE 7-10

Description	Load (psf)
Roof [16]	20
Meeting Room	40
Office Room	50
Reading Room	60
Corridor Above 1st Floor	80
Corridor 1st Floor	100
Mechanical Room	100
Stairs	100
Library Stacks	150

The application areas for live loads shown in Table 33 were determined using AutoCAD Software and a sample prototype of the second floor of the library is illustrated in Figure 24 below. For the design of each floor and its occupancy live loads, see Appendix C.1.

[16] ASCE 7-10 specifies a roof used for roof garden to have a live load of 100 psf: however, a live load of only 20 psf is more appropriate in roof gardens that do not need high maintenance and do not allow public access to it.

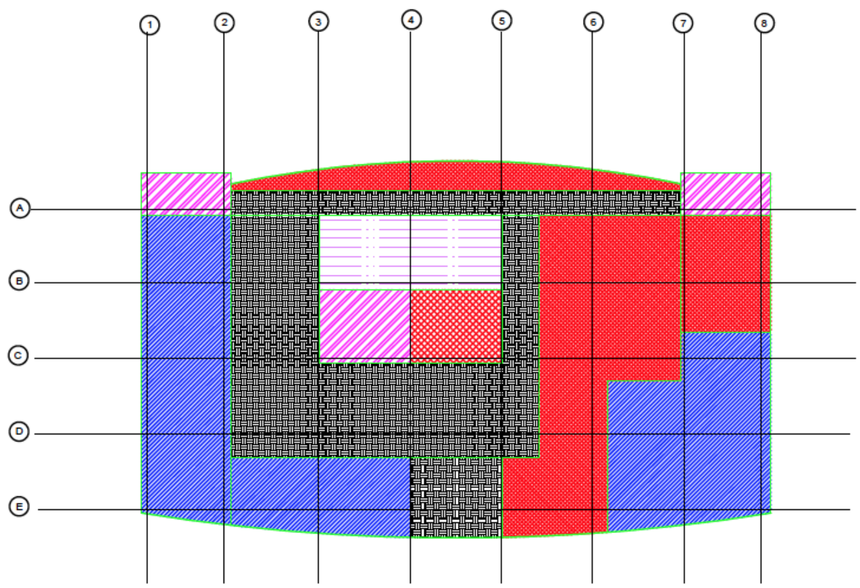


Figure 24: Live Loads Occupancy Area on 2nd Floor of Gordon Library

6.3.1.2 Dead Loads

The dead load was calculated by doing an analysis of the weight for each floor of the building. The loads for each floor includes the weight of the two-way dome slab, internal and external concrete walls, exit stairs, and roof penthouse. The dead load also included the designed green roof. An approximate weight per floor is shown in Table 34; see Appendix C.1 for extended dead load calculations.

Table 34: Weight per floor Gordon Library

Weight Library Results	
Floor	**Weight (kips)**
Ground - 1st	3,734.31
1st - 2nd	3,597.36
2nd - 3rd	3,479.40
3rd- Roof	2,818.64
Weight of Library (kips)	13,629.71

A more approximate approach to the weight of the two-way dome slab is shown in Table 35 below. These values are given by the CRSI according to the slab thickness and dome depth for a waffle slab. The weight of two-way dome slab is given in pounds per square feet.

Table 35: Weight of Two-Way Dome Slab in Accordance with CRSI[17]

Two-Way Dome Slab Dimensions		
Slab Thickness	**Dome Depth**	**Weight (psf)**
3 inches	10 inches	91
4 inches	10 inches	103

6.3.1.3 Rain Loads and Snow Loads

Rains and snow loads did not vary by implementing a green roof on the building. These loads were determined in accordance with FM Global and ASCE 7-10 Chapter 7 respectively.

6.3.1.4 Seismic Loads and Wind Loads

Seismic and wind loads were calculated in accordance with ASCE 7-10 and by using two step-by-step code masters. These loads were first calculated without considering the roof garden as shown in Table 36. After this calculation was done, some steps were re-done but this time including the weight of the green roof on the building, Table 37.

Table 36: Calculated Loads without Green Roof Technology

Loads	Value (psf)
Dead (D)	Table
Snow (S)	34.65
Wind (W)	Figure 27 and Figure 28
Seismic	Figure 30
Roof Live (L_r)	20
Live (L)	0
Rain (R)	32

Table 37: Total Weight of Gordon Library per Floor

Weight Library Results	
Floor	**Weight (kips)**
Ground - 1st	3734.31
1st - 2nd	3597.36
2nd - 3rd	3479.39
3rd- Roof	2818.64
Weight of Library (kips)	**13629.71**

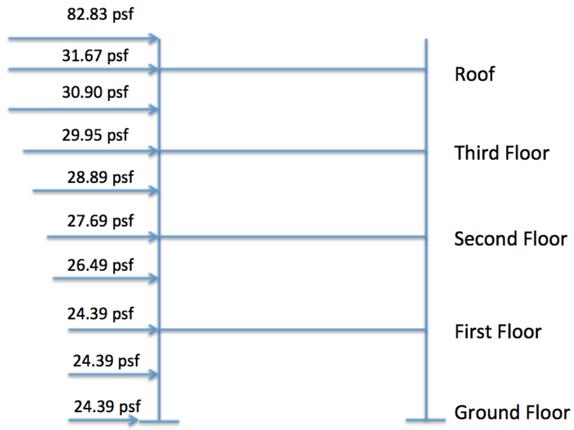
[17] Concrete Reinforcing Steel Institute

A simple calculation was done to determine the total weight of the roof garden on Gordon Library. It was assumed that the weight of the green roof technology was 35 pounds per square feet. This value was obtained in accordance to the depth of the soil, the type of green roof installed, and the layers used. Table 38 shows the overall weight of an extensive green roof with a six inch soil-depth.

Table 38: Total Weight of Green Roof

Green Roof System		
Description	**Value**	**Units**
Weight of Green Roof	35	psf
Gross Area	10,733.00	ft^2
Weight of Green Roof System	**375.66**	**kips**

The entire analysis and results of the seismic and wind load procedures are illustrated in Figures 25-30 below. These figures represent the total force in kips acting on the building laterally along its height. The figures represent wind forces in the North-South and East-West directions, the total resultant per floor for each direction, and seismic forces. For the wind load analysis, the forces also changed for the parapet on the roof of the building. As can be seen in Figure 27, the parapet has a distributed load of 82.83 pounds per square feet in the North-South direction, which drastically changed with respect to the other floors of the building. The parapet has a separate analysis and equation, causing this value to increase.



Figu **tion)**

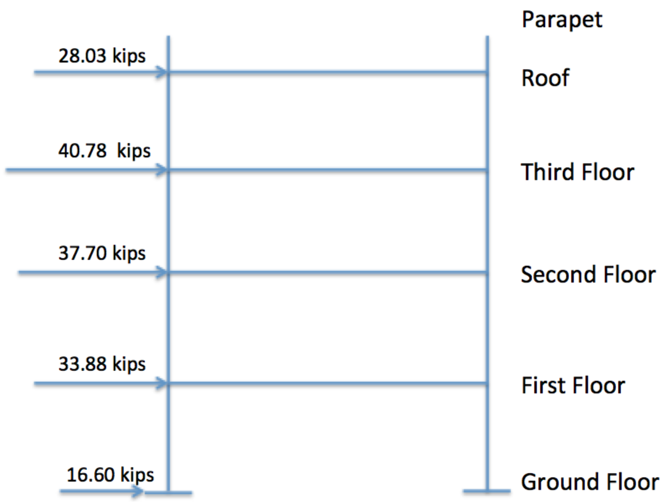


Figure 26: Resultant Wind Forces (North-South Direction)

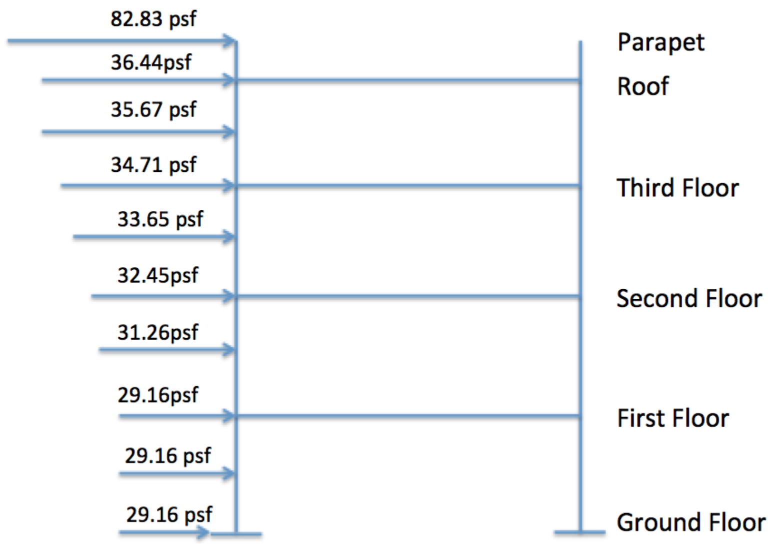


Figure 27: Wind Load Distribution per Floor (East-West Direction)

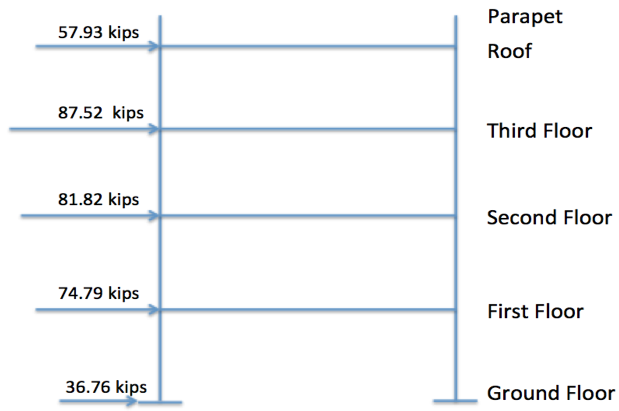


Figure 28: Resultant Wind Forces (East-West Direction)

As shown in the figures above, the resultant wind forces in the East-West direction are more critical than the wind forces in the North-South direction. For this reason, the resultant wind forces used for analysis were the ones from the East to West direction.

In addition to the resultant wind forces acting laterally on the building, a Components and Cladding analysis was done to understand how building components need to resist the wind forces. Table 39 shows the results for each zone in the building as stated in Chapter 30 of the ASCE 7-10. This table was generated with the guidance of Table 2 and 3 from Section 3.3.2.1, and the wind pressure at the roof.

Table 39: Wind Components and Cladding Results for Gordon Library

Results Components and Cladding										
AREA (SF)	ZONE 1		ZONE 2		ZONE 3		ZONE 4		ZONE 5	
	+	-	+	-	+	-	+	-	+	-
≤ 10 ft^2	15.83	-38.91	15.83	-65.30	15.83	-98.28	38.91	-42.21	38.91	-52.10
≥ 500 ft^2 walls & ≥ 100 ft^2 roof	12.53	-35.61	12.53	-42.21	12.53	-42.21	29.02	-32.32	29.02	-32.32

A similar approach was taken for the seisimc analysis of the building. Figures 29 and 30 represent the lateral loads in kips acting at each floor of the Gordon Library.

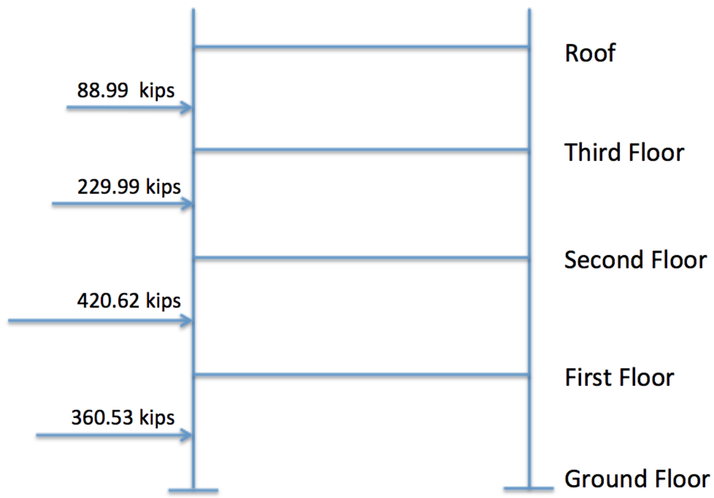


Figure 29: Seismic Forces Acting on Building

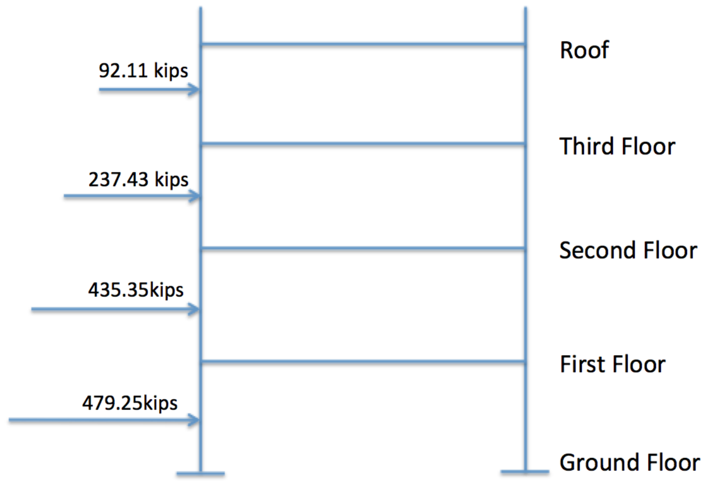


Figure 30: Seismic Forces Acting on Building with Green Roof Technology

6.3.2 Basic Load Combinatination Analysis

An assessment was performed for each floor of the structure with all the basic load combinations and with customized combinations that could be useful. All basic combinations were in accordance to ASCE 7-10, Chapter 2, However, some of the combinations used are not going to be stipulated in the ASCE 7-10 as they were modified to suit the structure more accurately (based on the more significant loads). See Table 40 for load combinations that were used in this structure and their values in pounds per square foot.

Table 40: Basic Load Combinations

Load Combinations	
Gravity Loads	1.4D
	1.2D + 1.6L + 0.5(L/S/R)
	1.2D + 1.6S + L
	1.2D + 1.6S + 0.5W
	1.2D + 1.0W + L + 0.5(L_r/S/R)
	1.2D + E_v + L + 0.2S
Lateral Loads	0.9D + 1.0W
	0.9D + 1.0E_h

By obtaining the values of all load combinations, the most significant combination was selected and therefore the most conservative load in pounds per square feet. This assessment was performed for each floor of the building as the loads accumulated from the roof to the first floor.

6.3.3 Structural Analysis and Capacity of Building

In order to determine if the building could resist the new loads imposed on its roof, a structural analysis was performed. The columns and the waffle slabs of each floor of the building were analyzed and then compared to their actual capacity.

6.3.3.1 Overturning Moment of the Building

Overturning moment of the structure relates to the capacity of the system to resist lateral loads due to seismic. For this structure, the overturning moment was not analyzed because the green roof technology imposes additional weight to the existing structure, and the overturning moment should not be a concern for the design.

6.3.3.2 Factored Design Load of Column (Axial Capacity ΦPn)

The first check consisted of simply analyzing the reinforced concrete capacity of each of the columns in the building. This calculation involved the following:

- Dimensions and shape of the column,
- Reinforcement properties (dowels[18] and spirals),
- Total area of concrete and steel in the column
- Concrete and steel strength.

Table 41: Concrete and Steel Strength Properties Gordon Library

Concrete and Steel Properties		
Steel Yield Strength (psi)	fy	60,000
Concrete Strength (psi)	f'c	4,000
Reduction Factor	ϕ	0.7

It was assumed that the steel strength is approximately 60,000 psi. This value was obtained through research of historic requirements of AISC for old buildings. Figure 31 below, illustrate

[18] A “dowel” refers to the vertical steel bars inside of the column.

the requirements per year of the AISC and the values for steel. The book titled, *Iron and Steel Beams 1873 to 1952*[19] also provides a table for structural steel specifications.

Table 1 ASTM and AISC History				
	ASTM			AISC
Year	Standard	T.S. (ksi)	Y.P. (ksi)	Basic Working Stress
1901	A9 Buildings	60-70	0.5 T.S.	—
1909	A9 Buildings	55-65	0.5 T.S.	—
1923	A9 Buildings	55-65	0.5 T.S.	18
1924	A9 Buildings	55-65	0.5 T.S.	18
1933	A9 Buildings	60-72	0.5 T.S. (not less than 33)	18
1936	A9 Buildings	60-72	0.5 T.S. (not less than 33)	20
1939	A7 Buildings (and Bridges)	60-72	0.5 T.S. (not less than 33)	20
1942	A7 WPB Emergency Standards			24
1960	A7	60-72	0.5 T.S. (not less than 33)	20
	A36 (Supp.)	58-80	36	22
1963	A7	60-72	0.5 T.S. (not less than 33)	20
	A36	58-80	36	$0.6F_y$
	A440	varied	varied	$0.6F_y$
	A441	varied	varied	$0.6F_y$
	A242	varied	varied	$0.6F_y$
1967	A7 discontinued			
1968	A36	58-80	36	$0.6F_y$
	A572	varied	varied	$0.6F_y$
	A588	varied	varied	$0.6F_y$

Figure 31: Typical Steel Standards for Old Buildings[20]

The following results were calculated for the axial capacity of the columns. The results are tabulated for each floor of the building and based on the column size and the reinforcement properties. The columns that have the most axial capacity, as illustrated in Table 42, are typically the most critical columns in the building. For this reason, the structural analysis was based on these columns to have a realistic idea of how the entire structure will behave when the new load of the green roof is imposed on the building.

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[19] https://www.aisc.org/globalassets/aisc/publications/out-of-print/iron-and-steel-beams-1873-1952.pdf
[20] Gustafson, Kurt. "Evaluation of Existing Structures." *Steel Solutions Center*, https://www.aisc.org/globalassets/modern-steel/steelwise/30762_steelwise_reno.pdf

Table 42: Axial Capacity of Columns Ground Floor with 60 Grade Steel Reinforcement

Ground to First Floor						
Column Size (in x in)	**Ag (in^2)**	**# of Bars**	**Size of Bar**	**Ast (in^2)**	**Pn_{max} (kips)**	**ϕPn_{max} (kips)**
24x24	576	6	11/9	7.68	2034	1423
24x24	576	6	10	7.62	2031	1421
24x24	576	8	10	10.16	2153	1507
24x24	576	8	11	12.48	2265	1586
24x24	576	6	9	6	1953	1367
24x18	432	6	9	6	1537	1076
29x29	841	8	11	12.48	3030	2121

Table 43: Axial Capacity of Columns First Floor with 60 Grade Steel Reinforcement

First Floor						
Column Size (in x in)	**Ag (in^2)**	**# of Bars**	**Size of Bar**	**Ast (in^2)**	**Pn_{max} (kips)**	**ϕPn_{max} (kips)**
16x16	256	8	10	10.16	1229	860
18x18	324	6	9/11	7.68	1306	914
23x23	529	8	11	12.48	2129	1490
24x24	576	6	9	6	1953	1367
24x18	432	6	9	6	1537	1076

Table 44: Axial Capacity of Columns Second Floor with 60 Grade Steel Reinforcement

Second Floor						
Column Size (in x in)	**Ag (in^2)**	**# of Bars**	**Size of Bar**	**Ast (in^2)**	**Pn_{max} (kips)**	**ϕPn_{max} (kips)**
16x16	256	8	10	10.16	1229	860
18x18	324	8	10	10.16	1425	998
18x18	324	8	11/10	11.32	1481	1037
18x18	324	6	9/8	5.37	1195	836
18x18	324	6	8	4.74	1164	815
23x23	529	8	11/10	11.32	2073	1451
23X23	529	8	10/7	7.48	1889	1322
23x23	529	8	10	10.16	2018	1412

Table 45: Axial Capacity of Columns Third Floor with 60 Grade Steel Reinforcement

Third Floor						
Column Size (in x in)	**Ag (in^2)**	**# of Bars**	**Size of Bar**	**Ast (in^2)**	**Pn_{max} (kips)**	**ϕPn_{max} (kips)**
12x12	324	8	10/6	6.84	1265	886
12x12	144	8	9	8	801	561
12x12	144	6	8	4.74	644	451
12x18	216	6	8	4.74	852	597
17x17	289	8	10/9	9.08	1272	890
17x17	289	8	7/5	3.64	1010	707
18x18	324	6	8	4.74	1164	815

As shown in the table above, the axial capacity of the columns ϕPn_{max} increases with increasing distance from the roof level. The strongest column is located in the lowest (first) floor of the building because that specific column needs to resist the entire load of the upper floors, plus the loads on that same floor. This table provides the values for Grade 60 Steel[21]. The axial capacities of each column would slightly change if the Grade of the steel changes. See Table 46 and Table 47 for an example of axial capacities ϕPn with a different Grade of steel in the first floor of the Gordon Library.

Table 46: Axial Capacities of Columns with 50 Grade Steel Reinforcement

First Floor						
Column Size (in x in)	**Ag (in^2)**	**# of Bars**	**Size of Bar**	**Ast (in^2)**	**Pn_{max} (kips)**	**ϕPn_{max} (kips)**
16x16	256	8	10	10.16	1142	799
18x18	324	6	9/11	7.68	1241	868
23x23	529	8	11	12.48	2023	1416
23x23	529	8	10	10.16	1931	1352
23x23	529	6	9	6	1766	1237
24x24	576	6	9	6	1902	1332
24x18	432	6	9	6	1486	1040

[21] Grade of steel determine the yield strength of the steel, (e.g. Grade 60 Steel; fy=60,000 psi)

Table 47: Axial Capacities of Columns with 40 Grade Steel Reinforcement

First Floor						
Column Size (in x in)	**Ag (in^2)**	**# of Bars**	**Size of Bar**	**Ast (in^2)**	**Pn_{max} (kips)**	**ϕPn_{max} (kips)**
16x16	256	8	10	10.16	1056	739
18x18	324	6	9/11	7.68	1175	823
23x23	529	8	11	12.48	1917	1342
23x23	529	8	10	10.16	1845	1291
23x23	529	6	9	6	1715	1201
24x24	576	6	9	6	1851	1296
24x18	432	6	9	6	1435	1005

Tables 46 and 47 show the difference in axial capacities when lower yield strength steel is used. This is important to consider, as come columns have a higher change due to this factor, and it can affect in the overall design of a building or the structural analysis.

As a result of the calculations of the axial capacity of the concrete columns, Table 45 was created. This table illustrates the three sections chosen in the building for analysis. The columns in the three sections were the most critical columns in the building and also the most appropriate columns to work with for symmetry purposes. Some axial capacities are repeated because their properties (dimension and reinforcement) were similar.

Table 48: Column Axial Capacities per Sections

Grade 60 Steel				
Column No.	**3rd Floor**	**2nd Floor**	**1st Floor**	**Ground**
	ΦPn (kips)	**ΦPn (kips)**	**ΦPn (kips)**	**ΦPn (kips)**
B5	890	1451	1490	1586
C5	707	1322	1490	1586
B6	890	1451	1490	2122
C6	707	1322	1490	1586
Column No.	**3rd Floor**	**2nd Floor**	**1st Floor**	**Ground**
	ΦPn (kips)	**ΦPn (kips)**	**ΦPn (kips)**	**ΦPn (kips)**
D1	890	997	1412	1422
E1	560	1413	1412	1422
D2	560	1036	1490	1507
E2	560	1451	1490	1507
Column No.	**3rd Floor**	**2nd Floor**	**1st Floor**	**Ground**
	ΦPn (kips)	**ΦPn (kips)**	**ΦPn (kips)**	**ΦPn (kips)**
B2	890	1451	1490	1507
C2	560	1036	1490	1507
B3	560	836	1367	1367
C3	451	1251	1272	1367

A similar table was created with the actual factored design loads (Pu) acting on each column section of the building. Table 46 provides the values of (Pu) calculated with the respective load combinations and superimposed loads acting on the building. See Appendix C.4 for procedure of the calculation.

Table 49: Factored Design Loads (Pu) on Different Sections

Grade 60 Steel				
Column No.	**3rd Floor**	**2nd Floor**	**1st Floor**	**Ground**
	Pu (kips)	**Pu (kips)**	**Pu (kips)**	**Pu (kips)**
B5	160	333	459	665
C5	160	329	459	665
B6	160	329	459	626
C6	160	303	459	626
Column No.	**3rd Floor**	**2nd Floor**	**1st Floor**	**Ground**
	Pu (kips)	**Pu (kips)**	**Pu (kips)**	**Pu (kips)**
D1	53	116	177	266
E1	26	58	88	133
D2	115	253	387	581
E2	72.949775	159.07565	243.9240125	366.2244375
Column No.	**3rd Floor**	**2nd Floor**	**1st Floor**	**Ground**
	Pu (kips)	**Pu (kips)**	**Pu (kips)**	**Pu (kips)**
B2	115.4885	265.57394	388.0166	581.65878
C2	115.4885	260.53394	388.0166	388.0166
B3	160.0177	317.36975	470.214225	630.0147
C3	160.0177	329.36975	466.214225	639.2147

From an axial load point of view, implementing a green roof technology on Gordon Library is not going to have critical conditions on the columns of the building. Structural reinforcement for the columns or additional reinforcement methods is not needed.

6.3.3.3 Combined Axial and Moment Capacities

Part of the structural analysis of the building included analyzing the most critical columns under a combined axial ΦPn and moment ΦMn capacities. Even though the columns satisfy the condition $\Phi Pn > Pu$, it was necessary to determine the effects and the capacity of the moments generated at the column due to the acting loads on the building. Interaction diagrams were created in accordance with the most critical column of the building, Column B5. The interaction diagrams for this column depend on the floor that is being analyzed. This was the only column selected for the analysis, and it would act as a basis for other critical columns in the building. Figures 32 to 34 illustrated below show the interaction diagram for column B5 for the first, second and third floor of the Gordon Library. The blue line in the figure represents the combined axial and moment capacity without the reduction factor *Φ*, while the green line represents the same axial and moment capacity including the reduction factor.

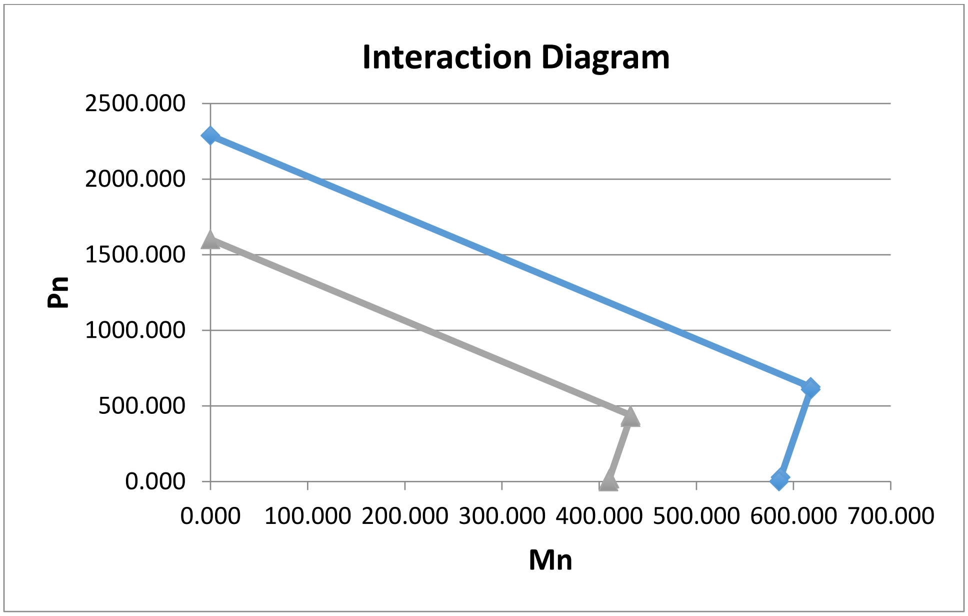


Figure 32: Interaction Diagram Column B5 First Floor

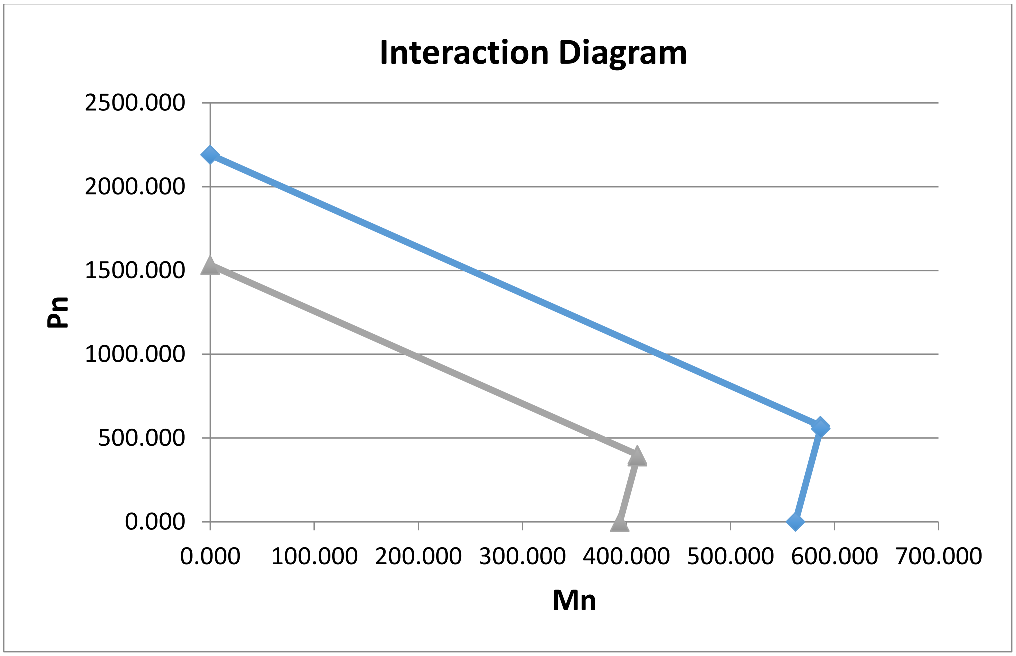


Figure 33: Interaction Diagram Column B5 Second Floor

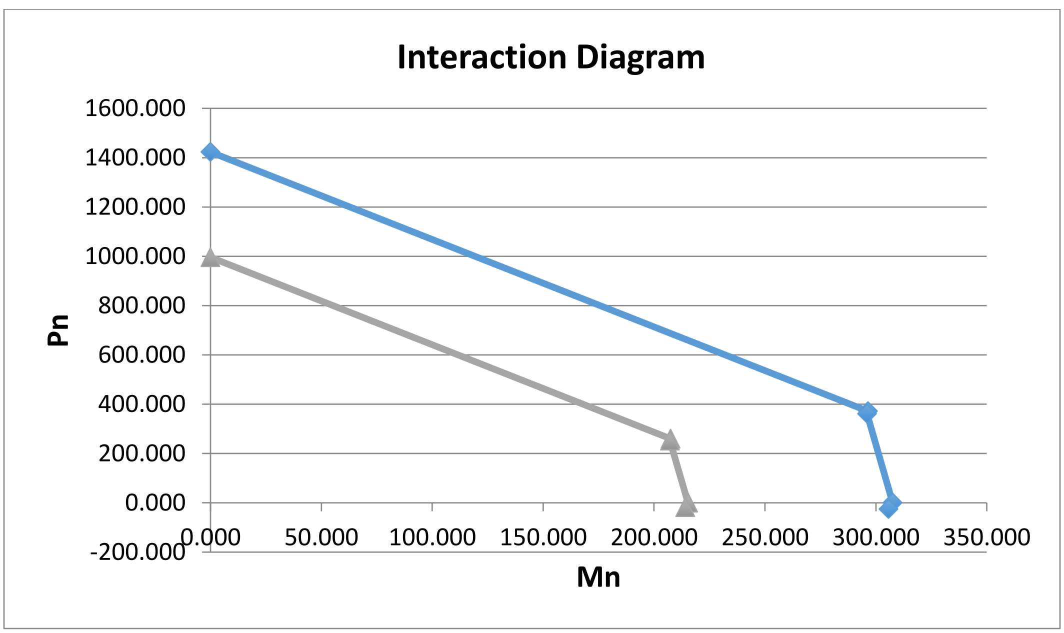


Figure 34: Interaction Diagram Column B5 Third Floor

In addition, the diagram was graphed by obtain five different points as previously noted in the methodology section. Table 47, displays an example of the interaction diagram points for the column in the first floor.

Table 50: Interaction Diagram Points Column B5 First Floor

Interaction Diagram Points				
a	**Pn**	**Mn**	**ΦPn**	**ΦMn**
inches	**kips**	**kips*ft**	**kips**	**kips*ft**
23.00	2287	0.0	1601	0.0
10.35	625	618	438	432
10.21	607	618	425	432
6.47	267	587	19	411
6.33	0.0	585	0.0	410

It is clear than by doing an axial and moment combined analysis of the columns of the building, the extra load of the green roof is not going to generate any structural damages. This column is assumed to be the most critical in the entire building, so satisfying the conditions for this particular column should make the system appropriate for the Gordon Library.

6.3.3.4 Two-Way Dome Waffle Slabs

The construction of waffle flat slabs allows a substantial reduction of the dead load of a building. The advantages of this type of slab construction include the overall weight reduction of a system while providing the building with an architecturally desirable structure. Ease of construction is another advantage of this type of slab, as a typical two-way dome slab is symmetric for the entire floor and it allows a building to be designed without any type of beams.

Table 51: Average Live Loads for each Floor

Live Loads	
Floor	**Average (psf)**
Roof	20
3rd	73
2nd	63.1
1st	102.2

The calculated minimum solid head over the columns of the Gordon Library was based on the minimum solid head equation, where the values of l_1 and l_2 were 25 feet and 20 feet respectively. The minimum solid head calculated was 7’6”, and the actual value of the solid head for the building was taken as 8’6” as a conservative value. This calculation was done, as the structural drawings did not provide specific details and dimensions for the actual solid head. The 8’6” value it is assumed based on specifications in accordance to the CRSI and its dimension is the same for all drop panels in the building. Figure 35, shows the side view of a column-to-column span for the slab on each floor of the Gordon Library. Each floor is going to have an exact arrangement of the slab except for the roof floor for which the slab will decrease in one inch. Figure 36 below shows a main section of the building that includes an edge, corner and interior column. The distribution of the domes in the slab is as illustrated in the figure, while the sections that do not include any domes are considered the drop panels.

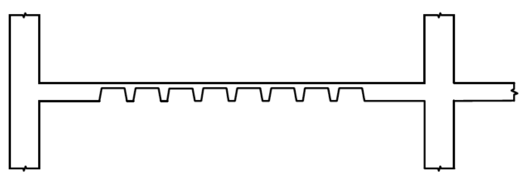


Figure 35: Side View of Waffle Slab on Gordon Library

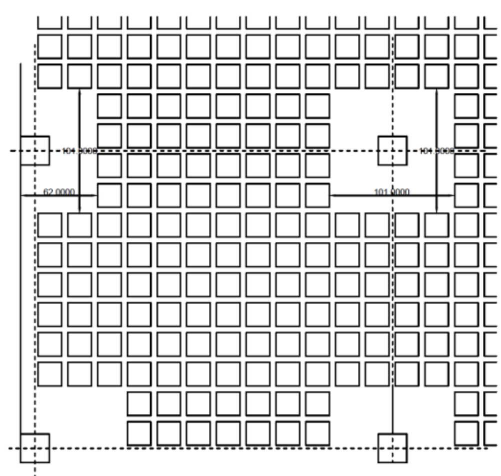


Figure 36: Plan View of Waffle Slab and Drop Panel Distribution

By obtaining the appropriate dimensions of the two-way dome slab and all the loads acting on the building, an analysis of the slab was done. This analysis included the calculations of the load combinations to obtain the most critical (Wu) values. Tables 52 and 53 below show the most critical values in kips per foot for each floor slab. The two tables differ in values for a 20’x25’ section and a 20’x21’ section. In addition, the (Wu) values increase for each floor, as additional weight has to be resisted. For this reason, the most critical slab is the one in the first floor as it is the one who has to support most of the loads of the building. The Factored Design Load (Wu) for the slab on the fist floor is the highest value for the entire building because the slab has to support the accumulated dead loads of the floors on top.

Table 52: Factored Design Load (Wu) 20’x25’ Section

20'x25' Section	
Floor	**Wu (kips/ft)**
Roof	8.42
Third	16.89
Second	24.25
First	33.00

Table 53: Factored Design Load (Wu) 20’x21’ Section

20'x21' Section	
Floor	**Wu (kips/ft)**
Roof	7.18
Third	14.40
Second	20.73
First	28.19

With the calculations of the (Wu) values on the floor slab the strength capacity of the slab was obtained.

Table 54 provides the distributed moments on the waffle slab. As the slabs for each floor of Gordon Library are two-way dome slabs, the distribution of moment is going to be different for interior, exterior, and edge columns. This table was used as a guideline to obtain the moments on the slab based on the Total Factored Moment (Mo) calculated[22].

Table 54: Distribution of Total Factored Moment (Mo) on Waffle Slab

	Exterior Edge Unrestrained	**Slab Without Beam Between Interior Supports**	**Exterior Edge Fully Restrained**
Interior (Negative factored Moment)	0.75	0.70	0.65
Positive Factored Moment	0.63	0.52	0.35
Exterior (Negative Factored Moment)	0	0.26	0.65

The calculations for the moment distribution on the slab and the appropriate shear and moment checks were done manually and are shown in Appendix C.6. The process for calculating the moment values for the slab can also be done by obtaining the shear constants from Table 11-5 from the CRSI Handbook as shown in Figure 37 below. The full table with different column sizes and slab dimensions can be obtained from the CRSI Handbook. The calculations and results for an end and interior span for the waffle slab of the first floor are shown Tables 51-54.

TABLE 11-5 PERIPHERAL SHEAR CONSTANTS AT COLUMNS

		Corner Column			Edge Column			Interior Column		
Square Column (in.)	Slab or Drop* (in.)	$c_{AB} = y_t$ (in.)	$A_c = b_o d$ (in.^{2})	J_c (in.^{4})	$c_{AB} = y_t$ (in.)	$A_c = b_o d$ (in.^{2})	J_c (in.^{4})	$c_{AB} = y_t$ (in.)	$A_c = b_o d$ (in.^{2})	J_c (in.^{4})
22	6	6.03	205	12,600	7.81	317	20,800	13.13	446	51,600
	7	6.16	259	16,600	7.93	402	27,600	13.63	572	71,500
	8	6.28	314	21,200	8.04	491	35,400	14.13	706	95,100
	9	6.41	372	26,200	8.16	584	44,100	14.63	848	122,800
	10	6.53	431	31,900	8.27	681	53,900	15.13	998	155,100
	11	6.66	493	38,100	8.39	782	64,900	15.63	1,156	192,300
	12	6.78	556	45,100	8.51	887	77,100	16.13	1,322	235,000

Figure 37: Shear Constants for Distributed Moment Calculations

[22] Appendix C.6 shows the calculation for the waffle slab and the Total Factored Moment (Mo)

Table 55: End Span Moment Distribution

Variable	Formula	Values
Total Static Moment (Mo)	$\frac{WuL^2}{8}$	1282.81 k*ft
Exterior Column (Negative Factored Moment) ACI 13.6.4.2		
Moment (Mu)	0.26Mo	333.53 k*ft
Bottom (Positive Factored Moment) ACI 13.6.4.4		
Moment (Mu)	0.52Mo	667.06 k*ft
Column Strip (Mu)	0.60Mu	400.24 k*ft
Middle Strip (Mu)	0.40Mu	266.82 k*ft
Interior Column (Negative Factored Moment) ACI 13.6.4.1		
Moment (Mu)	0.70Mo	897.97 k*ft
Column Strip (Mu)	0.75Mu	673.48 k*ft
Middle Strip (Mu)	0.25Mu	224.49 k*ft

Table 56: End Span Shear Calculations

Factored Shear (vu) Calculations		
Description/Variable	**Formula**	**Values**
Shear (Vu)	$\frac{wuL}{2} - \frac{Mu_{int} - Mu_{ext}}{L}$	239.35 kips
Factored Shear (vu)	$\frac{Vu}{Ac} + \frac{\gamma_v MuC_{AB}}{Jc}$	0.255 ksi
γ_v	(1-γ_f)	0.38
γ_f	$\frac{1}{(1+\left(\frac{2}{3}\right)\sqrt{b_1/b_2}}$	0.623
Mu	0.30Mo	384.84 k*ft
Shear Check at Exterior Column		
Shear Capacity (vc)	$(\frac{\alpha_s\, d}{b_o} + 2)\sqrt{f'c}$	0.382 ksi
α_s	For edge columns	30
Design Moment Strength (ΦMn)		
Area of Steel (As)	# of bars in column strip*Area of bars	6.60 in^2
a	$\frac{A_s f_y}{0.85 f'cb}$	2.28 in
Effective width (b)	Half the width of the panel	51 in

ΦMn	$\phi Asf_y(d - \frac{a}{2})$	344.82
Moment Check at Exterior Column		
$0.26\gamma_f$Mo		207.79 k*ft
ΦMn > $0.26\gamma_f$Mo		344.82 > 207.79
$\rho = \frac{As}{bd}$		0.010
ρ_{max}		0.011
$\rho_{max} > \rho$		0.011 > 0.010

Table 57: Interior Span Moment Distribution

Variable	Formula	Values
Total Static Moment (Mo)	$\frac{WuL^2}{8}$	1282.81 k*ft
Panel Moments		
Bottom (Positive Factored Moment) ACI 13.6.4.4		
Moment (Mu)	0.35Mo	448.98 k*ft
Column Strip (Mu)	0.60Mu	269.39 k*ft
Middle Strip (Mu)	0.40Mu	179.59 k*ft
Top (Negative Factored Moment) ACI 13.6.4.1		
Moment (Mu)	0.65Mo	833.83 k*ft
Column Strip (Mu)	0.75Mu	625.37 k*ft
Middle Strip (Mu)	0.25Mu	208.46 k*ft

Table 58: Interior Span Shear Calculations

Factored Shear (vu) Calculations		
Description/Variable	**Formula**	**Values**
Shear (Vu)	$\frac{wuL}{2} - \frac{Mu_{int} - Mu_{ext}}{L}$	298.52 kips
Factored Shear (vu)	$\frac{Vu}{Ac} + \frac{\gamma_v MuC_{AB}}{Jc}$	0.164 ksi
Shear Check at Exterior Column		
Shear Capacity (vc)	$(\frac{\alpha_s d}{b_o} + 2)\sqrt{f'c}$	0.264 ksi
α_s	For edge columns	40

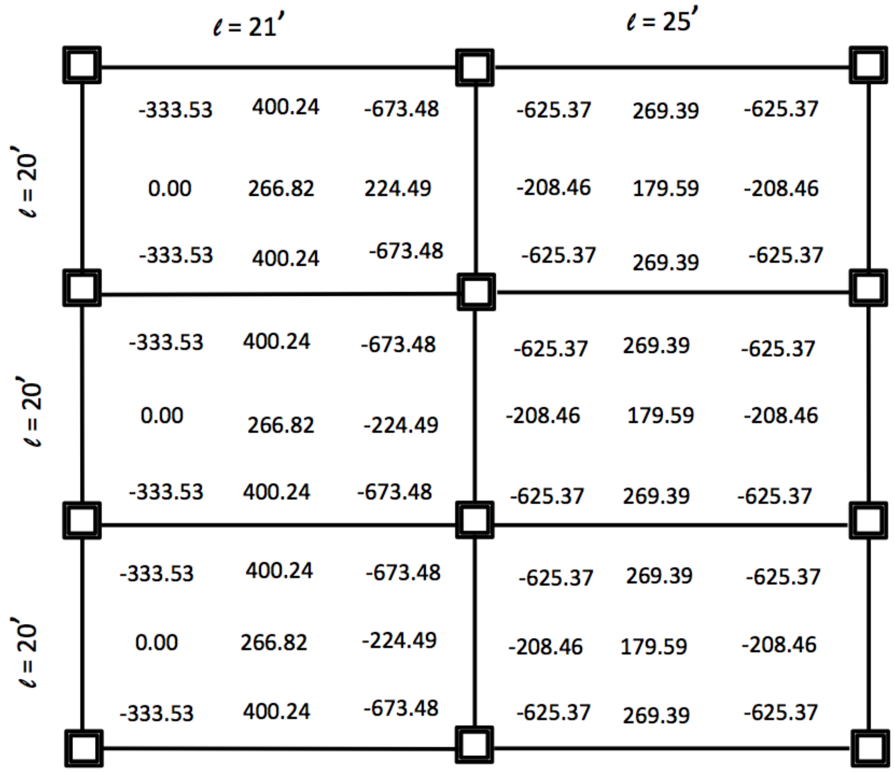


Figure 38: Moment Distribution Along Waffle Slab First Floor[23]

Figure 38 represents the moment distribution along the spans of the first floor waffle slab. These moments are distributed throughout the entire slab of the first floor of the building based on symmetry.

