



Worcester State University Sheehan Hall:
Project Management and Alternative Floor System
Design

A Major Qualifying Project

Submitted to the faculty of
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Abstract

This project proposes an alternative design for the floor system of Worcester State University's Sheehan Hall residence dormitory and compares it to the existing design in terms of scheduling and costs. It also reviews on-site project management practice including scheduling, cost and lean construction. Building Information Modeling is used to visualize the impacts of the alternative design and to create a 5D model of the building structure for the comparison between the planned and actual cost and schedule.

Capstone Design Statement

Sheehan Hall will be a state-of-the-art dormitory building on the campus of Worcester State University when it is completed in the summer of 2014. Our Major Qualifying Project consisted of reviewing the existing floor system design consisting of pre-fabricated concrete planks and exploring an alternative floor system design for the new dormitory consisting of reinforced cast-in-place concrete slabs on metal decking. This study also explores the effects of the alternative design components including schedule and cost. Building Information Modeling (BIM), Autodesk *Revit*, Autodesk *Navisworks* and *Primavera* scheduling software were used to determine design and constructability analysis processes.

The following constraints were addressed during the completion of this project: economic, environmental, health and safety, social and constructability.

The economic impact of the alternative floor system design is the first constraint. A structural analysis was performed, along with a cost and construction schedule impact analysis, to determine the most effective floor system that can support the required loads of the facility. This was completed with aid of the following software applications: *Primavera*, Autodesk *Revit*, and Autodesk *Navisworks*.

The environmental constraint was met through exploring how the existing property was prepared for construction. This included excavation, grading and site drainage.

The health and safety constraint was met by determining that the alternative design sustained the required loads of the facility. It was ensured that the designs met the appropriate provisions of the *Massachusetts State Building Code* with *ASCE 7* to complete the structural designs.

The next constraint explored was the social constraint. This constraint was addressed throughout the duration of our project because the dormitory will serve as housing to students of the Worcester State University.

The last constraint is the constructability of the alternative design. This constraint was met by researching the structural design of the alternative floor system. Design constructability was the principal consideration in proposing the alternative design, which affects the cost and schedule of the project.

Authorship

The following list indicates the primary areas of focus in the report for each team member:

Myo Latt – Primavera Scheduling, 5D Navisworks Model, Schedule Evaluations

Thomas Lacroix – Alternative Design Revit Model, Structural Calculations, Site Logistics

Matthew Blakeman – Cost Evaluations, Cost Calculations, Project Management Evaluation

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1.0 Introduction

Educational institutions all over the world are drawing in more and more students each year, partly owing to the fact that an increasing number of people are realizing the value and importance of higher education nowadays (*Admission Statistics*, 2013). This may be beneficial for universities and colleges in the sense that they are educating an increasing number of the population while generating greater revenue and growing in size, but an increasing student population also demands more on-campus facilities such as dormitories, cafeterias, etc. Many universities and colleges have very limited on-campus accommodation, meaning that a large number of students must live elsewhere and commute to campus, which is not ideal. Due to this increasing demand for construction within the education sector, the construction industry is witnessing a growing number of projects for educational buildings (*Construction Market Research*, 2013).

Such is the case of Worcester State University (WSU). Located in a residential neighborhood on the west of Worcester, MA, WSU is a commuter-heavy university that is currently facing the same problem of not being able to provide enough housing for its current student population. In an effort to address this problem and keep more students on campus, WSU is currently constructing a new facility, namely Sheehan Hall (Kotsopoulos, 2012). It is imperative that this new facility is completed on time and within budget because it needs to be ready for move-in by fall 2014. When completed, Sheehan Hall will rise six stories beside the football field and house 400 beds. In addition, the facility will also feature amenities such as a cafeteria capable of seating 575 people, a large community room, and offices for the residential and health services. The total budget for the design and construction of the project is \$60 million.

This study is based on the observation and analysis of the project during construction and is focused on exploring the impacts of an alternative floor system design on the total project duration and cost. It also includes a thorough analysis and evaluation of the construction management practice, in which the planned schedule and costs are compared with the actual construction schedule and costs.

The current structural design of the facility is comprised of a steel frame, with cast-in-place concrete slabs for the first floor, and pre-cast concrete slabs for floors two through six. This study proposes an alternative floor system design, in which the pre-cast slabs on floors two through six are entirely replaced with cast-in-place reinforced concrete slabs. The benefits of pre-cast slabs are that they could potentially speed up the construction process, eliminate the hassle and coordination involved with pouring concrete on-site, and since they are manufactured in a controlled environment, their quality is strictly monitored (Consigli, 2013). Cast-in-place concrete slabs, on the other hand, do not require equipment such as cranes to be installed, they can be poured to the exact required dimensions on-site, and do not cost as much as pre-cast slabs. By changing the existing pre-cast slabs to cast-in-place, the study examines the effects on the cost and schedule, and determines which method will be more beneficial for the project. The alternative design is first visualized through a 3D model, which is created using *Autodesk Revit* software, based on manual structural calculations. The impacts that this new design may have on the project are then analyzed in terms of cost and time by preparing a cost estimate and a schedule of activities, using *Primavera* scheduling software. This schedule and cost data are then incorporated with the 3D structural model using *Autodesk Navisworks* software to create a 5D Building Information Model (BIM). The BIM serves as a complete visual tool of the project and

aids in better understanding the alternative design, including its time and cost implications on the project.

The study also consists of observation and analysis of the overall project management process for the actual construction phase of the project, which entails evaluations of the relationships between different parties involved in the project, cost and schedule, safety practices, and the use of lean construction. However, for the intent of this study, the evaluations are limited to the site work, foundations, structural framing, and floors of the building only. A visual comparison of the baseline cost and schedule to the actual cost and schedule is presented in the form of a 5D BIM model. The 5D model is created through the integration of the *Primavera* schedules with the *Revit* model in *Navisworks*. Lastly, the study involves a site work review section, which provides a description of the existing layout and pre-construction site work.

2.0 Background

This chapter discusses the planning and need of a new dormitory on the campus of Worcester State. The section starts with an overview of the project as well as some information about Worcester State. Construction project management practices such as cost estimating and scheduling. The use of Building Information Modeling (BIM) for the construction of Sheehan Hall is reviewed. Structural analyses are discussed along with site implications.