With the results of these calculations and with the appropriate checks for the slab and columns it was determined that both the waffle slabs and the concrete columns on the building would be able to support the additional load of the green roof. Appendix C.5 show the resulted moment calculations on the slab and the appropriate check based on its capacity.

It is important to notice that the seismic loads acting on the building do not affect the existing structure. As an additional load is imposed on the roof of the Gordon Library, the extra load is going to make the effective seismic weight to increase.

[23] Moment values in Figure are in kips*feet.

CHAPTER 7: SOLAR COLLECTOR INSTALLATION ON STODDARD B

This chapter contains information on the specific type of solar collector technology chosen for Stoddard B. The technology was chosen based on ease of installation, energy production, and cost. Additionally, this chapter contains information about the rooftop layout and construction process for the installation of solar collectors. Such information includes the number of solar collectors, dimensions of the technology, as well as the specific location on the roof where the technology should be installed. Finally, this chapter contains associated structural analyses and minimum design specifications for the building's members. Since no blueprint with dimensions was obtained, a number of assumptions were made for the structural analyses as well as external research.

7.1 Selected Solar Collector Technology on Stoddard B

For the application of solar collectors on Stoddard B, evacuated tube solar collectors were chosen because of their efficiency, ease of installation, and insulation properties. Apricus is a leading designer and manufacturer of solar hot water and hydronic heating products. After researching their products, the ETC-30 model was chosen for the application of evacuated tube solar collectors. This model contains 30 double-glass solar tubes and is often used for commercial, rather than residential, projects (Apricus, 2016). For flat roofs, like Stoddard B, the solar collectors must be angled facing south in order to absorb the most amount of sunlight. The ETC solar collector converts sunlight into usable heat, heating the liquid in the header pipe. If the temperature in the header pipe is measured to be hotter than the water in the bottom of the solar tank, then the pump turns on. The liquid is slowly circulated through the header pipe in the collector, heating by approximately 13°F during each pass. Gradually throughout the day, the water in the solar tank is heated up, since hot water is less dense than cold water, the water at the top of the solar tank is distributed out to either a boost tank, or directly to the user (Apricus, 2016). Figure 39 displays the ETC solar system operation. Table 55 contains information on the type, size, weight, energy production, lifespan, and costs of the chosen Apricus evacuated tube solar collector.

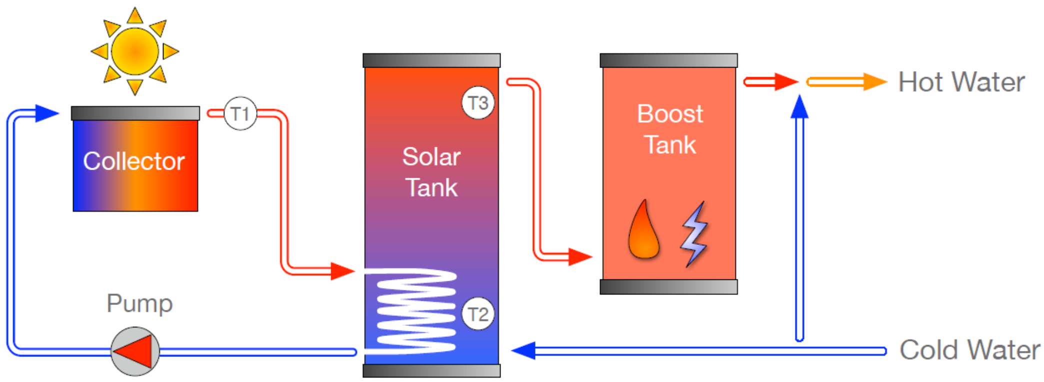


Figure 39: Evacuated Tube Solar Collector (ETC-30) Operation (Apricus, 2016)

Table 59: Type of Solar Collector Technology Information (Apricus, 2016)

Building	**Stoddard B**
Sustainable Rooftop Technology	Evacuated Tube Solar Collectors
Manufacturer	Apricus Solar Hot Water
Model	ETC-30
Dimensions	78.9" x 86.4" x 5.3"
Gross Area	47.34 square feet
Weight	209 lbs
Energy Production	12 kW/day (1kW = 7.5 gallons)
Lifespan	25 years
Cost of Technology	$3,080/collector (includes pump and solar tank)
Installation Cost	$1,000 (plumbing/tank installation)
	$70/hour (collector installation)
Annual Operational/Maintenance Cost	$50

7.2 Layouts and Construction Process for Solar Collectors on Stoddard B

This section contains information on the layout and construction process for the installation of solar collectors on Stoddard B. To determine the layout of the solar collectors, the number of solar collectors to produce the water consumption value of Stoddard B was calculated. According to the WPI Facilities Department, the annual water consumption of Stoddard B is

991,419 gallons. The chosen Apricus evacuated tube solar collector produces 32,850 gallons of water per year. By dividing the annual water consumption value of Stoddard B by the annual water production value of one solar collector, it was determined that Stoddard B would require at least 31 collectors to produce enough water for the entire building. All collectors will be angled at 40° above the horizontal and facing south. Table 56 contains information on energy, water, cost, number of collectors, and total area of collectors. Stoddard B contains two rectangular sections on its roof. The proposed design has 15 collectors on one section, and 16 collectors on the other section as seen in Figure 40.

Table 60: Installation of Solar Collectors on Stoddard B Information

STODDARD B	Current	Installation of Evacuated Tube Solar Collectors
Annual Energy Consumption/Production	232,800 kWh	4,380 kWh/panel
Annual Water Consumption/Production	991,419 gallons	32,850 gallons/panel
Annual Cost of Energy Paid/Saved	$32,592	$613.20/panel
Number of Collectors	31 collectors	
Total Area of Collectors	1,467.54 square feet	

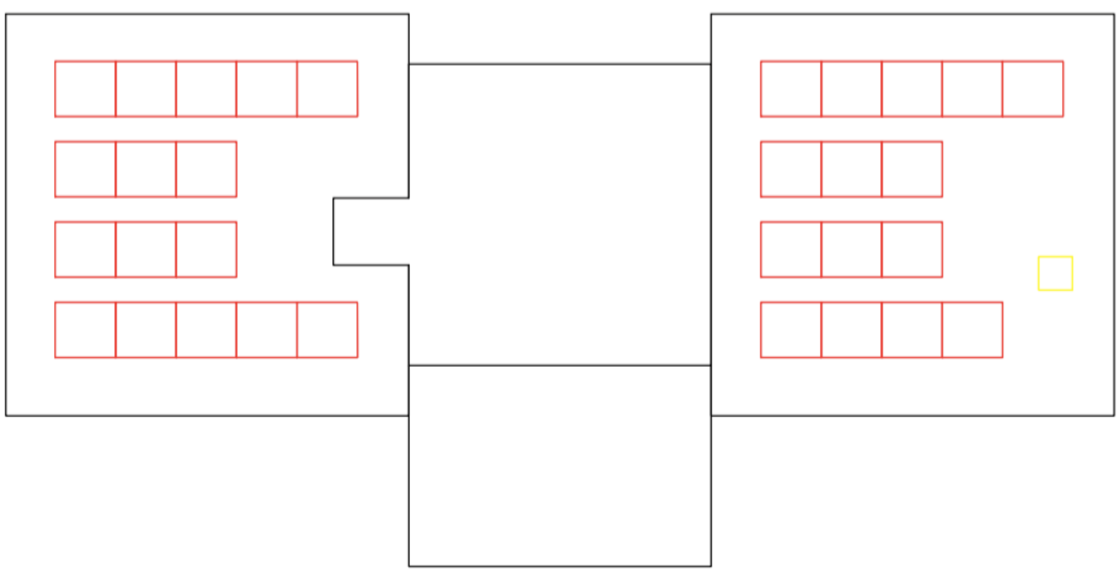


Figure 40: Solar Collectors Roof Layout

Table 57 contains information on the construction process for Apricus Evacuated Tube Solar Collectors. Figure 41 displays the mount's anchor that connects to the roof itself and some design requirements. Figure 42 shows the design of the mounting frame (as well as assembly instructions), maintenance and safety precautions.

Table 61: Construction Process for Apricus Solar Collectors (Apricus, 2008)

System Design	Mounting Frame	Maintenance and Safety Precautions
1) Installed at an angle between 20° and 80° above the horizontal	1) All Apricus solar collectors are supplied with a standard frame	2) Under no conditions, the Apricus ETC-30 system is maintenance free
2) Installed facing south with a deviation of up to 10°	2) Figure 41 below displays the roof attachment that should be followed for a flat roof.	2) Draining of the manifold is required for maintaining the system
3) Collector should be positioned as close to the storage cylinder as possible	3) Flat roofs require a high angle frame, which provides adjustments from 30° to 50° above the horizontal	3) Leaves should be removed regularly to ensure optimal performance and prevent life hazard
	4) Figure 41 displays the Apricus solar collector high angle frame kit, including safety considerations	

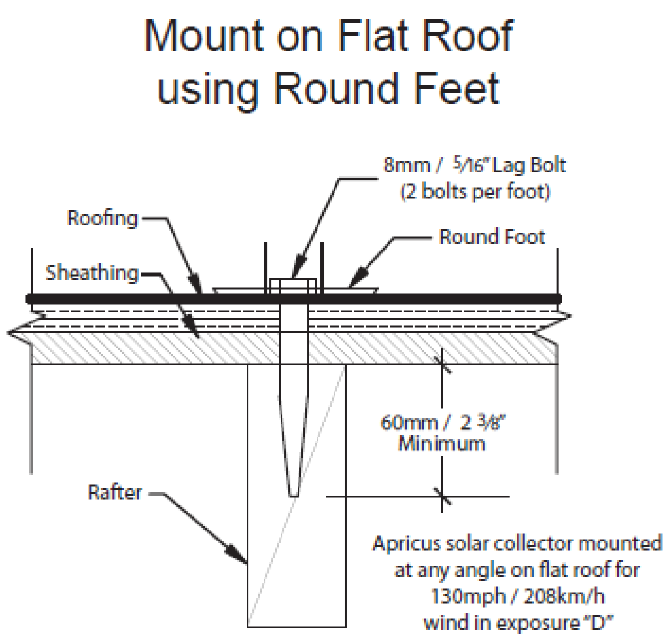
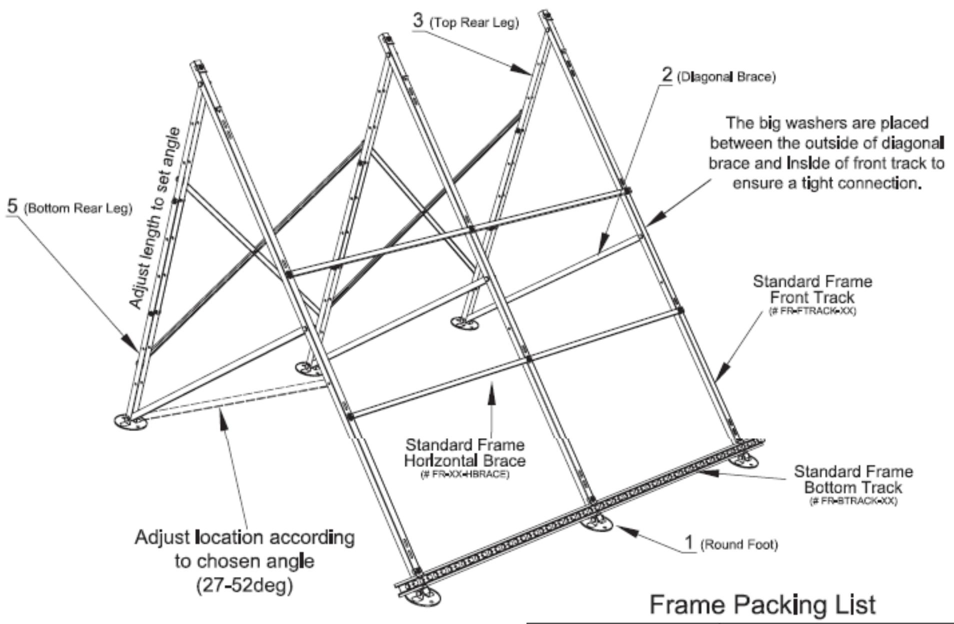


Figure 41: Attachment of Mounting Frame of Apricus ETC-30 (Apricus, 2008)

Apricus Solar Collector High Angle Frame Kit

Part #: FR-XX-HIGH-RFOOT

The components contained in this package combine with the standard frame to form the complete frame assembly shown below.



Frame Packing List

Part #	Component Quantities	
	10 & 20 Tube	22 & 30 Tube
1. FR-RFOOT	4	6
2. FR-DBRACE	2	3
3. FR-TRLEG	2	3
4. FR-RXB-HIGH-XX	2	4
5. FR-BRLEG	2	3
6. FR-BOLT-M8x50	*12*	*18*
7. FR-BOLT-M8x40	*1*	*4*
8. FR-BOLT-M8x20	*4*	*4*
9. FR-NUT-M8	*17*	*26*
10. FR-SWASH	*17*	*26*
11. FR-WASH-S	*29*	*44*
12. FR-WASH-B	*8*	*12*
13. FR-NLOCK	*5*	*8*
14. FR-SPAN-12/14	1	

Nuts and bolts are already attached to the appropriate components

SAFETY CONSIDERATIONS

- Wear gloves when handling frame components
- Feet must be bolted to ground
- Ensure attachment points are structurally sound
- Follow relevant safety regulations regarding working on roofs

Figure 42: Round Foot High Angle Frame Kit Assembly Diagram (Apricus, 2008)

7.3 Research and Estimates

In order to complete a structural analysis, the proper information about the building is required. This information requires a set of structural drawings with proper dimensions; unfortunately, the acquired drawings were not complete nor did they contain sufficient dimensions. In order to fill in the gaps as much as possible, extra research was conducted. This research involved reading relevant documents in WPI archives named *Stoddard Residence Center: Specifications* [24] as well as conducting as much measurements as possible in the actual building. Table 58 contains all the information obtained from this research.

Table 58: Information Obtained from Research

Stoddard Specifications		Measurements	
Concrete strength	3 Ksi	Columns Area (Measured)	Base= 12inches
Steel yield strength	60 Ksi		Height = 12 inches
Concrete cover	¾ inch	External Building Area (Measured)	Base= 48 feet
Slabs	Contain wire mesh		Height = 48 feet
		Beam Area (Eye estimate)	Base= 10 inches
			Height= 8 inches

This collected information was not sufficient to completely fill in the gaps, and estimates and assumptions had to be done to fill in the gaps in order to do a structure analysis. These estimates were done as objectively and as accurately as possible and the assumptions are the limitation of this aspect of the project. The assumptions made are regarding the building's members as well as their layout. The slabs were assumed to be a continuous one-way slab and have a metal deck weighting 3.5 psf and a MEP of 5psf; the beams were estimated to be 10 inches by 12 inches; the ties used for concrete were #3's; the columns are governed by axial forces; all members are the same as the ground's floor, with the exception of the roof slab; finally, the layout of the column and beam drawn and estimated as best as possible. All of these assumptions are found in Annex D.2.

[24] This book contains specifications for three dormitories for WPI to be known as the Stoddard Residence Center, Worcester, MA, O.E. Nault & Sons, Inc., Architects, May 1969 – includes Addendum Nos. 1, 2, and 3.

7.4 Structural Analysis and Member Designs for Solar Collectors on Stoddard B

After determining the layout and quantity of the Solar Collectors and compiling all the possible information on the building, the structure's analysis began. It started with determining the new loads acting on the flat roof. For the calculations, the side of the building with 16 collectors was chosen since it will introduce the largest load. After the new imposed load is a calculated, different member of the building are designed with all the loads acting on them, and these designs will establish the minimum requirement for each of the members. If any of the actual members are below these designs, the analysis is not adequate. The members designed consist of the first-floor slab, columns, beams, and the roof slab. The second and third floor members are assumed to have the same dimensions as the first floor for this analysis. The design of each member was done in the order that the weight is distributed along the building. To start, the new imposed load is directly above the roof slab, and then it is distributed into the beams. From the beams, the weight is distributed along the columns. The columns in turn rest on the first-floor slab.

7.4.1 Solar Evacuated Tubes Load Calculations

The first step in our analysis involved considering all loads acting on the solar panels: dead load, live load, rain load, snow load, wind load, and seismic load. For this system, the rain loads were considered negligible. Due to the 40° angle of the panels, all rain would runoff onto the roof and simply be drained. Live load was neglected since the system was not designed for people to walk on. Dead load was calculated by using the wet weight, area, and number of panels. Finally, snow load, wind load, and seismics load were calculated in accordance with the ASCE 7-10. In addition to the ASCE 7-10, wind and seismic load were calculated in accordance with documents from the Structural Engineers Association of California, which provided information specifically to solar photovoltaic arrays (Structural Engineers Association of California, 2012). All calculated design loads are shown below in Table 59, and all calculations can be found in Appendix D.1. Provided in the ASCE 7-10, there are seven load combinations that were considered when determining the governing load acting on the panels. These combinations accounted for both gravity and lateral loads. All load combination calculations were made in accordance with the Load and Resistance Factor Design (LRFD) method. The calculated load combinations are displayed in Table 60; the governing load for gravity effects is formula number three with a value of 81.55 psf.

Table 59: Calculated Design Loads for Solar Collectors

Loads	Value (psf)
Dead (D)	5.47
Snow (S)	34.65
Wind (W)	39.09
Seismic Horizontal (E_h)	0.9.92
Seismic Vertical (E_v)	0.21
Roof Live (L_r)	10
Live on technology (L)	0
Rain (R)	0

Table 60: Calculated LRFD Load Combinations per ASCE 7-10

Load Combinations		Value[12] (psf)
Gravity Loads	1.4D	7.66
	1.2D + 1.6L + 0.5(L/S/R)	39.89
	1.2D + 1.6(L_r/S/R) + (L/0.5W)	81.55
	1.2D + 1.0W + L + 0.5(L_r/S/R)	72.98
	1.2D + E_v + L + 0.2S	23.7
Lateral Loads	0.9D + 1.0W	44.01
	0.9D + 1.0E_h	14.84

7.4.1 Slab Design & Calculations

To begin the slab design, a sum of all the loads on top of the member is done and plugged into the governing load combination to get the factored load. This sum of forces includes the self-weight of the member. The next step is calculating maximum moment acting on the slab. In this case, for a continuous one-way slab with inner supports, the maximum moment occurs at the interior support. This moment represents the actual moment acting on the slab. From here on, different aspects of the slab were designed. Given the actual moment, an estimate of the effective depth, the allowed area of steel, and Whitney's stress block were made. This step can be thought of as a trial design. With the allowed area of steel, a bar is chosen with its required spacing using a book table[25]. This bar and spacing represent the actual area of steel per unit width. Given the actual area of steel, the effective depth and Whitney's stress block are recalculated to be more

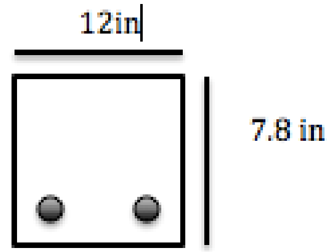
[25] Table A-9 found in the annex

accurate. The steel ratio is calculated and compared to the min and max to make sure that the reinforcement will yield before the concrete fails in compression. Finally, having a member design with every variable, the bending capacity ΦM_n of such slab is calculated. The moment capacity is compared to the design moment Mu and if it is larger than Mu, then the design for the slab can withstand the imposed load. Very similar to the moment, the shear Capacity ΦV_c and design shear Vu are calculated and compared. If all of these parameters are complied with, the proposed design is adequate to hold all of the loads acting on it.

This design was done twice, one for the roof slab as well as the first-floor slab. The slabs are very similar; the only difference is the reinforcement size and spacing. It is important to always have in consideration that this calculation is the minimum required design to withstand the load. Table 61 shows the design calculations for both slabs, and Figure 43 illustrates a cross section for the slab on the 1st floor. All calculations are found in Appendix D.2.

Table 61: Slab Values & Minimum Design

Roof Slab		**1ST Floor Slab**	
Design specification	Value	Design specification	Value
d-in	6.86	d-in	6.67
Thickness-in	8.0	Thickness-in	7.8
Area of steel- in^2	0.2	Area of steel- in^2	0.7
Bar number	3	Bar number	6
Spacing between bars-in	6.5	Spacing between bars-in	7.5
Min Steel Ratio (ρ_{min})	0.0018	Min Steel Ratio (ρ_{min})	0.0018
Design Steel Ratio (ρ)	0.0024	Design Steel Ratio (ρ)	0.009
Max Steel Ratio (ρ_{max})	0.016	Max Steel Ratio (ρ_{max})	0.016
Design Moment M_u- kips *ft	5.69	Design Moment M_u- kips *ft	18.33
Moment Capacity $ØM_n$- kips *ft	6.0	Moment Capacity $ØM_n$ - kips *ft	18.9
Design Shear (Vu) -kips	1.76	Design Shear (Vu) -kips	5.67
Shear Capacity ΦV-kips	6.7	Shear Capacity ΦV-kips	6.7
Whitney's Stress Block (a) -in	0.39	Whitney's Stress Block (a) -in	1.37



Cross section of slab with #6 bars spaced every 7.5 inches

Figure 41: Slab Cross-Section for 1st Floor

7.4.2 Beam Design & Calculations

After completing the slabs designs, the next members to be looked were the beams. For this section, it is important to remember that the beam measurements were estimated by eye since there was no way of measuring it properly. The layout of the beams is shown in Figure 44, with the X being the columns and the beams being the light blue rectangles. Similar to the slab design process, the factored load was calculated by summing all of the dead loads and live loads acting on it, including self-weight, and plugging them into the governing factored load combination. For a simply supported beam, the maximum moment formula is known. The area of steel is assumed to be close to the slab's so a proper bar placement can be taken out from a book table[26]. For this procedure, the steel stress is assumed to equal the specified minimum yield stress, meaning that the steel ratio is within parameters. After choosing the bar number and actual area of steel, the effective depth and Whitney's stress block are calculated. With the entire dimensions at hand, the initial assumption is checked to make sure that the beam is tension-controlled to prevent brittle effects. After making sure the assumptions are correct, the final step is to calculate the moment capacity ΦM_n and make sure it is larger than the design moment M_u. Having done this whole procedure, the design for a simply supported beam is adequate if the allowed moment and net tensile strain are larger than the actual moment and yield strain in tension, respectively.

This design was done once for the first-floor beams and assumed all other beams were the same. If the actual beams have other dimensions and bars, the design should be redone in

[26] Table A-8 of the annex.

order to get the allowed moment. Table 62 shows the design values and Figure 45 illustrates a cross section of the designed beam. All of the calculations are found in Appendix D.2.

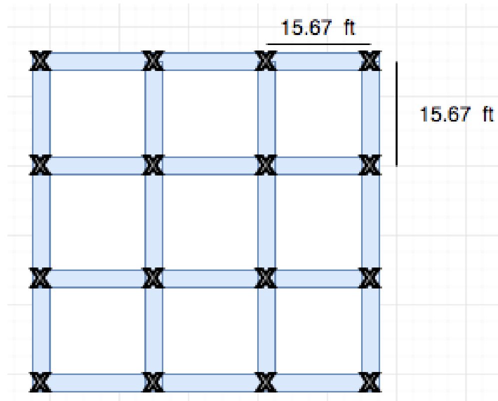


Figure 42: Beam Layout

Table 62: Beam Values and Minimum Design

First floor beams	
Design specification	**Value**
Factored Load (W_u) -psf	13.6
Area of steel (A_s)- in^2	0.79
Bar number	8
Thickness-in	10
Width (b) -in	8
Whitney's Stress Block (a) -in	2.3
Net tensile strain (ε_t)	0.0.0074
Steel Yield strain in tension (ε_y)	0.00207
Moment Capacity (ΦM_n) –kips*ft	24.3
Design Moment (M_u)- kips *ft	9.78
Safety Factor (Φ)	0.9
Steel Ratio (ρ)	0.0123

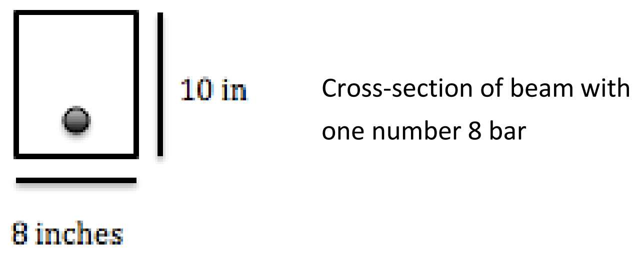


Figure 43: Beam Cross-Section for 1st Floor

7.4.3 Column Design & Calculations

The last members to be considered for design purposes were the columns. For this design, the layout of the columns is estimated to be 15.67 feet away from one another. Figure 46 shows the assumed layout with the tributary area. The columns were measured by hand to be 1foot by 1 foot. For the design of this member, the columns are considered to be governed by axial forces. Similar to the other members, the first step is calculating the factored load. Summing all the forces acting on the member, including self-weight, and plugging them in to the governing load combination gave the calculation. The design accounts for the column load effects from other floors. The layout of this column results with three different types of columns (Edge, Corner, and Middle), the difference being in the tributary area. The different columns are shown as E, C, and M in the layout. The column with the largest axial load Pu acting on it will be the governing one and the one that will be designed. The axial load is easily calculated by multiplying the factored load by the tributary area; the middle columns had the largest axial load acting on it. For the next step of the design, the axial load is considered to be equal to the axial load capacity ΦP_n. This results in a formula that allows the calculation of the required area of steel. Next, the layout of the reinforcement is chosen from the table A-8 of the book found in the annex. The chosen area of steel has to be similar to the required one; this selection will also provide the number of bars and quantity of bars in the column. In this case, two reinforcement arrays were selected which are acceptable: four # 7 bars, and two # 9 bars. After selecting the reinforcement, the steel ratio has to be checked to be within parameters. This ratio has to have a minimum of one to two percent. If it is within this range, the proposed design is acceptable. To finish the design, the ties and spacing were calculated. This step is fairly simple since the tie number is known and the spacing is the minimum of three straightforward equations.

This design was done once for the first floor columns and assumed all the others were the same. If the columns in any of the floors have other dimensions and reinforcement, the design should be redone in order to get the minimum size and reinforcement. Table 63 shows the design values and Figure 47 illustrates a cross section of the designed beam for the design with the # 7 bars. All of the calculations are found in Appendix D.2.

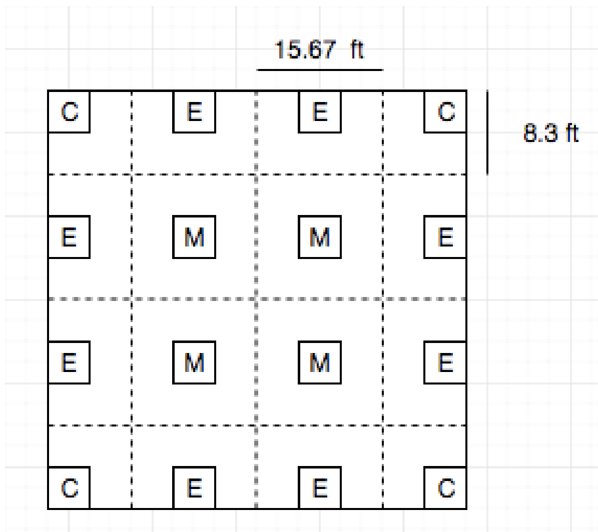
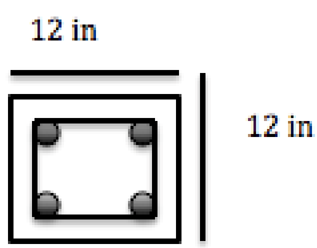


Figure 44: Assumed Column Layout

Table 63: Column Values & Minimum Design

First floor columns		
Design specification	**Value**	
Area (b x h)- ft^2	1	
Factored Load (W_u) -psf	544.62	
Governing Axial Load -kips	133.7	
Area of steel required (A_s)- in^2	1.92	
Bar number 7	Layout	4 bars
	Area of Steel (A_s)-in^2	2.40
Bar number 9	Layout	2 bars
	Area of Steel (A_s)-in^2	2.0
Steel Ratio (ρ) bar number 7	0.0167	
Steel Ratio (ρ) bar number 9	0.014	
Minimum steel ratio (ρ) -%	1-2	
Ties	#3	
Ties Spacing -in	12	
Safety Factor (Φ)	0.8	



Cross section of column with four #7 bars and #3 tie

Figure 45: Column Cross-Section for 1st Floor

CHAPTER 8: ECONOMIC ANALYSIS

This chapter contains information on the economic analysis to determine whether it is feasible to implement the chosen sustainable rooftop technologies. This chapter provides the values and results obtained from the calculation process outlined in section 3.5 of the Methodology chapter.

8.1 Economic Analysis of Solar Panels on the Gateway Parking Garage

The overall cost of the proposed solar panel design was calculated by adding the total cost of the following variables: steel framework, added 2 ft x 2 ft concrete columns, reinforcement within the concrete columns, and solar panels. The overall cost of the proposed solar panel design as well as the operational cost were compared to the net annual energy savings of the Gateway Parking Garage to determine how many years it will take to pay off the solar panel design and begin making a profit. These were compared since the chosen number of solar panels can produce the annual energy demand of the structure.

8.1.1 Total Cost of Steel Framework

The total weight of the primary steel members was calculated, as well as the total weight of the miscellaneous items in the framework, which includes steel plates, studs, and connections. The total weight of the miscellaneous steel was equal to 10% of the total weight of the primary members. The total weight of the steel members and miscellaneous items are shown in Table 64 below. The total cost of the steel framework was calculated using the unit costs provided in Table 65 below. The costs include unit cost values for labor, materials, and equipment. Labor rate accounts for the workers constructing and installing the steel framework, material rate accounts for the steel members and miscellaneous items, and equipment rate accounts for the tools used to construct the steel framework.

Table 64: Total Weight of Steel Members and Miscellaneous Items

Member Size	Member Weight (lb./ft)	Member Length (ft)	Quantity	Total Weight (tons)
W24 x 55	55	45.69	4	5.03
		28.21	2	1.55
W24 x 68	68	45.69	10	15.53
		28.21	5	4.80
W30 x 90	90	56.02	7	17.65
W30 x 108	108		1	3.03
W8 x 31	31	6.33	2	0.20
		14.26	2	0.44
		22.2	2	0.69
		27.1	2	0.84
TOTAL WEIGHT OF STEEL MEMBERS = 49.75 tons				
ESTIMATED TOTAL WEIGHT OF MISC. STEEL/PLATES/STUDS/CONNECTIONS = 4.97 tons				

Table 65: Total Steel Framework Cost

Steel Framework	*Steel Members*	*Misc. Steel/Plates/Studs/Connections*
Total Weight (tons)	49.75	4.97
Labor Unit Cost ($/ton)	400	400
Material Unit Cost ($/ton)	3,000	3,400
Equipment Unit Cost ($/ton)	200	200
Total Unit Cost ($/ton)	3,600	4,000
Total Cost ($)	179,100	19,880
TOTAL STEEL FRAMEWORK COST = $198,980		

8.1.2 Total Cost of Added 2 ft x 2 ft Concrete Columns

The total cost of the five added 2 ft x 2 ft concrete columns was calculated by multiplying the number of added concrete columns by the labor, material, and equipment unit costs for the installation of 24” x 24” concrete columns. Table 66 displays the cost data and total cost of the added concrete columns.

Table 66: Total Concrete Column Cost

Number of Added Concrete Columns	5
Labor Unit Cost ($)	400
Material Unit Cost ($)	241
Equipment Unit Cost ($)	32
Total Unit Cost ($)	673
TOTAL CONCRETE COLUMN COST = $3,365	

8.1.3 Total Cost of Reinforcement Within Concrete Columns

In order to support the steel framework columns, 6 #9 steel rebar was recommended to be placed within all eight concrete columns, including the three existing concrete columns. The material cost, labor cost, and equipment cost were determined and added together to produce the total cost of reinforcement within the concrete columns. The results are shown in Table 67 below.

Table 67: Total Cost of Reinforcement Within Concrete Columns

Number of Concrete Columns	8
Number of #9 Steel Rebar per Column	6
Material Cost ($)	64.50
Labor Cost ($)	60.50
Equipment Cost ($)	15.35
TOTAL STEEL REBAR COST = $6,736.80	

8.1.4 Total Cost of Solar Panels

The total cost of solar panels was based on the SPR-P17-350-COM model from the manufacturer SunPower. The total cost is displayed in Table 68 below, which is based on the unit cost of the technology and unit installation cost provided by SunPower. The unit cost of the technology and unit installation cost are values based on the Northeast region of the United States.

Table 68: Total Cost of Solar Panels

Number of Solar Panels	272
Cost of Solar Panels ($)	172,856
i. Unit Cost of Technology ($/panel)	635.50
Unit Installation Cost ($)	380,800
i. Panel Energy Production (watt/panel)	350
ii. Price per Watt ($/watt)	4.00
TOTAL SOLAR PANEL COST = $553,656	

8.1.5 Economic Analysis Results

The total cost of the steel framework, added concrete columns, steel rebar, and solar panels were added together to produce the overall construction cost of the proposed solar panel design. This is outlined in Table 69 below.

Table 69: Overall Cost of Proposed Solar Panel Design

Total Steel Framework Cost ($)	198,980
Total Concrete Column Cost ($)	3,365
Total Steel Reinforcing Rebar Cost ($)	6,736.80
Total Solar Panel Cost ($)	553,656
TOTAL COST = $762,738	

The net annual energy savings of the Gateway Parking Garage was calculated using the total annual solar panel energy production value as well as the energy operational cost obtained from the WPI Facilities Department. The simple payback period of the proposed solar panel design was calculated to determine if it is economically feasible for WPI to invest in this sustainable rooftop technology. The simple payback period of the proposed solar panel design is shown in Table 70 below.

Table 70: Simple Payback Period of the Proposed Solar Panel Design

Net Annual Energy Savings ($)	19,459
i. Annual Energy Demand (kWh)	137,207
ii. Annual Panel Energy Production (kWh)	138,992
iii. Cost per kWh of Energy ($/kWh)	0.14
Annual Operational Cost of Solar Panels ($)	2,593
Lifespan of Solar Panels (years)	25
Total Installation Cost of Design ($)	762,738
NUMBER OF YEARS TO PAY OFF DESIGN= 42 years and 7 months	

Based on this result, it is not economically feasible for WPI to invest in the proposed solar panel design. The number of years to pay off the design is much higher than originally expected. In addition, the lifespan of the solar panels is 25 years, which is over 15 years prior to when WPI would begin making a profit. This would require a new added solar panel cost, in addition to operational costs, taking it even longer for WPI to become profitable. The main contributing factor to the large cost of the design is the price of the solar panels. The cost for one solar panel is $635.50, and our proposed design contains 272 panels to produce energy for the entire parking garage.

8.2 Economic Analysis of Green Roof Installation on Gordon Library

8.2.1 Material and Labor

Implementing green roof systems can have many benefits on a building, including an overall energy reduction. However, the main concern comes when comparing the estimated costs of a normal flat roof and a green roof system on a structure. Even though green roofs have a vast quantity of benefits for the building and the environment, owners are not ready to assume such a high extra cost to build up this technology. A typical built-up flat roof can be extremely inexpensive versus a green roof. RS Means Software estimates the cost of a commercial roof without green technology to be around $5- $7 per square feet of construction. On the other hand, the estimate cost of a green roof ranges from $14 -$15 per square foot, in national averages. The additional $10 per square foot that this system costs includes, materials, installation process and labor.

A green roof system for a building such as the Gordon Library would have the estimated costs of construction and maintenance as shown in Table 71. This table illustrates the cost for the installation of a green roof on Gordon Library, assuming an area of 10,733 square feet. For the first days of green roof installation, some additional costs may be incurred due to the extra labor needed for an establishment period[27]. The annual maintenance of green roofs typically includes fertilization, weeding, drain inspection and removal of debris. [28]

Table 71: Costs of Installation and Maintenance for Green Roofs

Description	Unit Cost (per square foot)	Cost of System
Material & Installation	$15	$160,995.00
Maintenance/Annual	$0.27	$2,897.91
Total First Year	$15.27	$163,892.91

The typical costs for installation and maintenance of a conventional roof needed for the Gordon Library are shown in Table 72 below. This cost is given because at some point in time the building will need to have a full roof reconstruction, typically in 15-20 years (average service life of a roof). The area of the roof for the Gordon Library is 172’0”x 92’10”, which is equal to 14,002.38 square feet[29]. This roof area is different than the area of the green roof because not all the roof will include the green roof technology.

Table 72: Costs of Installation and Maintenance Gordon Library Roof

Description	Unit Cost (per square foot)	Cost of System
Material & Installation	$7	$98,016.66
Maintenance/Annual	$0.13	$1,820.31
Total First Year	$7.13	$99,836.97

This cost is assuming that there is a good and constant maintenance of the roof. If maintenance of the roof is not constant the price in the table can decrease but also the overall service life of the roof would decrease. The following conditions can be applied:

1. If the roof has no maintenance, the service life that is typically around 20 years can be reduced in half. This at the end will require an additional cost of the roof

[27] When vegetation of the green roof is in place and roots of plant start growing and adapting
[28] https://facilityexecutive.com/2017/10/greening-the-roof/
[29] The area of the roof does not include the penthouse system located on top of the roof.

2. If some maintenance is done, the roof will last a little less than 20 years, however the cost for maintenance is going to decrease as well.
3. If full maintenance is done, the average service life of the roof will be around 20 years, and its maintenance cost will be as shown in Table 72. At year 20 or less, there needs to be a full corrective and maintenance of the entire roof, which due to inflation and change in prices, it will typically cost more than the original cost of the roof shown in Table 72.

In addition, the cost of a commercial roof can change based on local contractor prices, type of labor used and other conditions. There is not an actual price for the reconstruction of a roof, but an average can be obtained from these values.

The main benefit of a green roof technology is the ability to increase twice the service life of a conventional roof. This would allow WPI to save some costs on full corrective and maintenance of the roof at year (20), which at the end is the biggest saving.

8.2.2 Stormwater Benefits

Some other savings and benefits occur as part of installing a roof garden on the Gordon Library, including stormwater benefits. Based on the building type (commercial) for Gordon Library, an estimate of $0.004 per square foot per year can be used for utility benefits for stormwater retention systems. This would be the best case-scenario for this particular saving due to stormwater retention. This value is not much and is not going to be used in cost reduction of the system. However, if this value can increase due to future technology, savings might be considerable high.

8.2.3 Energy and Insulation

The most significant cost benefit of a roof garden is the ability to reduce the overall energy consumption of a building by temperature control. For the Gordon Library, the green roof technology is going to act as an additional insulator of the roof and not as a replacement for insulation, creating energy savings for the building. As the information regarding annual energy consumption of the Gordon Library was not provided, an exact analysis of the savings was not done.

Implementing a green roof technology on the Gordon Library will reduce annual energy consumption by 12% for all types of structures (Andresen, et. al., 2004). The average cost of energy in Massachusetts is approximately 13.84 cents per kWh. With a 12% reduction of this the cost of energy would be around 12.18 cents per kWh.

On average, college and universities in the United States consume 18.9 kWh of electricity per square foot annually ("Managing Energy Costs in College and Universities"). This means than on average Gordon Library is consuming over 1,200,000 kWh annually, with an approximate cost of $156,921. Assuming an energy reduction of 12%, the annual savings for the building will be around $15,000 to $20,000. This is assuming that a constant energy reduction of 12% is applied for every month of the year. However, as this may not be true for certain months, a conservative value for annual savings would be around $10,000 to $12,000 a year. Based on these numbers it will take around 12 to 15 years to payback the actual cost of the green roof. However, this is only assuming the energy reduction savings by installing a green roof technology. If all the factors are considered, the payback period can be between 5 to 7 years. This is assuming that the reconstruction of the current roof of the Gordon Library includes the green roof technology. This value was obtained by concluding that at some point a full reconstruction of the roof is needed, and implementing a roof garden will be more expensive but it will increase the service life of the roof.

8.3 Economic Analysis of Solar Collectors on Stoddard B

This economic analysis results in the number of years required for the technology to pay itself off. The steps for this calculation is simply dividing the initial cost that WPI will have to spend by the annual savings that the technology will provide.

8.3.1 Fixed Cost

The first step in this analysis is determining the fixed cost of the system, which is the installation cost plus the price of each solar collector. The installation cost for the tank is known and the installation cost for the panels is $70 an hour. A period of two weeks is assumed for installation purposes and an estimate was calculated. The price of technology was fairly simple: multiplying price of panels times number of panels needed. These two values were added to get the fixed cost. The formulas and process for this process is listed in Table 73.

Table 73: Initial Cost of Solar Collectors

Step 1: Fixed Cost	
Variable:	**Reference/*Equation*:**
Number of Solar Panels	31
Cost of Solar Panels - $	*Unit Technology Cost*Number of Solar Panels*
i. Unit Cost of Technology ($/panel)	SunPower Corporation
	3,080
Installation Cost ($)	*Panel Energy Production*Unit Installation Cost*Number of Solar Panels*
i. Tank installation	Apricus Corporation
	1,000
ii. Collector installation (per hour)	Apricus Corporation
	70
	80 hours assumed
Cost of Solar Panels + Installation cost -$	

8.3.2 Savings Per Year

After having the fixed cost, the next step is calculating the total savings per year of WPI. The first step involved calculating the annual savings by subtracting the amount saved per year from energy by the maintenance cost per year. Finally, the payback period was calculated by adding the overall cost and the installation cost and dividing it by the annual savings amount. The formulas and process for this process is listed in Table 74.

Table 74: Annual Savings of WPI with Solar Collectors

Step 2: Annual Savings	
Variable:	**Reference/*Equation:***
Annual Energy Demand – kWh	WPI Facilities Department
	232,800
Energy Operational Cost - $/kWh	WPI Facilities Department
	0.14
Annual cost of energy	Annual Energy Demand* Energy Operational Cost
Annual Savings of system - $/ panel	613.20
Annual Total Savings of collectors	Number of Collectors*Savings per panel
Annual Maintenance Cost- $	50
Annual Total Savings of System – Annual Maintenance Cost	

8.4 Payback Period

To get the simple payback period, the annual savings of WPI is divided from the fixed cost. This value represents the number of years that the technology will take to pay itself off in savings. This number is compared with the remaining lifespan of the technology to finalize economic feasibility. The formulas and process for this process is listed in Table 75.

Table 75: Payback Period of Solar Collectors

Step 3: Simple Payback Period of the Proposed Solar Collector	
Variable:	**Reference/*Equation:***
Fixed Cost -$	Following step 1
	102,080
Savings per year -$	Following step 2
	18,959.3
Fixed Cost/Annual Savings - years	

The proposed technology will take 5.3 years to pay itself off and has a lifespan of 25 years. Meaning that the technology has a little less than 20 years to continue helping WPI saving a lot of money that is currently spending annually. This proposed system is very feasible and could aid WPI with its feasibility plan. All calculations are found in Appendix D.3.

CHAPTER 9: CONCLUSIONS

Our recommendation would be to redesign the solar panel system with less solar panels to reduce the overall cost. However, by doing this would not produce energy for the entire structure and a portion of the annual $19,209 of energy demand would have to be paid in addition to the cost of the solar panel design. A cost analysis would have to be done to determine how to gain a profit in the shortest time period. This would require analyzing how many solar panels would be needed to produce a certain percentage of energy demand required by the Gateway Parking Garage. Our second recommendation would be to choose a different model and manufacturer of solar panels, which are sold at a lower cost. However, the quality and energy production of these panels might not compare to the chosen SunPower model. For large-scale applications, such as the Gateway Parking Garage, it might not be economically feasible to install solar panels to produce energy to the entire structure.

As shown in this analysis, it would be economically feasible to implement a green roof on the Gordon Library, although many factors need to be considered. It is expected that the price of construction for green roofs is going to decrease throughout the years. As this technology becomes more popular in the country, material and labor costs will decrease. This will make the cost per square foot of the system more affordable and possibly get around the cost of a conventional roof. In addition, it will be more appropriate to implement the green roof technology when full corrective roof maintenance is needed in the building. This will reduce the additional cost of installing a green roof over a roof that is still working and in good shape and will increase the service life of the entire roof.

Like the green roof technology, it is economically feasible to invest in solar collectors for Stoddard B. According to calculations, the payback period of this technology is roughly 5.4 years, meaning that after that period the technology will start saving money for WPI until it reaches the lifespan. Annually, this system could save WPI nearly $19,000 after the payback period.

It is important to note that all these sustainable technologies are not extremely common in the United States. Even though their installation has increased dramatically in the last couple of years, the price for each technology is expected to decrease in the near future. This would reduce the overall cost for each sustainable rooftop technology: solar panels, solar collectors, and green roofs. In addition, tax incentives for implementing sustainable technologies should be determined to get the most approximate cost of installation. Each State will have a different tax credit for the installation of sustainable technologies, as well as a federal tax credit which can be applied. For example, for Massachusetts, the tax credit for implementing green roofs is 9.5%. However, WPI is tax exempt so tax incentives would not apply for sustainable rooftop technologies at WPI. All of these factors contribute to the overall price of the technology and whether or not it is feasible for implementation. Future work for this MQP would involve redesigning the solar panels on Gateway Parking Garage to make the installation economically feasible. This would require reducing the number of solar panels, or choosing a different solar panel manufacturer. Since the roof of the Gordon Library was recently reconstructed, it would be recommended to wait a couple of years until the roof begins to deteriorate to install a green roof, since a green roof will protect and increase the service life of the roof. Finally, in order to properly install the solar collector system, an analysis by a licensed professional structural engineer has to be done with adequate structural drawings.

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APPENDIX A: PROJECT PROPOSAL

WORCESTER POLYTECHNIC INSTITUTE

Sustainable Roofing Practices

A Major Qualifying Project submitted to the Faculty of Worcester Polytechnic Institute in partial fulfillment of the requirements for the Bachelor of Science degree

by

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Date

10/12/17

Proposal Submitted to

Leonard Albano

Worcester Polytechnic Institute

Abstract

This project will evaluate the feasibility of the installation of sustainable roofing practices on selected buildings at Worcester Polytechnic Institute (WPI). This report includes the structural analysis and design of three sustainable rooftop technologies: solar panels, green roofs, and solar collectors. These methods have the ability to alleviate the urban heat island effect, while contributing to WPI's sustainability plan. Additionally, an economic evaluation using a life-cycle cost analysis will be prepared to show the incentives for installing these sustainable rooftop technologies.

1 The Problem

This section contains an introduction to sustainable rooftop technologies, and their ability to mitigate global environmental problems. Additionally, this section lays out the goals and objectives for this project.

1.1 Problem Statement

Climate change, air pollution, and water pollution are a few of many environmental problems that the world is dealing with today. Specifically in urban areas, the heat island effect is another problem which is increasing temperatures. The negative impacts from the heat island effect in urban cities include an increase in energy usage, increase in gas emissions, impaired water quality, and health risks. It is the responsibility of our generation to explore ways to preserve the environment for future generations. Implementing sustainable rooftop technologies is one practice which can help reduce some of the environmental problems the world is dealing with today. Sustainable rooftop technologies include solar panels, solar collectors, green roofs, stormwater retention systems, and daylighting systems. All of these systems use the source of the problem, the sun, as a way to reduce environmental problems. Our objective is to explore three rooftop technologies, and investigate the structural impact these systems can have on buildings at Worcester Polytechnic Institute (WPI). The three technologies we have chosen are solar panels, green roofs, and solar collectors.

1.2 Goals and Objectives

The goal of this project is to provide recommendations and improvements for the installation of sustainable rooftop technologies on existing buildings at WPI. Additionally, we will investigate the impact of these technologies on the net energy demands. The objectives for this project include:

7. Determine the approach WPI has towards sustainable practices, as well as its current sustainable building practices.
8. Identify candidate buildings at WPI for the installation of certain sustainable rooftop technologies.
9. Perform an engineering analysis of the selected buildings.
10. Perform an energy analysis to determine the sustainable rooftop system which will result in the greatest reduction of energy usage.

11. Outline structural design activities for the selected buildings, which includes identifying structural reinforcements needed to withstand sustainable rooftop technologies.
12. Conduct a life-cycle cost analysis to determine whether it is economically feasible to implement sustainable rooftop technologies at WPI.

2 Background

This section provides information on the heat island effect, which is an environmental problem. The heat island effect can be reduced in urban areas through sustainable roofing practices. Additionally, this section contains background information on various sustainable rooftop technologies: solar panels, solar collectors, green roofs, stormwater retention systems, and day lighting systems.

2.1 The Heat Island Effect

The heat island effect describes urban regions which become hotter than its rural surroundings due to urban area development of buildings, roads, and other infrastructure which replaces open land and vegetation. The annual mean temperature of a city with one million people or more can be 1.8°F warmer than its surroundings. However, the temperature difference can be as much as 22°F during the nighttime due to the buildup of heat on infrastructure from the sun during the day, which is slowly released throughout the night. Shaded or moist surfaces in rural areas remain close to air temperatures. Elevated temperatures in urban areas can negatively impact a community's environment and quality of life (United States Environmental Protection Agency, 2017).

2.1.1 Negative Impacts

Some of the negative impacts of the heat island effect include increased energy consumption, elevated emissions of air pollutants and greenhouse gases, compromised human health and comfort, and impaired water quality (United States Environmental Protection Agency, 2017):

5. ***Increased Energy Consumption:*** When the temperature rises in urban areas during the summertime, there is an increase of energy demand for cooling. Starting from 68-77°F, the electricity demand for cooling increases 1.5-2.0% for every 1°F increase in air temperatures (United States Environmental Protection Agency, 2017).
6. ***Elevated Emissions of Air Pollutants and Greenhouse Gases:*** The burning of fossil fuel increases air pollutants and greenhouse gas emissions. Fossil fuel power plants are used to supply electricity, which in turn emit sulfur dioxide, nitrogen oxides, particulate matter, carbon monoxide, mercury, and carbon dioxide. All of these pollutants are

harmful to human health and contribute to air quality problems including smog, fine particulate matter, acid rain, and global climate change.

7. ***Compromised Human Health and Comfort:*** High temperatures affect human health and contribute to discomfort, respiratory difficulties, heat cramps and exhaustion, non-fatal heat strokes, and heat-related mortality. The Centers for Disease Control and Prevention estimated from 1979-2003 that excessive heat exposure contributed to more than 8,000 premature deaths in the United States (United States Environmental Protection Agency, 2017).
8. ***Impaired Water Quality:*** High pavement and rooftop surface temperatures can heat stormwater runoff. Tests have shown that 100°F pavement can elevate initial rainwater temperature from 70°F to over 95°F (United States Environmental Protection Agency, 2017). This heated stormwater will eventually runoff into storm sewers and raise the water temperature of streams, rivers, ponds, and lakes. Rapid temperature changes in aquatic ecosystems can be fatal to aquatic life.

2.1.2 Strategies to Reduce Urban Heat Islands

There are various strategies which help to reduce urban heat islands. One strategy is to increase tree and vegetation cover. This can provide shade and cooling to urban areas, as well as reduce stormwater runoff and protect against erosion. Another strategy is to implement more green roofs in urban areas. By growing a vegetative layer on a rooftop, the roof surface temperature will decrease and stormwater management will improve. Additionally, cool roofs are made of materials or coatings that reflect sunlight and heat away from a building. Cool roofs have the ability to reduce roof temperatures, increase the comfort of building occupants, and reduce energy demand. Vegetation cover, green roofs, and cool roofs are a few of many strategies that have the ability to reduce urban heat islands (United States Environmental Protection Agency, 2017).

2.2 Solar Panels

Solar energy is a renewable source of energy created from the sun. Solar energy produces energy through a process which is sustainable, inexhaustible, non-polluting, noise-free, and does not emit greenhouse gases (Energy Matters, 2016). Solar panels in the United States should face south to absorb the most sunlight; however, solar panels do not need direct sunlight to produce electricity. Solar power has the capacity to provide energy for air conditioners, hot water heaters, cooking and electrical appliances, natural gas, electricity, or oil fuels (Solar Power Authority, 2017). Solar technologies can be expensive and require a lot of land area to collect the sun's energy at useful rates; however, solar electricity can pay for itself in the long term, usually five to ten years with tax incentives (Imboden, 2009). When solar panels are purchased, the federal solar tax credit allows the owner to deduct 30% of the cost of installing a solar energy system from the owner's federal taxes. Not only has the cost of solar panels dropped by 80% since 2008 due to its high demand, but maintenance is minimal and returns are high once solar panels have been installed (Solar Power Authority, 2017).

2.2.1 How Solar Panels are Made

Solar panel systems (photovoltaic or PV system) are made up of semiconductor materials that convert sunlight into an electric current (Energy Matters, 2016). When sunlight hits the cells of the solar panels, electrons become loose from their atoms and flow through the cell generating electricity (Imboden, 2009). The semiconductor material is covered with an anti-reflective coating and made up of silicon wafers impregnated with impurities; impurities have the ability to improve electrical properties. The solar cells are joined together by electrical contacts, and located between a superstrate layer on top and a backsheet layer below (Energy Matters, 2016).

2.2.2 How Solar Panels Work

The photovoltaic effect is the process by which light is converted to energy at the atomic level. The majority of energy the solar cells produce goes into a grid connect inverter which converts the electric charge from a direct current (DC) into an alternating current (AC). This allows the solar electricity current to flow to and from the grid connect inverter. The solar electricity can power the appliances in a building when needed, and the leftover solar electricity will flow to the grid connect inverter where it is stored. If more energy is produced than used, then the owner is credited on their electricity bill, making this an incentive for building owners to implement renewable systems (Energy Matters, 2016).

2.2.3 Types of Solar Panel Systems

As the use of technology has increased over the years, different types of solar panels have been created. Of all these, approximately 90% of solar panels are made of silicon photovoltaic material (Battaglia, Cuevas & De Wolf, 2016). This section describes two different types of solar panel systems: crystalline silicon panels and thin-filmed panels.

Crystalline Silicon (Monocrystalline Silicon & Polycrystalline Silicon)

Crystalline silicon cells are the most common solar cells used in commercially available solar panels, consisting of more than 85% of world photovoltaic cell market sells. Crystalline silicon panels have two subtypes: Monocrystalline Silicon & Polycrystalline Silicon. The main difference between these types is the production technique. Each technique has its advantages and disadvantages. The cells have laboratory energy efficiencies of 25% for monocrystalline cells and over 20% for polycrystalline cells. However, industrially produced solar modules currently achieve efficiencies ranging from 18%–22% (Battaglia, et. al., 2016).

Monocrystalline solar panels have the highest efficiency rates since they are made out of the highest-grade silicon. Monocrystalline cells are produced from pseudo-square silicon wafers (substrates cut from boules grown by the Czochralski process), the float-zone technique, ribbon growth, or other emerging techniques. These other emerging techniques can have a specific reason for utilizing. For example, if produced using the ribbon growth technique, the production costs as well as the carbon footprint both decrease efficiency. These panels are also space-efficient. Since they yield the highest power outputs, they require less space compared to the other types. They also have a long life expectancy (25+ years) and tend to work better in low-light conditions. This type of panel is the most efficient and has a longer lifespan than other types of panels; however, it is the most expensive type of panel (Battaglia, et. al., 2016).

Polycrystalline silicon solar cells are a newer technology and vary in the manufacturing process. They are traditionally made from square silicon substrates cut from ingots cast in quartz crucibles. Polycrystalline cells are more cost effective to produce due to the fact that many cells can be created from a single block. However, every time silicon is cut, the edges become deformed, which results in a lower operating efficiency. Polycrystalline cells have become the dominant technology in the residential solar panels market because of their low operating efficiencies, and the cheap method by which they can be produced. In terms of efficiency, polycrystalline solar cells are now very close to monocrystalline cells (Battaglia, et. al., 2016).

Since crystalline cells were one of the first technologies, much of the production and manufacturing techniques have been refined to reach their maximum potential. Advantages of crystalline silicone cells include a high efficiency rate of about 12% to 24.2%, high stability, ease of fabrication, high reliability, and long lifespan. Other benefits include high resistance to heat and lower installation costs. Negatively, these panels are the most expensive, in terms of initial cost, and have a low absorption coefficient (Battaglia, et. al., 2016).

Thin-Film Panels

The differences between thin-film and crystalline silicon solar cells are the thin and flexible pairing of layers, and the photovoltaic material: either cadmium telluride or copper indium gallium dieseline instead of silicon. Thin-film solar panels are the least efficient type of solar panel. Depending on the technology, thin-film module prototypes have reached efficiencies between 7–13%, and production modules operate at about 9% (Battaglia, et. al., 2016).

Thin film panels are made by depositing a photovoltaic substance onto a solid surface, such as glass. Multiple combinations of substances have successfully and commercially been used for the photovoltaic substance. Typical thin-film solar cells are one of four types, depending on the material used: amorphous silicon (a-Si) and thin-film silicon (TF-Si); cadmium telluride (CdTe); copper indium gallium dieseline (CIS or CIGS); and dye-sensitized solar cell (DSC) plus other organic materials (Battaglia, et. al., 2016).

Despite being the least efficient, thin-film panels have advantages that should be considered when planning for solar roofing. Thin-film material is 100 times thinner than traditional solar panels, provides flexibility, and is lightweight. Thin-film panels are created by combining consecutive thin layers of material together. The result is a single film that is capable of being distributed in rolls or sheets making it easier to handle. Since they are becoming the lowest cost panels to produce because of their low material costs for thin film, they are quickly becoming the most economically efficient panel types. Some of thin-film panels' disadvantages include low efficiency, and they require the most space for producing the same amount of power as other solar panels. Additionally, the thin material's durability begins to suffer over time, requiring frequent replacement (Battaglia, et. al., 2016).

Table 1: Energy Output and Cost of Different Types of Solar Panel Systems

Type of Panel	Output (w)	Singular Panel Cost ($)
Monocrystalline	150	165
Polycrystalline	165	165
Thin-Film	100	135

2.2.4 Structural Considerations

Placing solar panels on the roof of a building adds various loads to the structure. To perform a structural analysis on the building involves to first define the loads, and then to determine how the loads affect the structure (Wrobel, 2017).

Solar panels add a dead load to the roof of a building. The dead load includes the self-weight of all the physical components of the solar panels. The dead load applied to the roof is a concentrated load located where the panels are supported by the roof, which is usually located at each corner of the panel (Wrobel, 2017).

In geographic regions where snow loads are present on roofs, warm roofs are constructed which can help decrease the snow load. If solar panels are raised above the roof, then they do not receive the benefit of the warm roof to decrease the snow load, which results in an increase of the snow load as well (Wrobel, 2017). The design of snow loads for roofs that include solar panels shall be determined in accordance with ASCE 7-10.

Wind loads are also considered as they have the ability to act in various directions, both upward and downward on solar panels. Wind loads also act on different locations of the solar panels depending on which direction the wind is blowing from (Wrobel, 2017). Some of the elements for which wind loads should be considered are: the ultimate design wind speed, risk category, wind exposure, internal pressure coefficient, and component and cladding.

Not only must we consider the various loads acting on the structure of a building, but we must also take into consideration the size, quantity, and location of solar panels on the roof of a building. All of these factors will determine the effect of the loads, and the existing structures' capacity for the addition of solar panels.

2.2.5 Wind Design for Solar Panels

A document by the Structural Engineers Association of California titled, *Wind Design for Low-Profile Solar Photovoltaic Arrays on Flat Roofs*, provides information on the step-by-step process for calculating wind loads on solar panels. There are many factors to consider when analyzing the effect of wind loads on solar panels. This document provides information on the determination of wind loads for solar photovoltaic arrays, which is not explicitly covered by the methods contained in the ASCE 7-10 (Structural Engineers Association of California, 2012). Steps to determine wind loads on rooftop equipment and other structures is located in Table 29.1-1 in ASCE 7-10. However, in Step 7 of this table, the equation provided needs to be changed for the consideration of solar panels. The design wind pressure for rooftop solar arrays can be determined by the formula below (Structural Engineers Association of California, 2012).

$$\mathbf{p = q_h * (GC_m)}$$

p = wind pressure for rooftop solar arrays

$\mathbf{q_h}$ = velocity pressure evaluated at mean roof height of the building (lb/ft^2)

$\mathbf{GC_m}$ = combined net pressure coefficient for solar panels (lb/ft^2)

Solar panels mounted on a roof are highly vulnerable to the speed and direction of the wind approaching the panel. There are three distinct regions or zones on a roof where the wind flow characteristics and resulting wind loading on solar panels are different: interior, edge, and corner zones. Wind loads on solar panels located in the corner zones of roofs are much greater than those in the middle of the roof. Higher tilt panels are particularly vulnerable to the vertical component of swirling winds in the corner vortices of the panels. Since solar panels in the northern hemisphere face south, the northeast and northwest corners of the panel create severe loading. The southeast and southwest corners of the panel still create loading, just not as strong as the other two corners (Structural Engineers Association of California, 2012).

Different restricting values for the size, height, spacing, and positioning of solar panels are located in Table 2. These values will help when designing the roof layout and calculating wind load values. *Wind Design for Low-Profile Solar Photovoltaic Arrays on Flat Roofs* provides more detailed information and application for these values.

Table 2: Solar Panel Design Restrictions (based on Structural Engineers Association of California, 2012)

Characteristic	Quantity
Height of gap between panels and roof surface (h_1)	$\leq$ 2 feet
Maximum height above the roof surface (h_2) for panels	4 feet
Panel chord length (l_p)	$\leq$ 6 feet 8 inches
Distance between solar panels and roof edge	$\leq 2*h_2$
Space between rows of solar panels	$\leq$ 2*panel characteristic height (h_c)
Panel tilt angle for typical installations	0-35 degrees

2.2.6 Seismic Requirements for Solar Panels

Similar to the previous section, a document by the Structural Engineers Association of California titled, *Structural Seismic Requirements and Commentary for Rooftop Solar Photovoltaic Array*, provides information on how to calculate and deal with seismic forces when designing solar panels. It is important to understand the effect of seismic forces on solar panels, and prepare for any type of loading. As described in the document, solar arrays can either be attached or unattached to the roof structure of a building (Structural Engineers Association of California, 2012). For our project, we plan on using attached solar arrays, therefore the information obtained has different values and procedures than that of unattached solar arrays.

Solar panels and their structural support systems shall be designed to provide life-safety performance in the design basis earthquake ground motion. Life-safety performance means that solar panels are expected not to create a hazard to life. For example, as a result of breaking free from the roof, sliding off the roof's edge, exceeding the downward load-carrying capacity of the roof, or damaging skylights, electrical systems, or other rooftop features or equipment in a way that threatens life-safety. Solar array support systems that are attached to a roof structure shall be designed to resist the lateral seismic force (F_p) specified in Chapter 13 of ASCE 7-10. In the computation of F_p, an evaluation of flexibility and ductility capacity of the support structure is permitted to be used to establish seismic coefficients of component amplification factor (a_p) and component response factor (R_p). These values can be found in Table 13.5-1 of ASCE 7-10. Additionally, friction is permitted to contribute in combination with the design lateral strength of attachments to resist the lateral force F_p (Structural Engineers Association of California, 2012).

2.3 Solar Collectors

Solar collectors convert energy from the sun into usable heat in a solar water heating system. This energy can be used for hot water heating, pool heating, space heating, or even air conditioning (Apricus Solar Water Company, 2017).

2.3.1 How Solar Collectors Work

Solar collectors can be mounted on a roof, wall, or the ground. A circulation pump moves liquid through the collector, which then carries heat back to the solar storage tank. Throughout the day, water in the solar storage tank is heated up. When hot water is used, the solar preheated water is fed into the traditional water heater and supplied for its desired usage (Apricus Solar Water Company, 2017).

2.3.2 Structural Considerations

Solar collectors impose similar loads to the roof structure as solar panels: dead loads, snow loads, wind loads, and seismic loads. Solar collectors add dead loads as a result from the weight of the collector, the mounting hardware, and the collector fluid. Typically, the collector has a dead load of approximately three to five pounds per square foot, but the exact weight considerations can be obtained from the manufacturer of the solar collectors (HTP, 2017).

In areas prone to heavy snowfall, such as Massachusetts, snow loads need to be considered in the design of the solar tubes. Ideally, solar collectors should be installed at an angle of 50º or greater to promote snow sliding off the tubes (HTP, 2017). Similarly, when installing solar tube collectors, wind and seismic resistance needs to be considered as well as the resultant stress on each of the attachment points. It is important to review the roof structure to ensure strength attachments of the solar collectors (HTP, 2017).

2.4 Green Roofs and Stormwater Retention Systems

A green roof is a roof of a building that is covered with vegetation. There are two characterizations of green roofs: extensive green roofs and intensive green roofs. Intensive green roofs use planting mediums that have a greater depth than extensive green roofs; this requires more maintenance because of the larger plant varieties intensive planting mediums can support. An extensive green roof has vegetation ranging from sedums to small grasses, herbs, and flowering herbaceous plants. Extensive green roofs are ideal for efficient stormwater management and low maintenance needs. An intensive green roof has vegetation ranging from herbaceous plants to small trees. Intensive green roofs require professional maintenance and advanced green roof irrigation systems. Rooftop farms fall under the intensive green roof category. The growing medium for an extensive green roof is 6” or less, while the growing medium for an intensive green roof is greater than 6” (Jörg Breuning & Green Roof Service LLC, 2017). Green roofs have the ability to reduce urban heat islands and can also serve as a stormwater retention system.

2.4.1 The Urban Problem

Urban areas generate more stormwater runoff than natural areas due to a greater percentage of impervious roof surfaces and paved surfaces that prevent water infiltration. The United States Environmental Protection Agency (USEPA) concluded that a typical city block generates more than five times as much runoff than a woodlot of the same area. Additionally, urban stormwater runoff carries pesticides, heavy metals, and contaminated nutrients which have the ability to flow into various bodies of water. According to USEPA, "The most recent National Water Quality Inventory reports that runoff from urbanized areas is the leading source of water quality impairments to surveyed estuaries and the third-largest source of impairments to surveyed lakes (Andresen, Fernandez, Rowe, Rugh, VanWoert & Xiao, 2004)."

2.4.2 Green Roof Stormwater Retention Success

Implementing green roofs in urban areas is a solution to reduce stormwater runoff. The Michigan State University Horticulture Teaching and Research Center conducted a 14-month study in which three simulated roof platforms were constructed. One of the roof platforms contained gravel, the other was vegetated, and the third was non-vegetated. Over a 14-month period, the vegetated roof had the greatest overall rainfall retention at 60.6%, while the non-vegetated roof had a rainfall retention of 50.4%, and the gravel roof had a rainfall retention of 27.2%. These percentages refer to the amount of rainfall which did not runoff the roof out of total amount of rainfall in the 14-month period. To conclude, vegetated roof platforms retain greater quantities of stormwater than conventional roofs. However, the study stated, "if the objective of a green roof is to maximize rainfall retention, then factors such as slope and media depth must be addressed (Andresen, et. al., 2004)."

2.4.3 Benefits of Green Roofs

Not only do green roofs control stormwater runoff, but their designs also have many other benefits (Andresen et. al., 2004):

- Insulate buildings, which saves on energy consumption.
- Increase the lifespan of a typical roof by protecting the roof membrane from damaging ultraviolet rays, extreme temperatures, and rapid temperature fluctuations.
- Filter harmful air pollutants.

- Contribute to aesthetically pleasing environment to live and work by controlling the temperature of a building.
- Provide habitat for a variety of living organisms.
- Contribute to reducing the Urban Heat Island Effect

2.4.4 Structural Considerations

Similar to solar panels, green roofs contribute dead loads, live loads, snow loads, rain loads, wind loads, and seismic loads to the roof of a structure. The most contributing factor to the loads on a green roof depend on the size and type of vegetation which is used. An intensive green roof contributes more load than an extensive green roof due to the larger trees, plants, and sometimes water features that are being used. Additionally, the location of the stormwater storage has an impact on the structure of a building. Depending on the green roof, stormwater can be stored within the green roof itself, in a tank below the building, or drained towards the local watershed.

The structural considerations for green roof design are typically attributed to the different components (layers) of green roofs. A typical modern vegetated roof requires a minimum of eight layers: plant level (vegetation), substrate layer, insulation layer, filter fabric, drainage layer, protection fabric, roof barrier, and waterproof layer as shown in Figure 1 (Gartner, 2008). To conclude, the overall design and layers of a green roof determine the effect of the various loads on the structure of a building.

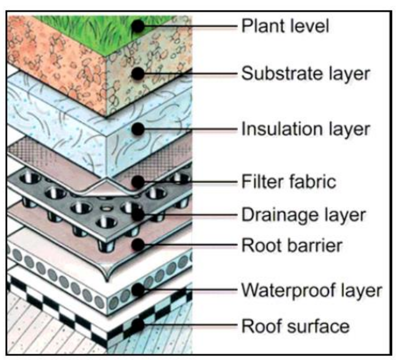


Figure 1: Layers of a Typical Modern Vegetated Roof (Gartner, 2008)

2.6 Types of Structural Reinforcements

Structural strengthening is used to reinforce structures due to deficiency, and to increase an existing element's capacity to carry new loads; new loads such as sustainable rooftop technologies. As with any structure or method of reinforcement, it is necessary to first identify and establish a good understanding of the structure through a structural condition assessment. The most common existing techniques to reinforce structural elements are mentioned below and classified into two different categories: passive systems and active systems. When selecting the appropriate strengthening method, it is important to consider the following factors: magnitude of strength increase, size of building and structures, environmental conditions, accessibility, concrete strength, construction, and maintenance and life cycle costs (Shaw, n.d.).

2.6.1 Passive Systems

Passive systems do not introduce any forces to the structure; they contribute to the overall resistance of an element when it deforms. Section enlargement strategies are mostly used to improve strength, stiffness, and to reduce cracks. Some types of section enlargement strategies are: span shortening, externally bonded steel shapes, and epoxy injection (Shaw, n.d.).

Externally bonded fiber reinforced polymer (FRP) reinforcement is a method of reinforcement which includes adhering additional reinforcement to the exterior faces of an element. The success of this strengthening method depends on both the durability and lifespan of the reinforcement material, and the properties of the material used to attach the new reinforcement (usually epoxy material). This method, if adopted correctly and with the appropriate materials, is able to: reduce deflection, increase carrying capacity, increase flexural strength, and increase resistance to shear (Shaw, n.d.).

2.6.2 Active Systems

Active strengthening systems are identified by adding external forces to structural elements, which can increase strength and improve the service performance. Service performance reduces tensile stress and cracking (Alkhrdaji & Thomas, 2017).

A post-tensioning system is an external force method which implements a structural member using high strength cables, bars, and strands. This system usually connects the reinforcement to the existing member at anchor points (typically at the end of the member). The reinforcement is profiled along the span at different locations (Shoultes, 2017).

3 Scope of Problem

After background research, the sustainable rooftop technologies we will further analyze are solar panels, green roofs, and solar collectors. We plan on analyzing one or more buildings at WPI for the application of each of these practices. This section includes the project activities as well as the range of topics and parameters that will be investigated for the chosen sustainable roofing practices.

3.1 Solar Panels

This section includes the range of topics and parameters that will be investigated for using solar panels as the sustainable rooftop technology on buildings at WPI. Information is defined for the following considerations: ease of construction, loads, structural analysis, energy output, and economic costs.

3.1.1 Ease of Construction

When investigating the structural impact solar panels have on buildings at WPI, we must first determine how solar panels are constructed and installed on roofs. We will be investigating multiple types of panel systems and assess their ease of installation. Many variables must be considered during the construction and installation process of solar panels. One variable is determining the type, size, and weight of the solar collectors. Additionally, the number of solar panels needs to be evaluated, which may vary per building based on the available space and the required energy output.

A second variable which needs to be considered is the safety of construction. We must determine the safety measures which must be taken when installing solar panels. Furthermore, we must figure out the time period for constructing and installing solar panels, which will vary depending on the quantity.

A third variable is the location on the roof where the solar panels need to be installed. This depends on the slope and shape of the roof, as well as the side of the roof which has the best exposure to the sun's rays. Another variable is how the panels will be installed at their desired location. Installation includes the blocking behind the roof, which supports the panels, and the process of mounting the panels. The position of the panels offset from the roof, the angle of the panels, and how the panels will be secured must also be considered (Radiantec Company, n.d.).

Finally, the last variable is figuring out where the energy will be supplied throughout the building. We must determine how the energy produced from the solar panels will be stored and distributed throughout the building. Depending on the functionality of the building, distributed amounts of energy will be required for various purposes.

3.1.2 Loads

There are many loads associated with installing solar panels on the roof of a building. These loads include dead loads, wind loads, snow loads, and seismic loads.

Dead load includes the self-weight of all the physical components of the solar panels. The dead load applied to the roof is a concentrated load located where the panels are supported by the roof; which is usually located at each corner of the panel (Wrobel, 2017). The dead load also depends on the size, type, and number of solar collectors placed on the roof.

Wind loads, snow loads, and seismic loads should be calculated in accordance with the guidelines provided in the ASCE 7-10. The reference chapters for these loads are displayed in Table 3. The following information related to wind load should be considered when performing an analysis: ultimate design wind speed, risk category, wind exposure, internal pressure coefficient, and component and cladding. For each selected wind direction at which the wind loads are to be evaluated, the exposure of the building should be determined for the two upwind sectors extending 45 degrees either side of the selected wind direction.

Table 3: Reference Chapters for Wind, Snow, and Seismic Load Information (Solar World, 2014, & Structural Engineers Association of California, 2012)

Chapter in ASCE 7-10:	**Information Provided:**
Chapter 7	Snow load calculations
Chapter 13	Seismic load calculations
Chapter 16	Determination of wind resistance using an effective wind area, based on dimensions of a single unit frame
Chapter 26-36	Determination and calculations of wind loads

3.1.3 Structural Analysis

An assessment of the buildings at WPI needs to be performed in order to determine whether the existing building can support the loads from solar panels, or if structural reinforcements need to be added to the building. To begin the structural analysis, the type, dimensions, gross area, and mass of the solar panels needs to be determined. Next, we must identify if the building has a flat or pitched roof, and the angle of the pitched roof. We will then need to analyze the material and dimensions of the roof structure and building frame. Once these variables have been identified, load combinations can be calculated using dead, wind, snow, and seismic loads with the guidelines outlined in ASCE 7-10 (Ridal, Garvin, Chambers, & Travers, 2010).

Based on a risk assessment of structural impacts on buildings of solar panels: "In order to establish a straightforward method of assessing, critical or affected members should not be loaded to more than 100% of their design capacity as a consequence of increased loading from solar collector products (Ridal, et. al., 2010)." To conclude, a structural analysis of the buildings can be performed by first determining the solar panel requirements, then using the resources and plans of the building to assess the adequacy of the structural load path, and finally decide whether the building can withstand the loads from the solar panels.

3.1.4 Energy Output

The number of solar panels to provide a desired amount of energy needs to be calculated. The number of solar panels correlates with total solar panel area. The global formula to estimate the energy generated in output of a photovoltaic system is (Photovoltaic Software, 2017):

$$\mathbf{E = A * r * H * PR}$$

E = Energy (kWh)

A = Total solar panel area (m^2)

r = Solar panel yield or efficiency (%)

H = Annual average solar radiation on tilted panels (kWh/m^2)

PR = Performance ratio (range of values: 0.5 - 0.9; default value: 0.75)

The value ‘**r**’ is equal to the electrical power (kWp) of one solar panel divided by the area of one panel. The value ‘**H**’ is a global radiation value, found online, which reflects seasonal effects and varies per geographic location. The value ‘**PR**’ is an important value to evaluate the quality of a photovoltaic installation because it gives the performance of the installation independent of the orientation and inclination of the panel. The ‘**PR**’ value is essentially a coefficient for losses (Photovoltaic Software, 2017).

We must determine the energy requirement of the building in order to calculate the total solar panel area. The energy value will vary depending on the purpose of the building. For example, recreational and residential buildings at WPI will most likely require more energy than an academic building. This information can be found from the WPI Facilities Department, but might not be given for each building. The energy requirements are most likely tracked for newer buildings, rather than the older ones. If this information is not available, we will research standard energy requirement values for recreational, residential, or academic buildings.

When the energy requirement for the building is obtained, we can use the global energy formula to calculate the required solar panel area. By calculating the required solar panel area, the number of solar panels for the building can be determined.

3.1.5 Economic Costs

Based on the energy output analysis described above, we will determine whether it is economically feasible to install and use solar panels on the chosen building. Using the required energy value of the building, we can either calculate or use available resources (WPI Facilities Department) to figure out the energy cost for the building. Then, from the design, the cost for installation of the solar panels can be calculated. The economic evaluation will not only include the initial product and construction costs, but will also include any costs that are incurred over time, such as maintenance costs. Additionally, we will need to figure out the lifetime of solar panels to see how long they will be able to effectively produce energy. Finally, a short-term and long-term financial analysis can be made to show the return of this investment over time.

For the life cycle cost analysis, we will also need to evaluate the time value of money. For example, something worth $200 in 40 years could be equivalent to $100 today. With that in mind, the energy cost of buildings will most likely increase in the future. The initial cost for the installation of solar panels will not be affected by the time value of money. However, the time value of money could have an effect on the maintenance costs of solar panels years after installation.

3.2 Green Roofs

This section includes the range of topics and parameters that will be investigated for using green roofs as the sustainable roofing practice on buildings at WPI. Information is defined for the following considerations: ease of construction, loads, structural analysis, energy output, and economic costs.

3.2.1 Ease of Construction

When investigating the structural impact green roofs have on buildings at WPI, we must first determine how they are constructed. Many variables come into hand for the installation of green roofs. One variable is determining the type of green roof that will be constructed. There are two main types of green roofs: intensive roofs and extensive roofs. Intensive roofs have a thick base and can support a wide variety of plants; however they are heavy and require maintenance. Extensive roofs have a shallow base, are light, and require minimal maintenance. Extensive roofs can support 10-25 pounds of vegetation per square foot and intensive roofs can support 80-150 pounds of vegetation per square foot. Some green roof designs incorporate both intensive and extensive elements. Comprehensive green roofs support plant varieties typically seen in intensive green roofs, but have the depth and weight of an extensive green roof system. A comparison of extensive and intensive green roofs is shown in Figure 2.

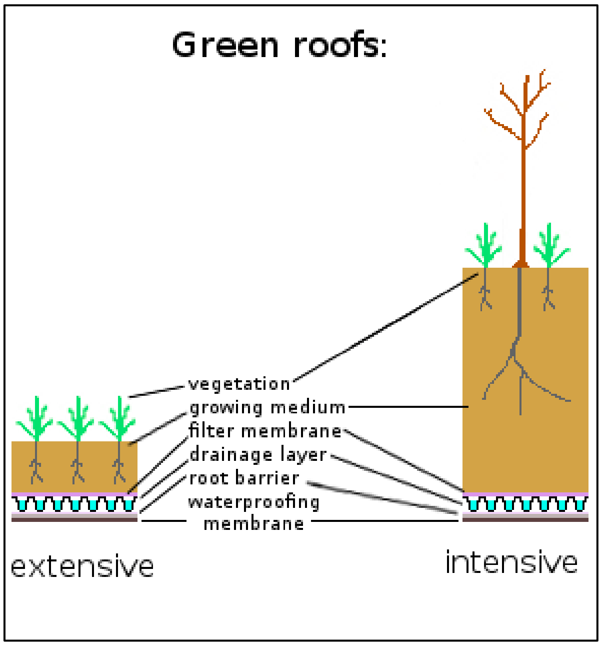


Figure 2: Extensive vs. Intensive Green Roofs (MGASE, 2008)

A second variable to consider when installing green roofs is location. Location of the green roof plays an important role in the design process. The height of the roof above grade, its exposure to wind, the roofs' orientation to the sun, and shading by surrounding buildings during parts of the day will all have an impact when deciding the location of the green roof. The general climate of the area and the specific microclimate on the roof must also be considered.

Another variable to consider is the type of plants that will be used. While most plants do well during the summer, they will likely die during the winter. Therefore, plants that thrive in winter should be highly considered. The last variable that should be considered is figuring out the amount of heating and cooling cost that will be saved. After the implementation of green roofs, the soil mixture and vegetation act as insulation, which reduces these costs by approximately 20%. This percentage varies depending on the type of green roof and amount of vegetation used.

3.2.2 Loads

There are many loads associated with green roofs. These loads include dead loads, live loads, transient live loads, snow loads, wind loads and seismic loads. A thorough analysis on how to determine these loads can be found in: *ASTM E2397-05 Standard Practice for Determination of Dead Loads and Live Loads Associated with Green Roof Systems.*

The dead loads associated with a green roof have the greatest contribution to the structure of the building. Dead loads include the weight of the roof system, all layers between the vegetation and roof, the capture water, and the vegetation itself. A 15% increase in the specified depth is recommended to account for future additions of growth media.

The live loads should be determined based on the type of occupancy and local building code requirements. It is recommended by FM Global that extensive green roofs be designed for no less than 12 psf when considering live load reduction, and a minimum of 20 psf for intensive green roofs. We will need to assess the adequacy of these values based on the range of parameters considered in our design. The live loads of green roofs include the weight of transient water contained in the drainage materials. This is the quantity of water that is required to completely fill the drainage layer of a green roof system.

The snow loads are based on the local jurisdictions building code requirements. In Worcester, buildings should be designed to withstand a snow load of 55 psf. For wind loads, the local building code requirements has to be followed, and roofs should be designed for the envelope of wind uplift on a bare roof and a saturated green roof. Seismic loads need to be calculated, in accordance with Chapter 13 of ASCE 7-10, since retained stormwater in the green roof produces a weight that is an inertia force.

3.2.3 Structural Analysis

The structural analysis of green roofs is similar to the structural analysis of solar panels. The structural implications of augmented loads on WPI buildings need to be analyzed to see whether additional reinforcements are needed to support the loads of green roofs.

The first step for this analysis is to determine the type of green roof that is going to be implemented. This is of extreme importance as different green roofs will generate different loads on a building. For example, extensive roofs usually require only minimal changes to the structural system of a building, while semi-intensive and intensive green roofs require a more detailed analysis and a need for a stronger building structure. Similarly, the layer design of a green roof will determine the exact load that the system will have on the structure. The layers of a green roof include, but are not limited to: roof barrier, protection fabric, drainage layer, filter fabric layer, insulation layer, substrate layer, and plant level (MGASE, 2008). Documents such as the *2002 Guideline for the Planning, Executing and Upkeep of Green-Roofs Sites,* and *Property Loss Prevention Data Sheet 1-35,* provide a comprehensive way on how to design green roof structures. These documents also indicate: “If a green roof assembly is not tested per ASTM standards, then the design load should be based on a saturated density of no less than 100 pcf (MGASE, 2008).” It is also important to consider several structural notes for green roof design:

1. Maintenance of the green roof and plant growth control to prevent structural overload.
2. Loading maps regarding different locations where a green roof is implemented.
3. Weights and thickness of all components.
4. Drainage plan and storage tank.
5. Specific tree data with weights and sizes.
6. Fabricate and test a mockup of the final green roof design (tested with ASTM E2397 and ASTM E2399).

When the design of the green roof is completed and all the necessary layers are determined, an analysis of the loads can be done to the selected buildings. The selected buildings will require to hold the different loads mentioned above in addition to the design dead load of the green roof.

3.2.4 Energy Saved

Green Roofs do not generate energy like solar panels, but its energy output is measured by the amount of energy saved after its implementation. Green roofs work as a form of insulation, thus improving the thermal performance of a roof. This allows buildings to better retain their heat during the cooler winter months while reflecting and absorbing solar radiation during the hotter summer months, allowing buildings to remain cooler. The insulated properties reduce energy demand for both heating and cooling; this reduced energy demand also reduces building energy costs. This means that energy requirements of the building are reduced year-round which allows the building temperature to be controlled at a lower cost.

There are only a small number of studies focused on quantifying the saved energy from green roofs. There is a study developed by Quantec that modeled the heating and cooling benefits of a green roof used in Portland, Oregon. The study found that a green roof reduced energy demand by 12%, with an annual cooling savings of 0.17 kWh/SF for electricity, and a heating savings of 0.02 therms/SF for natural gas. Roughly, the building saved around $1,500 a year. Other studies show similar results to this one; the reduction in the total energy demand for buildings ranges from 5-15%. If a green roof were to be implemented at WPI, then the energy requirements of that building should be expected to reduce around the same percentage as the studies. After choosing an appropriate location for the green roof at WPI, we will be able to determine an estimate of how much energy and money will be saved.