2.1 Worcester State's Plan

More students are attending colleges now more than ever. From 2000 to 2010 there has been an increase in enrollment in degree-granting institutions by 37% (Worcester State University 2013, August 1). Worcester State University (WSU) has been planning on adding more on-campus housing for their students to address this increase in students and students who live on campus. Sheehan Hall, the new dormitory on the campus will meet this need for the college. In the Worcester State University Master Plan from 2007 it was estimated that 700 new beds would be needed by 2014 (Sieniewicz, C. K, 2007). Sheehan Hall helps the university meet the needs of a growing student population. The college has many commuter students and the addition of this residence hall will help the process to have more students that stay and live on campus. Worcester State's President Maloney stated that "When Sheehan Hall is completed in August 2014, two out of every five of our students will be housed here on campus-and we know that residential students will both add vitality to our campus community and positively affect our retention and completion rates" (Reis, J, 2012). "Phase 3: beyond the framework horizon" section of the Campus Framework Plan states that a new residence hall would be implemented on the hillside of the sports field, six years later that plan was put into place. With this new residence hall the opportunity presented itself to enhance the "main street" of the campus

(Sieniewicz, C. K, 2007). The college campus lacks a clean pedestrian path or circulation pattern but this new building will add to the circulation pattern. The reason that the college wants a more prominent pedestrian path is to try to connect all campus buildings in one path, and this hall will fit into that path. **Figure 1** displays where the new residence hall will be located on the campus.

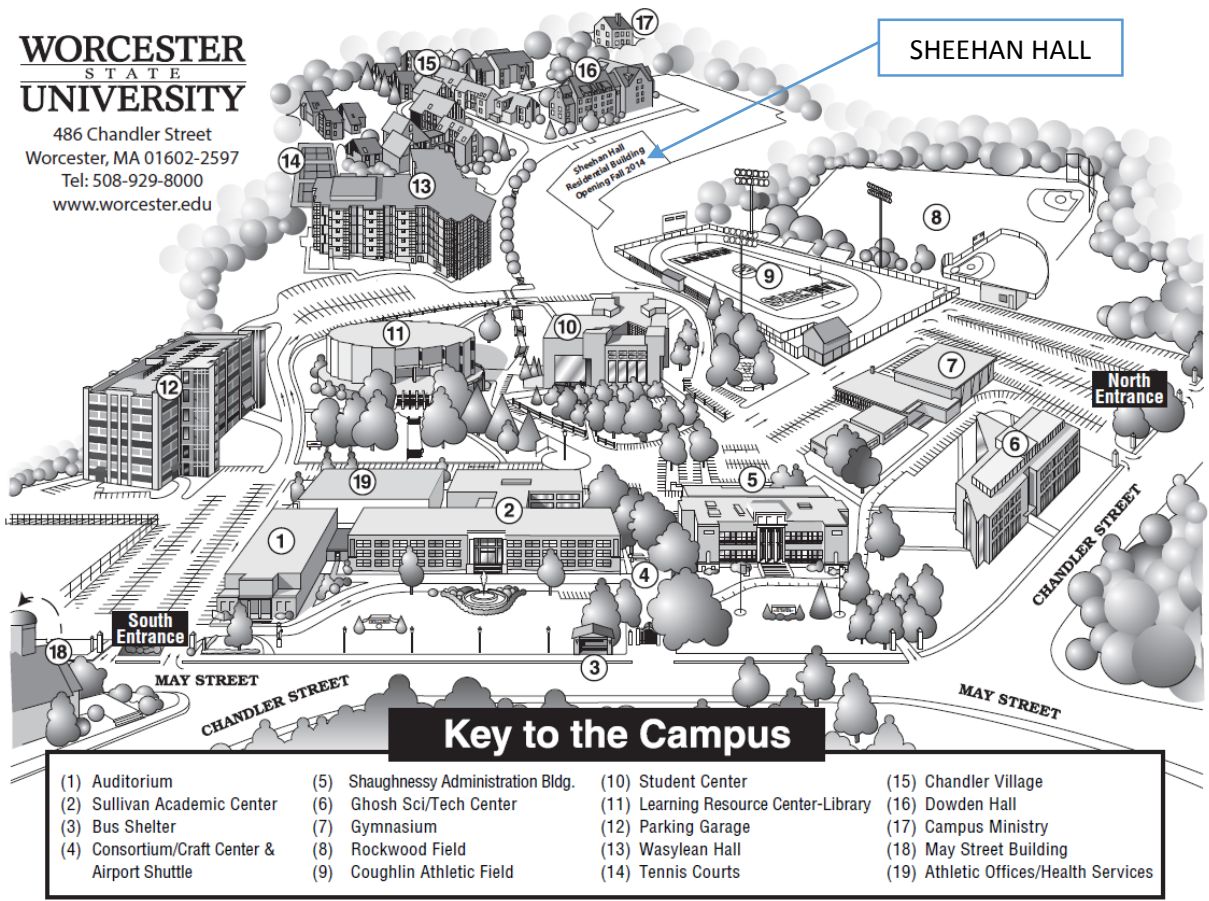


Figure 1: Campus Map of Worcester State University

2.2 Sheehan Hall

Worcester State University's new residence hall construction officially began in March of 2013, and has an expected completion date slated for the fall 2014. The new facility is designed to accommodate 400 students and also includes features such as a large community room, a dining hall with two-story windows capable of seating 575 students, faculty and staff, as well as additional outdoor seating overlooking the John F. Coughlin Field. This new residence hall will add approximately 10 percent to the University's on-campus housing capacity. Sheehan Hall will be named after Lt. Col. James F. Sheehan USMC (ret.) who graduated from the college in 1955. Over the years Lt. Sheehan has provided \$3.6 million in support for the college. Lt. Sheehan's support has gone towards scholarships, academic excellence and international study support. Massachusetts Higher Education Commissioner Dr. Richard M. Freeland stated that the support from Sheehan and the naming of the building was "truly a magnificent achievement for Worcester State and ... as a testament to his loyalty and gratitude towards the college" (Herrin, C 2013). Sheehan Hall will now become the fourth residential complex among those currently part of campus such as Wasylean and Dowden Halls, and the Chandler Village. Positioned on the hillside above the Coughlin Athletic Field, the new residential facility will serve as a clear anchor to the residential area of the campus, offering a panoramic view of the university grounds as well as creating a pedestrian core that integrates all residential life on campus.