3.2.5 Economic Costs

While the average cost of installing a green roof can run two or three times more than a conventional roof, it's likely to be a lower cost approach in the long run, due to energy savings. The growth medium and plantings of a green roof help protect the roof's waterproof membrane from ultraviolet radiation, extreme temperature fluctuations, and damage from use or maintenance. This protection may extend the life of the roof by two to three times that of a conventional roof. Conventional roofs have a life expectancy of around 20 years, while studies have found that the life expectancy of a green roof is close to 40 years. These studies were made in the United States, where green roofs are a fairly new practice. In Europe, where the development of green roofs has gone on for decades, some research shows that green roofs can protect the roof membrane upward of 50 years. For example, there are green roofs in Berlin that show a lifespan of more than 90 years before important repairs or replacement may be required (MGASE, 2008).

The study developed by Quantec, described in the previous Energy Saved section, also performed a cost and benefit report. In it, the cost and benefit analysis for a green roof is developed at different years after implementations (5 years, 20 years, and 40 years). The key findings from the analysis are: at five years, benefits accrued by a developer for green roof construction would only account for approximately half the cost of the green roof. Benefits do not appear to exceed costs until year 20 when an avoided cost of conventional roof replacement would be accrued. By forty years after development, the calculated economic benefits exceed costs by approximately $700,000. In both the five-year and forty-year time period, the public benefit of the green roof is positive.

3.3 Solar Collectors

Solar collectors convert energy from the sun to heat water. The construction process for installing solar collectors is similar to that for solar panels; however, the loads on the roof structure are different since solar collectors store water in their system. This affects the weight of the overall solar system, which contributes to the dead and seismic loads on the roof structure. Loads include dead, wind, snow, and seismic. Based on the calculated loads, proper structural reinforcements can be analyzed for the building. Additionally, it must be considered where to put the storage tank for the heated water. In order to install solar collectors, the hot water consumption value of the building and the amount of hot water the solar collectors can supply needs to be determined. Once the hot water consumption value of the building is collected, then the number of solar collectors can be calculated based on how much hot water each solar collector can produce. The cost to install solar collectors can be researched, followed by a life cycle cost analysis to determine whether it is worthwhile to install solar collectors on the roofs of specific buildings. For our project, we will look to install solar collectors on residential buildings, rather than academic buildings, since residential buildings consume more hot water than academic buildings. The athletic building at WPI has solar collectors on the roof, which heat the pool water. This saves more than $50,000 in operating costs and reduces carbon dioxide emissions by 4,400 pounds per year, as compared with conventional pool heating (WPI Sustainability Plan, 2017). Solar collectors will be an important technology for the application of our project.

4 Capstone Design

To fulfill the requirements of the Capstone Design, the team will complete a Major Qualifying Project focused on the plan and design of sustainable roofing practices on existing buildings at Worcester Polytechnic Institute (WPI). Structural analysis of different buildings, as well as feasibility of construction and costs will be addressed in this project. The Capstone Design constraints expected in this project include: economic, environmental, constructability, sustainability, ethical, and health and safety.

4.1 Design Problem

As Worcester Polytechnic Institute is committed to a sustainability plan of ecological stewardship, social justice, and economic security, every member of the WPI community should be engaged in this process. Our plan for sustainable rooftop technologies follows the same path of the already existing sustainability plan; it is our job to embrace this mission in the local community.

To approach the problem and support the WPI sustainability plan, our group will design sustainable rooftop technologies, solar panels, green roofs, and solar collectors, for a number of existing buildings on campus. Each proposed system has the ability to generate energy efficiency, water storage, and building cool-off.

4.2 Economic

The plan of implementing sustainable rooftop technologies comes at a cost. For each alternative that is considered, there is going to be a different design and therefore a different cost. Our group is going to provide costs for implementing each of these systems, which will include the actual cost of the system, maintenance costs, lifetime, and long-term net savings. Similarly, we are going to determine the return on investment of the desired project, and we will provide recommendations based on budget and costs.

4.3 Constructability

Constructability is one of the most important factors to consider for implementing these sustainable systems. Considerations regarding the type of building (academic/residential/recreational), type of roof (slope/flat), year built, and size of the building are all addressed under this criterion. Similarly, the following factors need to be analyzed and considered:

- Structural layout of the selected buildings.

- Zoning, permitting, and regulations.
- Construction schedule/time frame for each system.

4.4 Sustainability

Sustainability in this project consists of economic, environmental and social aspects. The design and construction of sustainable rooftop technologies includes all of these aspects and brings them together. Solar panels, green roofs, and solar collectors alleviate environmental concerns by implementing new technology in existing buildings at WPI. Sustainable practices reduce the consumption of energy, and they create more efficient buildings on campus.

4.5 Environmental

Through the development of this project, another constraint similar to sustainability is environmental. Implementing sustainable rooftop technologies on buildings at WPI can alleviate the urban heat island effect. This is accomplished by reducing energy usage and decreasing gas emissions with the use of natural sources of energy, such as the sun. However, installing each sustainable technology requires construction on the WPI campus, which can negatively impact the environment. Noise and dust can emit into the air during the construction processes for these systems. Our group will propose installation processes which will limit the impact on noise and air pollution.

4.6 Health and Safety

It is of extreme importance to protect the public and the community of WPI of any possible risks. Health and safety of all the people involved in this project is going to be considered, especially for potential users of the selected buildings. The design and construction of these systems will be in accordance with the *International Building Code* and all safety factors.

4.7 Ethical

Ethical practices play an important role in this project. It is crucial to consider ethical codes for the design and construction of sustainable rooftop technologies. All the appropriate codes and regulations are to be considered in the implementation of these systems. Furthermore, the team will complete confidentiality agreements for the information that it is going to be provided by WPI Facilities Department.

5 Professional Licensure

Civil engineering has been prevalent in human history since the beginnings of mankind. In addition to gathering food, society's main concern includes building a settlement, which requires civil engineering. Only a professional licensed civil engineer may prepare, sign, seal and submit engineering plans and drawings to a public authority for approval, or seal engineering work for public and private clients. The purpose of licensure is to protect the health and welfare of the public by regulating requirements to restrict engineering practice to qualified individuals that have obtained a professional license. In order to get licensed, engineers must complete a number of requirements. First, one must complete a four or five-year college undergraduate degree. Following graduation, the individual must work under a professional engineer for at least four years, pass an intensive exam, and earn a license from their state's licensure board. Having a professional engineer's license means you have accepted both the technical and the ethical obligations of the engineering profession. Once a professional engineer is licensed, the individual is free to practice the discipline of civil engineering, and may stamp documents of any kind within the practice and expertise. This licensure is important since it is legally required to be a consulting engineer or a private practitioner. It can also raise prestige and accelerate career development.

The process of preparing a sustainable roofing plan for WPI will expose our group to the concept of structural design and analysis, which is also required by professional licensed civil engineers. Our project explores alternative rooftop technologies that could possibly be employed by the WPI community. These alternative systems consist of installing solar panels, green roofs, and solar collectors to the roofs of chosen buildings at WPI. A structural analysis of the buildings will be executed, as well as a proposed sustainable roofing plan which will be given to the school. In order to install solar panels, green roofs, and solar collectors, one must make sure that the building can carry the loads imposed by these technologies. Additionally, our analysis will include how efficient solar panels, green roofs, and solar collectors are, and how much money they can save the school in the long run.

Solar panels, green roofs, and solar collectors have the ability to deal with the negative impacts of the urban heat island effect by making the problem part of the solution. This project reflects the meaning of a professional licensed civil engineer. There are technical aspects to this project: designing the layout of solar panels and green roofs, choosing a building and analyzing the structure’s support, and producing an economic evaluation. Finally, our project relates to the nature of a professional licensed engineer by promoting health and welfare in an ethical manner and making the WPI community more sustainable.

6 Methodology

The methods section outlines the criteria for completing our MQP project and accomplishing our objectives. Each criteria contains information relating to steps, specific tasks, references, and person responsible for completing the task. The following criteria are defined in this section: identify buildings to begin analyzing, meet with WPI Facilities Department, identify type of solar panel, green roof, and solar collector systems to install, define solar panel, green roof, and solar collector layout, structural analysis and design, and evaluation and recommendation.

6.1 Identify Buildings for Consideration

The goal for this criteria is to make a list of potential buildings at WPI for consideration. This involves starting with a list of all the buildings at WPI, categorizing these buildings based on different criteria, and then narrowing down the list. An outlined list of requirements for buildings to have in order to support solar panels, green roofs, and solar collectors will be outlined, as well as the initial list of all the buildings at WPI with information pertaining to the requirements. By comparing the list of buildings and seeing if they meet the requirements outlined, we will be able to make a narrowed down list of buildings to begin analyzing. Table 4 shows a breakdown of steps and tasks for identifying buildings for consideration.

6.2 Meet with WPI Facilities Department

The goal for this criteria is to narrow down the list even further, and identify one or more buildings for each of the three technologies for our final analysis. This involves getting in contact with a representative within the WPI Facilities Department to obtain information about the buildings identified in the previous criteria. Our plan during the meeting is to give the representative the list of buildings we currently believe can support solar panels and/or green roofs, and explain the process and requirements for identifying these buildings. Once the representative understands and approves of this process, we will attempt to obtain different information on the energy consumption and design drawings for each building. Our objective is to choose the final buildings which have available design drawings, and have available energy consumption values. If we are not able to obtain energy consumption for the buildings, then we will use researched standard energy consumption values. Table 5 shows a breakdown of steps and tasks for meeting with WPI Facilities Department.

Table 4: Steps and Tasks for Identifying Buildings for Consideration

Steps	Specific Tasks	References	Person(s) Responsible
1) Outline list of requirements for buildings to have in order to support solar panels, green roofs, and solar collectors	a. Categorize list into following sections:	Online research	Ryan
	i. Age of building		
	ii. Exposure to sun		
	iii. Slope of roof		
	iv. Existing sustainable roofing practice		
2) Begin categorizing and observing buildings at WPI	a. List all of the buildings at WPI	Online research: WPI website	Sebastian and Ian
	b. Categorize buildings into academic, residential, recreational, or administrative		
	c. Identify year the building was constructred		
	d. Identify the number of stories the building has (not including basement)		
	e. Physical observation: identify if there are any trees or buildings which block the roof's exposure to the sunlight (south side of roof)		
	f. Identify type of roof on building:		
	i. Flat or sloped		
	ii. If sloped, identify if a portion of the sloped roof is facing south		
	g. Identify buildings which currently use sustainable practices:		
	i. Buildings that have solar panels, green roofs, or solar collectors		
3) Make list of potential buildings at WPI for consideration	a. See which buildings in Step 2 meet the requirements outlined in Step 1 (steps above)	Step 1 + Step 2	All

Table 5: Steps and Tasks for Meeting with WPI Facilities Deparment

Steps	Specific Tasks	References	Person(s) Responsible
1) Get in contact with someone within WPI Facilities Department	a. Potential contact: William Paul Spratt - Director of Facilities	Email: wpspratt@wpi.edu Phone: 508-831-5904	All
2) Obtain information about created list of buildings	a. Energy consumption of building:	Obtain from interview	All
	i. If cannot obtain information, will have to research standard values depending on type of building		
	b. Cost of energy consumption of building:		
	i. If cannot obtain information, will have to research standard values depending on type of building		
	c. Design drawings (plans) of building		
3) Narrow down list even further for buildings to analyze	a. Identify final buildings:	Obtain from interview	All
	i. Design drawings must be available for the building		
	ii. Energy consumption values must be available for the building		

6.3 Identify Types of Solar Panels, Green Roofs, and Solar Collectors to Install

The goal for this criteria is to choose two types of solar panel systems, one type of green roof system, and one type of solar collector system to use for our analysis. This requires researching different types of solar panel, green roof, and solar collector systems, and choosing the types based on ease of installation, low weight to reduce loads, sufficient energy production, and low cost of installation. Table 6 shows a breakdown of steps and tasks for identifying types of solar panels, green roofs, and solar collectors to install.

6.4 Define Solar Panel, Green Roof, and Solar Collector Layout

The goal for this criteria is to calculate the number of solar panels or solar collectors for each building, choose the specific location on the roof for solar panel, green roof, solar collector installation, outline the construction process, and consider safety of construction. Calculating the number of solar panels or solar collectors will depend on the energy production for each building, and the type of solar panel or solar collector system used. By calculating the number of solar panels or solar collectors, we can determine the location on the roof by assessing the available space of proper size. Similarly for green roofs, we will need to use the energy consumption of the building to determine what size green roof will save energy greater than or equal to the building's energy consumption value. Then we can determine the location on the roof by assessing the available space of proper size. Table 7 shows a breakdown of steps and tasks for defining the solar panel, green roof, and solar collector layout.

6.5 Structural Analysis and Design

This criteria is a major portion of our project. The goal is to perform a structural analysis for each considered building, and determine whether it is feasible to install solar panel, green roof, or solar collector system on the roof of the building. If the current structure of the building cannot support the solar panels, green roof, or solar collectors, then structural reinforcements will be designed for the supporting elements of the building. Table 8 shows a breakdown of steps and tasks for the structural analysis and design.

6.6 Evaluation and Recommendation

The goal of this criteria is to determine whether it is both structurally and economically feasible to install solar panels, green roofs, or solar collectors on the roof of each building. After the structural analysis is performed in the previous criteria, we will need to perform an economic evaluation by comparing current energy consumption cost values for the building and cost values for the installation and long term maintenance of solar panels, green roofs, and solar collectors. Whichever value is greater will determine whether or not it is economically feasible to install solar panel, green roof, or solar collector system on the building. There is also a revenue source due to production of electricity or increased insulation that reduces energy demand for cooling. Table 9 shows a breakdown of steps and tasks for the evaluation and recommendation.

Table 6: Steps and Tasks for Identifying Types of Solar Panels, Green Roofs, and Solar Collectors to Install

Steps	Specific Tasks	References	Person(s) Responsible
1) Research different types of solar panel, green roof, and solar collector systems	a. Choose two types of solar panel systems, one type of green roof system, and one type of solar collector system:	Online research	Sebastian
	i. Ease of installation		
	ii. Low weight to reduce loads		
	iii. Suffecient energy production		
	iv. Low cost of installation		
	b. Identify size (dimensions, gross area) and weight for each type of solar panel, green roof, and solar collector system		
	i. Will need to identify type of vegetation used for green roof system		
2) Determine cost for installation of solar panel, green roof, and solar collector systems	a. Determine installation costs for each type of solar panel, green roof, and solar collector system	Online research	Ian
3) Determine lifetime of each type of solar panel, green roof, and solar collector system	a. Determine the maintenance costs overtime	Online research	Ryan

Table 7: Steps and Tasks for Defining the Solar Panel, Green Roof, and Solar Collector Layout

Steps	Specific Tasks	References	Person(s) Responsible
1) Using global energy output formula, calculate number of solar panels and solar collectors for each building	a. Based on energy consumption of building and type of solar panel or solar collector system used	Global energy output formula	Ryan
	b. Number of solar panels or solar collectors will vary per building and for each type of solar system		
2) Identify size of green roof system for each building	a. Identify the building's energy consumption value	Information from interview	Ryan
	b. Identify size of green roof which will produce energy greater than or equal to the building's energy consumption value	Online research	
3) Determine available space on roof for calculated number of solar panels, solar collectors, or size of green roof system	a. Calculate area of different sections of roof to determine if solar panels, green roof, or solar collectors can be installed:	Online research and design drawings	Sebastian
	i. Will require calculating total area of all solar panels or solar collectors		
	ii. Total area of solar panels or solar collectors will depend on type of solar system		
	iii. Compare to size of green roof determined in previous step (Step 2)		
	b. Needs to be completed for each chosen building		
4) Consider safety of construction	a. Identify safety measures needed to be considered when installing solar panels, green roofs, and solar collectors	Online research	Ian
	b. Time it will take to install solar panels, green roofs, and solar collectors		
	i. When can they be installed (season dependent)?		
	ii. Depends on number of solar panels, solar collectors, or size of green roof being installed		
5) Choose specific location on roof where solar panels, green roof, or solar collectors will be installed	a. Solar panels and solar collectors depend on pitch of roof	Design drawings	All
	b. Solar panels and solar collectors must be positioned to face south		
	c. Roof must be flat for green roof		
6) Outline construction process of installing solar panels, green roof, or solar collectors	a. Database research on step by step construction process for installing solar panel, green roof, or solar collector system	Online research	Ryan

Table 8: Steps and Tasks for the Structural Analysis and Design

Steps	Specific Tasks	References	Person(s) Responsible
1) Set up Excel sheet which will calculate dead loads, wind loads, snow loads, and seismic loads of solar panels, green roofs, and solar collectors	a. Identify calculation process in accordance with ASCE 7-10	ASCE 7-10	Sebastian and Ian
2) Input values to calculate different loads	a. Depends on type of solar panel, green roof, or solar collector system, dimensions, weight, and number of solar panels and solar collectors	Excel sheet from Step 1	Ryan
	b. Calculate for each building and type of solar panel, green roof, or solar collector system		
3) Analyze structure of each building to determine feasibility of installing solar panels, green roof, or solar collectors on roof of building	a. Identify material and dimensions of roof structure and building frame	Design drawings & design specifications (AISC, ACI, NDS)	All
	b. Benchmark the capacity of the exisiting roof structure and supporting elements (columns)		
	c. Use design drawings of building to perform analysis		
4) Determine if structural reinforcements are needed to support solar panels, green roof, or solar collectors	a. Identify which structural reinforcements should be applied to the building	Online research	All
	b. Design added structural reinforcements		

Table 9: Steps and Tasks for the Evaluation and Recommendation

Steps	Specific Tasks	References	Person(s) Responsible
1) Determine the current cost of energy requirement of the building	a. Will vary per building	Obtain from interview	All
	b. Previously determined from meeting with WPI Facilities Department		
2) Compare cost values to determine if it is economically feasible to install solar panels, green roofs, and solar collectors	a. Cost values include:	Obtain from interview and online research	All
	i. Current cost of energy consumption of the building		
	ii. Cost for installation of solar panels, green roofs, and solar collectors		
	iii. Lifetime of solar panels, green roofs, and solar collectors: maintenance costs overtime		
	b. Compare values i and ii+iii above and see which is greater to determine feasibility		
	i. Will involve life-cycle cost analysis with time value of money		
	c. Determine for each chosen building		
3) Create deliverables	a. Comprehensive proposal handbook:	Microsoft Word	Ryan
	i. Outline plan for implementing sustainable roofing practices		
	ii. Type of solar panel, green roof, or solar collector system		
	iii. Roof layout		
	iv. Structural reinforcements		
	v. Economic evaluation		
	b. Engineering design drawings	AutoCAD and Revit	Sebastian and Ian

7 Deliverables

Our deliverables will include recommendations for the WPI Facilities Department on ways they can implement sustainable rooftop technologies on a defined set of buildings at WPI. The vision of WPI's sustainability plan states: "We at WPI will demonstrate our commitment to the preservation of the planet and all its life through the incorporation of the principles of sustainability throughout the institution (WPI Sustainability Plan, 2017)." We can contribute to this vision by providing the school with a set of recommendations for the implementation of sustainable rooftop technologies.

The recommendations given to WPI will be in the form of a proposal handbook, which will outline a plan for implementing sustainable rooftop systems on specific buildings at WPI. The handbook will outline a plan for implementing a sustainable roofing practice on one or more buildings at WPI. The plan will contain information on the type of solar panel, green roof, or solar collector system, the roof layout, structural reinforcements, and an economic evaluation which will identify how much money is saved for the building overtime. By providing WPI Facilities Department with a handbook outlining a plan for sustainable rooftop technologies on different buildings, the Department has the opportunity to further contribute to the vision outlined in WPI's sustainability plan.

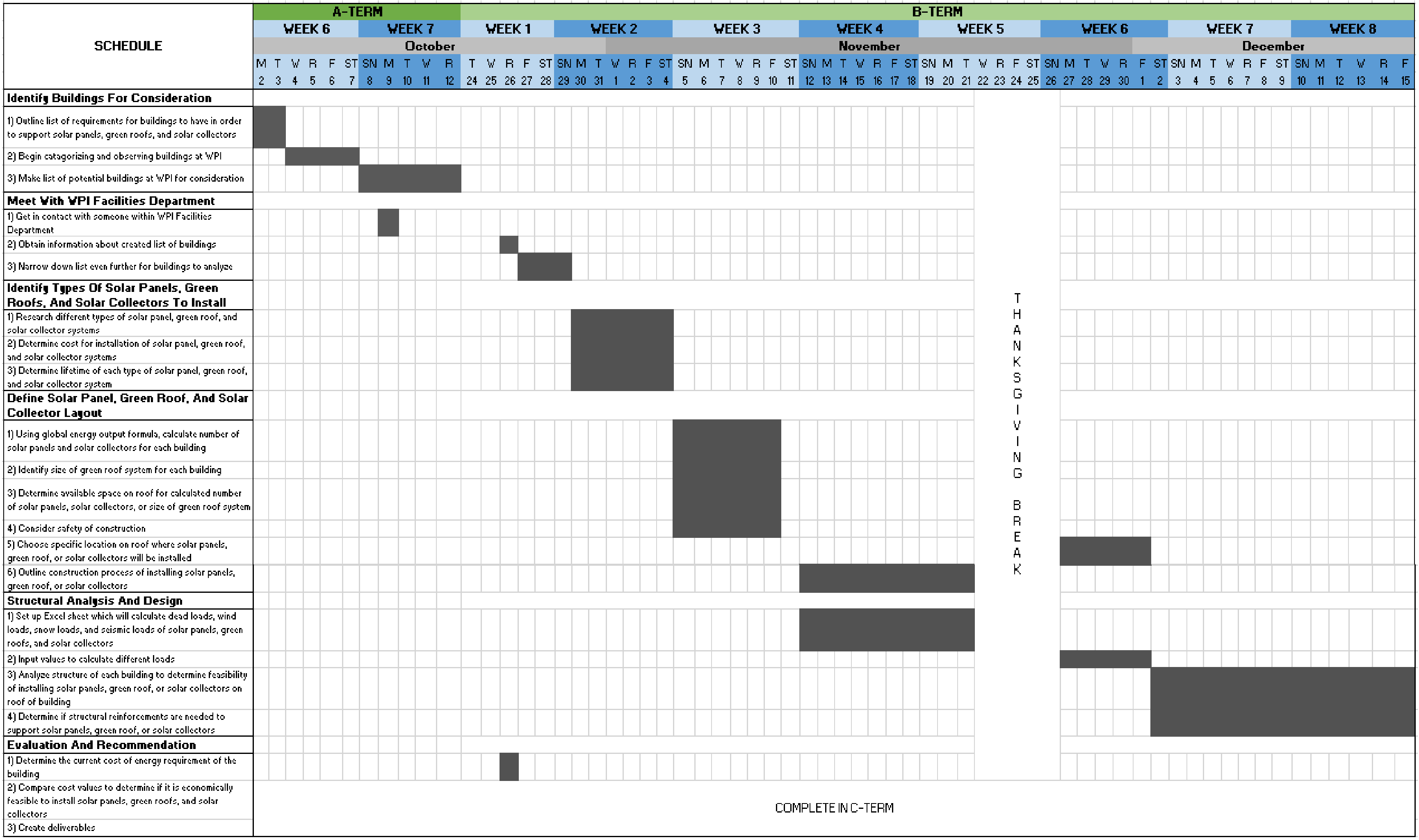
Additionally, we will create engineering drawings using Revit and/or AutoCAD to present and document the proposed sustainable roofing practice on each of the buildings. This will include the location and dimensions of the sustainable roofing practice, as well as any structural reinforcements on the columns or roof structure of the building. These drawings will be created in accordance with the actual design drawings of the building.

As described, the products of our project will include a handbook and engineering drawings. By producing these deliverables, we can provide WPI Facilities Department with a comprehensive outlined plan for implementing sustainable roofing practices on different buildings at WPI. Our MQP provides an opportunity for WPI to further enhance its sustainability plan, and commit to the vision they have set out in the plan.

8 Conclusions

The expectation of this project is to identify buildings at WPI where an analysis will be performed for the installation of solar panels, green roofs, and solar collectors. The analysis will be completed on one or more buildings for the installation of solar panels, green roofs, and solar collectors. A structural analysis and economic evaluation will be performed to determine the feasibility of installing the sustainable roofing system for each building. Finally, a comprehensive proposal handbook and engineering design drawings will outline and display the process for installing solar panels, green roofs, and solar collectors on each of the chosen buildings. These deliverables will be given to the WPI Facilities Department at the end of the project. By completing this project, we will contribute to WPI's sustainability plan and serve the community in an environmental, economic, and ethical way.

9 Schedule


A-TERM
B-TERM
WEEK 6
WEEK 7
WEEK 1
WEEK 2
WEEK 3
WEEK 4
WEEK 5
WEEK 6
WEEK 7
WEEK 8
SCHEDULE
October
November
December
Identify Buildings For Consideration
1) Outline list of requirements for buildings to have in order to support solar panels, green roofs, and solar collectors
2) Begin catagorizing and observing buildings at WPI
3) Make list of potential buildings at WPI for consideration
Meet With WPI Facilities Department
1) Get in contact with someone within WPI Facilities Department
2) Obtain information about created list of buildings
3) Narrow down list even further for buildings to analyze
Identify Types Of Solar Panels, Green Roofs, And Solar Collectors To Install
1) Research different types of solar panel, green roof, and solar collector systems
2) Determine cost for installation of solar panel, green roof, and solar collector systems
3) Determine lifetime of each type of solar panel, green roof, and solar collector system
Define Solar Panel, Green Roof, And Solar Collector Layout
1) Using global energy output formula, calculate number of solar panels and solar collectors for each building
2) Identify size of green roof system for each building
3) Determine available space on roof for calculated number of solar panels, solar collectors, or size of green roof system
4) Consider safety of construction
5) Choose specific location on roof where solar panels, green roof, or solar collectors will be installed
6) Outline construction process of installing solar panels, green roof, or solar collectors
Structural Analysis And Design
1) Set up Excel sheet which will calculate dead loads, wind loads, snow loads, and seismic loads of solar panels, green roofs, and solar collectors
2) Input values to calculate different loads
3) Analyze structure of each building to determine feasibility of installing solar panels, green roof, or solar collectors on roof of building
4) Determine if structural reinforcements are needed to support solar panels, green roof, or solar collectors
Evaluation And Recommendation
1) Determine the current cost of energy requirement of the building
2) Compare cost values to determine if it is economically feasible to install solar panels, green roofs, and solar collectors
3) Create deliverables
THANKSGIVING BREAK
COMPLETE IN C-TERM

10 References

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APPENDIX B: SOLAR PANEL CALCULATIONS

Appendix B.1: Solar Panel Load Calculations

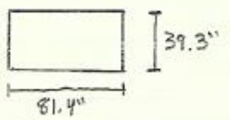
Gateway Parking Garage | Solar Panel Load Calculations | Appendix B.1 | 1/5

Solar Panel Layout

272 panels - Sunpower SPR-P17-350-COM

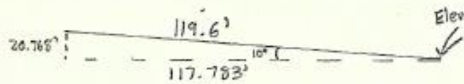
Dimensions of Panels: 81.4" x 39.3" x 1.8" (1/4" spacing b/w panels)

Orientation:



Landscape Orientation (Required for model type)

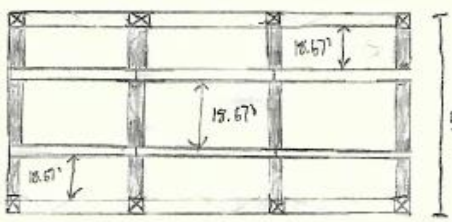
Angle of System:



$\measuredangle = 10°$

Original Overhead View (Flat):

NOT DRAWN TO SCALE



⊠ = Columns, ▯ = Girders, ▯ = Beams — Size TBD

Flat: 27.783' | 45' | 45'

@ $\measuredangle$ 10°: 28.21' | 45.69' | 45.69'

34 panels

Live Load and Rain Load

Negligible for solar panels

Dead Load - D

One Panel = 51 lbs

51 lbs × 272 panels = 13,872 lbs

Area of Panels = 119.6ft × 56.0167ft = 6,699.59732 ft²

13,872 lbs / 6699.59732 ft² = 2.07 psf

$\boxed{D = 2.07\ psf}$

Snow Load - S

Sloped Roof Snow Loads (Section 7.4)

$p_s = C_s p_f$ for an unobstructed slippery surface

Thermal Factor (Table 7-3)

$C_t = 1.0$ for all structures except indicated below

Cold Roof Slope Factor (Section 7.4.2)

Since $C_t = 1.0$

=> Dashed line on Fig. 7-2a

Using 10° roof slope

$C_s \doteq 0.92$

Exposure Factor (Table 7-2)

=> Terrain Category B (Section 26.7)

Fully Exposed Roof

$\therefore C_e = 0.9$

Importance Factor (Table 1.5-2; Table 1.5-1)

Risk Category II

$I_s = 1.00$

Ground Snow Loads (Figure 7-1)

Worcester, MA:

$p_g = 50\,psf$

Flat Roof Snow Load (Section 7.3)

$p_f = 0.7 C_e C_t I_s p_g$

$p_f = 0.7(0.9)(1.0)(1.0)(50psf)$

$p_f = 31.5\,psf$

$\therefore p_s = C_s p_f$

$p_s = (0.92)(31.5\,psf)$

$p_s = 28.98\,psf$ $\boxed{S = 28.98\,psf}$

Wind Load - W

From ASCE 7-10:

1) Risk Category (Table 1.5-1)

Risk Category II

2) Basic Wind Speed (Fig. 26.5-1A)

Worcester, MA:

$V = 120\,mph$

3) Wind Directionality Factor (Table 26.6-1)

Structure Type - Buildings

$K_d = 0.85$

Exposure Category (Section 26.7)

Terrain Category B

Topographic Factor (Section 26.8)

No topographic effects

$K_{zt} = 1.0$

Gust Effect Factor (Section 26.9)

$G = 0.85$

4) Velocity Pressure Exposure Coefficient (Table 29.3-1)

Height above ground level = 60ft

Exposure Category B

$K_z = 0.85$

5) Velocity Pressure (Section 29.3.2)

$q_z = 0.00256 K_z K_{zt} K_d V^2$

$q_z = 0.00256(0.85)(1.0)(0.85)(120mph)^2$

$q_z = 26.63\ psf$

From Wind Design For Low-Profile Solar Photovoltaic Arrays on Flat Roofs Document:

1) Compute a_{pv}

$a_{pv} = 0.5\sqrt{h W_L}$ h = height of building

$a_{pv} = 0.5\sqrt{(60ft)(268ft)}$ W_L = width of building on the longest side

$a_{pv} = 63.4ft$

=> a_{pv} must be $\le h$; since inequality not satisfied:

$a_{pv} = h = 60ft$

2) Calculate Normalized Wind Area

$A_n = \left(\frac{1000}{(\max(a_{pv}, 15ft))^2}\right) A$ A = tributary area of beam

$A_n = \left(\frac{1000}{(60ft)^2}\right)(18.67ft)(45ft)$

$A_n = 233.375$

3) Determine Nominal Net Pressure (Figure 29.9-1)

$15° \le \omega \le 35°$

$(GC_m)_{nom} = 1.1$ for $A_n = 233.375$

$0° \le \omega \le 5°$

$(GC_m)_{nom} = 0.75$ for $A_n = 233.375$

=> since our application $\omega = 10°$

$\frac{1.1 + 0.75}{2} = 0.925 = (GC_m)_{nom}$

4) Calculate Panel Chord Length Factor

$\gamma_c = 0.6 + 0.06\,\ell_p$ ℓ_p = chord length of solar panel

$\gamma_c = 0.6 + 0.06(3.275ft)$

$\gamma_c = 0.8$

5) Determine Parapet Height Factor

$\frac{10ft + 30.768ft}{2} = 20.384ft = h_{pt}$ h_{pt} = Mean parapet height above adjacent roof surface

For $h_{pt} > 4ft$:

$\gamma_p = 0.25(20.384ft)$

$\gamma_p = 5.096$

=> γ_p must be ≤ 1.3; since inequality not satisfied:

$\gamma_p = 1.3$

6) Determine Characteristic Height

$h_c = \min(h_1, 1ft) + \ell_p \sin(\omega)$ h_1 = solar panel height above roof at low edge

$h_c = \min(10ft, 1ft) + (3.275ft)\sin 10°$

$h_c = 1.57ft$

7) Determine Array Edge Factor (Figure 29.9-1)

$d_x/h_c = 1ft/1.57ft$ d_x = horizontal distance from edge of panel to edge of roof

$d_x/h_c = 0.64$

$\therefore E = 1.0$

8) Calculate Net Pressure Coefficient

$GC_m = \gamma_p E[(GC_m)_{nom}(\gamma_c)]$

$GC_m = 1.3(1.0)[(0.925)(0.8)]$

$GC_m = 0.962$

9) Calculate Design Wind Pressure

$p = q_z G C_m$

$p = (26.63 psf)(0.962)$

$p = 25.618 psf$ $\boxed{W = 25.618 psf}$

Seismic Load – E

Risk-Targeted Maximum Considered Earthquake Spectral Response Accelerations

S_S: (Fig. 22-1)

Worcester, MA: $MCE_R = 18\%\,g$

S_1: (Fig. 22-2)

Worcester, MA: $MCE_R = 7\%\,g$

Seismic Design Category

Soil Classification (Section 20)

Site D – Details unknown

Design Spectral Acceleration Parameters (Section 11.4.4)

$S_{DS} = \frac{2}{3} S_{MS}$

$S_{D1} = \frac{2}{3} S_{M1}$

Spectral Response Acceleration Parameter (Section 11.4.3)

$S_{MS} = F_a S_s$

$S_{M1} = F_v S_1$

Site Coefficients

For Site D and $S_s \le 0.25$

$F_a = 1.6$ (Table 11.4-1)

For Site D and $S_1 \le 0.1$

$F_v = 2.4$ (Table 11.4-2)

$\therefore S_{MS} = (1.6)(0.18)$ $S_{M1} = (2.4)(0.07)$

$S_{MS} = 0.288$ $S_{M1} = 0.168$

$\therefore S_{DS} = \frac{2}{3}(0.288)$ $S_{D1} = \frac{2}{3}(0.168)$

$S_{DS} = 0.192$ $S_{D1} = 0.112$

Risk Category (Table 1.5-1)

Risk Category II

Seismic Design Category (Table 11.6-1)

For $0.167 \le S_{DS} < 0.33$ and Risk Category II

SDC = B

Seismic Importance Factor (Table 1.5-2)

For Risk Category II

$I_e = 1.00$

Seismic Base Shear (Section 15.4.1.2)

$V = 0.30 S_{DS} W I_e$ For rigid nonbuilding structures

$V = 0.30(0.192)(2.07 psf)(1.00)$

$V = 0.119232 psf$

Fundamental Period (Section 12.8.2.1.)

$T_a = C_t h_n^x$

$\left.\begin{array}{l} C_t = 0.02 \\ x = 0.75 \end{array}\right\}$ All other structural systems (Table 12.8-2)

$h_n = \frac{10ft + 30.768ft}{2} = 20.384 ft$ – Structural height

$T_a = (0.02)(20.384 ft)^{0.75}$

$T_a = 0.19s = T$

Vertical Distribution Factor (Section 12.8.3)

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

$w_x = 2.07$ psf – weight

$h_x = 20.384$ ft

$k = 1$ – since $T \le 0.5s$ $(T = 0.19s)$

$C_{vx} = \frac{(2.07 psf)(20.384 ft)^1}{(2.07 psf)(20.384 ft)^1}$

$C_{vx} = 1.0$

Lateral Seismic Force (Section 12.8.3)

$F_x = C_{vx} V$

$F_x = (1.0)(0.119232 psf)$

$F_x = 0.119232$ psf

Seismic Load Effect (Section 12.4.2)

$E = E_h + E_v$

Horizontal Seismic Load Effect (Section 12.4.2.1)

$E_h = \rho Q_E$

ρ – Redundancy Factor (Section 12.3.4)

$\rho = 1.0$ – Seismic Design Category B

$Q_E = 0.119232$ psf $= F_x$

$E_h = (1.0)(0.119232 psf)$

$\boxed{E_h = 0.119232 psf}$

Vertical Seismic Load Effect (Section 12.4.2.2)

$E_v = 0.2 S_{DS} D$

$E_v = 0.2(0.192)(2.07psf)$

$\boxed{E_v = 0.079 psf}$

Summary

D = 2.07 psf

S = 28.98 psf

W = 25.618 psf

E_h = 0.119232 psf E_v = 0.079 psf

L = 0 psf

R = 0 psf

Load Combinations:

1. 1.4 D
2. 1.2 D + 1.6L + 0.5(Lr or S or R)
3. 1.2 D + 1.6(Lr or S or R) + (L or 0.5 W)
4. 1.2D + 1.0W + L + 0.5(Lr or S or R)
5. 1.2D + Ev + L + 0.2S
6. 0.9D + 1.0 W
7. 0.9D + 1.0 Eh

Gravity Loads

Lateral Loads

Appendix B.2: Beam Calculations

Load Combinations

$1.4D = 1.4(2.07psf) = 2.90\ psf$

$1.2D + 1.6L + 0.5(Lr/S/R) = 1.2(2.07psf) + 1.6(0psf) + 0.5(28.98psf) = 16.97\ psf$

$1.2D + 1.6(Lr/S/R) + (L/0.5W) = 1.2(2.07psf) + 1.6(28.98psf) + 0.5(25.618psf) = 61.61\ psf$

$1.2D + 1.0W + L + 0.5(Lr/S/R) = 1.2(2.07psf) + 1.0(25.618psf) + 0psf + 0.5(28.98psf) = 42.48psf$

$1.2D + Ev + L + 0.2S = 1.2(2.07psf) + 0.079psf + 0psf + 0.2(28.98psf)$
$= 8.36psf$

$0.9D + 1.0W = 0.9(2.07psf) + 1.0(0\ psf) = 1.86\ psf$

$0.9D + 1.0Eh = 0.9(2.07psf) + 1.0(0.119232\ psf) = 1.98psf$

Governing Load = 61.61 psf

Exterior Beams

Tributary Width $= 18.67ft/_2 = 9.335ft$

$W_u = 61.61psf \times 9.335ft \times (k/1000lb) = 0.58\ k/ft$

Max. Beam Length = 45.69ft

$M_u = \frac{w_u L^2}{8} = \frac{(0.58k/ft)(45.69ft)^2}{8} = 150.073\ k\cdot ft$

$Z_x = \frac{M_u}{\phi F_y} = \frac{(150.073\ k\cdot ft)(12in/ft)}{(0.9)(50ksi)} = 40.0in^3$

AISC Table 3-2: W14×26 => $Z_x = 40.2in^3 \geq 40.0in^3$ ✓

Selected Beam Weight = $26^{lb}/ft$

$w_u = 0.58\ k/ft + 1.2(26 lb/ft)(k/1000lb) = 0.606\ k/ft$

$M_u = \frac{w_u L^2}{8} = \frac{(0.606k/ft)(45.69ft)^2}{8} = 158.214\ k\cdot ft$

$Z_x = \frac{M_u}{\phi F_y} = \frac{(158.214\ k\cdot ft)(12in/ft)}{(0.9)(50ksi)} = 42.2in^3 \geq 40.2in^3$

AISC Table 3-2: W24×55 => $Z_x = 134in^3 \geq 42.2in^3$ ✓

New Selected Beam Weight = $55^{lb}/ft$

$w_u = 0.606\ k/ft + 1.2(55lb/ft)(k/1000lb) = 0.672\ k/ft$

$M_u = \frac{w_u L^2}{8} = \frac{(0.672k/ft)(45.69ft)^2}{8} = 175.437\ k\cdot ft$

$Z_x = \frac{M_u}{\phi F_y} = \frac{(175.437\ k\cdot ft)(12in/ft)}{(0.9)(50ksi)} = 46.8in^3 \leq 134in^3$ ✓

Flange Local Buckling (FLB):

$b_f/2t_f \leq 0.38\sqrt{\frac{E}{F_y}} = 0.38\sqrt{\frac{29{,}000ksi}{50ksi}}$ AISC Table 1-1 for W24×55: $b_f/2t_f = 6.94$

$6.94 \leq 9.15$ ✓

Web Local Buckling (WLB):

$h/t_w \leq 3.76\sqrt{\frac{E}{F_y}} = 3.76\sqrt{\frac{29{,}000ksi}{50ksi}}$ AISC Table 1-1 for W24×55: $h/t_w = 54.6$

$54.6 \leq 90.6$ ✓

Total Service Load:

$\Delta_T = \frac{5 w_T L^4}{384 E I_x}$

$\Delta_T = \frac{5(344.857 \text{ lb/ft})(45.69 \text{ ft})^4}{384(29\times10^6 \text{ psi})(1350 \text{ in}^4)} \times \frac{1728 \text{ in}^3}{\text{ft}^3}$

$\Delta_T = 0.86$ in

$0.86 \text{ in} < 2.28 \text{ in}$ ✓

$W_T = (D+S)$ Tributary Width + Weight of Beam

$W_T = (2.07 \text{ psf} + 28.98 \text{ psf})(9.335 \text{ ft}) + 55 \text{ lb/ft}$

$W_T = 344.857 \text{ lb/ft}$

I_x: from AISC Table 3-3 for beam size

$\text{Limit} = L/240 = \frac{(45.69 \text{ ft})(12 \text{ in/ft})}{240} = 2.28 \text{ in}$

Snow Deflection:

$\Delta_S = \frac{5 w_S L^4}{384 E I_x}$

$\Delta_S = \frac{5(270.528 \text{ lb/ft})(45.69 \text{ ft})^4}{384(29\times10^6 \text{ psi})(1350 \text{ in}^4)} \times \frac{1728 \text{ in}^3}{\text{ft}^3}$

$\Delta_S = 0.68$ in

$0.68 \text{ in} < 1 \text{ in}$

$W_S = S \times$ Tributary Width

$W_S = (28.98 \text{ psf})\, 9.335 \text{ ft}$

$W_S = 270.528 \text{ lb/ft}$

$\text{Limit} = L/360 \text{ or } 1" = \frac{(45.69 \text{ ft})(12 \text{ in/ft})}{360} = 1.5 \text{ in}$

Limit = 1 in

Due to satisfication of FLB, WLB, Total Service Load, Snow Deflection:

$\boxed{\text{Exterior Beam Size} = W24\times55}$

<u>Interior Beams</u>

Governing Load = 61.61 psf

Tributary Width = 18.67 ft

$W_u = 61.61 \text{ psf} \times 18.67 \text{ ft} \times (\text{k}/1000 \text{ lb}) = 1.15 \text{ k/ft}$

Max Beam Length = 45.69 ft

$M_u = \frac{w_u L^2}{8} = \frac{(1.15 \text{ k/ft})(45.69 \text{ ft})^2}{8} = 300.145 \text{ k}\cdot\text{ft}$

$Z_x = \frac{M_u}{\phi F_y} = \frac{(300.145 \text{ k}\cdot\text{ft})(12 \text{ in/ft})}{(0.9)(50 \text{ ksi})} = 80.0 \text{ in}^3$

AISC Table 3-2: $W21\times44 \Rightarrow Z_x = 95.4 \text{ in}^3 \geq 80.0 \text{ in}^3$ ✓

New Selected Beam Weight = 44 lb/ft

$W_u = 1.15 \text{ k/ft} + 1.2(44 \text{ lb/ft})(\text{k}/1000 \text{ lb}) = 1.203 \text{ k/ft}$

$M_u = \frac{W_u L^2}{8} = \frac{(1.203 \text{ k/ft})(45.69 \text{ ft})^2}{8} = 313.923 \text{ k}\cdot\text{ft}$

$Z_x = \frac{M_u}{\phi F_y} = \frac{313.923 \text{ k}\cdot\text{ft}(12 \text{ in/ft})}{(0.9)(50 \text{ ksi})} = 83.7 \text{ in}^3 \leq 95.4 \text{ in}^3$ ⇒ However Deflection Performance Not Satisfied

AISC Table 3-2: $W24\times68 \Rightarrow Z_x = 177 \text{ in}^3 \geq 83.7 \text{ in}^3$ ✓

New Selected Beam Weight = 68 lb/ft

$W_u = 1.203 \text{ k/ft} + 1.2(68 \text{ lb/ft})(\text{k}/1000 \text{ lb}) = 1.285 \text{ k/ft}$

$M_u = \frac{W_u L^2}{8} = \frac{(1.285 \text{ k/ft})(45.69 \text{ ft})^2}{8} = 335.217 \text{ k}\cdot\text{ft}$

$Zx = \frac{M_u}{\phi F_y} = \frac{335.217\ k\cdot ft\ (12in/ft)}{(0.9)(50ksi)} = 89.4in^3 \leq 177in^3$ ✓

Flange Local Buckling (FLB):

$b_f/2t_f \leq 0.38\sqrt{\frac{E}{F_y}}$ AISC Table 1-1: $b_f/2t_f = 7.66$ for W24x68

$7.66 \leq 9.15$ ✓

Web Local Buckling (WLB):

$h/t_w \leq 3.76\sqrt{\frac{E}{F_y}}$ AISC Table 1-1: $h/t_w = 52$ for W24x68

$52 \leq 90.6$ ✓

Total Service Load:

$\Delta_T = \frac{5 w_T L^4}{384 E I_x}$

$w_T = (D+S)$ Tributary Width + Weight of Beam

$w_T = (2.07psf + 28.98psf)(18.67ft) + 68\ lb/ft$

$w_T = 647.714\ lb/ft$

$\Delta_T = \frac{5(647.714\ lb/ft)(45.69ft)^4}{384(29\times10^6 psi)(1830in^4)} \times \frac{1728 in^3}{ft^3}$

$\Delta_T = 1.20$ in

I_x: From AISC Table 3-3 for beam size

$1.20in < 2.28in$ ✓

$Limit = L/240 = (45.69ft)(12in/ft)/240 = 2.28in$

Snow Deflection:

$\Delta_S = \frac{5 w_S L^4}{384 E I_x}$

$W_S = S \times$ Tributary Width

$W_S = (28.98psf)(18.67ft)$

$W_S = 541.057\ lb/ft$

$\Delta_S = \frac{5(541.057\ lb/ft)(45.69ft)^4}{384(29\times10^6 psi)(1830in^4)} \times \frac{1728 in^3}{ft^3}$

$\Delta_S = 1.00in$

$Limit = L/360$ or $1" = \frac{(45.69ft)(12in/ft)}{360} = 1.5in$

$1.00in \leq 1.00in$ ✓

$Limit = 1in$

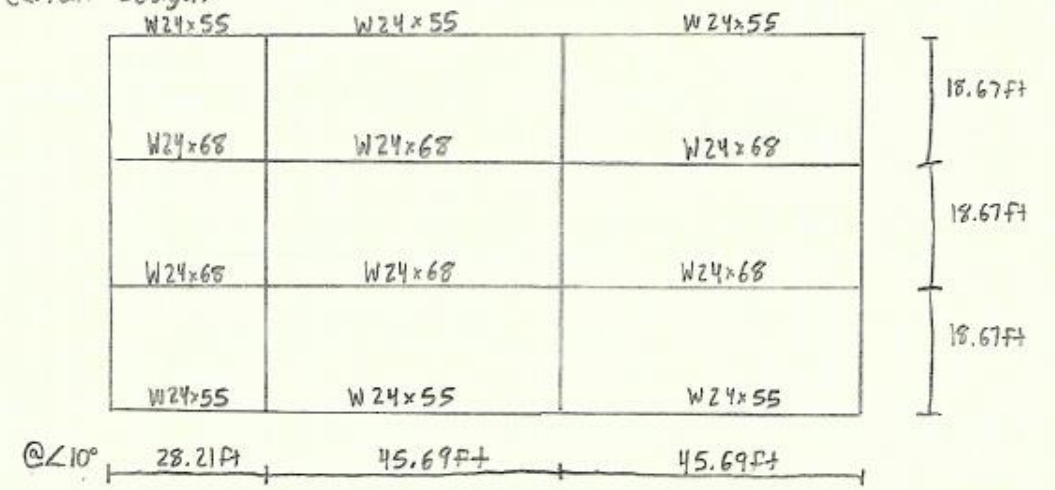
Due to satisfaction of FLB, WLB, Total Service Load, Snow Deflection:

$\boxed{\text{Interior Beam Size} = W24\times68}$

Appendix B.3: Laterally Unsupported Beam Calculations

Laterally Unsupported Beams

Current Design:



Unbraced Length = 45.69 ft (L_b)

W24x55: $M_u = 175.437$ k·ft ($L = 45.69$ft)

$L_p = 1.76 r_y \sqrt{E/F_y}$

$L_p = 1.76(1.34\text{in})\sqrt{29000\text{ksi}/50\text{ksi}}$

$L_p = 56.80\text{in} = 4.73\text{ft}$

$$L_r = 1.95 r_{ts}\left(\frac{E}{0.7F_y}\right)\sqrt{\frac{Jc}{S_x h_o} + \sqrt{\left(\frac{Jc}{S_x h_o}\right)^2 + 6.76\left(\frac{0.7F_y}{E}\right)^2}}$$

$$L_r = 1.95(1.72\text{in})\left(\frac{29000\text{ksi}}{0.7(50\text{ksi})}\right)\sqrt{\frac{(1.18\text{in}^4)(1.0)}{(114\text{in}^3)(23.1\text{in})} + \sqrt{\left(\frac{(1.18\text{in}^4)(1.0)}{(114\text{in}^3)(23.1\text{in})}\right)^2 + 6.76\left(\frac{0.7(50\text{ksi})}{29000\text{ksi}}\right)^2}}$$

$L_r = 167.15\text{in} = 13.92\text{ft}$

4.73ft < 13.92ft < 45.69ft

$L_p < L_r < L_b$

∴ Zone 3 (Elastic Buckling)

W24x68: $M_u = 335.217$ k·ft ($L = 45.69$ft)

$L_p = 1.76(1.87\text{in})\sqrt{29000\text{ksi}/50\text{ksi}}$

$L_p = 79.26\text{in} = 6.61\text{ft}$

$$L_r = 1.95(2.30\text{in})\left(\frac{29000\text{ksi}}{0.7(50\text{ksi})}\right)\sqrt{\frac{(1.87\text{in}^4)(1.0)}{(154\text{in}^3)(23.1\text{in})} + \sqrt{\left(\frac{(1.87\text{in}^4)(1.0)}{(154\text{in}^3)(23.1\text{in})}\right)^2 + 6.76\left(\frac{0.7(50\text{ksi})}{29000\text{ksi}}\right)^2}}$$

$L_r = 226.27\text{in} = 18.86\text{ft}$

6.61ft < 18.86ft < 45.69ft

$L_p < L_r < L_b$

∴ Zone 3 (Elastic Buckling)

$M_n = F_{cr} S_x$

$$M_n = \left[\left(\frac{C_b \pi^2 E}{(L_b/r_{ts})^2}\right)\sqrt{1 + 0.078\frac{Jc}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2}\right] S_x$$

For Elastic Buckling

For W24×55: Elastic Buckling

$$M_n = \left[\left(\frac{(1.0)\pi^2(29000\,ksi)}{(45.69ft\times 12^{in}/_{ft}/1.72in)^2}\right)\sqrt{1+0.078\frac{(1.18in^4)(1)}{(114in^3)(23.1in)}\left(\frac{45.69ft\times 12^{in}/_{ft}}{1.72in}\right)^2}\right](114in^3)\left(\frac{ft}{12in}\right)$$

$M_n = 57\,k\cdot ft$

$\phi M_n = (0.9)(57\,k\cdot ft) = 51.38\,k\cdot ft$

$175.437\,k\cdot ft \nless 51.38\,k\cdot ft$

$M_u \nless \phi M_n$

$\therefore$ Decrease L_b

For W24×68: Elastic Buckling

$$M_n = \left[\left(\frac{(1.0)\pi^2(29000\,ksi)}{(45.69ft\times 12^{in}/_{ft}/2.30in)^2}\right)\sqrt{1+0.078\frac{(1.87in^4)(1)}{(154in^3)(23.1in)}\left(\frac{45.69ft\times 12^{in}/_{ft}}{2.30in}\right)^2}\right](154in^3)\left(\frac{ft}{12in}\right)$$

$M_n = 118\,k\cdot ft$

$\phi M_n = (0.9)(118\,k\cdot ft) = 106.16\,k\cdot ft$

$335.217\,k\cdot ft \nless 106.16\,k\cdot ft$

$M_u \nless \phi M_n$

$\therefore$ Decrease L_b

Changed Unbraced Length = 15.23ft (L_b)

W24×55: $M_u = 175.437\,k\cdot ft$ ($L = 45.69ft$)

$L_p = 4.73ft \quad L_r = 13.92ft$

$L_p < L_r < L_b \quad \therefore$ Zone 3 (Elastic Buckling)

W24×68: $M_u = 335.217\,k\cdot ft$ ($L = 45.69ft$)

$L_p = 6.61ft \quad L_r = 18.86ft$

$L_p < L_b < L_r \quad \therefore$ Zone 2 (Inelastic Buckling)

For W24×55: Elastic Buckling

$$M_n = \left[\left(\frac{(1.0)\pi^2(29000\,ksi)}{(15.23ft\times 12^{in}/_{ft}/1.72in)^2}\right)\sqrt{1+0.078\frac{(1.18in^4)(1)}{(114in^3)(23.1in)}\left(\frac{15.23ft\times 12^{in}/_{ft}}{1.72in}\right)^2}\right](114in^3)\left(\frac{ft}{12in}\right)$$

$M_n = 284.4\,k\cdot ft$

$\phi M_n = (0.9)(284.4\,k\cdot ft) = 256\,k\cdot ft$

$175.437\,k\cdot ft < 256\,k\cdot ft$

$M_u < \phi M_n$ ✓

For W24×68: Inelastic Buckling

$$M_n = C_b\left[M_p - (M_p - 0.7F_yS_x)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right]$$

$$M_n = (1.0)\left[(177in^3)(50ksi) - \left((177in^3)(50ksi) - 0.7(50ksi)(154in^3)\right)\left(\frac{15.23ft(^{12in}/_{ft}) - 6.61ft(^{12in}/_{ft})}{18.86ft(^{12in}/_{ft}) - 6.61ft(^{12in}/_{ft})}\right)\right]$$

$M_n = 6428\,k\cdot in\left(\frac{ft}{12in}\right)$

$M_n = 535.67\,k\cdot ft$

$\phi M_n = (0.9)(535.67\,k\cdot ft) = 482.1\,k\cdot ft$

$335.217\,k\cdot ft < 482.1\,k\cdot ft$

$M_u < \phi M_n$ ✓

$\boxed{\text{For W24×55 and W24×68 member @ } L = 45.69ft \Rightarrow L_b = 15.23ft}$

Unbraced Length = 28.21 ft (L_b)

W24×55: $M_u = \frac{w_u L^2}{8} = \frac{(0.672\,k/ft)(28.21\,ft)^2}{8} = 66.85\,k \cdot ft$

$L_p = 4.73\,ft \quad L_r = 13.92\,ft$

$L_p < L_r < L_b$

$\therefore$ Zone 3 (Elastic Buckling)

$$M_n = \left[\left(\frac{(1.0)\pi^2(29000\,ksi)}{(28.21\,ft \times 12\,in/ft / 1.72\,in)^2}\right)\sqrt{1 + 0.078\,\frac{(1.18\,in^4)(1)}{(114\,in^3)(23.1\,in)}\left(\frac{28.21\,ft \times 12\,in/ft}{1.72\,in}\right)^2}\,\right](114\,in^3)\left(\frac{ft}{12\,in}\right)$$

$M_n = 107.70\,k \cdot ft$

$\phi M_n = (0.9)(107.70\,k \cdot ft) = 96.93\,k \cdot ft$

$66.85\,k \cdot ft < 96.93\,k \cdot ft$

$M_u < \phi M_n$ ✓

W24×68: $M_u = \frac{w_u L^2}{8} = \frac{(1.285\,k/ft)(28.21\,ft)^2}{8} = 127.83\,k \cdot ft$

$L_p = 6.61\,ft \quad L_r = 18.86\,ft$

$L_p < L_r < L_b$

$\therefore$ Zone 3 (Elastic Buckling)

$$M_n = \left[\left(\frac{(1.0)\pi^2(29000\,ksi)}{(28.21\,ft \times 12\,in/ft / 2.30\,in)^2}\right)\sqrt{1 + 0.078\,\frac{(1.87\,in^4)(1)}{(154\,in^3)(23.1\,in)}\left(\frac{28.21\,ft \times 12\,in/ft}{2.30\,in}\right)^2}\,\right](154\,in^3)\left(\frac{ft}{12\,in}\right)$$

$M_n = 233\,k \cdot ft$

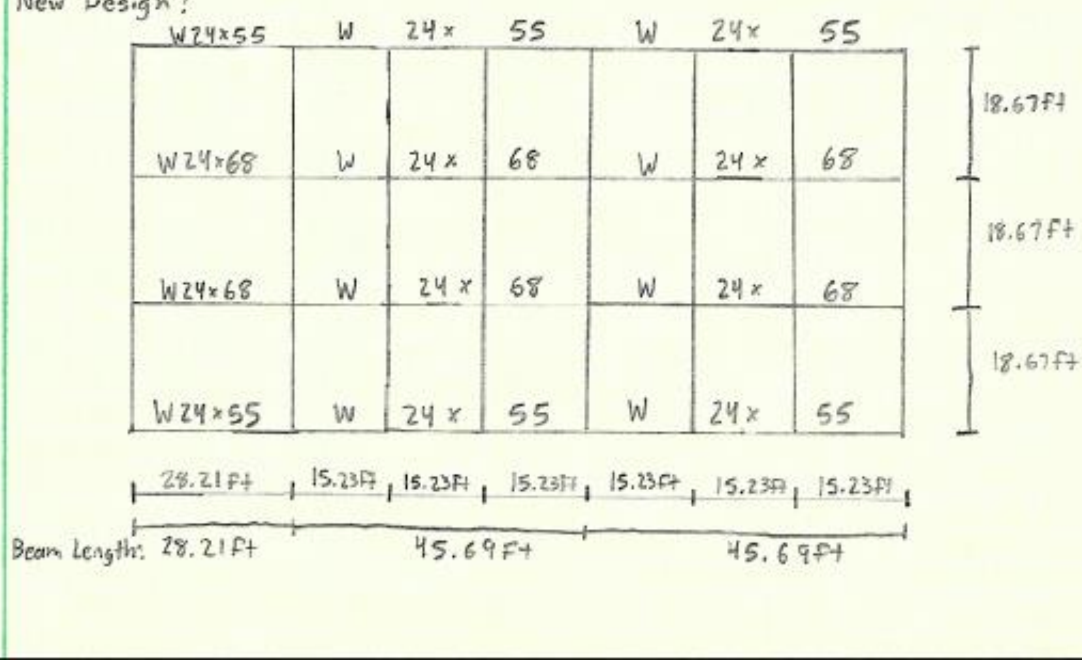
$\phi M_n = (0.9)(233\,k \cdot ft) = 209.7\,k \cdot ft$

$127.83\,k \cdot ft < 209.7\,k \cdot ft$

$M_u < \phi M_n$ ✓

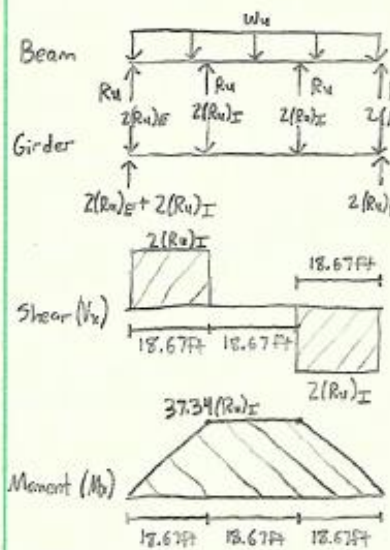
$\boxed{\text{For W24×55 and W24×68 member @ } L = 28.21\,ft \Rightarrow L_b = 28.21\,ft}$

New Design:



Appendix B.4: Girder Calculations

Girders



Exterior $(R_u)_E = \frac{(w_u)_E L}{4} = \frac{(0.672\ k/ft)(45.69ft)}{4}$

$(R_u)_E = 7.68k$

Interior $(R_u)_I = \frac{(w_u)_I L}{4} = \frac{(1.285\ k/ft)(45.69ft)}{4}$

$(R_u)_I = 14.68k$

M_x = Area Under V_x

$M_u = 37.34(R_u)_I$

$M_u = 37.34(14.68k)$

$M_u = 548.15\ k\cdot ft$

$Z_x = \frac{M_u}{\phi F_y} = \frac{(548.15\ k\cdot ft)(12\ in/ft)}{(0.9)(50\ ksi)} = 146.17\ in^3$

AISC Table 3-2: $W21\times68 \Rightarrow Z_x = 160\ in^3 \geq 146.17\ in^3$ ✓

Selected Girder Weight $= 68\ lb/ft$

$W_u = 1.2(68\ lb/ft)\times(k/1000lb) = 0.0816\ k/ft$

$M_u = 548.15\ k\cdot ft + \frac{w_u L^2}{8}$ Length of Girder = 56.0167 ft

$M_u = 548.15\ k\cdot ft + \frac{(0.0816\ k/ft)(56.0167ft)^2}{8} = 580.16\ k\cdot ft$

$Z_x = \frac{M_u}{\phi F_y} = \frac{(580.16\ k\cdot ft)(12\ in/ft)}{(0.9)(50\ ksi)} = 154.7\ in^3 \leq 160\ in^3$ ⇒ However Deflection Performance Not Satisfied

AISC Table 3-2: $W30\times108 \Rightarrow Z_x = 346\ in^3 \geq 146.17\ in^3$ ✓

Selected Girder Weight $= 108\ lb/ft$

$W_u = 1.2(108\ lb/ft)(k/1000lb) = 0.1296\ k/ft$

$M_u = 548.15\ k\cdot ft + \frac{(0.1296\ k/ft)(56.0167ft)^2}{8} = 598.98\ k\cdot ft$

$Z_x = \frac{M_u}{\phi F_y} = \frac{(598.98\ k\cdot ft)(12\ in/ft)}{(0.9)(50\ ksi)} = 159.7\ in^3 \leq 346\ in^3$ ✓

Flange Local Buckling (FLB):

$b_f/2t_f \leq 0.38\sqrt{\frac{E}{F_y}}$

$6.89 \leq 9.15$ ✓

AISC Table 1-1, for $W30\times108$: $b_f/2t_f = 6.89$

Web Local Buckling (WLB):

$h/t_w \leq 3.76\sqrt{\frac{E}{F_y}}$

$49.6 \leq 90.6$ ✓

AISC Table 1-1, for W30×108: $h/t_w = 49.6$

Total Service Load:

$\Delta_T = \frac{M_u L^2}{C_1 I_x}$

$\Delta_T = \frac{(598.99 k \cdot ft)(56.0167 ft)^2}{(158)(4470 in^4)}$

$\Delta_T = 2.66$ in

$C_1 = 158$ for acting point loads

I_x: From AISC Table 3-3 for girder size

Limit $= L/240 = \frac{(56.0167 ft)(12 in/ft)}{240} = 2.80 in$

$2.66 in < 2.80 in$ ✓

Snow Deflection:

$\Delta_S = \frac{5 w_S L^4}{384 E I_x}$

$\Delta_S = \frac{5(441.37 lb/ft)(56.01 ft)^4}{384(29\times10^6 psi)(4470 in^4)} \times \frac{1728 in^3}{ft^3}$

$\Delta_S = 0.75$ in

$w_S = S \times$ Tributary Width

$W_S = (28.98 psf)(15.23 ft)$

$W_S = 441.37$ lb/ft

Limit $= L/360$ or $1'' = \frac{(56.01 ft)(12 in/ft)}{360} = 1.9 in$

Limit = 1 in

$0.75 in < 1 in$ ✓

=> One girder has Tributary Width = 21.72 ft

$W_S = (28.98 psf)(21.72 ft)$

$W_S = 629.45$ lb/ft

$\Delta_S = \frac{5(629.45 lb/ft)(56.01 ft)^4}{384(29\times10^6 psi)(4470 in^4)} \times \frac{1728 in^3}{ft^3}$

$\Delta_S = 1$ in

$1 in \leq 1 in$ ✓

Due to satisfaction of FLB, WLB, Total Service Load, Snow Deflection:

$\boxed{\text{Girder Size = W30} \times 108}$

Appendix B.5: Laterally Unsupported Girders

Laterally Unsupported Girders

Unbraced Length = 18.67 ft (L_b)
W30×108 ⇒ $L_p = 7.59$ ft $L_r = 22.1$ ft $M_u = 598.98\ k\cdot ft$
$L_p < L_b < L_r$ ∴ Zone 2 (Inelastic Buckling)

$$M_n = C_b\left[M_p - (M_p - 0.7F_yS_x)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right]$$

$$M_n = (1.0)\left[(346in^3)(50ksi) - \left((346in^3)(50ksi) - 0.7(50ksi)(299in^3)\right)\left(\frac{18.67ft - 7.59ft}{22.1ft - 7.59ft}\right)\right]$$

$M_n = 434.94\ k\cdot ft$
$\phi M_n = (0.9)(434.94\ k\cdot ft) = 391\ k\cdot ft$

$598.98\ k\cdot ft \nless 391\ k\cdot ft$
∴ Decrease L_b

Changed Unbraced Length = 9.335 ft (L_b)
W30×108 ⇒ $L_p = 7.59$ ft $L_r = 22.1$ ft $M_u = 598.98\ k\cdot ft$
$L_p < L_b < L_r$ ∴ Zone 2 (Inelastic Buckling)

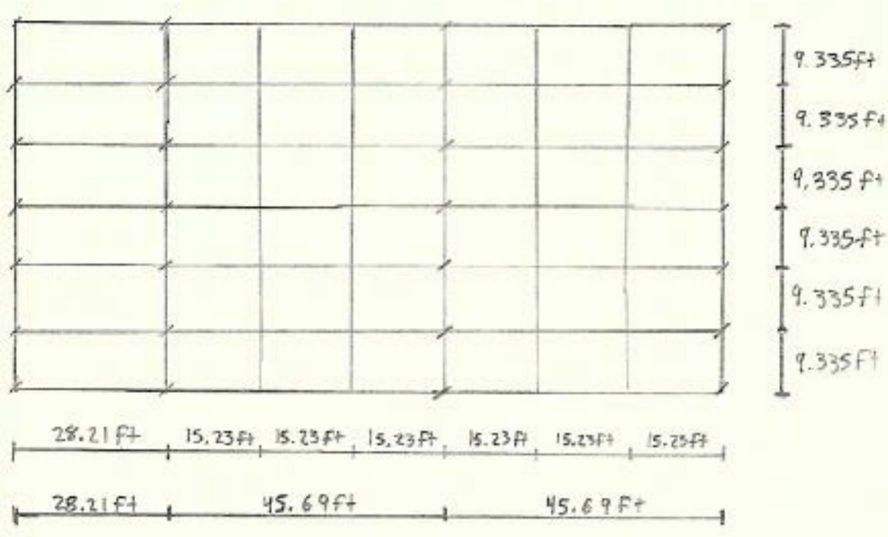
$$M_n = (1.0)\left[(346in^3)(50ksi) - \left((346in^3)(50ksi) - 0.7(50ksi)(299in^3)\right)\left(\frac{9.335ft - 7.59ft}{22.1ft - 7.59ft}\right)\right]$$

$M_n = 1373\ k\cdot ft$
$\phi M_n = (0.9)(1373\ k\cdot ft) = 1236\ k\cdot ft$

$598.98\ k\cdot ft < 1236\ k\cdot ft$ ✓

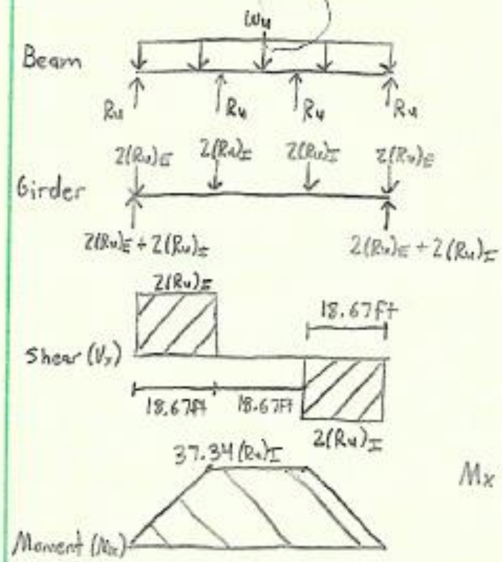
$\boxed{\text{For W30×108 member @ } L = 56.01\text{ ft} \Rightarrow L_b = 9.335\text{ ft}}$

New Design:



Appendix B.6: Column Calculations

Columns



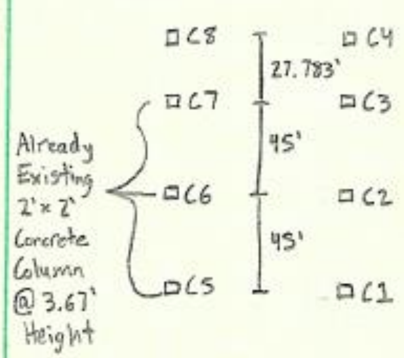
Previously Calculated in Girder Calculations:
Exterior $(R_u)_E = 7.68\,k$
Interior $(R_u)_I = 14.68\,k$

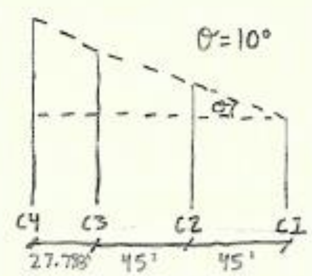
M_x = Area Under V_x

Column

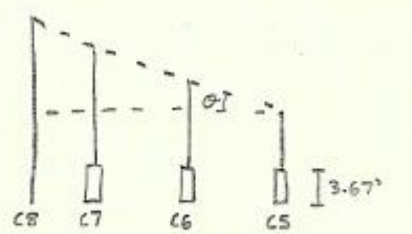
$P_u = 2(R_u)_E + 2(R_u)_I$ + Girder Weight
$P_u = 2(7.68k) + 2(14.68k) + 1.2(108\,lb/ft)(k/1000lb)(56.0167ft/2)$ — Tributary Width
$P_u = 48.35k$

Column Placement and Information:





$L_1 = 10'$ (Minimum Clearance)
$L_2 = 10' + (45')\tan(10°) = 17.93'$
$L_3 = 10' + (90')\tan(10°) = 25.87'$
$L_4 = 10' + (117.783')\tan(10°) = 30.77'$



$L_5 = 10' - 3.67' = 6.33'$
$L_6 = 17.93' - 3.67' = 14.26'$
$L_7 = 25.87' - 3.67' = 22.20'$
$L_8 = 30.77'$

AISC Table 4-1a:

C1: $L_1 = 10'$
$\phi_c P_n = 317k > 48.35k$ ✓
$\therefore$ W8×31

C2: $L_2 = 17.93' \rightarrow 18'$
$\phi_c P_n = 178k > 48.35k$ ✓
$\therefore$ W8×31

C3: $L_3 = 25.87' \rightarrow 26'$
$\phi_c P_n = 86.5k > 48.35k$ ✓
$\therefore$ W8×31

C4: $L_4 = 30.77' \rightarrow 31'$
$\phi_c P_n = 61.0k > 48.35k$ ✓
$\therefore$ W8×31

C5: $L_5 = 6.33' \rightarrow 7'$
$\phi_c P_n = 362k > 48.35k$ ✓
$\therefore$ W8×31

C6: $L_6 = 14.26' \rightarrow 15'$
$\phi_c P_n = 230k > 48.35k$ ✓
$\therefore$ W8×31

C7: $L_7 = 22.20' \rightarrow 23'$
$\phi_c P_n = 111k > 48.35k$ ✓
$\therefore$ W8×31

C8: Same as C4
$\therefore$ W8×31

Column Number	Member Size	Member Length
1	W8×31	10 ft
2	W8×31	17.93 ft
3	W8×31	25.87 ft
4	W8×31	30.77 ft
5	W8×31	6.33 ft
6	W8×31	14.26 ft
7	W8×31	22.20 ft
8	W8×31	30.77 ft

Column Recommendation

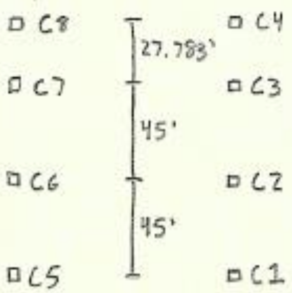
=> For convenience, we recommend placing 2ft x 2ft concrete columns with a height of 3.67 ft under the following columns: C1, C2, C3, C4, C8. By doing this, C1, C2, C3 will be the exact same as C5, C6, C7, respectively, since there are already existing 2ft x 2ft concrete columns under C5, C6, C7. C4 will be the same as C8, causing each to be resized because neither has an existing concrete column under them. Resizing of the columns is shown below:

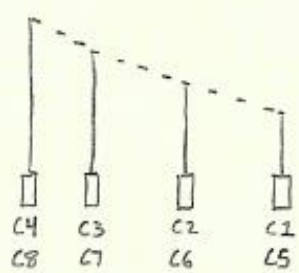
C4 and C8: $L = 30.77ft - 3.67ft = 27.10ft \rightarrow 28ft$

$\phi_c P_n = 74.5k > 48.35k$ ✓

$\therefore$ W8 x 31

Column Number	Member Size	Member Length
1	W8x31	6.33ft
2	W8x31	14.26ft
3	W8x31	22.20ft
4	W8x31	27.10ft
5	W8x31	6.33ft
6	W8x31	14.26ft
7	W8x31	22.20ft
8	W8x31	27.10ft

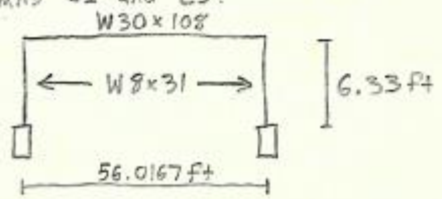




Appendix B.7: Second-Order Elastic Analysis

Risa Analysis:

Columns C1 and C5:



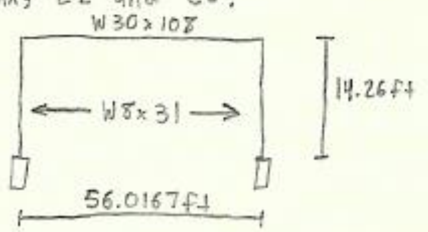
Girder Tributary Width = $\frac{15.23ft}{2}$

Girder Tributary Width = 7.615ft

DL = 2.07psf(7.615ft) = 15.8 lb/ft
SL = 28.98psf(7.615ft) = 220.7 lb/ft
WL = 25.618psf(7.615ft) = 195.1 lb/ft
] Gravity Loads ↓

Eh = 0.82psf(7.615ft)(56.0167ft) = 349.8 lb] Lateral Load →

Columns C2 and C6:

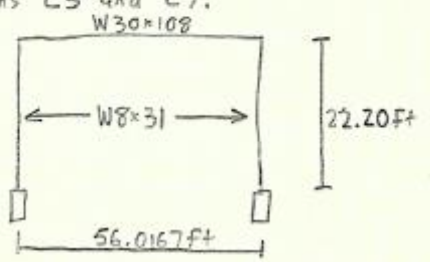


Girder Tributary Width = 15.23ft

DL = 2.07psf(15.23ft) = 31.5 lb/ft
SL = 28.98psf(15.23ft) = 441.4 lb/ft
WL = 25.618psf(15.23ft) = 390.2 lb/ft
] Gravity Loads ↓

Eh = 0.82psf(15.23ft)(56.0167ft) = 699.6 lb] Lateral Load →

Columns C3 and C7:



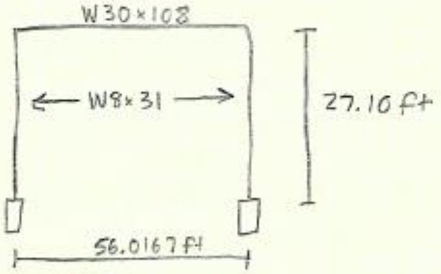
Girder Tributary Width = $\frac{15.23ft}{2} + \frac{28.21ft}{2}$

Girder Tributary Width = 21.72ft

DL = 2.07psf(21.72ft) = 44.96 lb/ft
SL = 28.98psf(21.72ft) = 629.45 lb/ft
WL = 25.618psf(21.72ft) = 556.42 lb/ft
] Gravity Loads ↓

Eh = 0.82psf(21.72ft)(56.0167ft) = 997.68 lb] Lateral Load →

Columns C4 and C8:



Girder Tributary Width = $\frac{28.21\,ft}{2}$

Girder Tributary Width = 14.105 ft

$DL = 2.07\,psf\,(14.105\,ft) = 29.2\,lb/ft$
$SL = 28.98\,psf\,(14.105\,ft) = 408.8\,lb/ft$
$WL = 25.618\,psf\,(14.105\,ft) = 361.3\,lb/ft$
] Gravity Loads ↓

$E_h = 0.82(14.105\,ft)(56.0167\,ft) = 647.9\,lb$] Lateral Load →

Second-Order Elastic Analysis

Governing Load Combination for Investigation: $1.2D + 1.6S + 0.5W$ (Gravity) ↓
$1.0E$ (Lateral) →

Column Load Effects from Risa Analysis:

1) Factored axial force P_{nt} from no-sway analysis (gravity loads)
2) Factored axial force P_{lt} from sway analysis (lateral loads)
3) Factored moment M_{nt} from no-sway analysis (gravity loads)
4) Factored moment M_{lt} from sway analysis (lateral loads)

Lateral Deflection from Risa Analysis:

1) Total story shear ΣH
2) Lateral deflection (drift) for story Δ_H

Amplifier B_2:

1) Total elastic critical buckling load for the story

$$P_{estory} = R_M \frac{\Sigma HL}{\Delta_H} \qquad R_m = 0.85 \text{ (conservative)} \qquad L = \text{story height}$$

2) P_{story} = total vertical load supported by the story
3) $$B_2 = \frac{1}{1 - \frac{\alpha P_{story}}{P_{estory}}} \geq 1 \qquad \alpha = 1.0 \text{ (LRFD)}$$

Amplifier B_1:

1) M_1 = smaller factored column end moment due to gravity load (no sway) analysis
2) M_2 = larger factored column end moment due to gravity load (no sway) analysis
3) Indicate: single or reverse curvature
4) $C_m = 0.6 \pm 0.4\,(M_1/M_2)$
   + : single
   − : reverse
5) Required second-order axial strength
   $P_r = P_{nt} + B_2 P_{lt}$
6) Elastic critical buckling load for column
   $$P_{e1} = \frac{\pi^2 EI}{(K_1 L)^2} \qquad K_1 = 1.0$$

7) $B_1 = \dfrac{C_m}{1 - \frac{\alpha P_r}{P_{e1}}} \geq 1 \qquad \alpha = 1.0 \text{ (LRFD)}$

Required Second-Order Strength Values:

1) $P_r = P_{nt} + B_2 P_{\ell t}$
2) $M_r = B_1 M_{nt} + B_2 M_{\ell t}$

Effective Length Factor K:

1) Rotational resistance at joints

$$G = \frac{\Sigma(I_c/L_c)}{\Sigma(I_g/L_g)} \qquad \begin{array}{l} G_t: \text{top joint} \\ G_b: \text{bottom joint} \end{array}$$

2) AISC Fig. C-A.7.2.

Determine K_x

Axial Capacity P_c

1) $\dfrac{K_x L}{r_x} \qquad \dfrac{K_y L}{r_y} \qquad (K_y = 1.0)$

Larger value governs

2) $\dfrac{KL}{r} < 4.71\sqrt{E/F_y}$ => short to intermediate column

$\dfrac{KL}{r} > 4.71\sqrt{E/F_y}$ => long column

3) AISC Table 4-1a

- Using column length and size => ϕP_n

$P_c = \phi P_n$

Bending Capacity M_{cx}

1) Web Local Buckling

$h/t_w \leq 3.76\sqrt{E/F_y}$

2) Flange Local Buckling

$b_f/2t_f \leq 0.38\sqrt{E/F_y}$

3) Lateral Torsional Buckling

$L_b < L_p < L_r$:

$M_n = M_p = F_y Z_x$

$\phi M_n = (0.9) M_n$

$L_p < L_b < L_r$:

$$M_n = C_b \left[M_p - (M_p - 0.7 F_y S_x)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \qquad C_b = 1.0$$

$\phi M_n = (0.9) M_n$

$L_p < L_r < L_b$:

$$M_n = S_x \left[\frac{(C_b \pi^2 E)}{(L_b/r_{ts})^2} \sqrt{1 + 0.078 \frac{J_c}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2}\right]$$

$\phi M_n = (0.9) M_n$

4) $M_{cx} = \phi M_n$

5) $\dfrac{P_r}{P_c} \geq 0.2 \;\Big|\; \dfrac{P_r}{P_c} + \dfrac{8}{9}\left(\dfrac{M_{rx}}{M_{cx}}\right) \leq 1.0 \quad$ (H1-1a)

$\dfrac{P_r}{P_c} < 0.2 \;\Big|\; \dfrac{P_r}{2P_c} + \left(\dfrac{M_{rx}}{M_{cx}}\right) \leq 1.0 \quad$ (H1-1b)

Appendix B.8: New Girder Calculations

Gateway Parking Garage	New Girder Calculation	Appendix B.8	1/2

New (Lighter) Girder Sizes From Risa Analysis:

New Max. Moment = 487.14 k·ft

$$Z_x = \frac{M_u}{\Phi F_y} = \frac{(487.14\ k\cdot ft)(12^{in}/_{ft})}{(0.9)(50\ ksi)} = 129.9\ in^3$$

AISC Table 3-2: W21x62 => $Z_x = 144 in^3 \geq 129.9 in^3$ ✓

Selected Girder Weight = $62\ ^{lb}/_{ft}$

$W_u = 1.2(62^{lb}/_{ft}) \times (^{K}/_{1000lb}) = 0.0744\ ^{K}/_{ft}$

$M_u = 487.14 k\cdot ft + \frac{W_u L^2}{8}$

$$M_u = 487.14 k\cdot ft + \frac{(0.0744\ ^{k}/_{ft})(56.0167 ft)^2}{8} = 516.32 k\cdot ft$$

$$Z_x = \frac{M_u}{\phi F_y} = \frac{(516.32k\cdot ft)(12 in/ft)}{(0.9)(50ksi)} = 137.7 in^3 \leq 144 in^3$$ => However Deflection Performance Not Satisfied

AISC Table 3-2: W30x90 => $Z_x = 283 in^3 \geq 129.9 in^3$

Selected Girder Weight = $90^{lb}/_{ft}$

$W_u = 1.2(90^{lb}/_{ft})(^{k}/_{1000lb}) = 0.108\ ^{k}/_{ft}$

$$M_u = 487.14k\ ft + \frac{(0.108^{k}/_{ft})(56.0167ft)^2}{8} = 529.5\ k\cdot ft$$

$$Z_x = \frac{M_u}{\phi F_y} = \frac{(529.5k\cdot ft)(12^{in}/_{ft})}{(0.9)(50ksi)} = 141.2 in^3 \leq 283 in^3$$ ✓

Flange Local Buckling (FLB):

$b_f/2t_f \leq 0.38\sqrt{\frac{E}{F_y}}$

$8.52 \leq 9.15$ ✓

Web Local Buckling (WLB):

$h/t_w \leq 3.76\sqrt{\frac{E}{F_y}}$

$57.5 \leq 49.6$ ✓

Total Service Load:

$\Delta_T = \frac{M_u L^2}{C_1 I_x}$ $C_1 = 158$ For acting point loads

$$\Delta_T = \frac{(487.14k\cdot ft)(56.0167ft)^2}{(158)(3610 in^4)}$$

$\Delta_T = 2.68 in < 2.80 in$ ✓

$Limit = L/240 = \frac{(56.0167ft)(12^{in}/_{ft})}{240} = 2.80 in$

Snow Deflection

$\Delta_S = \frac{5 w_S L^4}{384 E I_x}$

$$\Delta_S = \frac{5(441.37^{lb}/_{ft})(56.01ft)^4}{384(29\times10^6 psi)(3610 in^4)} \times \frac{1728 in^3}{ft^3}$$

$\Delta_S = 0.93\ in$

$w_S = 5 \times$ Tributary Width

$w_S = (28.98 psf)(15.23 ft)$

$W_S = 441.37\ ^{lb}/_{ft}$

Limit $= L/360$ or $1'' = \frac{(56.01ft)(12 in/ft)}{360} = 1.9 in$

Limit = 1 in

$0.93 in < 1 in$ ✓

$\Rightarrow$ One girder has Tributary Width = 21.72 ft

$W_s = (28.98 psf)(21.72 ft)$

$W_s = 629.45 \ lb/ft$

$$\Delta_s = \frac{5(629.45 \ lb/ft)(56.01 ft)^4}{384(29 \times 10^6 psi)(3610 in^4)} \times \frac{1728 in^3}{ft^3}$$

$\Delta_s = 1.3 \ in$

$1.3 in \not\leq 1 in$

$\therefore$ $\boxed{\text{Girder with Tributary Width = 21.72ft} \Rightarrow \text{W30} \times 108}$

Due to satisfaction of FLB, WLB, Total Service Load, Snow Deflection:

$\boxed{\text{All Other Girder Size = W30} \times 90}$

Appendix B.9: Baseplate Design

Baseplate Design

C2 and C5: W8x31 L = 6.33ft

P_u = (Previously Calculated P_u) + (Column Weight)(Column Length)

$P_u = 48.35k + 1.2(31^{lb}/_{ft})(6.33ft)(^{K}/_{1000lb})$

$P_u = 48.59k$

$M_u = 13.3\,k \cdot ft$ (Risa Analysis)

A_2 = Footing area $= (2ft \times {}^{12in}/_{ft})(2ft \times 12^{in}/_{ft}) = 576\,in^2$

$A_1 = (8in)(8in) = 64in^2 \Leftarrow$ bfd (A_1 min)

$\sqrt{\frac{A_2}{A_1}} = \sqrt{\frac{576in^2}{64in^2}} = 3 \geq 2$

$\therefore \sqrt{\frac{A_2}{A_1}} = 2$

$A_1 = \frac{P_u}{\phi_c(0.85f'_c)\sqrt{A_2/A_1}} = \frac{48.59k}{(0.65)(0.85)(4ksi)(2)}$