Sheehan Hall received an allocation of a budget of \$60 million for design and construction, the bulk of which is financed through the Massachusetts State College Building Authority (MSCBA). The MSCBA is responsible for the financing, designing, constructing and also the management of all revenue-funded projects including housing, dining, athletics, parking and other student recreational facilities with the goal to support the academic mission of the nine

Massachusetts state universities. The Authority receives no appropriation from the Commonwealth. All revenues to support facility design, construction and operation are derived from the rents and fees paid by students for the use of these facilities and services (MSCBA, 2013).

2.3 Construction Project Management (CPM) Overview

Construction Project Management (CPM) is the art of directing and coordinating human and material resources throughout the life of a project by using modern management techniques to achieve predetermined objectives of scope, cost, time, quality and participation satisfaction (What is Construction Project Management, 2014). There are many different components that are critical to completing the project on time and within budget. The CPM overview section explains the main components of the CPM methods that were used for this project. This section includes the contract type that was used for this project, the organization breakdown structure of the people and companies that are working on this project, the CPM practices that were used for cost estimating and scheduling, how Building Information Modeling (BIM) was used in project management and how the concept of Lean Construction and how it was used in this project.

2.3.1 Organizational Breakdown Structure (OBS)

Construction Management at Risk is the contract type used for this project. Under this contract type the Owner hires a design firm to design the project for the owner. Firms that offer construction management (CM) services then bid on the project before the construction drawings are complete. The owner then chooses the best CM contractor to complete the project based on variables such as bid price, projected schedule, and contractor qualifications. The work is being done for Worcester State University, which is a state school and becomes the end user, the owner

is the Massachusetts State College Building Authority (MSCBA). The MSCBA finances, helps design and oversees construction and operation of the residence halls and student activity facilities on the nine State University campuses in Massachusetts (MSCBA, 2013). The Authority uses all revenues derived from the rental and fees of these buildings to the students to support facility design, construction, and operation (MSCBA, 2013). The MSCBA chose Goody Clancy and Associates from Boston, MA as the architectural firm for the design of this project. The CM firm that was chosen for this project is Consigli Construction Co. based out of Milford, MA. Consigli is a Construction Manager and General Contractor that also has offices in Williamstown, MA, Portland, ME and Hartford, CT and Boston, MA as well as having affiliates in NY. Once Consigli was awarded the project they began hiring the subcontractors for the job. There are also many engineering design consultants hired by the MSCBA who are involved with many different trades on the project. A list of all of these consultants can be seen in **Appendix B**.

Figure 2 displays the organization breakdown for this project.

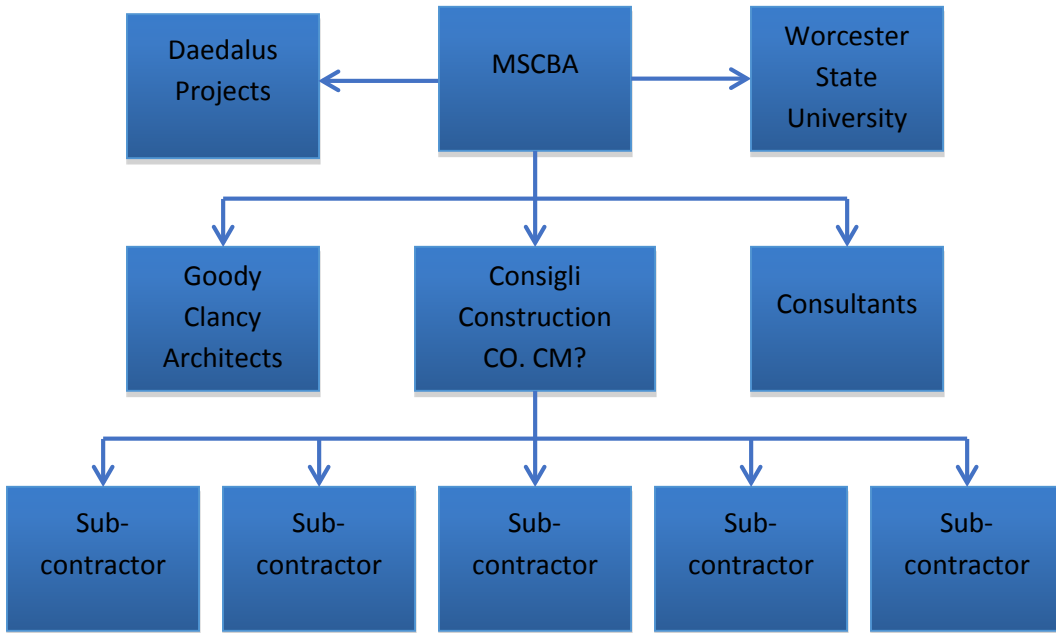


Figure 2: Organizational Breakdown Structure of Sheehan Hall

2.3.2 CPM Contract

The project delivery system for this project is Construction Management (CM) at risk with a Guaranteed Maximum Price (GMP) as the contract type. A GMP is the maximum possible cost to the owner for total construction of the project, however it is a cost reimbursable contract so that if the cost to complete the project is under the capped GMP amount the owner gets back the remaining amount of money not spent. The main difference between a GMP and a Lump Sum contract type (Lump Sum is the other typical contract type) is the CM's contingency. The contingency is a portion of money in the contract that is used for unforeseen changes that occur to the project due to lack of scope, incomplete drawings or specifications or to cover unforeseen costs to a project. If a change has to be made to the project that is not specified through the scope of work then money from the contingency can be used for this change and it will not change the overall cost of the project. It is called a Guaranteed Max Price because of the contingency aspect so the max price does not change. However the GMP can be subjected to change if the owner or the Architect/Engineer makes a change to the scope of work. The CM is at risk in this contract because after the money from the contingency is used the CM has to pay for unexpected costs that come up on a project, other than owner approved scope changes. The MSCBA likes using this contract type because they receive the remaining amount of contingency back once the project is done if the cost does not exceed the GMP (Consigli, 2013). The initial GMP bid for this project was \$50,262,375 (Consigli, 2013). This cost to complete bid will change through the project based on changes and unforeseen expenditures.

2.3.3 Scheduling

Scheduling is one of the most important tasks involved in construction project management. A carefully planned and well-defined schedule, endorsed by all parties involved, is a necessary component of any project in order to ensure that the project gets completed within the specified time and cost estimate. Construction projects involve a myriad of activities that need to be completed by many different subcontractors and professional teams in order to properly finish the project. A well-coordinated schedule not only helps in determining all the activities in the project as well as the sequence in which the activities are to be performed, but it is also necessary for identifying the critical activities of the project that will determine the overall project duration, as well as the order and timing in which each subcontractor is expected to start and complete their tasks. A schedule can also be used to gauge the progress of the entire project by comparing the activities planned on the schedule with the activities that have been completed. If an activity falls behind schedule and could potentially delay the completion of the project, it is the job of the project management team to manipulate the schedule, reallocating resources and task sequencing in order to finish on time. In the case of Sheehan Hall, finishing on time is essential because WSU needs to have the building ready for move-in by fall 2014.

The design of the Sheehan Hall project began in November of 2012, and the entire project is expected for completion in July of 2014, with a total project duration of 20 months (Consigli, 2013). The project is on a fast-track schedule, meaning that the design and construction phases are overlapped in order to compress the total duration of the project. For example, the construction can begin as soon as the structural design is complete, while the rest of the details and designs can be finalized as the project moves along. This enables the project management to significantly expedite the construction process since they don't have to wait for

the complete design to commence construction. A fast-track schedule saves time but it demands greater coordination and communication between the designers and the project management team.

In any schedule, it is important to identify the critical activities whose completion is absolutely necessary in order for the project to be finished on time. The Critical Path Method (CPM) is commonly used in construction schedules to identify the tasks that are critical to the project, and based off these tasks, the total project duration. In the CPM, all activities that have a total float of zero are considered critical while activities whose total floats are greater than zero are considered non-critical. The path with the longest total duration along these critical activities is known as the critical path and the duration of the critical path determines the duration of the entire project. Total float is the leeway between the earliest date at which an activity can start and the latest date it can start without resulting in a delay for the entire project (Halpin & Senior, 2011). Therefore, delaying a critical activity (zero total float) will result in the total duration being extended as well. On the other hand, non-critical activities (total float greater than zero) can be delayed by up to a number of days equal to their total float without impacting the total duration of the project. The CPM is a very useful tool for the project management team in planning and controlling a project from start to finish: critical activities indicate which tasks require continuous and immediate attention and resources. Shortening the duration of the critical path can shorten the total duration of the entire project.

The larger the project, the greater the number of activities involved in the schedule of the project. Large construction projects involve tens of thousands of individual activities and scheduling all these activities can be very complex and time-consuming. For this reason, various computer software exist that make scheduling a project fast, simple, and manageable. Programs

such as *Primavera Project Manager* are very capable of organizing and performing calculations on many information, and can handle various tasks, from planning and generating a simple timeline for all the activities of a project, to evaluating entire projects and portfolios (Primavera Works, 2013). *Primavera* is widely used by many construction and contracting firms to create schedules for projects because the program is also capable of tracking many important aspects of a project such as costs, duration of individual activities, and the relationship between activities. It can even be used to manage risks, keep track of all the contracts, documents, and change orders pertaining to the project, and monitor Requests for Information (RFIs) and unresolved issues (Oracle, 2013). A part of the *Primavera* baseline schedule developed by Consigli for the Sheehan Hall project can be seen in **Figure 3**, with the list of activities on the left and a bar chart showing the activities in sequence on a timeline on the right (please refer to **Appendix D** for a complete display of the baseline schedule that contains all the project activities).

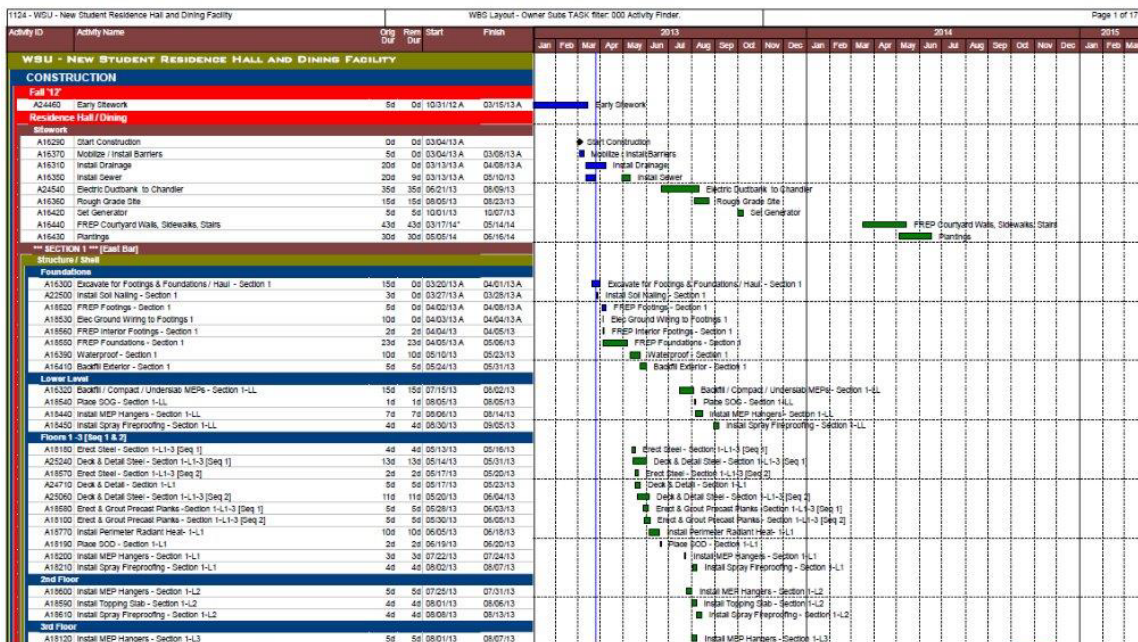


Figure 3: Consigli's Baseline Schedule

2.3.4 Building Information Modeling in Project Management

Building Information Modeling (BIM) is an emerging computer-based approach in the construction industry that is being adopted by an increasing number of construction firms. BIM enables firms to virtually construct in a digital fashion, a structure or facility before the actual construction occurs, thus minimizing the chances for error and spatial clashes between building components that would likely occur during construction (Consigli, 2013). BIM is mainly based on a 3D model, to which large amounts of information and other models can be added as desired. The BIM of a construction project usually incorporates into a single model significant amount of information from different components of the project such as the architectural details, the structural design, the HVAC and MEP designs, as well as geotechnical information. Different parts of this complete model can then be exported into special application software, such as *Autodesk Robot*, to be structurally analyzed. It also allows to conduct a 3D-spatial verification to detect potential clashes between components so these can be identified and resolved, thus enabling the project management team to eliminate costly adjustments on site.

In addition to being capable of providing a complete 3D model of a facility, BIM can also incorporate other information such as the schedule of the project and the costs associated with the construction of the building into the same model. A BIM model with incorporated cost and schedule data is known as a 5D model. BIM models are great tools for project management because they enable the project management team to simulate the actual construction process and prepare cost estimates along different project phases (Autodesk, 2013).