$A_1 = 11.0\ in^2$

$A_1\ min = 64in^2$

$\Rightarrow A_1 = 64in^2$

$\Delta = \frac{0.95d - 0.8b_f}{2} = \frac{0.95(8in) - 0.8(8in)}{2} = 0.6in$

$N = \sqrt{A_1} + \Delta = \sqrt{64in^2} + 0.6in = 8.6in$

$N = 9in$

$B = {}^{A_1}/_{N} = {}^{64in^2}/_{9in} = 7.1in$

$B = 8in \Rightarrow 9in$

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

$\phi_c P_p = (0.65)0.85(4ksi)(9in \times 9in)(2)$

$\phi_c P_p = 358.02k > 48.59k$ ✓

$m = \frac{N - 0.95d}{2} = \frac{9in - 0.95(8in)}{2} = 0.7in$

$n = \frac{B - 0.8b_f}{2} = \frac{9in - 0.8(8in)}{2} = 1.3in$

$n' = \frac{\sqrt{db_f}}{4} = \frac{\sqrt{(8in)(8in)}}{4} = 2.0in$

$\ell = 2.0in$ (largest of m, n, n')

$t_{req} = \ell\sqrt{\frac{2P_u}{0.9F_yBN}} = (2.0in)\sqrt{\frac{2(48.59k)}{0.9(36ksi)(9in)(9in)}} = 0.38in$

$\boxed{\text{A36 Baseplate} = 9in \times 9in}$

C2 and C6: W8×31 L = 14.26ft

$P_u = 48.35k + 1.2(31^{lb}/_{ft})(14.26ft)(^{K}/_{1000lb})$ $M_u = 18.6k\cdot ft$ (Risa Analysis)

$P_u = 48.88k$

=> Since P_u is similar to C1 and C5 P_u

$\underline{48.88k \approx 48.59k}$

$\boxed{\text{A36 Steel Baseplate} = 9in \times 9in}$

C3 and C7: W8×31 L = 22.20ft

$P_u = 48.35k + 1.2(31^{lb}/_{ft})(22.20ft)(^{K}/_{1000lb})$ $M_u = 18.8k\cdot ft$ (Risa Analysis)

$P_u = 49.18k$

=> Since P_u is similar to C1 and C5 P_u

$\underline{49.18k \approx 48.59k}$

$\boxed{\text{A36 Steel Baseplate} = 9in \times 9in}$

C4 and C8: W8×31 L = 27.10ft

$P_u = 48.35k + 1.2(31^{lb}/_{ft})(27.10ft)(^{K}/_{1000lb})$

$P_u = 49.36k$ $M_u = 10.3k\cdot ft$ (Risa Analysis)

$A_2 = 576in^2$ (2ft × 2ft)

$A_1 = (8in)(8in) = 64in^2$ ⇐ $b_f d$ (A_1 min)

$\sqrt{\frac{A_2}{A_1}} = \sqrt{\frac{576in^2}{64in^2}} = 3 \geq 2$

$\therefore \sqrt{\frac{A_2}{A_1}} = 2$

$A_1 = \frac{P_u}{\phi_c(0.85f'_c)\sqrt{A_2/A_1}} = \frac{49.36k}{(0.65)(0.85)(4ksi)(2)}$

$A_1 = 11.17in^2$

$A_1\ min = 64.0in^2$

$\Rightarrow A_1 = 64.0in^2$

$\Delta = \frac{0.95d - 0.8b_f}{2} = \frac{0.95(8.00in) - (0.8)(8.00in)}{2} = 0.60in$

$N = \sqrt{A_1} + \Delta = \sqrt{64.0in^2} + 0.60in = 8.60in$

$\therefore N = 9in$

$B = A_1/N = 64.0in^2/9in = 7.11in$

$B = 8in \Rightarrow 9in$

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

$\phi_c P_p = (0.65)(0.85)(4ksi)(9in \cdot 9in)(2)$

$\phi_c P_p = 358.02k > 49.36k$ ✓

$m = \frac{N - 0.95d}{2} = \frac{9in - 0.95(8.00in)}{2} = 0.70in$

$n = \frac{B - 0.8b_f}{2} = \frac{9in - 0.8(8.00in)}{2} = 1.30in$

$n' = \frac{\sqrt{db_f}}{4} = \frac{\sqrt{(8.00in)(8in)}}{4} = 2.0in$

$\ell = 2.0in$

$t_{req} = \ell\sqrt{\frac{2P_u}{0.9F_yBN}} = (2.00in)\sqrt{\frac{2(49.36k)}{0.9(36ksi)(9in)(9in)}} = 0.39in$

$\boxed{\text{A36 Steel Baseplate} = 9in \times 9in}$

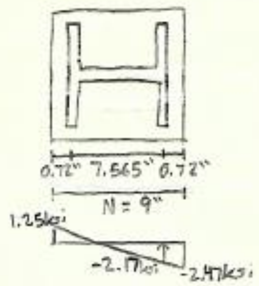
Moment-Resisting Thickness:

⇒ Largest M_u and P_u chosen from all columns

$$e = \frac{M_u}{P_u} = \frac{18.8k\cdot ft\,(12in/ft)}{49.36k} = 4.57in$$

$$f = -\frac{P_u}{A} \pm \frac{P_u e c}{I} = -\frac{49.36k}{(9in)(9in)} \pm \frac{(49.36k)(4.57in)(4.5in)}{1/12\,(9in)(9in)^3}$$

$$f = -0.61ksi \pm 1.86ksi = \begin{matrix} 1.25\,ksi \\ -2.47\,ksi \end{matrix}$$



$$M_u = (2.17ksi)(0.72in)(0.72in/2) + (2.47ksi - 2.17ksi)(0.72in)(2/3 \times 0.72in)$$

$$M_u = 0.67k\cdot in$$

$$t \geq \sqrt{\frac{6M_u}{\phi_b F_y}} = \sqrt{\frac{6(0.67k\cdot in)}{(0.9)(36ksi)}}$$

$$t \geq 0.35in$$

$$\text{Average } f_p = \frac{-2.47ksi + 1.25ksi}{2}$$

$$\text{Avg. } f_p = -0.61ksi = 0.61ksi$$

$$M_u' = f_p\, n\,(n/2) = 0.61ksi\,(1.3in)(1.3in/2)$$

$$M_u = 0.52k\cdot in < 0.67k\cdot in \checkmark$$

$\boxed{\text{All Baseplates will have a thickness } (t) = 0.5\ in}$

A36 Steel Baseplate			
Column Number	B	N	t_{req}
1	9in	9in	0.5in
2	9in	9in	0.5in
3	9in	9in	0.5in
4	9in	9in	0.5in
5	9in	9in	0.5in
6	9in	9in	0.5in
7	9in	9in	0.5in
8	9in	9in	0.5in

Appendix B.10: Recalculation of Seismic Load

Recalculation of Seismic Load

Check to see if designed structure weight ≤ 25% of current structure weight

Area of Top Floor of Garage = (Length)(Width)(Thickness)
= (268 ft)(113 ft)(4 ft)
= 121,136 ft^3

Weight of Top Floor of Garage = (Weight of Concrete)(Area)
= $(150\ lb/ft^3)(121{,}136\ ft^3)(K/1000\ lb)$
= 18,170 kips

Weight of Selected Beams = $4[(55\ lb/ft)(45.69\ ft)] + 2[(55\ lb/ft)(28.21\ ft)] + 10[(68\ lb/ft)(45.69\ ft)] + 5[(68\ lb/ft)(28.21\ ft)]$
= 53,815.5 lb

Weight of Selected Girders = $8[(90\ lb/ft)(56.0167\ ft)]$
= 40,332.02 lb

Weight of Selected Columns = $31\ lb/ft\,(2(6.33\ ft) + 2(14.26\ ft) + 2(22.20\ ft) + 2(27.10\ ft))$
= 4,333.18 lb

Combined Weight of Selected Members = $[53{,}815.5\ lb + 40{,}332.02\ lb + 4{,}333.18\ lb](K/1000\ lb)$
= 98.48 Kips

98.48 kips ≤ 0.25 (18,170 kips)
98.48 kips ≤ 4,542.5 kips ✓ (Section 15.3.1)

Therefore, horizontal design force (F_p):

$$F_p = \frac{0.4 a_p S_{DS} W_p}{(R_p/I_p)}\left(1 + 2\frac{z}{h}\right) \quad \text{(Section 13.3.1)}$$

S_{DS} = 0.192 ⇐ previously calculated (Section 11.4.1) – Spectral Acceleration

a_p = 1.0 (Table 15.1 – Other Rigid Components) – Component Amplification Factor

I_p = 1.0 (Section 13.1.3) – Component Importance Factor

$$W_p = (98.48\ Kips)\left(\frac{1000\ lb}{K}\right)\left(\frac{1}{6{,}699.60\ ft^2}\right) + 2.07\ psf$$

Component Operating Weight — Solar Panel Area — Solar Panel Weight

W_p = 16.77 psf

R_p = 2.5 (Table 13.5-1) – Component Response Modification Factor

z = 60 ft – Height of Attachment Roof

$$h = \frac{10\ ft + 30.77\ ft}{2} = 20.384\ ft + 60\ ft = 80.384\ ft$$ – Average Roof Height of Structure With Respect to the Base

h = 80.384 ft

$$F_p = \frac{0.4(1.0)(0.192)(16.77\ psf)}{(2.5/1.0)}\left(1 + 2\frac{60\ ft}{80.384\ ft}\right)$$

$$\boxed{F_p = 1.28\ psf}$$

Lower Limit: $0.3 S_{DS} I_p W_p = 0.3(0.192)(1.0)(16.77\ psf) = 0.97\ psf$

Upper Limit: $1.6 S_{DS} I_p W_p = 1.6(0.192)(1.0)(16.77\ psf) = 5.15\ psf$

0.97 psf ≤ 1.28 psf ≤ 5.15 psf ✓

Vertical seismic force (F_v):

$F_v = 0.2 S_{DS} W_p = 0.2(0.192)(16.77\,psf)$

$\boxed{F_v = 0.64\,psf}$

$\boxed{\begin{array}{l}\text{Horizontal Seismic Load } (F_p) = 1.28\,psf \\ \text{Vertical Seismic Load } (F_v) = 0.64\,psf\end{array}}$

Appendix B.11: Concrete Reinforcement

Reinforcement in 2ft × 2ft Concrete Columns

Columns C1 and C5:

$P_u = 13.15k$ (Axial Force acting on concrete column)
$M_u = 32.857\,k\cdot ft$ (Moment acting on concrete column) } Risa Analysis

$$K_n = \frac{P_u}{\phi f'_c A_g} = \frac{13.15k}{(0.9)(4ksi)(24in \times 24in)} = 0.006$$

$$R_n = \frac{M_u}{\phi f'_c A_g h} = \frac{32.857k\cdot ft}{(0.9)(4ksi)(24in \times 24in)(2ft)} = 0.008$$

Columns C2 and C6:

$P_u = 26.30k$
$M_u = 18.568k\cdot ft$

$$K_n = \frac{26.30k}{(0.9)(4ksi)(24in \times 24in)} = 0.013$$

$$R_n = \frac{18.568k\cdot ft}{(0.9)(4ksi)(24in \times 24in)(2ft)} = 0.004$$

Columns C3 and C7:

$P_u = 37.51k$
$M_u = 18.794\,k\cdot ft$

$$K_n = \frac{37.51k}{(0.9)(4ksi)(24in \times 24in)} = 0.018$$

$$R_n = \frac{18.794k\cdot ft}{(0.9)(4ksi)(24in \times 24in)(2ft)} = 0.005$$

Columns C4 and C8:

$P_u = 24.36\,k$
$M_u = 10.299\,k\cdot ft$

$$K_n = \frac{24.36k}{(0.9)(4ksi)(24in \times 24in)} = 0.012$$

$$R_n = \frac{10.299k\cdot ft}{(0.9)(4ksi)(24in \times 24in)(2ft)} = 0.002$$

For All Columns:

$\rho_g < 0.01 \Rightarrow \rho_g = 0.01$ (min. value)
$A_s = \rho_g A_g$
$A_s = (0.01)(24in \times 24in)$
$A_s = 5.76in^2$

$N(\pi/4\, d_b^2) = A_s$
$6(\pi/4\, d_b^2) = 5.76in^2$
$d_b = 1.1in \Rightarrow 6$ #9's $(A_s = 5.96in^2)$

Appendix B.11 $\quad$ 2/2

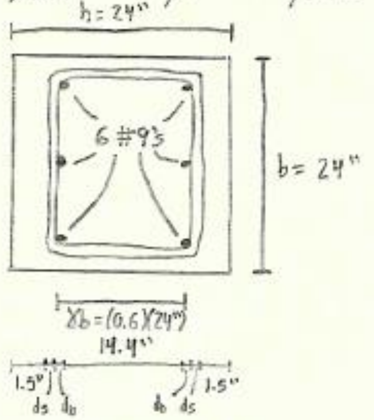
$\rho_{min} = \frac{3\sqrt{f'c}}{f_y} = \frac{3\sqrt{4000psi}}{60,000psi}$ $\qquad$ $\frac{200}{f_y} = \frac{200}{60,000psi}$

$\rho_{min} = 0.0032$ $\qquad$ $\rho_{min} = 0.0033$ ⇐ Larger governs

$\rho_{max} = 0.85\beta_1 \frac{f'c}{f_y} \frac{\varepsilon_u}{\varepsilon_u + 0.004} = 0.85(0.85)\left(\frac{4ksi}{60ksi}\right)\frac{0.003}{0.003+0.004}$

$\rho_{max} = 0.021$

$\underset{\rho_{min}}{0.0033} < \underset{\rho}{0.01} < \underset{\rho_{max}}{0.021}$ ✓



$2\,Cover + 2d_s + \gamma b + d_b = 24\,in$

$2(1.5") + 2d_s + 14.4" + 9/8" = 24"$

$d_s = 2.7\,in \Rightarrow 2\,in$

$Cover = 2.2\,in$

Reinforcement in 2ft x 2ft Concrete Column Under Each Steel Column: 6 #9's w/ 2.0 in ties

APPENDIX C: GREEN ROOF CALCULATIONS

Appendix C.1: Live, Dead, Snow and Rain Load Calculations

Live Loads (per floor) ASCE 7-10 Chapter 4

Roof = 100 psf roof gardens or 12 psf (FPM Global, Green roof system)

1st, 2nd, and 3rd = stack rooms = 150 psf

reading rooms = 60 psf

corridors = 80 psf (First floor = 100 psf)

stairs and exit ways = 100 psf

meeting rooms = 50 psf

Live loads will vary per each floor on library regarding the usage of each floor.

A live load of 50 psf its assumed for meeting rooms in library or as we call them "tech suites".

Dead Loads

• For 6" soil depth: (Mass.gov)

D = 35 psf → captured and retained water part of load

• Green roof dead load will be added to additional dead loads obtained from Excel spreadsheet calculations for effective weight of building

• Other dead loads will be neglected

• Floor dead load (psf): (Roof) = 91 psf

• Floor dead load (psf): (for 1st, 2nd and 3rd floor)

- Dome size = 19in
- Dome Depth (in) = 10in
- slab thickness = 4in

} 103 psf.

↳ Table 11-1 CRSI Design Handbook

Snow loads : ASCE 7-10 Chapter 7

$p_f = 0.7\, C_e C_t I_s p_g$ $\quad$ p_f = snow load on flat roof

- Exposure Factor : (C_e) → Table 7-2

  Surface Roughness B: urban and suburban areas (Figure 26.7)

  Fully Exposed : roofs exposed w/ no shelter afforded by terrain, higher structures or terrains

  $\therefore \boxed{C_e = 0.9}$

- Thermal Factor : (C_t) → Table 7-3

  $\therefore \boxed{C_t = 1.0}$

- Importance Factor (I_s) : Table 1.5-1

  Risk Category III

  $\therefore \boxed{I_s = 1.10}$

- Ground snow loads : p_g (Figure 7-1)

  $\therefore \boxed{p_g = 50 \text{ psf}}$

$$p_f = 0.7(0.9)(1.0)(1.10)(50\text{ psf})$$

$$\boxed{p_f = 34.65 \text{ psf} = S}$$

Rain Loads : (FM Global Data Sheets)

$\boxed{R = 32 \text{ psf}}$ → (DS 1-54 Roof Loads for New Construction Data Sheet 2.5.2.8)

Appendix C.2: Seismic Load Calculations

SC I

seismic load library w/o green roof

step 1: Risk - Targeted Max Considered Earthquake

S_s : Fig 22-1 -> 18%

S_1 : Fig 22-2 -> 7%

} MSBC 9th Edition

step 2: The structure being analyzed IS exept from seismic requirements

Step 3: Seismic Design Category (SDC)

1. Soil Classification

Table 20.3-1 -> Site D (stiff soil)

↳ soil properties are not known

2. $S_{DS} = (2/3)\underbrace{(F_a)(S_s)}_{S_{MS}}$

Table 11.4-1 -> F_a = 1.6

$S_{DS} = (2/3)(1.6)(0.18) \Rightarrow \boxed{S_{DS} = 0.192}$

$S_{D1} = (2/3)\underbrace{(F_v)(S_1)}_{S_{M1}}$

Table 11.4-2 -> F_v = 2.4

$S_{D1} = (2/3)(0.07)(2.4) \Rightarrow \boxed{S_{D1} = 0.112}$

3. Risk Category III -> structures w/ large number of persons

From Table 11.6-1 -> $\boxed{SDC = B}$

and Table 11.6-2

Sc II

Step 4: Determine Analysis Procedure

Determining Period T of a structure ($T = T_a$)

$T_a = C_t h_n^x$

h_n = structural height (vertical distance from base to highest level of the seism.

↳ concrete moment resisting frame force resisting system)

h_n = from ground floor = $(\underbrace{14'-7\frac{7}{8}'' \times 4}_{14.656'}) + (\underbrace{10\frac{1}{2}''}_{0.875'}) \Rightarrow \boxed{h_n = 59.5 \text{ ft}}$

C_t: 0.016

x = 0.9

$T_a = (0.016)(59.5')^{0.9} \Rightarrow \boxed{T_a = 0.6327}$

Determining T_s

↳ $\boxed{T_s = 0.5}$

Step 5: Determine R, Response Modification coefficient

Table 12.2-1

C 7. ordinary reinforced concrete moment frame

$\boxed{R = 3}$

Step 6: Seismic Importance Factor (I_e)

Risk Category III

↳ $\boxed{I_e = 1.25}$

SC III

here it changes for Green Roof Design
↳ account for 100% of weight of landscaping and other materials

* Step 7: Seismic Base Shear (V) -> w/o Green Roof weight.

$V = C_s W$; W = weight of structure

C_s = seismic response coefficient

include in seismic weight = where flat roof snow loads are greater than 30 psf
20% of the design snow load, regardless roof slope

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} \Rightarrow C_s = \frac{0.192}{\left(\frac{3}{1.25}\right)} \Rightarrow \boxed{C_s = 0.08}$$

C_s shall not be less than: $C_s = 0.044 S_{DS} I_e \geq 0.01$

C_s shall not exceed:

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} \quad \text{for } T \leq T_L$$

$$C_s = \frac{S_{D1} T_L}{T^2\left(\frac{R}{I_e}\right)} \quad \text{for } T > T_L$$

$\boxed{T_L = 6}$ -> Figure 22-12 Transition Period

W = 20% of snow load + effective weight of library (excel spreadsheet)

W = 6.93 psf + 13,629.71 kips

W = 6.93 (178 ft × 92.87 ft) + 13,629.71 kips

W = 114.559 kips + 13,629.71 kips => $\boxed{W = 13,744.269 \text{ kips}}$

$\boxed{V = 1,099.54 \text{ kips}}$

SCIV

Weight of structure: (from commentary C.12.7.2)

- calculations done in excel spreadsheet
- Effective seismic weight only portion of mass tied to the structure. Hence, live loads such as loose furniture, loose equip, and human occupants are not included.
- Certain types of live loads such as storage loads are included
- Other contributors to effective weight are:
  - All permanent equip. (air conditioners, elevator equip, mechanical systems)
  - 20% of snow load ($P_f > 30$ psf)
  - weight of landscaping and similar materials
  - Partitions to be erected or rearranged as specified in Section 4.3.2

* Step 8: Distribute V, over height of structure

$$F_x = C_{vx} V$$

Per each story ←

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

w_i and w_x = portion of total effective seismic weight assigned to level i or x

h_i and h_x = height from base to level i or x

Assume $k = 2$ as $0.5 \text{ sec} < T_a < 2.5 \text{ sec}$

See excel spreadsheet for all values

$$C_{vx} = \frac{(W_{3-roof}\, h_{3-roof})^2}{(W_{G-1st} \times h_{G-1st})^2 + (W_{1st-2nd} \times h_{1st-2nd}) + (W_{2nd-3rd} \times h_{2nd-3rd}) + (W_{3rd-roof}\, h_{3rd-roof})}$$

$$C_{vx} = \frac{(165{,}242.15)^2}{(82{,}096.26 \text{ k ft})^2 + (131{,}809 \text{ k ft})^2 + (178{,}411.55 \text{ k ft})^2 + (165{,}242.15)^2}$$

SCV

Step 9: Redundancy factor (ρ)

$\boxed{\rho = 1.0} \rightarrow$ SDC A, B, or C

Step 10: Seismic Load Effects E and E_m

$S_{DS} = 0.192 \rightarrow$ step 3

$E = \overbrace{\rho Q_E}^{E_{horizontal}} \pm \overbrace{0.2 S_{DS} D}^{E_{vertical}}$ D= dead load $Q_E = F_x$

$E = (1.0)(\quad) + 0.2(0.192)($

$E =$

Repeat from Step 7: assuming Green Roof system

$V = C_s W \Rightarrow \boxed{C_s = 0.08}$

$W = 13,744.27$ kips $+ 375.66$ k $\Rightarrow \boxed{W = 14,119.93 \text{ kips}}$

$V = 1,129.59$ kips $+ 114.559$ kip

$\boxed{V = 1,244.15 \text{ kips}}$

Green Roof weight doesn't account for 25% or more of whole building weight.

Step 8: Distribute V, over height of structure

Appendix C.3: Wind Load Calculations

W1

Wind load Analysis (see also Ch. 29.5 ASCE 7-10)

Step 2: Wind Speed

↳ IBC Figure 1609B or ASCE 7-10 Figure 26.5-1B

$\boxed{V = 134 \text{ mph}}$ (check MSBC 9th Edition)

↳ State code governs this value

Step 3: Mean Roof Height

- same as height above ground

Step 4: Exposure Category → may change w/ green roof

Exposure B

↳ check commentary

Step 5: Enclosure classification

↳ enclosed building

Step 6: Wind Directionality (k_d), Topographic factor (k_{zt}), Gust Effect (G)

$K_d = 0.85$ → always for buildings

$K_{zt} = 1.0$ → no topographic effects on building

$G = 0.85$ → Low rise building (rigid)

Step 7: Wind Design MWFRS

- Method 1 → Chapter 27 Par 1

↳ to analyze MWFRS

Step 8: Determine Wind Design Method for C & C

- Method 3 → Chapter 30 Par 1

WII Wind loads shall not be less than 16 psf multiplied by the wall area of the building and 8 psf multiplied by the roof area of the building.

MWFRS Wind loads

- Velocity Pressure Coefficient

$$q_z = 0.00256\ K_z\ K_{zt}\ K_d\ V^2 \quad (Eq\ 27.3\text{-}1)$$

For Expose B Building

↳ height above ground = $\boxed{K_z = 0.85}$

$$q_z = 0.00256\,(0.85)(1.0)(0.85)(134\text{ mph})^2$$

$\boxed{q_z = 33.21\text{ psf}}$ $\boxed{q_h = 33.21\text{ psf}}$ → same as q_z b/c mean height is the same

- External pressure coefficient (C_p)
  - flat roof ($L = 167$ ft, $B = 103$ ft, $h = 59.5$ ft)
  - Wall pressure coefficient (C_p) Figure 27.4-1
    - Windward $\boxed{C_p = 0.8}$ → for q_z
    - Leeward wall $\boxed{C_p = -0.38}$ → for q_h
    - Side wall $\boxed{C_p = -0.7}$ → for q_h
  - Roof pressure coefficients (C_p)

$$\boxed{p = qGC_p - q_i\,\underbrace{(GC_{pi})}_{\text{internal pressure coefficient}}\ [\text{psf}]}$$ → rigid buildings.

- From Table 26.9-1

↳ $GC_{pi} = +0.18,\ -0.18$ → enclosed building.

- Gust Effect Factor (G)

↳ for rigid building gust-effect factor is $\boxed{0.85}$

VIII

Wind calculations done at top of building assuming $q_h = q_z$

For windward walls:

$p = (33.21\ psf)(0.85)(0.8) - (33.21\ psf)(0.18)$

$\boxed{p = 16.61\ psf}$ $\boxed{p = 28.56\ psf}$ → assuming GCpi to be negative

For Leeward walls:

$p = (33.21\ psf)(0.85)(-0.38) - (33.21\ psf)(0.18)$

$\boxed{p = -16.70\ psf}$ $\boxed{p = -4.75\ psf}$

For Side walls:

$p = (33.21\ psf)(0.85)(-0.7) - (33.21\ psf)(0.18)$

$\boxed{p = -25.73\ psf}$ $\boxed{p = -13.78\ psf}$

C & C Wind Load (shall not be less than 16 psf)

- Velocity Pressure Coefficient (k_z)
  - → from Table 30.3-1 $\boxed{k_z = 0.85}$
- Wind pressure

$p = q_h[(GC_p) - (GC_{pi})]$

$GC_{pi} = 0.18, -0.18$ → Table 26.11-1

- External Pressure coefficient (GC_p)

Walls → Figure 30.4-1 a = 10% of least horizontal dimension = 9.87 ft

4 and 5 → $\boxed{GC_p = +0.8}$

4 and 5 effective wind area ≈ 100 ft² ⇒ $\boxed{GC_p = 0.7}$

Roof → Figure 30.4-2

a = 10% of least horizontal dimension = 9.87 ft

3 effective wind area ≈ 40 ft² → $\boxed{GC_p = -1.1}$

2 effective wind area ≈ 150 ft² → $\boxed{GC_p = -1.1}$

1 effective wind area ≈ 1000 ft² → $\boxed{GC_p = -0.9}$

WIV

①, ② & ③ effective wind area $\simeq 1000 ft^2 \Rightarrow \boxed{GC_p = 0.2}$

Design Wind Pressures

$$p = q_h [(GC_p) - (GC_{pi})]$$

For Flat Roof

Zone ①, ② and ③ $\begin{cases} p = 31.26\ psf\,[(0.2) - (0.18)] \Rightarrow \boxed{p = 0.625\ psf} \\ p = 31.26\ psf\,[(0.2) - (-0.18)] \Rightarrow \boxed{p = 11.88\ psf} \end{cases}$

For individual zones (Figure 30.4-2A)

Zone ③ $p = 31.26\ psf\,[(-1.6) - (0.18)] = \boxed{-40.01\ psf}$

Zone ② $p = 31.26\ psf\,[(-1.1) - (0.18)] = \boxed{-40.01\ psf}$

Zone ① $p = 31.26\ psf\,[(-0.9 - (0.18)] = \boxed{-33.76\ psf}$

GCp for zone ② and ③ are the same as parapet is higher than 3 ft.

Zone ③ and ② $p = 31.26\ psf\,[(-1.1) - (-0.18)] \Rightarrow \boxed{p = -28.75\ psf}$

Zone ① $p = 31.26\ psf\,[(-0.9) - (-0.18)] \Rightarrow \boxed{p = -22.50\ psf}$

For walls

Zone ④ and ⑤ $p = 31.26\ psf\,[(-0.8) - (0.18)] \Rightarrow \boxed{p = -30.63\ psf}$

Zone ④ and ⑤ $\Rightarrow p = 31.26\ psf\,[(0.7) - (0.18)] \Rightarrow \boxed{p = 16.26\ psf}$

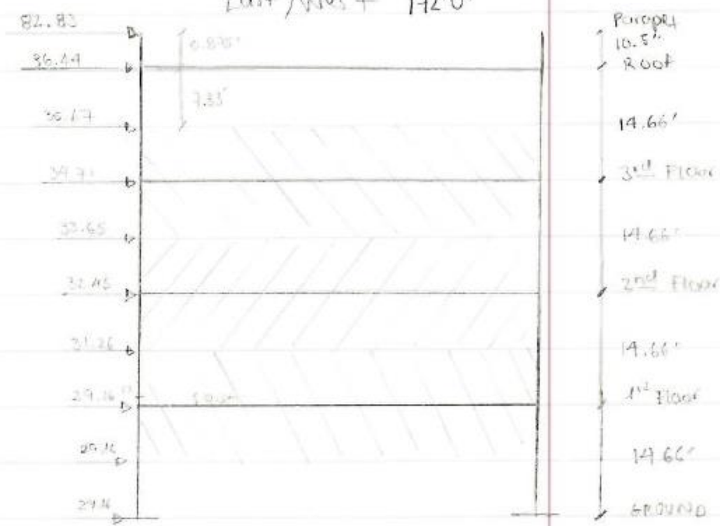
$p = 31.26\ psf\,[(0.7) - (-0.18)] \Rightarrow \boxed{p = 27.51\ psf}$

$p = 31.26\ psf\,[(-0.8) - (-0.18)] \Rightarrow \boxed{p = -19.38\ psf}$

WSII

Wind Load Distribution

East / West 172'0"



CALCULATIONS:

Roof : 82.83 psf (172 ft × 0.875 ft) + $\frac{36.64 + 36.67}{2}$ psf (172 ft × 7.33 ft)

Roof level = 12.47 kips + 45.46 kips => $\boxed{\text{Roof level} = 57.93\ K}$

3rd Floor : 34.71 psf (172 ft × 14.66') => $\boxed{\text{3rd Floor} = 87.52\ \text{kips}}$

2nd Floor : 32.45 psf (172 ft × 14.66') => $\boxed{\text{2nd Floor} = 81.82\ \text{kips}}$

1st Floor : 29.16 psf (172 ft × 7.67 ft) + $\frac{29.16 + 31.26}{2}$ (172 ft × 6.99 ft)

1st Floor = 38.47 kips + 36.32 psf => $\boxed{\text{1st Floor} = 74.79\ \text{kips}}$

Ground Floor : 29.16 psf (172 ft × 7.33 ft) => $\boxed{\text{Ground Floor} = 36.76\ \text{kips}}$

WIII

Base Shear (V):

Σ Story Forces => $V = 57.93k + 87.52k + 81.82k + 74.79k + 36.76k$

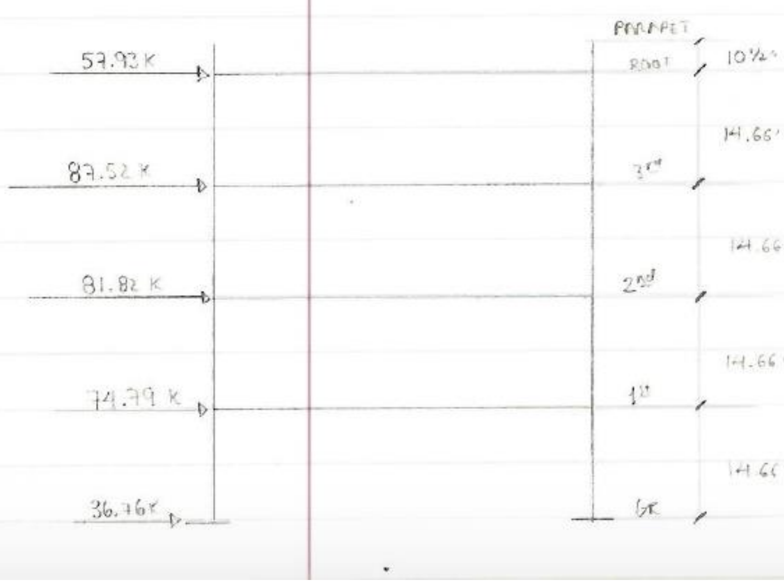
$\boxed{V = 338.82 \text{ Kips}}$

Overturning Moment (Mo):

$Mo = (57.93k \times 58.64ft) + (87.52k \times 43.97ft) + (81.82k \times 29.31ft) + (74.79k \times 14.66ft)$

$\boxed{Mo = 10,739.26 \text{ k.ft}}$

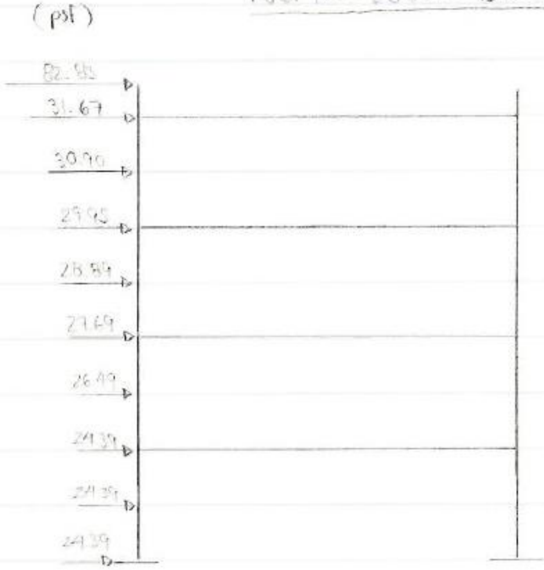
- Overturning will not be an issue with green roof sytems as the extra load that the system imposes on the building is going to be used to resist the overturning moment.



WVIII

Wind Load Distribution

North - South Direction



CALCULATIONS:

Roof : 82.82 psf (92.87' x 0.875) + $\frac{31.67 \text{ psf} + 30.90}{2}$ (92.87ft x 7.33 ft)

Roof level = 6.73 kips + 21.30 kips => $\boxed{\text{Roof} = 28.03 \text{ kips}}$

3rd Floor : 29.95 psf (92.87' x 14.66') => $\boxed{\text{3rd Floor} = 40.78 \text{ kips}}$

2nd Floor : 27.69 psf (92.87' x 14.66') => $\boxed{\text{2nd Floor} = 37.70 \text{ kips}}$

1st Floor : 24.39 psf (92.87' x 7.67') + $\frac{24.39 + 26.49}{2}$ (92.87' x 6.99')

1st Floor = 17.37 kips + 16.51 kips => $\boxed{\text{1st Floor} = 33.88 \text{ kips}}$

Ground Floor 24.39 psf (92.87' x 7.33') => $\boxed{\text{Ground Floor} = 16.60 \text{ kips}}$

WIX

Base Shear (V):

$\sum$ story forces => $V = 28.03K + 40.78K + 37.70K + 33.88K + 16.60K$

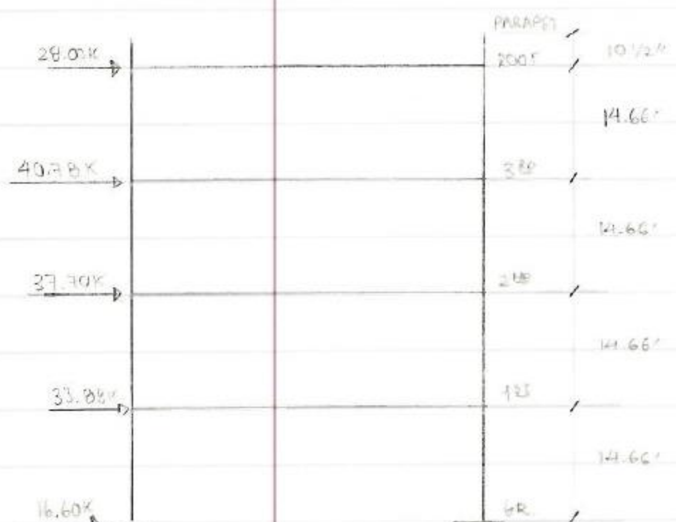
$\boxed{V = 156.99 \text{ Kips}}$

Overturning Moment (Mo):

$Mo = (28.03K \times 58.63 ft) + (40.78K \times 43.97 ft) + (37.70K \times 29.31 ft) + (33.80K \times 14.66 ft)$

$\boxed{Mo = 5{,}036.99 \text{ k.ft}}$

• Overturning moment in North-south direction is almost half of East-West Direction

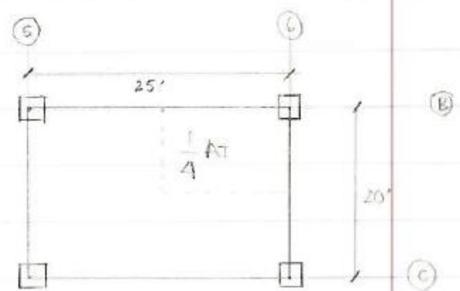


Appendix C.4: Factored Design Load (Pu) Calculations

Pu I

Column Load Calculations

Load combinations are the same as for slabs.



Typical interior section for column

AT= Tributary Area of column

Roof Floor:

Column, B5

AT= 25 ft × 20 ft => $\boxed{AT= 500\ ft^2}$ $\boxed{AT\ Drop\ Panel= 8.5' \times 8.5' = 72.25 ft^2}$

Dead Load:

* MEP= 5 psf
* Green Roof = 35 psf → may not apply for this section.
* Two-way dome slab = 91 psf
* Penthouse = 58.37 psf
* Solid head = 71 psf
* Columns = 4.20 psf

Dead load = (5 psf + 35 psf + 91 psf + 58.37 psf + 4.20 psf) × (500 ft²)
+ (71 psf × 72.25 ft²)

$\boxed{\text{Dead load} = 101.91\ \text{Kips}}$

Live Load: live loads are based on occupancy areas based on the location of each column analyzed.

Pu II

Live load = 20 psf for entire roof

Live load = (20 psf)(500 ft^2)/1000 => $\boxed{\text{Live load} = 10 \text{ kips}}$

Snow load:

Snow load = (34.65 psf x 500 ft^2) => $\boxed{\text{Snow load} = 17.33 \text{ kips}}$

Load combinations Colum B5

(1) $Pu_1 = 1.4D$ => $Pu = 1.4(101.91 \text{ kips})$ => $\boxed{Pu = 142.67 \text{ kips}}$

(2) $Pu_2 = 1.2D + 1.6L + 0.5S$ => $Pu_2 = 1.2(101.91\ K) + 1.6(10) + 0.5(17.33K)$

$\boxed{Pu_2 = 146.96\ K}$

(3) $Pu_3 = 1.2D + 1.6S + L$ => $Pu_3 = 1.2(101.91K) + 1.6(17.33K) + 10K$

$\boxed{Pu_3 = 160.02 \text{ kips}}$

In this case column B5, B6, C5 and C6 will have the same as all their tributary areas, and loads are exactly the same, only for the Roof Floor.

Pu.III

3rd Floor

Column. B5 → $\boxed{A_T = 500\ ft^2}$ $\boxed{A_T\ Drop\ Panel = 72.25\ ft^2}$

Dead load:

* Roof combined = 193.57 psf
* MEP = 5 psf
* Roof solid head = 71 psf
* Two way Dome slab = 103 psf
* Solid head 3rd floor = 59 psf
* Walls / partition / columns = 105.52 psf

Dead load = (193.57 psf + 5 psf + 103 psf + 105.52 psf)(500 ft²)
+ (71 psf + 59 psf)(72.75 ft²)

$\boxed{\text{Dead load} = 213.00\ \text{Kips}}$

Live load:

* Live loads acting on tributary area of column are:
    - corridor
    - mechanical room
    - meeting room

$$Live\ load = \underbrace{(80\ psf)}_{corridor} \times (20' \times 10') + (100\ psf) \times (20' \times 12.5') + (40\ psf)(20' \times 2.5')$$

Live load = 16,000 lbs + 25000 lbs + 2000 lbs ⇒ $\boxed{LL = 43\ \text{Kips}}$

Load Combination Column B5

(1) $P_{u_1} = 1.4D \Rightarrow P_{u_1} = 1.4(213\ k) \Rightarrow \boxed{P_{u_1} = 298.2\ \text{Kips}}$

(2) $P_{u_2} = 1.2D + 1.6L + 0.5S \Rightarrow P_{u_2} = 1.2(213K) + 1.6(43K) + 0.5(17.33k)$

$\boxed{P_{u_2} = 333.07\ \text{Kips}}$

(3) $P_{u_3} = 1.2D + 1.6S + L \Rightarrow P_{u_3} = 1.2(213K) + 1.6(17.33K) + 43K$

$\boxed{P_{u_3} = 326.33\ \text{Kips}}$

Pu IV

Column C5

- Dead load is the same
- Live load consists of:
  - Corridor
  - mechanical room
  - meeting room

Live Load = $(80\ psf)(10' \times 22.5') + (80\ psf)(10' \times 10') + (40\ psf)(2.5' \times 20) + (100\ psf)(12.5' \times 10')$

$\boxed{\text{Live load} = 40.5\ \text{kips}}$

Load combination Column C5

① $Pu_1 = 1.4D \Rightarrow \boxed{Pu_1 = 298.2\ kips}$

② $\boxed{Pu_2 = 329.07\ kips}$

③ $\boxed{Pu_3 = 323.83\ kips}$

Column B6 and Column C6

- Dead load is the same
- Live load consists of:
  - meeting room

Live load = $(40\ psf)(500\ ft^2) \Rightarrow \boxed{LL = 20\ kips}$

Load combinations Column B6 and C6

① $Pu_1 = 1.4D \Rightarrow \boxed{Pu_1 = 298.2\ kips}$

② $\boxed{Pu_2 = 296.27\ kips}$

③ $\boxed{Pu_3 = 303.33\ kips}$

PuV

2nd Floor

Column B5 → $\boxed{A_T = 500 ft^2}$ $\boxed{A_T \text{ Drop panel} = 72.25 ft^2}$

Dead load:

* Roof and 3rd floor combined = 407.09 psf
* MEP = 5 psf
* Roof and 3rd floor solid head = 130 psf
* Two-way dome slab = 105 psf
* Solid head = 59 psf
* walls/columns = 113.68 psf

Dead load = (407.09 psf + 5 psf + 105 psf + 113.68 psf)(500 ft²) + (130 psf + 59 psf)(72.25 ft²)

$\boxed{\text{Dead load} = 328.04 \text{ kips}}$

Live load:

* Live load acting on A_T of column B5
  * corridor
  * mechanical room
  * meeting room

Live load = (80 psf) × (20' × 10') + (125 psf)(12.5' × 10') + (10' × 12.5')(40 psf) + (40 psf)(20' × 2.5')

Live load = 16,000 K + 12500 K + 5000 K + 2240 K

$\boxed{\text{Live load} = 35.74 \text{ K}}$

Load combinations Column B5

① $P_{u1} = 1.4D \Rightarrow P_{u1} = 1.4(328.04 \text{ K}) \Rightarrow \boxed{P_{u1} = 459.26 \text{ kips}}$

② $P_{u2} = 1.2D + 1.6L + 0.5S \Rightarrow P_{u2} = 1.2(328.04 \text{ K}) + 1.6(35.74 \text{ K}) + 0.5(17.33 \text{ K})$

$\boxed{P_{u2} = 459.50 \text{ kips}}$

③ $P_{u3} = 1.2D + 1.6S + L \Rightarrow P_{u3} = 1.2(328.04 \text{ K}) + 1.6(17.33 \text{ K}) + 35.74 \text{ K}$

$\boxed{P_{u3} = 457.11 \text{ kips}}$

Pu II

Column C5

Dead load is the same as for Column B5

- Live loads consists of:
  - Corridor
  - meeting room.

Live load = (80 psf)(20'×10') + (80 psf)(10'×12.5') + (40 psf)(10'×12.5') + (40 psf)(20'×7.5')

$\boxed{\text{Live load} = 33 \text{ kips}}$

Load combinations Column C5

① $P_{u1} = 1.4D \Rightarrow P_{u1} = 1.4(328.04\text{ k}) \Rightarrow \boxed{P_{u1} = 459.26\text{ k}}$

② $P_{u2} = 1.2D + 1.6L + 0.5S \Rightarrow \boxed{P_{u2} = 453.11\text{ kips}}$

③ $P_{u3} = 1.2D + 1.6S + L \Rightarrow \boxed{P_{u3} = 454.38\text{ kips}}$

Column B6

Live load = (40 psf)(500 ft²) = $\boxed{20\text{ kips}}$

Load combination Column B6

① $P_{u1} = 1.4D \Rightarrow \boxed{P_{u1} = 459.26\text{ kips}}$

② $P_{u2} = 1.2D + 1.6L + 0.5S \Rightarrow \boxed{P_{u2} = 434.31\text{ k}}$

③ $P_{u3} = 1.2D + 1.6S + L \Rightarrow \boxed{P_{u3} = 441.38\text{ k}}$

Column C6

Live load = (40 psf)(20'×17.5') + (40 psf)(7.5'×15') + (50 psf)(5'×7.5')

$\boxed{\text{Live load} = 20.38\text{ K}}$

Load combination Column C6

① $P_{u1} = 1.4D \Rightarrow \boxed{P_{u1} = 459.26\text{K}}$

② $P_{u2} = 1.2D + 1.6L + 0.5S \Rightarrow \boxed{P_{u2} = 434.92\text{ K}}$

③ $P_{u3} = 1.2D + 1.6S + L \Rightarrow \boxed{P_{u3} = 441.76\text{K}}$

Pu VIII

1^{st} Floor

Column B5, C5 $A_T = 500 ft^2$ A_T PANEL = 72.25 ft2

Dead load :

* Roof, 3rd and 2nd floor combined = 628.77 psf   * MEP = 5 psf
* Roof, 3rd and 2nd floor solid head = 189 psf   * Dome slab = 103 psf
* solid head = 59 psf   * walls/columns = 121.94 psf

Dead load = (628.77 psf + 5 psf + 102 psf + 121.94 psf)(500 ft2) + (189 psf + 59 psf)(72.25 ft2)

$\boxed{\text{Dead load} = 447.27 \text{ kips}}$

Live load :

· Live load acting on column B5 and C5 consist of:
· library stacks

Live load = (500 ft2)(150 psf) => $\boxed{LL = 75 \text{ kips}}$

Load combination Column B5 and C5

(1) $Pu_1 = 1.4D$ => $Pu_1 = 1.4(447.27 k)$ => $\boxed{Pu_1 = 626.18 \text{ kips}}$

(2) $Pu_2 = 1.2D + 1.6L + 0.5S$ => $\boxed{Pu_2 = 665.39 \text{ kips}}$

(3) $Pu_3 = 1.2D + 1.6S + L$ => $Pu_3 = 1.2(447.27k) + (1.6)(14.35) + 75k$

$\boxed{Pu_3 = 639.45 \text{ kips}}$

Pu VIII

Column B6 and C6

Live load consist of :

- library stacks
- meeting room

Live load = (150 psf)(20' x 12.5') + (40 psf)(20' x 12.5')

$\boxed{LL = 47.5 \text{ k}}$

Load combinations for column C6 and B6

(1) $Pu_1 = 1.4D \Rightarrow Pu_1 = 1.4(447.27 \text{ k}) \Rightarrow \boxed{Pu_1 = 626.18 \text{ k}}$

(2) $Pu_2 = 1.2D + 1.6L + 0.5S \Rightarrow Pu_2 = 1.2(447.27\text{k}) + 1.6(47.5\text{k}) + 0.5(17.33\text{k})$

$\boxed{Pu_2 = 621.39 \text{ kips}}$

(3) $Pu_3 = 1.2D + 1.6S + L \Rightarrow Pu_3 = 1.2(447.27\text{k}) + 1.6(17.33\text{k}) + (47.5\text{k})$

$\boxed{Pu_3 = 611.95 \text{ kips}}$

AxI

Column Dimensions – Axial Load

- An analysis of the strength capacity of interior and edge columns was developed. All calculations were based on column dimensions per each floor and reinforced steel bars.
- All columns are assumed to be circular concrete with spiral reinforcement.

First Floor Internal columns (23" x 23") → C5 in S-14 sheet

Nominal Axial load Capacity:

$$P_{n\,max} = \left[A_s f_y + 0.85 f'_c \left(A_g - A_s\right)\right] 0.85$$

A_g = gross section area (in^2)

A_s = Area of steel reinforcement bars (in^2)

f_y = Strength of steel

f'_c = strength of concrete

P = axial force of column

ϕ = 0.75 for spiral columns

$$\phi P_{n,max} = 0.85\phi\left[0.85 f'_c\left(A_g - A_{st}\right) + A_{st} f_y\right]$$

→ Reference: ACI 318-14

From structural drawings:

8 No. 11 bars on a 23" X 23" concrete column

d_b = diameter of bar = 1.41 in A_{st} = 1.56 in^2

f'_c = assuming 5000 psi for columns

f_y = 50,000 psi → assumption

A_g = 23" x 23" ⇒ $\boxed{A_g = 529 \text{ in}^2}$

A_{st} = 8 x 1.56 in^2 ⇒ $\boxed{A_{st} = 12.48 \text{ in}^2}$

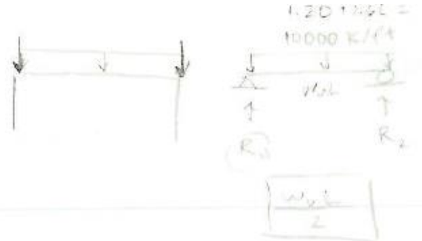
AX II

$$\phi P_{n,max} = 0.85(0.75)\left[0.85(5000\,psi)(529\,in^2 - 12.48\,in^2) + 50{,}000\,psi(12.48\,in^2)\right]$$

$$\boxed{\phi P_{n,max} = 1{,}797.25\ kips}$$

Appendix C.5: Factored Design Load (Wu) Calculations

Wu I



Load Combinations

1. 1.4D
2. 1.2D + 1.6L + 0.5S
3. 1.2D + 1.6S + L
4. 1.2D + 1.0W + L + 0.5S
5. 1.2D + 1.0E + L + 0.2S
6. 0.9D + 1.0W
7. 0.9D + 1.0E

Gravity Load Combinations (1–3)

3a: 1.2D + 1.6S + 0.5W

Load Combinations Calculations For Slabs

Roof: (Gravity Load Combinations only)

Dead load: Green roof = 35 psf

Two-way dome slab = 10" + 3" slab = 91 psf → Table 11.1 CRSI

Penthouse and walls / area of roof = 932370.21 lbs / (172' × 92.87')

Penthouse = 58.37 psf

MEP = 5 psf (assumption)

solid head = (13 in / 12 in/ft) × 150 pcf = 162 psf

solid head = (162 psf − 91 psf) × 8.5 ft = 603.5 lb/ft

columns = 4.20 psf

$l_1 = 25'$ $l_2 = 20'$

(1) $W_u = 1.4D \Rightarrow W_u = 1.4\overbrace{(35\text{ psf} + 5\text{ psf} + 62.57\text{ psf} + 91\text{ psf})(25')}^{4839.25\text{ lb/ft}} + 1.4(603.5\text{ lb/ft})$

$\boxed{W_{u_1} = 7.62\text{ K/ft}}$

(2.) $W_u = 1.2D + 1.6L + 0.5S$

snow load = 34.65 psf

Live load = 20 psf

$W_{u_2} = 1.2\,(4839.25\ lb/ft + 603.5\ lb/ft)(25') + 1.6(20\ psf \times 25') + 0.5(34.65\ psf)$

$W_u = 6{,}531.3\ lbs/ft + 800\ lbs/ft + 433.13\ lbs/ft$

$W_u = 7764.43\ lb/ft \Rightarrow \boxed{W_{u_2} = 7.76\ k/ft}$

(3.) $W_u = 1.2D + 1.6S + L$

$W_u = 6531.3\ lbs/ft + 1.6(34.65\ psf \times 25') + (20\ psf \times 25')$

$W_u = 6531.3\ lbs/ft + 1386\ lbs/ft + 500\ lbs/ft$

$W_{u_3} = 8417.3\ lbs/ft \Rightarrow \boxed{W_{u_3} = 8.41\ k/ft}$

<u>2nd Floor</u>

Dead load: Roof Dead load = 193.57 psf

Roof solid head = 71 psf

MEP = 5 psf

Dome slab = 103 psf

solid head 3rd Floor = 162 psf - 103 psf = 59 psf

Walls and partitions = 1496466.34 / (172' x 92.72')

= 94 psf.

columns = 11.52 psf

(1.) $W_u = 1.4D \Rightarrow W_u = 1.4\,(\underbrace{193.57\ psf + 5\ psf + 103\ psf + 94\ psf + 11.52\ psf)(25')}_{10{,}179.25\ lb/ft}$

$+ 1.4\,(56\ psf + 71\ psf)(8.5')$

$W_{u_1} = 14{,}248.15\ lb/ft + 1511.3\ lb/ft \Rightarrow \boxed{W_{u_1} = 15.76\ k/ft}$

WJII

(2.) $W_u = 1.2D + 1.6L + 0.5S$

Snow load = 34.65 psf

live load = A. 3741.80 ft² + 907.61 ft² = 40 psf → meeting room

B. 1752.0 ft² = 150 psf → library stacks.

C. 4299.42 = 50 psf → office room

D. 3174.5 ft² = 80 psf → corridor 3rd floor

E. 1786.07 ft² = 100 psf → mechanical room

F. 591.0 ft² + 2 × 281.75 ft² = 100 psf → stairs.

Total Area 1st Floor = 172 ft × 92.87 ft = $\boxed{15,973.64 \text{ ft}^2}$

$\boxed{A = 24.40\%}$ $\boxed{B = 10.97\%}$ $\boxed{C = 26.92\%}$ $\boxed{D = 19.87\%}$

$\boxed{E = 11.18\%}$ $\boxed{F = 6.66}$ → percentages based on live load occupancy

Average live load = 73.0 psf.

$W_{u_2} = 1.2\,(10,177.25 \text{ lb/ft} + 1079.5 \text{ lb/ft}) + 1.6\,(73 \text{ psf})(25')$
$+ 0.5\,(34.65 \text{ psf})(25')$

$W_{u_2} = 13,508.1 \text{ lb/ft} + 2,920 \text{ lb/ft} + 433.12 \text{ lb/ft}$

$W_u = 16,861.23 \text{ lb/ft} \Rightarrow \boxed{W_{u_2} = 16.86 \text{ k/ft}}$

(3) $W_{u_3} = 1.2D + 1.6S + L$

$W_{u_3} = 13,508.1 \text{ lb/ft} + 1.6\,(34.65 \text{ psf})(25') + (73 \text{ psf} \times 25')$

$W_{u_3} = 13,508.1 \text{ lb/ft} + 1386 \text{ lb/ft} + 1825 \text{ lb/ft}$

$W_{u_3} = 16,719.7 \text{ lb/ft} \Rightarrow \boxed{W_{u_3} = 16.72 \text{ k/ft}}$

Wu IV

2nd Floor

Dead load: - Roof + 3rd Floor = 407.09 psf

- Roof + 3rd Floor solid head = 130 psf

- MEP = 5 psf

- Dome slab = 103 psf

- Solid head 2nd floor = 59 psf

- Walls / partitions / columns = 113.68 psf

(1.) $W_u = 1.4D \Rightarrow W_u = 1.4\ (407.09\ psf + 5\ psf + 103\ psf + 113.68\ psf)(25')$
$+ 1.4\ (59\ psf + 130\ psf)(8.5')$

$W_{u1} = 1.4\ (15{,}719.25\ lb/ft) + 1.4\ (1606.5\ lb/ft)$

$W_{u1} = 24{,}256.05\ lb/ft \Rightarrow \boxed{W_{u1} = 24.26\ k/ft}$

(2.) $W_{u2} = 1.2D + 1.6L + 0.5S$

Snow load = 34.65 psf

Live load = A. 2046.78 ft² + 2720.51 ft² = 50 psf → office room

B. 3172.77 ft² + 500 ft² + 685.50 ft² = 40 psf → meeting room

C. 1013.87 ft² = 100 psf → mechanical room

D. 501 ft² + 2 (281.75 ft²) = 100 psf → stairs

E. 4099.42 ft² + 814.58 ft² = 80 psf → corridor

Total area of Floor = 172' x 92.87' = $\boxed{15{,}973.64\ ft^2}$

Percentages based on occupancy ⇒ $\boxed{A = 29.84\ \%}$ $\boxed{B = 27.73\ \%}$

$\boxed{C = 6.34\ \%}$ $\boxed{D = 6.66\ \%}$ $\boxed{E = 30.76\ \%}$

Average Live load = 63.10 psf

WuⅡ

$W_{u_2} = 1.2(15{,}719.25 \text{ lbs/ft} + 1606.5 \text{ lbs/ft}) + 1.6(63.10 \text{ psf})(25')$
$+ 0.5(34.65 \text{ psf})(25')$

$W_{u_2} = 20{,}790.9 \text{ lbs/ft} + 2524 \text{ lbs/ft} + 433.13 \text{ lbs/ft}$

$W_{u_2} = 23{,}748.03 \text{ lbs/ft} \Rightarrow \boxed{W_{u_2} = 23.75 \text{ K/ft}}$

(3) $W_u = 1.2D + 1.6S + L$

$W_u = 20{,}790.9 + 1.6(34.65 \text{ psf})(25') + (63.10 \text{ psf})(25')$

$W_{u_3} = 23{,}754.4 \text{ lbs/ft} \Rightarrow \boxed{W_{u_3} = 23.75 \text{ K/ft}}$

1st Floor (Gravity Load Combinations)

Dead load: Roof, 3rd Floor, 2nd Floor = 628.77 psf

Roof, 3rd, 2nd solid head = 189 psf

MEP = 5 psf

Dome slab = 103 psf

Solid head = 59 psf

Walls / partitions / columns = 121.94 psf

(1) $W_u = 1.4D \Rightarrow W_u = 1.4(628.77 \text{ psf} + 5 \text{ psf} + 103 \text{ psf} + 121.94 \text{ psf})(25')$
$+ 1.4(189 \text{ psf} + 59 \text{ psf})(8.5')$

$W_u = 1.4(21{,}467.75 \text{ lbs/ft}) + 1.4(2108 \text{ lbs/ft})$

$W_{u_1} = 33{,}006.05 \text{ lbs/ft} \Rightarrow \boxed{W_{u_1} = 33.00 \text{ k/ft}}$

W_u II

(2) $W_{u_2} = 1.2D + 1.6L + 0.5S$

Snow load = 34.65 psf

Dead Load = 21,467.75 + 2108 lbs / ft

live load = (A) 4664.27 ft² + 910.21 ft² = 40 psf → meeting room

(B) 3269.25 ft² + 4023.19 ft² = 150 psf → library stack

(C) 946.46 ft² = 60 psf → reading room

(D) 522.75 ft² = 100 psf → corridor 1st floor

(E) 1221.53 ft² = 100 psf → mech room

(F) 501.0 + 2(281.75 ft²) = 100 psf → stairs.

Total Area 1st floor = 172 ft × 92.57 ft = $\boxed{15,923.64\ ft^2}$

$\boxed{A = 30.89\%}$ $\boxed{B = 45.65\%}$ $\boxed{C = 5.93\%}$ $\boxed{D = 3.27\%}$

$\boxed{E = 7.65\%}$ $\boxed{F = 6.66\%}$

Average live load 1st Floor = 102.20 psf

$W_{u_2} = 1.2(21,467.75\ lbs/ft + 2108\ lbs/ft) + 1.6(102.20\ psf)(25')$

$+ 0.5(34.65\ psf)(25')$

$W_{u_2} = 28,290.9\ lbs/ft + 4088\ lbs/ft + 433.13\ lbs/ft$

$W_{u_2} = 32,812.03\ lbs/ft \Rightarrow \boxed{W_{u_2} = 32.81\ k/ft}$

(3) $W_{u_3} = 1.2D + 1.6S + L$

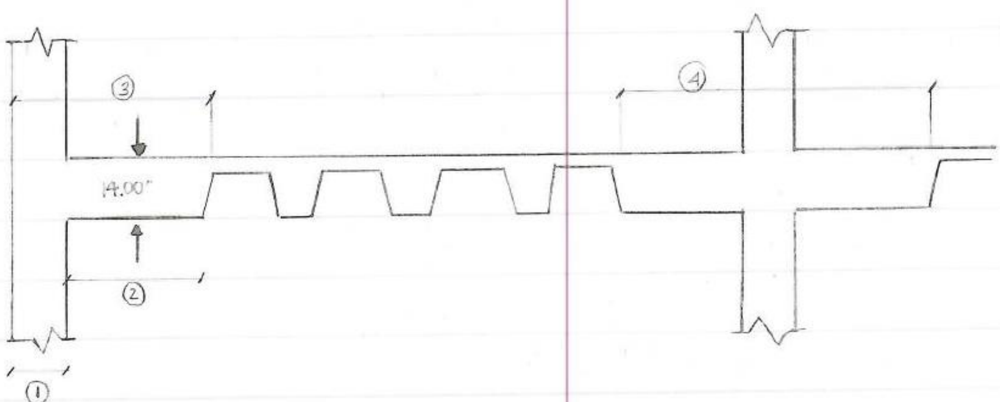
$W_{u_3} = 28,290.9\ lbs/ft + 1.6(34.65\ psf)(25') + (102.20\ psf)(25')$

$W_{u_3} = 32,231.9\ lbs/ft \Rightarrow \boxed{W_{u_3} = 32.23\ k/ft}$

Appendix C.6: Two-Way Dome Slab Calculations

SL

Typical Two-Way Dome Slab with Columns



① Size of column (will change for each floor)

② Separation from column to start of waffle slab (one side)

③ Distance from edge of column to the start of waffle slab

④ Total distance of full slab thickness (solid head)

SL I

t_e = avg. equivalent thickness.

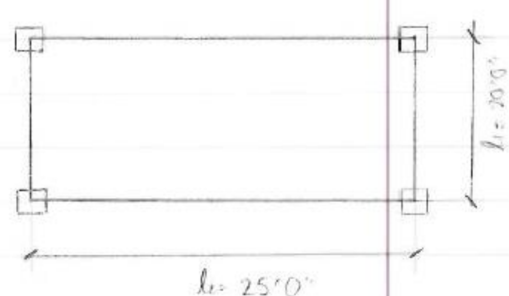
See Example 1 Chapter 11-10 and Ex 2.

$(t_e)^3 = \frac{1}{10}\left[2(t_s)^3 + 8(t_r)^3\right]$ where, t_r = thickness of ribbed area

t_s = thickness of solid head

- The solid head over columns is treated as if it were a drop panel in conventional flat slab construction (CRSI, Ch 11-10)
- Ribs are designed as joists (each rib contains (2) two bottom bars
- straight top bars are used and are of two lengths in the column strip to account for the negative moment reinforcement

Center-to-center span $l_1 = 20'$ $l_2 = 25'$ (interior columns)



Solid head: shall extend in each direction from the centerline of support a distance not less than 1/6 the span length measured from center to center of supports in that direction

Minimum solid head = $\frac{1}{6}l_1 + \frac{1}{6}l_2$

Minimum solid head = $\frac{1}{6}(25') + \frac{1}{6}(20') \Rightarrow 4.17' + 3.33'$

$\boxed{\text{min. solid head} = 7.50'}$

SL-II

Structural Analysis Two-Way Dome slab

↳ This analysis works for the 1st, 2nd and third floor of the Gordon Library Building. A separate analysis will be made for the Two-way Dome slab for the roof.

Width of ribs of Dome slab:

Used ribs 5 inches → Sheet S-11, Structural Drawings

Overall depth of ribs:

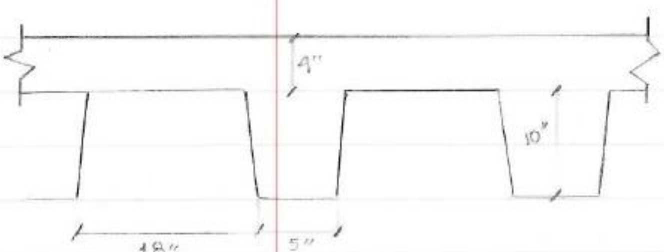
Used 10 inches for overall depth.

Clear spacing between ribs:

Used 19 inches as a clear spacing between ribs

Slab Thickness:

Used 4 inches for slab thickness



The Two-Way Slab is the same for each floor described above, however the size of the columns will differ per each floor. A separate analysis is necessary for each floor.

SL III

See pg 248 ACI

Based on structural drawings of Gordon Library
- Assume a square drop panel or solid head
- Assume a $\underline{\underline{8'6''}}$ solid section for drop panel based on dimensions given in S-14 sheet structural Drawing gordon Library. This includes the dimensions of the column
- All drop panel will have same dimensions for ease of construction and for calculations.

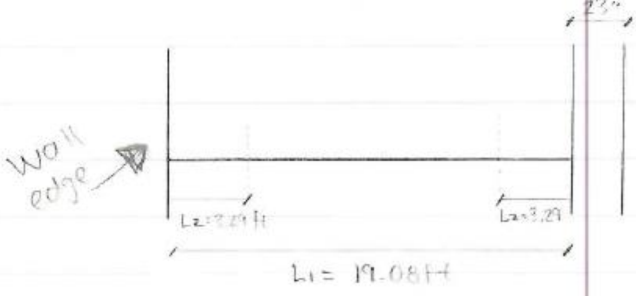
Two-Way Dome slab Capacity (End span)

Total static Moment (Mo):

M_{o1} (outside head) = $W_u L_1^2 / 8$ $\quad$ L_1 = column to column distance

M_{o2} (head) = $(W_{u\,solid} - W_u)(L_2)^2 / 2$ $\quad$ L_2 = column to end of solid head

Total M_o = M_{o1} + M_{o2}



Total M_o = $(28.19\ K/ft)(19.08\ ft)^2/8$ => $\boxed{M_o = 1282.81\ K\,ft}$

SL III

Exterior Column (Negative Moment) (ACI 13.6.4.2)

$Mu = 0.26\ Mo \Rightarrow \boxed{Mu = 333.53\ ft}$

Column strip resist 100% of Mu, therefore 1.0Mu

Bottom (Positive Moment) (ACI 13.6.4.4)

$Mu = 0.52\ Mo \Rightarrow Mu = 0.52(1282.81) \Rightarrow \boxed{Mu = 667.06\ K.ft}$

Column strip resists 60% of Mu, therefore 0.6Mu

$Mu = 400.24\ K.ft$

Middle strip resists 40% of Mu, therefore 0.40Mu

$Mu = 266.82\ K.ft$

Interior Column (Negative Moment) (ACI 13.6.4.1)

$Mu = 0.70\ Mo \Rightarrow \boxed{Mu = 897.97\ K.ft}$

Column strip resists 75% of Mu, therefore 0.75Mu

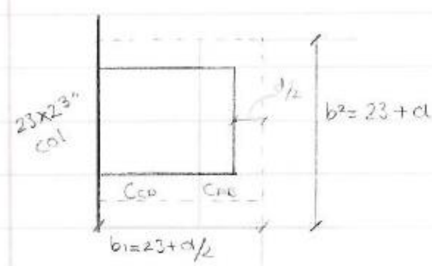
$0.75(897.97) \Rightarrow Mu = 673.48\ K.ft$

Middle strip resists 25% of Mu, 0.25Mu

$Mu = 224.49\ K.ft$

SL VI

Shear at External column



$d = (4 + 10) -$ concrete cover $- d_b$

where $d_b =$ diameter of bar

$d = 14 - 0.75 - 0.50$

$\boxed{d = 12.75 \text{ in}}$

$b_1 = 23 + 12.75/2 \Rightarrow \boxed{b_1 = 29.38 \text{ in}}$

$b_2 = 23 + 12.75 \text{ in} \Rightarrow \boxed{b_2 = 35.75 \text{ in}}$

$b_o = 2 \times b_1 + b_2 \Rightarrow \boxed{b_o = 94.51 \text{ in}}$

$A_c = b_o \times d \Rightarrow \boxed{A = 1205.0 \text{ in}^2}$

$\boxed{C_{AB} = 9.07 \text{ in}} \Rightarrow \boxed{C_{CD} = 20.31 \text{ in}}$

Factored Shear (V_u) at centerline of exterior column:

$$V_u = W_u L/2 - \frac{(M_{u,int} - M_{u,ext})}{L}$$

$$V_u = (28.19 \text{ k/ft})(19.08 \text{ ft})/2 - \left[\frac{(897.97 - 333.53 \text{ kft})}{19.08 \text{ ft}}\right]$$

$$V_u = 268.93 \text{ k} - 29.58 \text{ k}$$

$$\boxed{V_u = 239.35 \text{ kip}}$$

SL VI

Shear check at Exterior Column

$V_c = (\alpha_s d/b_o + 2)\sqrt{f'c} \leq 4\sqrt{f'c}$ (ACI Eq. 11-32)

$\alpha_s = 30$ for edge columns

$V_c = ([30 \times 12.75 / 94.51] + 2)\sqrt{f'c} \Rightarrow 6.05\sqrt{f'c} = \boxed{382.64 \text{ psi}}$

A: 382.64 psi $\geq 4\sqrt{4000\text{psi}} = 253$ psi We need to use 253 psi

→ ACI Eq. 11-35

$\phi V_c = 0.75(253) \Rightarrow \boxed{\phi V_c = 190 \text{ psi}}$ or $\boxed{\phi V_c = 0.191 \text{ ksi}}$

$v_u = \frac{V_u}{A_c} + \frac{\gamma_v M_u C_{AB}}{J_c}$

where $\gamma_v = (1 - \gamma_f)$ (ACI Eq. 11-37)

$\gamma_f = 1/(1+(2/3)\sqrt{b_1/b_2})$ (ACI Eq. 13-1)

$= 1/(1+(2/3)\sqrt{29.38/35.75}) = \boxed{0.623}$

$\gamma_v = (1 - 0.623) \Rightarrow \boxed{\gamma_v = 0.38}$

$M_u = 0.3 M_o$ (ACI 13.6.3.6)

$M_u = 0.3\,(1282.81 \text{ k}\cdot\text{ft}) \Rightarrow \boxed{M_u = 384.84 \text{ k}\cdot\text{ft}}$

$J_c = \frac{b_1 d^3}{6} + \frac{2d[(C_{AB})^3 + (C_{CD})^3]}{3} + b_2 d (C_{AB})^2$

$J_c = \frac{(29.38)(12.75)^3}{6} + \frac{2(12.75)[(9.07)^3 + (20.31)^3]}{3} + (35.75)(12.75)(9.07)^2$

SL VIII

$J_c = 10149.18 + 77553.48 + 37497.37$

$\boxed{J_c = 125{,}200 \text{ in}^4}$

$$v_u = \frac{275.94\text{k}}{1205.0\text{ in}^2} + \frac{(0.38)(384.84)(9.07\text{ in})}{125{,}200\text{ in}^4}$$

$V_u = 0.245 + 0.0106 \qquad \boxed{v_u = 0.255 \text{ ksi}}$

Design Moment strength (ϕM_n)

A_s = # of bars in column strip x Nominal Area of bars

$A_s = (33) \times (0.2 \text{ in}^2) \Rightarrow \boxed{A_s = 6.6 \text{ in}^2}$

$$a = \frac{A_s f_y}{0.85 f'_c b} \Rightarrow a = \frac{(6.6\text{ in}^2)(60\text{ ksi})}{0.85(4\text{ ksi})(51\text{ in})} \Rightarrow \boxed{a = 2.28 \text{ in}}$$

where b = effective width = half the width of the panel = 51 in

$$\phi M_n = \phi A_s f_y (d - a/2) \Rightarrow \phi M_n = 0.90(6.6\text{ in}^2)(60\text{ ksi})\left(12.75 - \frac{2.28}{2}\right)$$

$\boxed{\phi M_n = 344.82 \text{ k·ft}} \qquad \phi M_n > M_u$ ✓

$M_u = 0.26 M_o = 0.26(1282.81) \Rightarrow M_u = 333.53 \text{ k ft}$

$\gamma_f M_u = 0.623(333.53 \text{ k·ft}) \Rightarrow \boxed{207.79 \text{ k·ft}}$ ✓

SL VIII

$\rho = \frac{A_s}{bd} \Rightarrow \rho = \frac{6.6 in^2}{(51 in)(12.75 in)} \Rightarrow \boxed{\rho = 0.010}$

$\rho_{max} = 0.011$ $\quad$ $\rho \leq \rho_{max}$ ✓

Interior Span

Panel Moments

- Bottom (positive moment)

$M_u = 0.35 M_o \Rightarrow M_u = 0.35(1282.81) \Rightarrow \boxed{M_u = 448.98 \text{ k ft}}$

Column strip resist 60% of Mu

$0.60 M_u \Rightarrow 0.6(448.98) \Rightarrow \boxed{M_u = 269.39 \text{ k ft}}$

Middle strip resists 40% of Mu

$0.40 M_u = 0.40(448.98) \Rightarrow \boxed{M_u = 179.59 \text{ k ft}}$

- Top (negative moment)

$M_u = 0.65 M_o \Rightarrow M_u = 0.65(1282.81) \Rightarrow \boxed{M_u = 833.83 \text{ k ft}}$

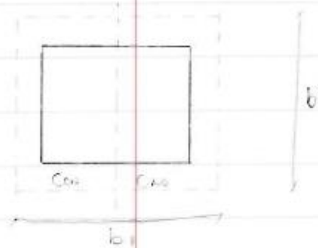
Column strip resist 75% of Mu

$0.75 M_u = \boxed{625.37 \text{ k Mu}}$

Middle strip resist 25% of Mu

$0.25 M_u \Rightarrow \boxed{M_u = 208.46 \text{ k ft}}$

Shear at Interior Column



SL IX

$b_1 = b_2 = 23 + 12.75 \Rightarrow 35.75 \text{ in}$

$b_o = 4 \times 35.75 \text{ in} \Rightarrow b_o = 143 \text{ in}$

$A_s = b_o \times d \Rightarrow \boxed{A_s = 1823.25 \text{ in}^2}$

$C_{AB} = C_{CD} = 35.75/2 \Rightarrow \boxed{C_{AB} = 17.88 \text{ in}}$

$J_c = (d b_1 / 6)[d^2 + 4 b_1^2]$

$$J_c = \frac{(12.75 \times 35.75)}{6}\left[(12.75)^2 + 4(35.75)^2\right]$$

$$\boxed{J_c = 400{,}722.23 \text{ in}^4}$$

Shear (V_u) of interior column

$$V_u = W_u L/2 + \frac{(M_{u,ext} - M_{u,int})}{L}$$

$$V_u = \frac{(28.19 \text{ k/ft})(19.08 \text{ ft})}{2} + \frac{(897.97 - 333.53 \text{ k ft})}{(19.08 \text{ ft})}$$

$$\boxed{V_u = 298.52 \text{ kips}}$$

Shear check

$V_c = (\alpha_s d / b_o + 2)\sqrt{f'c} \leq 4\sqrt{f'c}$

$V_c = (40 \times 12.75/143 \text{ in} + 2)\sqrt{f'c}$

$V_c = 5.56\sqrt{f'c}$ $\quad V_c = 352 \text{ psi}$

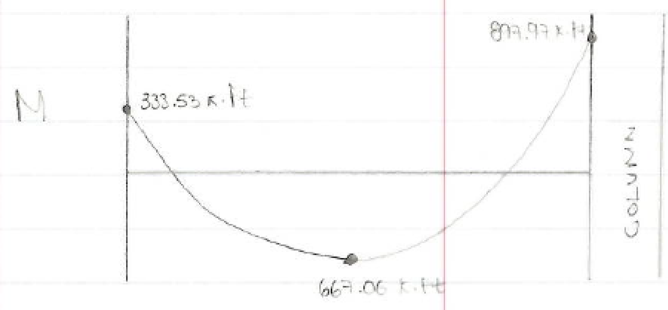
$$\boxed{\phi V_c = 264.04 \text{ psi}}$$

SLX

$v_u = \frac{V_u}{A_c} \Rightarrow v_u = \frac{298.52 \text{ kips}}{1823.25 \text{ in}^2} \Rightarrow \boxed{v_u = 0.164 \text{ ksi}}$

$\phi v_c \geq v_u$ ✓

Factored Moment at End span



SL XI

Slab Calculation Section 2 $L = 25'$; $L_1 = 23.08'$

Slab size = (20' × 25' c/c) = 23.08 ft × 18.08 ft

$M_o = W_u L^2/8 \Rightarrow M_o = \frac{(33.00 \text{ k/ft})(23.08 \text{ ft})^2}{8}$

$\boxed{M_o = 2{,}197.33 \text{ k}\cdot\text{ft}}$

Interior Support

$M_{int}^- = 0.65 M_o \Rightarrow 0.65(2197.33) \Rightarrow \boxed{M_{int}^- = 1428.26 \text{ k}\cdot\text{ft}}$

$M^+ = 0.35 M_o \Rightarrow \boxed{M^+ = 769.07 \text{ k}\cdot\text{ft}}$

Total Column Strip Moments

$M_{cint}^- = 0.75 M_{ext}^- = 0.75(1428.26) \Rightarrow \boxed{1071.19 \text{ k ft}}$

$M_c^+ = 0.60 M^+ \Rightarrow M_c^+ = 0.60(769.07) \Rightarrow \boxed{M_c^+ = 461.44 \text{ k}\cdot\text{ft}}$

$\boxed{M_{cint}^- = 1071.19 \text{ k-ft}}$

Total middle strip moments

$M_{mint}^- = 1428.26 - 1071.19 \Rightarrow \boxed{M_{mint}^- = 357.09 \text{ k ft}}$

$M_m^+ = 769.07 \text{ k ft} - 461.44 \Rightarrow \boxed{M_m^+ = 307.64 \text{ k ft}}$

$M_{mint}^- = \boxed{357.09 \text{ k}\cdot\text{ft}}$

Appendix C.7: Interaction Diagram Calculations

ID I

Computation of Nominal Axial and Nominal Moment Strength.

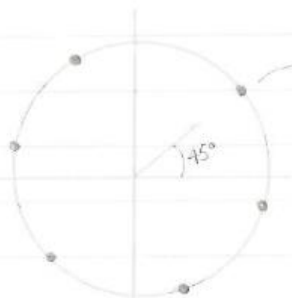


Figure 1: 6 bars

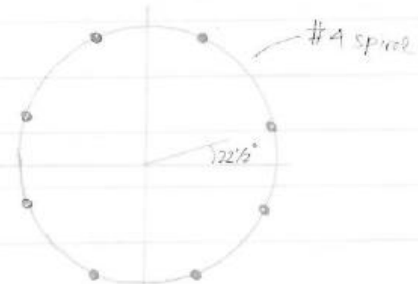


Figure 2: 8 bars

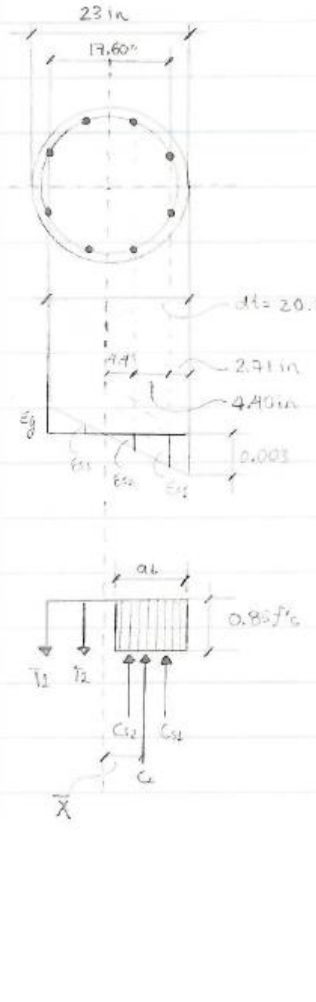
Example: Column Type Figure 2

- 1½" cover
- 8 #11 bars
- #4 spirals
- 2½" pitch
- $f'_c = 4$ ksi
- $f_y = 60$ ksi
- $E_s = 29{,}000$ ksi
- A_s #11 bar = 1.56 in²
- Diameter #4 bar = 0.50 in
- Diameter #11 bar = 1.41 in

$\varepsilon_y = f_y / E_s$

$\varepsilon_y = 60 \text{ ksi} / 29{,}000 \text{ ksi}$

$\boxed{\varepsilon_y = 0.00207}$



10 II

Location of NA:

$dt = 23\,in - 1\tfrac{1}{2}'' - 0.50'' - \frac{1.41\,in}{2} \Rightarrow \boxed{dt = 20.29\,in}$

cover; diameter of spiral

$\varepsilon_y = 0.00207$

$$x_b = \frac{0.003}{0.003 + \varepsilon_y} \times dt \Rightarrow x_b = \frac{0.003}{0.003 + 0.00207}(20.29'')$$

$$\boxed{x_b = 12.00\,in}$$

Depth of Whitney stress block

$a_b = \beta_1 x_b$

Where, $\beta_1 = 0.85$ for $f'c \leq 4000$ psi or 4 ksi

$a_b = 0.85\,(12.00'') \Rightarrow \boxed{a_b = 10.2\,in}$

Circular compression Block Properties:

$$\alpha = \cos^{-1}\left(\frac{h/2 - a_b}{h/2}\right) \Rightarrow \cos^{-1}\left(\frac{\frac{23\,in}{2} - 10.2\,in}{23\,in/2}\right) \Rightarrow \cos^{-1}(0.113)$$

$\boxed{\alpha = 83.51°}$ $\boxed{\alpha = 1.46\,rad}$

Area of circular compression block

$$A = \frac{h^2}{2}\left(\frac{\alpha_{rad}}{2} - \frac{1}{4}\sin 2\alpha\right)$$

$$A = \frac{(23\,in)^2}{2}\left(\frac{1.46}{2} - \frac{1}{4}\sin 167.02°\right) \Rightarrow \boxed{A = 178.23\,in^2}$$

ID III

Location of centroid of compression block

$$\bar{X} = \frac{\frac{h^3}{4}\left(\frac{\sin^3\alpha}{3}\right)}{A} \Rightarrow \bar{x} = \frac{\frac{(23\,in)^3}{4}\left(\frac{\sin^3 83.51^\circ}{3}\right)}{178.23\,in^2}$$

$$\boxed{\bar{X} = 5.58\,in}$$

Compressive force in compression Block (C_c)

$$C_c = 0.85 f'_c A \Rightarrow C_c = 0.85(4\,ksi)(178.23\,in^2)$$

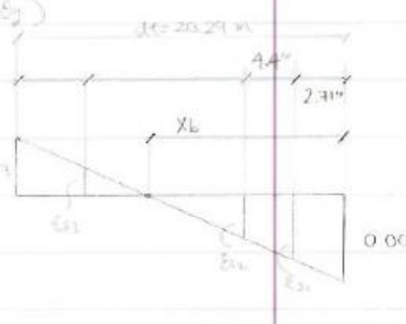
$$\boxed{C_c = 605.98\ Kips}$$

Strains and forces in tension and compression steel

$\varepsilon_{s1} = 0.0023 \quad (> \varepsilon_y)$

$\varepsilon_{s2} = 0.0012 \quad (< \varepsilon_y)$

$\varepsilon_{s3} = 0.00097 \quad (< \varepsilon_y)$



Linear Interpolation to get strain value

Calculate forces in steel

$$T_1 = A_s f_y \Rightarrow T_1 = 2(1.56\,in^2)(60\,ksi) \Rightarrow \boxed{T_1 = 187.2K}$$

$$T_2 = A_s\,\varepsilon_{s3}\,E_s \Rightarrow T_2 = 2(1.56\,in^2)(0.00097)(29000\,ksi)$$

$$\boxed{T_2 = 87.77\ kips}$$

1D IV

$$C_{s1} = A_s\left(f_y - 0.85f'_c\right) \Rightarrow C_{s1} = 2\left(1.56\,in^2\right)\left(60\,ksi - 0.85\left(4\,ksi\right)\right)$$

$$\boxed{C_{s1} = 176.59\ kips}$$

$$C_{s2} = A_s\left(\varepsilon_{s2}E_s - 0.85f'_c\right) \Rightarrow C_{s2} = 2\left(1.56\,in^2\right)\left(0.0012 \times 29000\,ksi - 0.85(4k)\right)$$

$$\boxed{C_{s2} = 97.97\ kips}$$

Nominal axial compression strength

$$P_n = C_c + C_{s1} + C_{s2} - T_1 - T_2$$

$$\boxed{P_n = 605.57\ kips}$$

Nominal moment strength

$$M_n = \left[C_c \times \bar{X} + \left(T_1 + C_{s1}\right)(D) + \left(T_2 + C_{s2}\right)(D_1)\right]$$

D = distance from PNA to C_{s1}

D_1 = distance from PNA to C_{s2}

$$M_n = \left[(605.98\,k)(5.58\,in) + (176.59\,k + 187.2\,k)(8.80\,in) + (97.97\,k + 87.77\,k)(4.40\,in)\right]$$

$$M_n = 3381.37\ k\cdot in + 3201.35\ k\cdot in + 817.26\ k\cdot in$$

$$\boxed{M_n = 7399.98\ k\cdot in} \quad \text{or} \quad \boxed{M_n = 616.66\ k\cdot ft}$$

10 V

Nominal Axial and Moment strength (Full Axial)

↳ assuming different "a" value for interaction diagram purposes

Depth of Whitney Stress Block

$\boxed{a_b = 23\,in}$

Circular compression Block Properties

$$\alpha = \cos^{-1}\left(\frac{h/2 - a_b}{h/2}\right) \Rightarrow \cos^{-1}\left(\frac{23/2 - 23\,in}{23/2}\right) \Rightarrow \cos^{-1}(-1).$$

$\boxed{\alpha = 180°}$ $\boxed{\alpha = 3.1415}$

Area of compression Block

$$A = h^2/2\left(\frac{\alpha rad}{2} - \frac{1}{4}\sin 2\alpha\right) \Rightarrow A = \frac{(23)^2}{2}\left(\frac{3.1416}{2} - \frac{1}{4}\sin 360\right)$$

$\boxed{A = 415.46\,in^2}$

Location of centroid of compression block

$$\bar{X} = \frac{h^3/4\left(\frac{\sin^3\alpha}{3}\right)}{A} \Rightarrow \boxed{\bar{X} = 0}$$

Compressive force (C_c)

$C_c = 0.85 f'c\,A \Rightarrow \boxed{C_c = 1412.56\text{ kips}}$

Strain Values (ε)

strains will have a value of $\varepsilon = 0.003$ all across the whitney stress block