BIM is a great way of communicating various aspects and objectives of a project with everyone involved, from the owner to the field workers, because it provides a visual model with integrated time and cost data. The complexity of these models enables information from all the

different trades of the project to be stored in a single file, from which data can be pulled as necessary and each individual component of the project can be analyzed. BIM has dramatically enhanced the capabilities of the construction industry with its versatility. It is becoming increasingly popular.

This study incorporates the use of BIM for two purposes; to compare the baseline schedule for the actual construction of the structure to the as-built schedule, and to help with the visualization of the alternative floor system design and its impacts on the schedule and cost. Hence, the 3D model created by the designer using *Autodesk Revit* software has been modified to include the foundations and the structural design only. **Figure 4** displays the complete BIM model that Consigli uses. Consigli's use of BIM in the Sheehan Hall project is much more comprehensive than just for visualization and comparison purposes. Their main uses are primarily for co-location, as a digital mock-up, and for modeling site logistics as well as costs. Co-location is the process of bringing together all the designers for each of the different building systems (MEP, HVAC, plumbing, etc.) in one room and making them design the systems jointly and cooperatively. This ensures that everyone's input is taken into consideration in the designs, thus eliminating chances for errors, omissions, and clashes on site.

Using BIM for digital mock-up purposes is highly advantageous for Consigli because they can virtually go through the entire construction process before actual construction begins on site. This is beneficial because creating a digital mock-up using BIM tools forces the project management team to take a more in-depth look and identify and resolve any issues in their design documents, schedules, and construction and shop drawings. A digital mock-up also helps with detecting possible spatial clashes, as well as ensuring a proper sequencing of construction activities. Consigli also uses BIM to model the site logistics of Sheehan Hall. A site logistics

model is excellent for making sure everyone on the project team understands how to use the site efficiently and effectively, as well as the layout of the site. It takes into consideration factors such as effective location and use of cranes as well as other equipment, accessible drop-off sites for material deliveries, temporary placement locations for steel, slabs, etc., and locations of garbage and waste disposals. Consigli also modeled the costs of the building into the architectural model during the design process so that if the client or designer decides to make a change to the model, the cost data will be automatically updated. This makes the evaluation of alternative designs easier, faster, and more effective (Consigli, 2013).

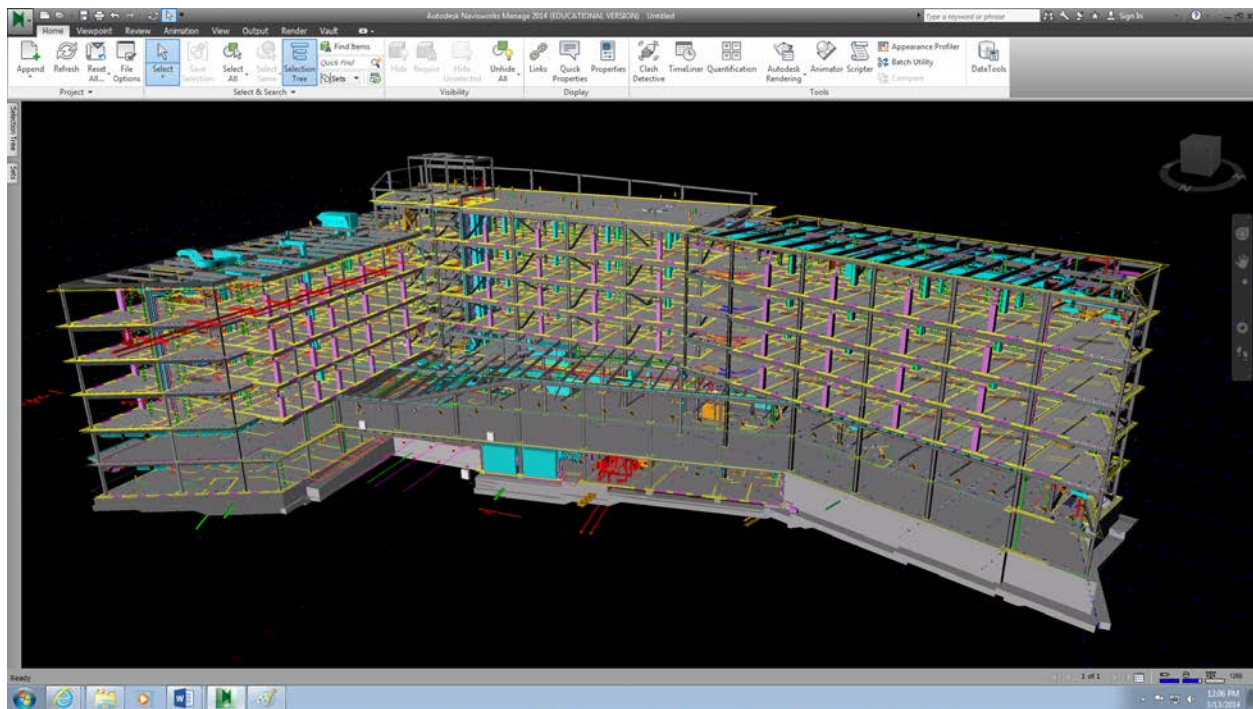


Figure 4: Consigli's Coordination BIM Model

2.3.5 Lean Construction

Lean Construction is an increasingly popular method for efficiently managing construction that is being employed by many construction firms nowadays (Consigli, 2013). In lean construction, a production management-based approach is used to help streamline the process of designing and building new facilities, in order to minimize the waste of materials, time and effort, and maximize value (Lean Construction Institute, 2013). Lean construction is especially useful for projects that are complex, uncertain and quick because the techniques used in lean construction call for enhanced collaboration among the different parties involved, reduced waste and redundancy, and improved efficiency and project outcome (Turner Construction, 2013).

Consigli also decided to adopt lean construction practices in the WSU New Dormitory and Cafeteria project in order to make the construction process more efficient and to tighten up the schedule (Consigli, 2013). In order to implement lean construction practices in a project, each work area is sub-divided into smaller sections, in which a single trade focuses on the work they need to complete before the next trade takes over the section. This method of dividing up the work areas into sections and having trades work in these smaller sections over a certain period of time creates a production-line type of effect and increases efficiency, as opposed to giving the work area to just a single trade at a time. This is true due to the fact that each trade is under the responsibility of completing their work properly and on time so that the next trade can move in and begin their work as scheduled with a minimum of wasted time. The added benefit of having multiple trades working simultaneously on different sections of a work area is that there is increased communication among the trades.