$C_{s1} = 4A_s\,(f_y - 0.85 f'c) \Rightarrow \boxed{C_{s1} = 353.18\,K}$

$C_{s2} = A_s\,(\varepsilon E_s - 0.85 f'c) \Rightarrow C_{s2} = 4(1.56\,in^2)(0.003 \times 29000\,ksi - 0.85(4ksi))$

$\boxed{C_{s2} = 521.66\,K}$

$P_{n\,max} = C_c + C_{s1} + C_{s2}$

$P_{n\,max} = 1412.56\text{ K} + 353.18\text{ K} + 521.66\text{ K}$

$\boxed{P_{n,max} = 2287.40\text{ kips}}$

$\boxed{M_n = 0}$

<u>Nominal Axial and Moment strength (Pure Flexure Point P=0)</u>

$P = C_{s1} + C_{s2} + C_c - T_1 - T_2 = 0$

$0 = A_s\left(f_y - 0.85f'c\right) + A_s\left(0.003\left(\frac{x_b - d_1}{x_b}\right)E_s - 0.85f'c\right) + 0.85f'c\,A$

$- A_s f_y - 0.00207A_s\left(\frac{(d_t - x_b) - d_1}{d_t - x_b}\right)E_s$

$A = \frac{h^2}{2}\left(\frac{\alpha_{rad}}{2} - \frac{1}{4}\sin 2\alpha\right) \Rightarrow A = \frac{h^2}{2}\left(\frac{\overbrace{\cos^{-1}\left(\frac{h/2 - B_1 x_b}{h/2}\right)}^{\alpha_{rad}}}{2}\right) - \frac{1}{4}\sin 2\left(\cos^{-1}\left(\frac{h/2 - B_1 x_b}{h/2}\right)\right)$

$A = \frac{(23)^2}{2}\left(\frac{\cos^{-1}\left(\frac{\frac{23}{2} - 0.85x_b}{23/2}\right)}{2}\right) - \frac{1}{4}\sin 2\left(\cos^{-1}\left(\frac{\frac{23}{2} - 0.85x_b}{23/2}\right)\right)$

$A = 264.5\left(\frac{\cos^{-1}\left(\frac{11.5 - 0.85x_b}{11.5}\right)}{2}\right) - 0.25\sin 2\left(\cos^{-1}\left(\frac{11.5 - 0.85x_b}{11.5}\right)\right)$

10 VII.

$$\boxed{A = 264.5\left(\frac{\cos^{-1}\left(1-0.0739\,xb\right)}{2} - 0.25\sin 2\left(\cos^{-1}\left(1-0.0739\,xb\right)\right)\right)}$$

$$0 = 2(1.56)\left(60-0.85(4)\right) + 2(1.56)\left(0.003\left(\frac{xb-2.71}{xb}\right)29000 - 0.85(4)\right)$$

$$+ 0.85(4)A - 2(1.56)(60) - 0.0027(2\times1.56)\left(\frac{(20.29-xb)-4.4}{20.29-xb}\right)29000$$

$$0 = 176.59 + 3.12\left(87\frac{(xb-2.71)}{xb} - 3.4\right) + 3.4A - 187.2$$

$$- 187.3\left(\frac{(20.29-xb)-4.4}{(20.29-xb)}\right)$$

$$0 = 176.59 + 3.12\left(87 - \frac{235.77}{xb} - 3.4\right) + 3.4A - 187.2 - 187.3\left(\frac{(20.29-xb)-4.4}{(20.29-xb)}\right)$$

$$0 = 3.12\left(83.6 - \frac{235.77}{xb}\right) + 3.4A - 10.61 - 187.3\left(\frac{(20.29-xb)-4.4}{(20.29-xb)}\right)$$

$$0 = 260.83 - \frac{735.60}{xb} + 3.4A - 10.61 - 187.3\left(\frac{(20.29-xb)-4.4}{(20.29-xb)}\right)$$

$$0 = 250.22 - \frac{735.60}{xb} + 899.3\left(\frac{\cos^{-1}\left(1-0.0739\,xb\right)}{2} - 0.25\sin 2\left(\cos^{-1}\left(1-0.0739\,xb\right)\right)\right)$$

$$- 187.3\left(\frac{(20.29-xb)-4.4}{(20.29-xb)}\right)$$

ID VIII

$0 = 250.22 - \frac{735.60}{xb} - (3800.32 + 187.3\,xb)((20.29 - xb) - 4.4) + \ldots$

$0 = 250.22 - \frac{735.6}{xb} - (3800.32 + 187.3xb)(15.89 - xb) + \ldots$

$0 = 250.22 + \frac{735.6}{xb} - 60387.08 + 3800.32xb - 2976.2xb + 187.3xb^2 + \ldots$

$0 = 187.3\,xb^2 - 60136.86 + \frac{735.6}{xb} + 824.12\,xb + \ldots$

see spreadsheet for completion

ID TX

Tension Control Point ($\varepsilon_c = 0.003$ and $\varepsilon_t = 0.005$)

$$x_b = \frac{0.003}{0.003 + 0.005} \times d_t \Rightarrow \boxed{x_b = 7.60\ in}$$

$$a_b = \beta_1 x_b \Rightarrow a_b = 0.85(7.60\,in) \Rightarrow \boxed{a_b = 6.47\ in}$$

Block properties

$$\alpha = \cos^{-1}\left(\frac{h/2 - a_b}{h/2}\right) \Rightarrow \cos^{-1}\left(\frac{23/2 - 6.47}{23/2}\right) \Rightarrow \cos^{-1}(0.44)$$

$$\boxed{\alpha = 63.67°} \quad or \quad \boxed{\alpha = 1.11\ rad}$$

Area of compression Block

$$A = \frac{h^2}{2}\left(\frac{\alpha_{rad}}{2} - \frac{1}{4}\sin 2\alpha\right) \Rightarrow A = \frac{(23)^2}{2}\left(\frac{1.11}{2} - \frac{1}{4}\sin 127.34\right)$$

$$\boxed{A = 94.22\ in^2}$$

Location of centroid of compression block

$$\bar{x} = \frac{\frac{h^3}{4}\left(\frac{\sin^3\alpha}{3}\right)}{A} \Rightarrow \bar{x} = \frac{\frac{(23)^3}{4}\left(\frac{\sin^3 63.67°}{3}\right)}{94.22\,in^2}$$

$$\boxed{\bar{x} = 7.74\ in}$$

10 X

Compression force in Block (Cc)

$C_c = 0.85 f'_c A \Rightarrow C_c = 0.85(4\,ksi)(94.22\,in^2)$

$\boxed{C_c = 320.35\ kips}$

Strains and forces in tension and compression

$\varepsilon_{s1} = 0.0019$

$\varepsilon_{s2} = 0.00019$

$\varepsilon_{s3} = 0.0033$

Forces in Steel

$T_1 = A_s f_y \Rightarrow T_1 = 2(1.56\,in^2)(60\,ksi) \Rightarrow \boxed{T_1 = 187.2\,k}$

$T_2 = A_s \varepsilon_{s3} E_s \Rightarrow T_2 = 2(1.56\,in^2)(0.0033)(29000\,ksi)$

$\boxed{T_2 = 298.58\,k}$

$C_{s1} = A_s(f_y - 0.85 f'_c) \Rightarrow C_{s1} = 2(1.56\,in^2)(60\,ksi - 0.85(4\,ksi))$

$\boxed{C_{s1} = 176.59\,k}$

$C_{s2} = A_s(\varepsilon_{s2} E_s - 0.85 f'_c) \Rightarrow C_{s2} = 2(1.56\,in^2)(0.00019 \times 29000 - 0.85(4\,ksi))$

$\boxed{C_{s2} = 6.58\,k}$

$P_n = C_c + C_{s1} + C_{s2} - T_1 - T_2 \Rightarrow P_n = 320.35\,K + 176.59\,K + 6.58\,K - 187.2\,K - 298.6\,K$

$\boxed{P_n = 17.74\ kips}$

RF I

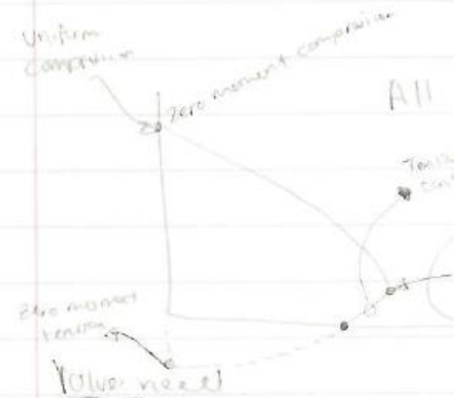
Interaction Diagram Reference

Purpose: ~~combine~~ evaluate loading effect

↳ we want to satisfy $\phi P_n \geq P_u$ and $\phi M_n \geq M_u$

$$1 \geq \frac{P_u}{\phi P_n} \quad \text{and} \quad 1 \geq \frac{M_u}{\phi M_n}$$

$$\frac{P_u}{\phi P_n} + \frac{M_u}{\phi P_n} \leq 1$$



All based on different Cn values

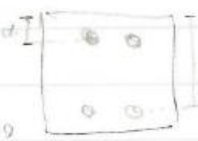
strain steel = yield

Value need

$\beta_1 = 0.85 =$

$d_1 =$ cover from top bar to edge of concrete

$d_2 =$ distance from lower bars to edge of concrete



Five Points in diagram

1. Zero moment compression point $P_n = \max$ $M_n = 0$
2. Balance point:

$f_y = 60$ ksi; $E_s = 29000$ ksi

$\varepsilon_{cu} = 0.003$ when $\varepsilon_s = \varepsilon_y$

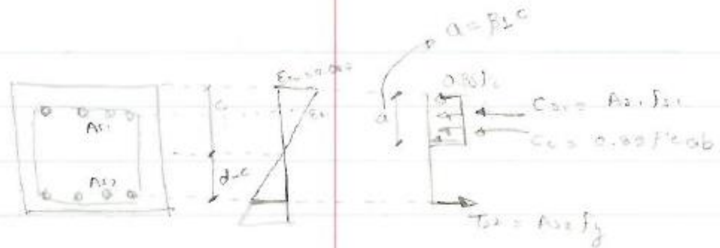
$\varepsilon_{s2} = f_y / E_s = 0.00207$

$$c = d \left(\frac{0.003}{\varepsilon_s + 0.003} \right)$$

↳ distance from lower bars to edge of top concrete

$\boxed{c = \text{value}}$

RF II



$$\boxed{\varepsilon_{s1} = 0.003\left(\frac{c-d_1}{c}\right) = \text{value}}$$

$$\boxed{a = \beta_1 c = 0.85(c)}$$

A_{s1} = area of top steel

A_{s2} = area of bottom steel

C_{s1} = compression steel $= A_{s1} f_{s1} = A_{s1}$ (60ksi)

$C_c = 0.85 f'_c ab = 0.85$ (4ksi)(a)(b)

$T_{s2} = A_{s2} f_y = A_{s2}$ (60ksi)

$$\boxed{\text{Total force} = C_{s1} + C_c + T_{s2}}$$ → P force of column at balance point

Lever arms

Y_{s1} = distance from PNA to edge − d_1 =

Y_{cc} = distance from PNA to edge − $\frac{a}{2}$ =

Y_{s2} = distance from bottom bar to edge − distance from PNA to edge =

$$\boxed{M_n = (C_{s1} \times Y_{s1}) + (Y_{cc} \times C_c) + (Y_{s2} \times T_{s2})}$$

$$\boxed{e = \text{eccentricity} = \frac{M}{P}}$$

RF III

Point 3: Pure flexure Point $(P=0)$

$$P = \underbrace{0.003\,A_{s1} E_s \left(\frac{c-d_1}{c}\right)}_{C_{s1}} + \underbrace{0.85 f'_c \beta_1 c b}_{C_c} - \underbrace{A_{s2} f_y}_{T_{s2}} = 0$$

$$\to \boxed{\left[0.85 f'_c \beta_1 b\right] c^2 + \left[0.003\,A_{s1} E_s - A_{s2} f_y\right] c - 0.003\,A_{s1} E_s d_1 = 0}$$

$\to$ solve for quadratic equation

Get (c) value and based on that $(a) \to a = \beta_1 c$

Get C_{s1}, C_c and T_{s2}

$C_{s1} = 0.003\,A_{s1} E_s \left(\frac{c-d_1}{c}\right)$

$C_c = 0.85 f'_c \beta_1 c b$

$T_{s2} = A_{s2} f_y$

Find Lever arm

y_c = distance from PNA to edge $- \frac{a}{2}$

The other ones are going to be the same as previous y_{s1} and y_{s2}

$$\boxed{M_n = (y_c \times C_c) + (C_{s1} \times y_{s1}) + (T_{s2} \times y_{s2})}$$

Point 4 Pure Tension

$$\boxed{P = (\overset{\text{top}}{A_{s1}} + \overset{\text{bottom}}{A_{s2}})\, f_y}$$

$$\boxed{M_n = 0}$$

Point 5: Tension - Controlled ($\varepsilon_{cu} = 0.003$ and $\varepsilon_{s} = 0.005$)

$$\frac{\varepsilon_{s2}}{d-c} = \frac{0.003}{c} \Rightarrow \boxed{c = d\left(\frac{0.003}{\varepsilon_{s2} + 0.003}\right)}$$

$$\varepsilon_{s1} = 0.003\left(\frac{c - d_1}{c}\right)$$ → strain on top steel

$$a = \beta_1 c \Rightarrow \boxed{a = 0.85c}$$

$C_{s1} = A_{s1} f_y$

$C_c = 0.85 f'_c a b$

$T_{s2} = A_{s2} f_y$

$$\boxed{P = C_{s1} + C_c + T_{s2}}$$

Lever arm

Y_{cc} = PNA to edge $- \frac{a}{2}$

Other ones are the same.

$$\boxed{M_n = (Y_{cc} \times C_c) + (Y_{s1} \times C_{s1}) + (T_{s2} \times Y_{s2})}$$

APPENDIX D: SOLAR COLLECTOR CALCULATIONS

Appendix D.1: Solar Evacuated Tube Load Calculations

Solar Evacuated Collectors Info

- Dimensions : 78,9" x 86,4" x 5,3"
- Weight : 209 lbs (dry)
- angle : 40° south
- Number of collectors : 31, 16 on one side, 15 on another
- Fluid Capacity : 0,2 gal → (0,2 gal)(8,34 lb/gal) = 1,67 lb (30 tubes) = 50,1 lb
- Snow load capacity : 60 lb/ft². angle = 45° or higher
- Wind load capacity : up to winds of 196 mph

- Dead Load D

↳ One collector = 209 lbs + 50.1 lbs = 259.1 lbs (16) = 4145.6 lbs (one side)

↳ Area = (78.9"/12')(86.4"/12) = 47.34 ft^2

$$D = \frac{259.1\ lbs}{47.34\ ft^2} = \underline{5.47\ psf}$$ * one collector

- Snow Load S

↳ Flat roof snow load (section 7.3)

* $P_F = 0.7\ C_e\ C_t\ I_F\ P_g$ *

↳ Exposure factor (C_e) (table 7-2)

Terrain Category B (Section 26.7); fully exposed roof

$C_e = 0.9$

↳ Ground Snow load (P_g) (Figure 7-1)

Worcester = 50 psf

↳ Thermal factor (C_t) (table 7-3)

$C_t = 1.0$

↳ Importance Factor (I_s) (table 1.5-2)

Risk category <u>III</u>

$I_s = 1.10$

↳ Flat Roof Snow Load

$P_F = 0.7\ C_e\ C_t\ I_F\ P_g$

$P_F = 0.7(0.9)(1.0)(1.10)(50\ psf)$

$P_F = 34.65\ psf$

$\underline{S = 34.65\ psf}$

- Wind Load W (Rooftop Solar Arrays buildings $\theta \leq 7°$)

1) Risk Category (table 1.5-1)

risk category III

2) Basic Wind Speed (Fig 26.5-1 B)

Worcester, Ma → $V = 135$ mph

3) Wind directionality factor (Fig 26.6-1)

→ Structure type: building

$K_d = 0.85$

→ Exposure Category (26.7 Section)

Exposure B

→ Topographic Effects (Section 26.8)

no topographic factor

$K_{zt} = 1.0$

→ Gust Effect factor (Section 26.9)

Rigid building

$G = 0.85$

4) Velocity Pressure Exposure Coefficient (table 29.3-1)

Height above ground level = 26 ft ; Exposure B

$K_z = 0.668$

5) Velocity Pressure (Section 29.3.2)

$q_z = 0.00256\, K_z\, K_{zt}\, K_d\, V^2$

$q_z = 0.00256\,(0.668)(1.0)(0.85)(135\text{ mph})^2$

$q_z = 26.49$ psf

- From wind design for low profile solar photovoltaic arrays on flat roofs

1) Compute A_{PV}

$A_{PV} = 0.5\sqrt{h\,W_L}$ — h = building height

$A_{PV} = 0.5\sqrt{26ft\,(48')}$ — W_L = building width on longest side

$A_{PV} = 17{,}66\ ft$

* $A_{PV} \le h$. if $A_{PV} > h$, then $A_{PV} = h$

2) Normalized Wind area A_n

$$A_n = \left(\frac{1000}{(\max(17{,}66',\ 15ft))^2}\right) A$$

A = [illegible] area of [illegible]

$$A_n = \left(\frac{1000}{(17{,}66ft)^2}\right)(48')(48')$$

$A_n = 7387{,}56$

3) Normalized Net pressure (Figure 29,9-1)

$15° \le \omega \le 35°$

* 5000 is max A_n

for $A_n = 0{,}3$ ⟶ $(GC_{rn})_{nom} = 0{,}42$

4) Panel length Chord Length factor

$\gamma_c = 0{,}6 + 0{,}6\ l_p$ — l_p = chord length of system = 7,2 ft

$\gamma_c = 0{,}6 + 0{,}6(7{,}2')$

$\gamma_c = 4{,}92$

5) Parapet Heigth factor

$h_{pt} \le 4\ ft$

$\gamma_p = 1.0$

☆ 6) Determine Characteristic Height

$h_c = \min(h_1,\ 1ft) + l_p \sin\omega$ — h_1 = solar panel height above roof at low edge

$h_c = \min(5{,}3'',\ 1ft) + (7{,}2')\sin(40°)$

$h_c = 6{,}31\ ft$

7) Determine Array Edge Factor (Fig 29.9-1)

$d_x/h_c = \frac{3 ft}{6.3 ft}$

d_x = horizontal distance from edge of panel to edge of roof or other panel

$d_x/h_c = 0.5$

- $E = 1.0$

8) Net Pressure Coefficient

$(GC_{rn}) = \gamma_p E [(GC_{rn})_{nom} (\gamma_c)]$

$(GC_{rn}) = 1.0(1.0)[(0.3)(4.92)]$

$(GC_{rn}) = 1.47$

9) Design Wind Pressure

$p = q_z (GC_{rn})$

$p = 26.49(1.47)$

$p = 39.09 psf$

- Seismic Load E

- S_S : (Figure 22-1)
  Worcester, Ma : MCER - 18% g
- S_1 : (Figure 22-2)
  Worcester, Ma : MCER = 7% g

↳ Seismic Design Category
  Soil classification (Section 20)
    Site D - unknown details

↳ Design spectral acceleration Parameters (Section 11.4.4)

$$S_{DS} = \frac{2}{3} S_{MS}$$

$$S_{D1} = \frac{2}{3} S_{M1}$$

- Spectral response acceleration Parameter (Section 11.4.3)

$$S_{MS} = F_a S_s$$

$$S_{M1} = F_v S_1$$

- Site Coefficients
  For site D & $S_s \leq 0.25$ (table 11.4-1) & $S_1 \leq 0.1$ (table 11.4-2)

$F_a = 1.6$	$F_v = 2.4$
$- S_{MS} = 1.6(0.18)$	$- S_{M1} = 2.4(0.07)$
$S_{MS} = 0.288$	$S_{M1} = 0.168$
$- S_{DS} = 2/3(0.288)$	$- S_{D1} = 2/3(0.168)$
$S_{DS} = 0.192$	$S_{D1} = 0.112$

↳ Risk Category (table 1.5-1)
  Risk Category III

↳ Seismic Design Category (table 11.6-1)
  For $0.167 \leq S_{DS} \leq 0.33$ & risk category III
  SDC = B

↳ Seismic Importance Factor (table 1.5-2)

Risk category III

$I_e = 1.25$

↳ Seismic Response Coefficient (Section 12.8.1.1)

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{0.192}{3/1.25} = 0.8$$

↳ Response modification factor (R) (table 12.2-1)

$R = 3$

↳ Seismic base shear (V) (Section 12.8.1)

$V = C_s W$ $\quad$ $W = D + 0.2S$

$V = 0.8(12.4\text{psf})$ $\quad$ $W = 5.47\text{psf} + 0.2(34.65\text{psf})$

$V = 9.92\text{ psf}$ $\quad$ $W = 12.4\text{psf}$

↳ Fundamental Period (Section 12.8.2.1)

$T_a = 0.1N$ $\quad$ N= stories above base

$T_a = 0.1(3)$

$T_a = 0.3 = T$

↳ Vertical Distribution Factor (C_{vx}) (Section 12.8.3)

$$C_{vx} = \frac{W_x h_x^k}{\sum_{i=1}^{n} W_i h_i^k}$$

$k = 1$ since $T \leq 0.5$

$W_x = 12.4\text{psf}$

$h_x = 26\text{ft}$

$$C_{vx} = \frac{(12.4\text{psf})(26\text{ft})}{(12.4\text{psf})(26\text{ft})}$$

↳ $C_{vx} = 1.0\text{ psf}$

↳ Lateral Seismic force (F_x) (Section 12.8.3)

$F_x = C_{vx} V$

$F_x = 1.0(9.92\text{psf})$

$F_x = 9.92\text{ psf}$

L

4. Seismic Load Effect (Section 12.4.2)

$E = E_h + E_v$

- Horizontal Seismic Load Effect (E_h) (Section 12.4.2.1)

$E_h = \rho Q_e$ $\rho = 1.0$ because SDC B

$E_h = 1.0 (9.92 psf)$ $Q_e = F_x$

$E_h = 9.92 psf$

- Vertical Seismic Load Effect (E_v) (Section 12.4.2.2)

$E_v = 0.2 S_{DS} D$

$E_v = 0.2 (0.192)(5.47 psf)$

$E_v = 0.210 psf$

$E = \underline{10.13 psf}$

Summary

D = 5.47psf + self weight = 55.47psf E = 10.13psf

S = 34.65 psf L = 0

W = 39.09psf R = 0

Building L = 10psf

Load Combinations

1. 1.4D =
2. 1.2D + 1.6L + 0.5(L_r or S or R)
3. 1.2D + 1.6 (L_r or S or R) + (L or 0.5W)
4. 1.2D + 1.0W + L + 0.5 (L_r or S or R)
5. 1.2D + 1.0E + L + 0.2S
6. 0.9D + 1.0W
7. 0.9D + 1.0E =

Load Combinations

1. $1{,}4D = 1{,}4(5{,}47\ psf) = \underline{7{,}66\ psf}$
2. $1{,}2D + 1{,}6L + 0{,}5(L_r/S/R) = 1{,}2(5{,}47) + 1{,}6(0) + 0{,}5(34{,}65) = \underline{39{,}89\ psf}$
3. $1{,}2D + 1{,}6(L_r/S/R) + (L/0{,}5W) = 1{,}2(5{,}47) + 1{,}6(34{,}65) + 0{,}5(39{,}09) = \underline{81{,}55\ psf}$
4. $1{,}2D + 1{,}0W + L + 0{,}5(L_r/S/R) = 1{,}2(5{,}47) + 1{,}0(39{,}09) + 0 + 0{,}5(34{,}65) = \underline{70{,}98\ psf}$
5. $1{,}2D + 1{,}0E_v + L + 0{,}2S = 1{,}2(5{,}47) + 1{,}0(0{,}21) + 10 + 0{,}2(34{,}65) = \underline{23{,}704}$
6. $0{,}9D + 1{,}0W = 0{,}9(5{,}47\ psf) + 1{,}0(39{,}09) = \underline{44{,}013\ psf}$
7. $0{,}9D + 1{,}0E_h = 0{,}9(5{,}47) + 1{,}0(9{,}92) = \underline{14{,}843\ psf}$

Governing load = 81,55 psf

Appendix D.2: Member Design Calculations

- Assumptions
  - Slab: • one way continous slab with inner support
    - Metal Deck = 3.5 psf
    - MEP = 5 psf
    - All slabs are the same except for the roof's
  - Column: • 12" x 12" (estimated by measuring) & #3 stirrups
    - Column placement & number of columns estimated as best as possible given the amount of information
    - Governed by axial forces only
  - Beam: • 8" x 10" (estimated by eye)
    - Layout estimated as best as possible
    - As needs to be similar to the slab's

- Research
  - $f'_c = 3$ ksi
  - A15 steel & A432 steel → $F_y = 60$ ksi
  - 3/4" cover for slabs

1

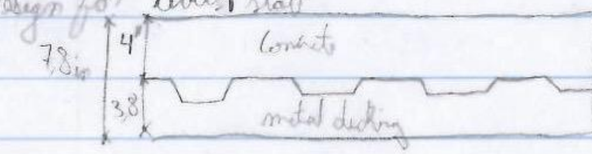
Slab on Roof (4 level): Design for level 4 slab

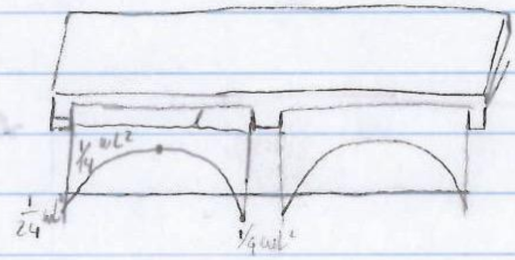
MEP = 5 psf ☆

Metal decking = 3,5 psf *

$l = 15{,}67$ ft *

Cover = 3/4"





1) Estimate h

$$h \geq L/24 = \frac{15{,}67\,ft\,(12\,in/ft)}{24} = 7{,}8\,in$$

2) Factored Load

$$\text{Self weight} = 7{,}8\,in\left(\frac{1\,ft}{12\,in}\right)(150\,lb/ft^3) = 97{,}5\,psf$$

$$W_U = 1{,}2D + 1{,}6S + 0{,}5W$$

$$W_U = 1{,}2(5{,}47 + 5 + 3{,}5 + 97{,}5) + 1{,}6(34{,}65) + 0{,}5(39{,}09) = 208{,}75\,psf$$

3) $M_{U\,max}$

$$M_U = \frac{W_U L^2}{9} = \frac{208{,}75\,psf\,(15{,}67\,ft)^2}{9} = 5{,}69\ K\cdot ft$$

4) ρ_{max} & $\rho_{0{,}005}$

$$\cdot\ \rho_{max} = 0{,}75\rho_b = 0{,}75\left[0{,}85\beta_1 \frac{f'c}{Fy}\left(\frac{87}{87+Fy}\right)\right] = 0{,}75\left[0{,}85(0{,}85)\left(\frac{3}{60}\right)\left(\frac{87}{87+60}\right)\right] = \underline{0{,}016}$$

$$\cdot\ \rho_{0{,}005} = 0{,}85\beta_1 \frac{f'c}{Fy}\frac{\varepsilon_u}{\varepsilon_u + 0{,}005} = 0{,}0136$$

$\rho = \frac{A_s}{bd} = 0{,}005$

5) d @ M_U assume $\rho = 0{,}0136$

$$M_U = \phi\rho F_y b d^2 (1 - 0{,}59\rho\, F_y/f'c)$$

$$d^2 \geq \frac{5{,}69\,K\cdot ft\,(12\,in/ft)}{0{,}9(0{,}0136)(60)(12\,in)(1 - 0{,}59(0{,}0136)(60/3))} = 9{,}23$$

$$d_{avg} \geq 3{,}04\,in$$

$\cdot d_{des} = h - cover - 0.25$

$= 7.8'' - 0.75'' - 0.25$

$= 6.8$

$d_{des} \geq d_{reg}$ (ρ not efficient)

6) A_s per unit width

@ interior support

assume $a = 1$

$$A_s = \frac{M_u}{\phi F_y (d - \frac{a}{2})} = \frac{5.69\,(12)}{0.9(60)(6.8 - \frac{1}{2})} = \boxed{0.201\ in^2}$$

$$a = \frac{A_s F_y}{0.85 f'_c b} = \frac{0.201\,(60)}{0.85(3)(12)} = 0.39\ in^2$$

@ Midspan (Satisfactory to use same "a" for other locations)

$$M_u = \frac{1}{14} W_u L^2 = \frac{208.75\,(15.67)^2}{14} = 3.66\ K\text{-}ft$$

$$A_s = \frac{3.66\,(12)}{0.9(60)(6.8 - \frac{0.39}{2})} = \boxed{0.12\ in^2}$$

@ Exterior

$$M_u = \frac{1}{24} W_u L^2 = \frac{208.75(15.67)^2}{24} = 2.14\ K\text{-}ft$$

$$A_s = \frac{2.14 \cdot 12}{0.9(60)(6.8 - \frac{0.39}{2})} = \boxed{0.072\ in^2}$$

7) $\rho > \rho_{min} = 0.0018$

$$\rho_{min} = 0.0018 = \frac{A_s}{bh} \rightarrow A_{s,min} = 0.0018\,(12)(7.8) = \boxed{0.17\ in^2}$$

8) Shear

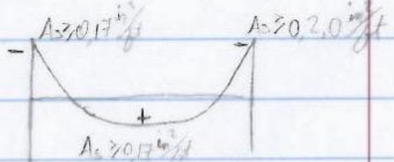
$$V_u = 1.15\,\frac{w_u L}{2} - w_u\,d = 1.15\left(\frac{0.20875 \cdot 15.67}{2}\right) - 0.20875\left(\frac{6.8}{12}\right)$$

$V_u = 1{,}76$ Kips

$$\phi V_c = \phi\, 2\sqrt{f'c}\, b\, d = 0{,}75(2)\left(\frac{\sqrt{3000}}{1000}\right)(12)(6{,}8) = 6{,}7 \text{ Kips}$$

$$\phi V_c > V_u$$

9)



* Bar #3 spaced every 6,5"; $A_s = 0{,}20$ in²

10) New d

$d_{dv} = h -$ cover $-$ half bar diameter

$= 7{,}8 - 0{,}75 - 0{,}375/2$

$d_{dv} = 6{,}86''$ * Doesn't change

11) actual ρ

$$\rho = \frac{A_s}{bd} = \frac{0{,}20}{12(6{,}86)} = \boxed{0{,}0024}$$

$$\rho_{max} > \rho > \rho_{min}$$

12) ϕM_n check

$$a = \frac{A_s F_y}{0.85 f'_c b} = \frac{0.20(60)}{0.85(3)(12)} = 0.39''$$

$$\phi M_n = \phi A_s F_y \left(d - \frac{a}{2}\right) = 0.9(60)(0.20)\left(6.86 - \frac{0.39}{2}\right) = 71.9 \text{ in-kips}$$

$\phi M_n = 6.0$ k-ft $\geq M_u$ ✓

Slab on 1st floor : Design for levels 1 - 3 slab

MEP = 5psf

Metal Deck = 3.5psf

$l = 15.67'$

$4^{th} - 2^{nd}$ level slab = 318psf

$3^{rd} - 1^{st}$ level columns = 27.09psf

$h = 7.8"$ $d = 6.8"$

$3^{rd} - 1^{st}$ level beams = 40.8

1) Factored load

Self weight = 106 psf * including MEP & metal Deck

$W_u = 1.2D + 1.6S + 0.5W$

$W_u = 1.2(106 + 5.47 + 318psf + 27.09 + 40.8) + 1.6(34.65) + \frac{1}{2}(39.09)$

$W_u = 671.82psf$

2) M_u:

@ Interior support : $M_u^- = -\frac{1}{9}(671.82)(15.67^2) = 18.33$ k·ft

@ Midspan : $M_u = \frac{1}{14}(671.2)(15.67^2) = 11.82$ k·ft

@ Exterior : $M_u^- = -\frac{1}{24}(671.2)(15.67^2) = 6.97$ k·ft

3) ρ_{max} & $\rho_{.005}$

- $\rho_{max} = 0.75\rho_b = 0.75\left(0.85\beta_1\frac{f'c}{Fy}\left(\frac{87}{87+Fy}\right)\right) = 0.016$
- $\rho_{.005} = 0.85\beta_1\left(\frac{f'c}{Fy}\right)\frac{\varepsilon_u}{\varepsilon_u + 0.005} = 0.0136$

4) $d_{req} \geq d_{des}$

$$d_{req}^2 \geq \frac{M_u}{\phi\rho_{.005}F_y b\left(1 - 0.59\rho_{.005}\frac{F_y}{f'_c}\right)}$$

$d_{req} \geq 5.5"$

$\therefore d > d_{req}$ ✓

5) A_s per unit width

@ interior ; $a = 1$

$$A_s = \frac{18.33(12)}{0.9(60)(6.8 - \frac{1}{2})} = 0.64 \text{ in}^2$$

$$a = \frac{0.64(60)}{0.85(3)(12)} = 1.27$$

@ Midspan

$$A_s = \frac{11.8(12)}{0.9(60)(6.8 - \frac{1.3}{2})} = 0.38 \text{ in}^2$$

@ Exterior

$$A_s = \frac{6.97(12)}{0.9(60)(6.8 - \frac{1.3}{2})} = 0.25 \text{ in}^2$$

6) $\rho > \rho_{min} = 0.0018$

$$A_{s\,min} = 0.0018\, b\, h = 0.17 \text{ in}^2$$

7) Shear

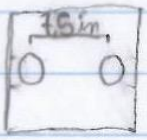
$$V_u = 1.15\left(\tfrac{1}{2}\right) W_u l - d W_u = 1.15\left(\tfrac{1}{2}\right)(0.67182)(15.67) - 6.8\left(\tfrac{1}{12}\right)(0.67182)$$

$V_u = 5.67$ K

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

$\phi V_c = 6.7$ K

$\therefore \phi V_c > V_u$

8) 

#6 bar spaced every 7.5 inches
$A_s = 0.7 \ in^2$

9) Actual d

$d = h - cover - half \ bar \ diameter$
$d = 7.8 - 0.75 - \frac{0.75}{2}$
$d = 6.67''$

10) actual ρ

$$\rho = \frac{A_s}{bd} = \frac{0.7 \ in^2}{12 in (6.67)} = \underline{0.009}$$

$$\rho_{max} > \rho > \rho_{min}$$

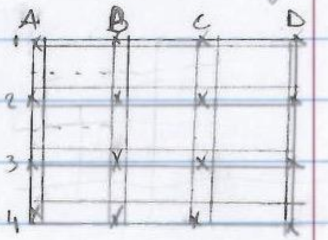
11) ϕM_n check

$$a = \frac{A_s F_y}{0.85 f'_c b} = \frac{0.76 (60)}{0.85(3)(12)} = 1.37''$$

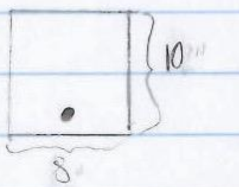
$$\phi M_n = \phi A_s F_y \left(d - \frac{a}{2}\right) = 0.9(0.76)(60)\left(6.67 - \frac{1.37}{2}\right) = 226.2 \ kip \cdot in$$
$\phi M_n = 18.9 \ kips \cdot ft \geq M_u$

7

Beams 1 floor



X = columns



Length = 15,67 ft
Cover = 1,5"

Self weight = $0{,}56\,ft^2 (15{,}67\,ft)(150\,lb/ft^3) = 1305{,}83\,lb$

Average floor self load $= \dfrac{(1305{,}83\,lb)24}{(48\,ft)(48\,ft)} = 13{,}6\,psF$

1) Loads actin at each beam

Slabs = 3(106 psf)
Columns = 2(9,03 psf)
Beams = 3(13,6 psF)
Evacuated tubes = 1(5,47 psF)

$W_u = 1{,}2D + 1{,}6S + 0{,}5W = 1{,}2\left[5{,}47 + 3(106psf) + 2(9{,}03) + 3(13{,}6)\right] + 1{,}6(34{,}65) + 0{,}5(39{,}09)$

$W_u = 382{,}33\ psF\ (10/12) = 318{,}61\ lb/ft$

2) Assumed A_s = A_s from slab

$A_s = 0{,}7$

1 #8 bar

$A_s = 0{,}79$

3) $d = h -$ cover $-$ half bar diameter $-$
$= 10 - 1.5 - 1.0/2 = 8''$

4) a

* $F_s = F_y$ ($\rho \leq \rho_b$)

$$a = \frac{A_s F_y}{0.85 f'_c b} = \frac{0.79\ (60)}{0.85(3)(8)} = 2.3''$$

5) ϵ_t Net tensile strain

$$c = a/\beta_1 = 2.3/0.85 = 2.71$$

$$\epsilon_t = \left(\frac{d - c}{c}\right)0.003 = \left(\frac{8 - 2.3}{2.3}\right)0.003 = 0.0074$$ ~ net tensile strain

$$\epsilon_y = \frac{60000}{29000000} = 0.00207$$ ~ yield strain in tension

$\epsilon_t > \epsilon_y$ then $F_s = F_y$
$\phi = 0.9$

6) allowed Moment

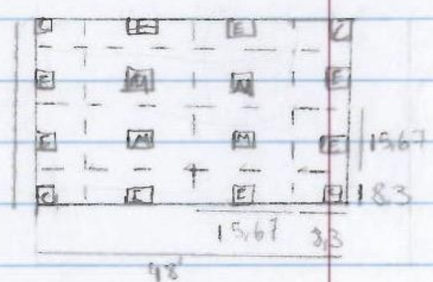
$$\phi M_n = \phi A_s F_y \left(d - \frac{a}{2}\right)$$
$$= 0.9(0.79 in^2)(60 ksi)\left(8'' - \frac{2.3}{2}\right)$$
$$= 292.22\ k \cdot in \approx 24.3\ kips/ft > M_u$$

7) $M_u = \frac{W_u L^2}{8} = \frac{318.61\ (15.67)^2}{8} = 9.78\ k \cdot ft$

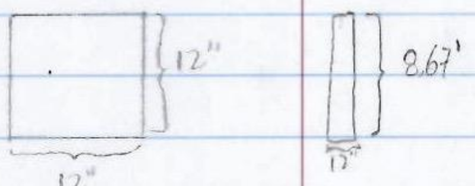
8) $\rho = \frac{A_s}{bd} = \frac{0.79\ in^2}{8''(8'')} = 0.0123$

9

Columns 1st floor



4 Middle columns
8 Edge columns
4 Corner Columns



* gravity column how much As

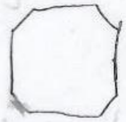
Self weight $= (1\,ft^2)(8{,}67\,ft)(150\,lb/ft^3) = 1300{,}5\,lb$

Average floor self load $= \dfrac{(1300{,}5\,lb)\,16}{(48\,ft)(48\,ft)} = 9{,}03\,psf$

1) Loads acting each column = # of × self weight
Slabs = 3(106psf)   Beams = 3(13,6)
Columns = 3(9,03psf)
Evacuated tubes = 1(5,47psf)

$W_u = 1{,}2D + 1{,}6S + 0{,}5W = 1{,}2(5{,}47 + 3(106) + (3)(9{,}03) + 3(13{,}6)) + 1{,}6(34{,}65) + 0{,}5(39{,}09)$
$W_u = 544\ 2{,}621\,psf$

- @ Middle : $P_u =$ (tributary area) $W_u = (15{,}67\,ft)^2\ 502{,}221\,psf = 133{,}7$ Kips
- @ Edge : $P_u = (15{,}67\,ft)(8{,}3\,ft)(544{,}621\,psf) = 70{,}83$ Kips
- @ Corner : $P_u = (8{,}3\,ft)^2 (544{,}621\,psf) = 37{,}52$ Kips



2) A_{st}

$P_U = \phi P_{n\ max} = 0{,}80\ \phi \left[A_{st} f_y + 0{,}85\ f'_c (A_g - A_{st}) \right]$

$133{,}7\ kips = 0{,}80(0{,}65) \left[A_{st} \cdot 60 + 0{,}85(3)(144 - A_{st}) \right]$

$133{,}7 = 0{,}52 \left[60 A_{st} + 367{,}2 - 2{,}55 A_{st} \right] = 29{,}87 A_{st} + 190{,}94$

$A_{st} = \dfrac{-57{,}24}{29{,}87} = -1{,}92\ in^2$

*Negative sign means column is too big

Use 4#7 bars A_{st} 2,40 in²
or 2#9 bars A_{st} = 2,0 in²

3) Ratio

$A_s / A_g = 2{,}40/144 = 0{,}0167$

$A_s / A_g \quad 2{,}0/144 = 0{,}014$

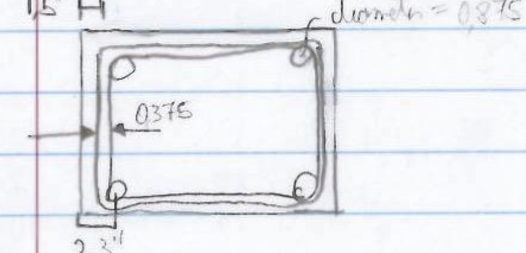
$P_{min} = 1\text{-}2\%$

4) Ties

#3 ties

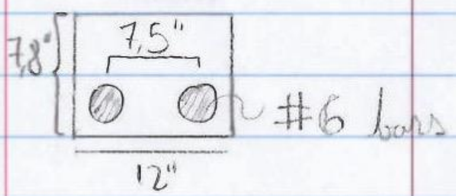
spacing < off :
- 16 (#7 bar diameter) = 16 × 0,875 = 14"
- 48 (tie diameter) = 48 (0,375) = 18"
- Least dimension of column = 12"

*Summary : 12" × 12" columns with Fy = 60ksi & F'c = 30ksi with 4#7 bars (two in each face) & ties #3 spaced 12" apart. alternative, 2#9 bars could also be used

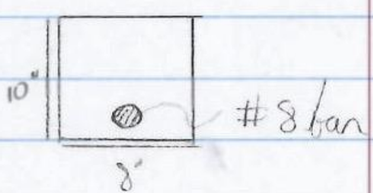


11

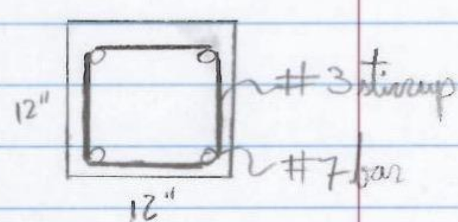
Final Design crossection



• 1st floor slab



• 1st floor beam



• 1st floor column

Appendix D.3: Economic Analysis

Economic feasibility

1) Fixed cost = Price of tech + Installation Cost

$= (3080)31 + 1000 + 70(80)$

$= 102080$ \$

2) Savings per year

Current spending $= 35592$ \$ per year

tech saving $= 613.20(31) = 19009,2$

Future $= 35592 - 19009,2 + 50 = 16632,8$

Saving per year $= 35592 - 16632,8 = 18959,2$

3) Payback

$X = \dfrac{\text{Fixed cost}}{\text{Savings per year}} = \dfrac{102080}{18959,2} = 5,38 \approx 5,4$ years