The practice of lean construction can also be applied to equipment and resources in order to ensure a better flow of work among the trades and to reduce costs; this is achieved through careful scheduling and allocation of the equipment and resources among the various trades involved in the project. It allows the project management to reduce the planning, coordination, and clutter that would otherwise be involved with moving the equipment frequently from place to place on site among different trades. There are many benefits to incorporating the principles of lean construction in a project. Lean construction achieves better efficiency in the use of materials, time, and effort by streamlining the traditional construction process and making it more like an assembly-line of a manufacturing plant. Consigli realized the benefits of lean construction in the Sheehan Hall project by dividing up each floor into multiple smaller work areas and having different trades work simultaneously on a floor as opposed to giving each trade a floor at a time. In order to ensure a better work flow in this kind of setup, they employed the use of a pre-deficiency log, which looks at potential problems six weeks in advance. Foremen are forced to look at shop drawings and identify problems beforehand, and trades are forced to better understand the scope of work as a result.

2.4 Structural Components Overview

This project as it pertains to the structural components of WSU's new dormitory building, Sheehan Hall, is based on proposing an alternative design for the current concrete floor system. The alternative design was developed to determine the impact of using a more traditional method in the design of the structural floor systems on the overall construction period as well as on the building's total cost of construction. To achieve this, an alternative to the current floor system's concrete method that uses a girder-slab system utilizing hollow-core precast planks with dissymmetric open-web steel beams. In our proposed alternative, our objective was to design a

cast-in-place concrete slab on metal deck supported by Vulcraft K-series open-web steel floor joists added for additional support on the girders. Our design features a non-composite acting reinforced concrete slab (as opposed to the precast composite-acting slab on d beams) as well as the addition of steel floor joists for additional support on the girders. The existing and proposed designs are compared both (precast planks versus composite slab) to identify the advantages and limitations of both systems in terms of the project's constructability and cost.

2.4.1 Precast Concrete vs. Cast-in-Place

Precast Concrete is a type of construction material that is typically used for both architectural and structural applications on a variety of buildings (PCI, 2013). This material is commonly used as the primary structural system for many high rise or multi story buildings because of its ability to transfer roof, floor, and lateral loads while also reducing the overall weight of the entire system (PCI, 2013). The use of precast hollow core planks allows for designers to integrate both the architectural and structural systems while reducing the total amount of materials, detailing, costs and also construction complexity (PCI, 2013). Precast is also valued for its high versatility, because it can serve many needs for the structure of a building and most importantly, in terms of its growing popularity, precast is more than just a very good building material because it can take almost any form and shape. Other beneficial traits for precast concrete is that there are different types of precast materials such as prestressed concrete- which is a type of structural member that is known for its exceptional load-carrying capacity. Due to having such high load-carrying capacity, this typically results in the use of smaller sections, longer spans, or even both when compared to other structural systems (ACP Co., 2013).

What makes this building material so advantageous to use during construction is its ability to be transported to a construction site where it can then be lifted and set into place all in

the same day. During the production of precast concrete, the controlled environment it is mixed in is typically referred to as a precast plant. At this plant, the production process is done on ground level, which has been proven to help with production safety (ACP Co., 2013). Also this provides a greater ability to control the quality of materials being added to the mixture while also affecting the workmanship in a precast plant versus being on a congested construction site (ACP Co., 2013). After the mixture has been poured and shaped, it begins the curing process where it is closely monitored to reduce the possibility of deformities from being created within the structure that would typically be caused by unnecessary exposure to inclement weather or other disturbances found on any construction site.

This type of concrete is widely being used for construction projects today because it offers numerous positive advantages during construction scheduling and also requires less coordination between the project manager and designers during construction, but most importantly the installation process; Furthermore, in terms of differentiating the differences between poorly structured projects versus smooth and exceptionally well run projects, a project that is managed properly and executed to satisfy both the expectations set by the owners and the demands set by the designers, directly correlates to a reduction in the probability of complications and set-backs from occurring. This idea is reinforced in the example of WSU's new residence hall "Sheehan hall" as it shows many of today's cutting-edge building, construction management and design techniques. Some of these cutting-edge techniques include the projects usage of LEAN construction, the structural design of a precast plank on dbeam girder-slab floor system and also with the project's establishment of a persistent coordination process between ownership, the designers and the projects managers. These techniques all contribute directly to a project's ability to achieve its full and expected potential (an accelerated

project schedule at reduced project cost, simplified installation and closely managed construction processes) when building any high-rise multi-story building.

While spectating the installation of the first level, the use of these hollow-cored planks allowed for them to just lift the material to its desired location and set them in place on the Dissymmetric beams all in the same day. An important installation technique that was used in this project was the way in which each of the floors were turned into a composite system. To establish composite action between the planks and D-beams a process called grouting was used. Grouting (“Grout” also known as super-strength concrete) is the process of filling the hollow cores with this high-strength concrete, and it was done by passing the grout through the open web of the D-beams and into the cores. As it cures, this will essentially connect the two materials together making it possible for the floor system to successfully transfer loads throughout each of the precast planks, to their supporting steel members, down through the system’s columns and into the buildings foundation and soil. This grouting technique uses similar steps as in the ordinary cast-in-place concrete, but in terms of this project, the girder-slab system design and its use of open-web D-beams with hollow core planks in combination with high strength grout are the premier contributing factors to a quick and efficient structural erection period; alternatively with the use of site-casted concrete (CIP), additional time is needed for the placement of steel decking, reinforcement and also concrete forming before the actual pouring of concrete can begin. This explains why this project limited the usage of CIP concrete to more effectively satisfy its strict schedule and meet critical deadlines.

Lastly, from more of a financial standpoint, the prep work needed for the use of precast concrete members is very small and consists of the following: the excavation (if needed and is typically done for foundations and footings) of soil for pre determinedly sized members to be

placed in, and the use of a boom lift or tower crane to lift the members off the delivery truck and lowered into place, like what was seen for WSU's Sheehan Hall and their use of prefabricated HC Planks.



Figure 5: Boom Lift



Figure 6: Tower Crane

Precast concrete can be used to expedite a significant portion of the construction process and listed below is a summarization of all the main points previously mentioned in this section (ACP Co., 2013):

- Made easily available by a variety of precast suppliers.
- Manufactured to accommodate almost every construction project need.
- Controlled environment it is made in, inclement weather is not a factor in the planning process, which will help to avoid any unnecessary delays due to unworkable conditions.

Cast-in-Place concrete (also known as ready-mix concrete) is brought onsite in its un-hardened liquid state where upon arrival it is poured into site-specific forms (typically “molds”) and cured on site. Concrete is typically mixed in a factory or batching plant (according to standard design-mix-proportions), and is then delivered to a site by a truck mounted in-transit mixers. The result from a precise batch provides the ability to create special concrete mixtures and with the convenience of making other alterations to the mix and implemented on a construction site to change properties like handling and strength.

Cast-in-place (also known as ready mix concrete) is the material of choice for slab-on-ground and foundations as well as on steel or metal decking because of the material’s long-term durability as well as its structural support.

CIP concrete can serve many needs for a variety of different types of buildings, some of the common many applications of CIP consist of beams, columns, floors, walls and roofs. Additionally, widely used building material has been shown to have environmental attributes during construction and have also been known for being present during the structure’s life span. These environmental benefits during construction are as follows:

- There is very little wasting of material due to the specific state that the material is in during construction applications, it can really only be used and placed on an as-needed basis. This material can't be left around on-site as it will begin to harden unless continuously stirred or mixed.
- Additionally, this material is very easily recycled and used for the creation of other structures like jersey barriers or retaining wall blocks (Mineral Industry, 2011).

Some projects actually prefer the use of cast-in-place concrete instead of precast members because of the precision of the mixture and also due to its reduced worksite confusion. The use of a predetermined concrete mixture (typically associated with concrete suppliers) helps to reduce any inconsistencies as well as the flexibility of both the supply chain and the actual concrete components. Ready mix concrete (Cast-In-Place) is known for its customizability in the type of concrete product being produced for commercial as well as private purposes. Also, ready mix concrete companies typically offer different variations of concrete according to the user's mix design or industrial standard. Each of the variations of RMC can be manufactured to meet the demand specified for each new delivery or project. Some disadvantages from using RMC are (Mineral Industry):

- The materials are batched at a central plant where the concrete is mixed before being shipped to the site. This poses a critical time period beginning from when supply truck leaves the plant and ending once the supply truck reaches its destination. This critical time period becomes increasingly difficult to manage over longer distances. This is the reason for supply trucks to be built not only to ensure a quick and safe delivery but also to prevent the concrete from losing its ideal pouring state through means of installing a continuously rotating holding tank.

- The travel route taken by the supplier, as high levels of road traffic can become an issue for not only the supplier but can also add delays to construction where deadlines are not met due to late arrivals. Additionally Site access for supply trucks is an unavoidable issue for construction projects, Amongst being a contributing factor in a projects site development plan, access roads must be provided and able to support workers, emergency vehicles as well as large and heavy supply trucks; However this not usually an issue and can be avoided by utilizing what's called a "mini-mix company"- a company that deals with using smaller 4m³ capacity mixers that have the ability to reach more-restricted construction sites.

Cast-In-Place Slab on Steel Deck versus Precast (HC Planks) Girder-Slab Floor Systems

A Precast Girder-Slab floor systems consists of interior girders (also known as an open-web-dissymmetric beam or D-beam) and prestressed hollow-core slabs that are connected using cementitious grout. The use of a Girder-Slab system allows for the concrete slabs, being supported by the steel frame, to resist all gravity and lateral loads. Once the hollow core slabs are placed on the D-beams, the process of creating composite action is done by grouting through the web openings and into the hollow slab cores and is completed once the grout has been cured properly. Similar to the floor system chosen for WSU's new Sheehan Hall, a Girder-Slab system is typically used for mid to high-rise residential structures such as hotels, apartments and condominiums. There are two basic D-beam girder sections available for use with an 8" thick precast slab (generally spanning as long as 28 ft.) and they are a DB-8 and DB-9. The DB-8 provides an 8" thick slab assembly, while the DB-9 is designed to be installed with a 2" concrete topping layer resulting in a 10" total slab thickness. A Girder-Slab system is constructed in

accordance with the “Underwriters Laboratories Inc. Floor-Ceiling Design K912” (Construction Field, 2011). The reason why this system is so highly valued is because it has been shown to greatly improve a projects construction operations as well as a project’s ability to stay on schedule and meet critical deadlines. An example of the Floor System used for WSU’s new dormitory building can see below in **Figure 7**.

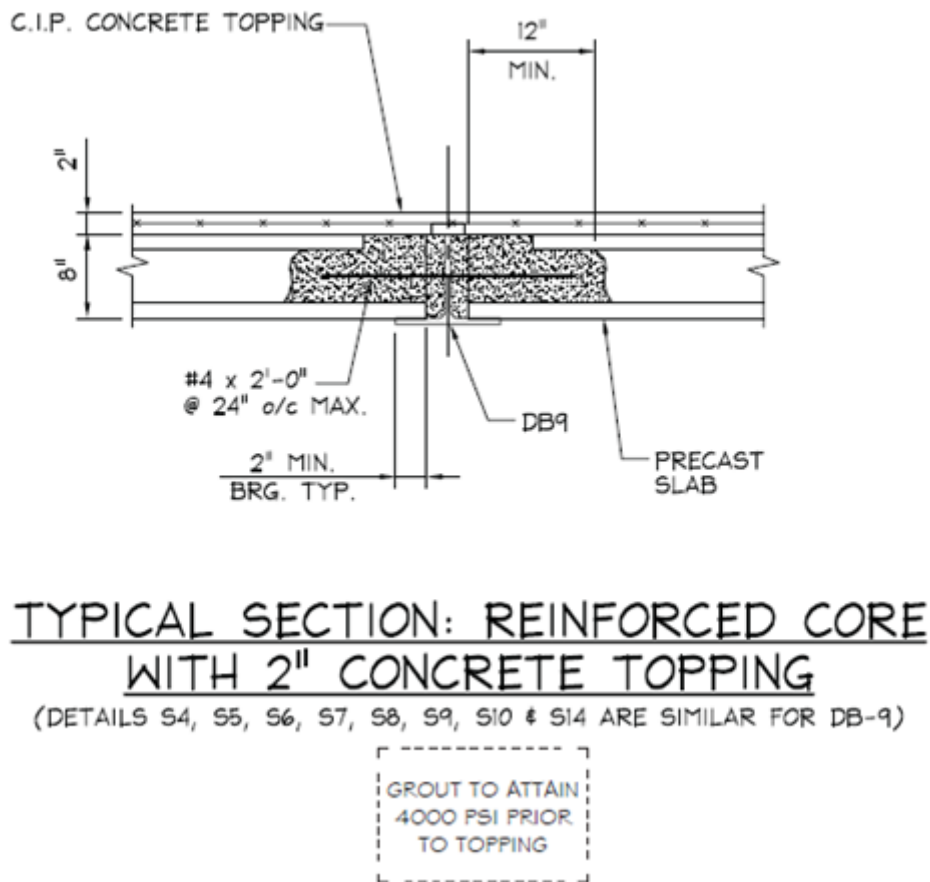


Figure 7: Typical Girder-Slab Section Detail- Reinforced Core with 2-inch Concrete Topping

The use of a pre-topped system allows for faster construction at a slightly more reduced cost than with field-topped systems (Cudney, 1998). However, field topped systems offer less floor vibration, positive drainage (easier to achieve), and also a lower maintenance cost for joint sealants.

A cast-in-place, post-tensioned concrete system is typically constructed by pouring concrete into temporary forms (typically either plywood or steel) that are made on site. This system utilizes a one-way, post-tensioned slab that is supported by long spanned, post-tensioned beams (Cudney, 1998). These beams are typically located at the column line and are about 14 to 18 inches wide by about 32 to 36 inches deep (Cudney, 1998). The advantages and disadvantages for the use of each type of system are listed below in table-3. When properly designed, detailed, constructed and maintained, the durability of the CIP, post-tensioned and precast systems are very similar. Both systems include elements such as expansion joints, joint sealants, and exposed painted metal connections as well as railings that will require preventative maintenance, and even reparations; however, because of the increased number of sealant joints, the precast system would require more maintenance than would a CIP system. Both structural concrete systems are cost effective and durable, but the decision on which structural system to select comes down to the following points (Cudney, 1998):

- The Owner's preference
- Requirements of the structural component's-lateral load system, foundation, flexibility of the framing, ramping, expansion joints, site dimensions, etc.
- Maintenance considerations
- Aesthetics, facade treatment
- Openness, visibility and lighting
- Economics, including first cost and life cycle maintenance costs.
- Construction schedule
- Ability to utilize local labor
- Availability of competitive contractors

Cost advantages

Among the many differences found in each type of concrete construction (production and distribution methods for example) the most important difference is the cost of the material. For many contractors and project managers there is a big difference between Price and Cost. Price only happens to be one element of cost; it is the initial and the easier of the two to understand along with being the most visible. Focusing on price is not a preferred strategy in any business, especially when it comes to a material's quality, and the reliability of manufactured goods.

Instead, the prime focus should be set on the "Total Cost of Ownership". TCO is equal to the sum of the four cost components: quality, service, delivery, and price (NPCA, 2010).

In terms of cost elements, a clear advantage of using precast concrete over cast-in-place (CIP) is the speed of its delivery and also its ease of installation, or service (NPCA, 2010). These collectively contribute to a lower TCO. Precast concrete, especially when produced in controlled plants, boasts the additional benefit of higher quality. The controlled batch proportions placed under uniform conditions consistently creates a better product than can be cast in place (NPCA, 2010).

On any construction site, scheduling is an important but unpredictable and expensive risk. Nature stacks the cost odds against CIP concrete because it is much easier to order precast concrete structures (assembled ahead of time) and have them delivered and installed the same day than it is to have to excavate, form, pour, and strip, the CIP concrete which is then followed by having to cure it, damp proof and backfill each structure. Depending on the type of project and the different constraints present, research shows that on average "the use of precast concrete structures over cast-in place structures can save roughly 5-6 days in construction scheduling" (NPCA, 2010). CIP requires three separate days to pour the base, walls and top of each structure;

additionally, curing and stripping adds one day to the CIP process, totaling seven working days of open-hole time. The TCO of precast is a fixed cost; however the TCO of CIP just begins at an initial cost of the product itself (does not include its delivery and installation costs, etc.) which makes the choice of using precast actually cheaper even though its fixed cost can be higher than CIP's initial cost. It is this concept of TCO that our group plans to implement in our alternative floor system design of Sheehan Hall.

3.0 Project Management

Project management entails many components that must coincide in order for the construction process to be executed to the desired manner. Many of the components interact with each other and therefore all of these components must be done correctly. This chapter evaluates some of the project management components that were important to the construction process. The first topic that is discussed is the evaluation of the project management which entails looking into the communication of the PM and the safety of the project site. The next sections analyze the cost and schedule for the current. The chapter ends with an analysis of how Building Information Technology and Lean Construction are used to complete the project.

3.1 Project Management Evaluation

During construction it is imperative that every party that is involved in the construction process is informed and up to date with the progress and problems that are occurring for the project. These parties include the owner, the design team, and subcontractors. While the corresponding party should be informed of any problems when they happen, a weekly meeting is important so that every party can be informed of any occurrences that have occurred for the project. Every week the project manager, in this case Consigli, has held meetings on site to inform all the parties of the progress and problems that have occurred. One of the important aspects of project management is the communication and the ability to resolve any issues that have come up. From attending the meetings it has been clear to the MQP group that Consigli has handled the issues that have occurred because of their good communication and problem solving ability. It is imperative that all parties are informed of any issues and that every party is involved in making the correct decision in how to handle the issue so that everybody is on the same page. One of the issues that the MQP group has seen handled in a professional way was the delay in

delivery of the windows for the exterior of the building. Consigli did a good job in informing every one of the issue, communicating with the window manufacturer on when the windows will eventually be delivered, and working with the subcontractors to work around the delay so that the project stayed on schedule.

Another important aspect of project management is the safety of the project site and the workers on the project. Consigli has safety officers that visit each site every week and provide a safety score each week for a project. These officers observe and record safety aspects that include workers safety, equipment usage and site safety. If any of these aspects are not being followed to the correct specifications or not followed at all the officer will deduct points from the overall score that is provided at the end of the visit. The safety officer will also inform the PM of the issues so that they will be resolved. The PM can also earn extra points for going above and beyond the safety requirements. For this project Consigli has received safety scores that range between 95 and 102. This is a great indication that all the safety requirements have been followed and any safety issues that have arose were handled correctly.

3.2 Cost

The original contract for completed design and construction for this project was \$50,293,915, which included an original contingency amount of \$500,000. Throughout the project changes have been made to the original design that have affected the cost of completion for the project. Change requests and the PM's contingency are used when changes need to be made to the original design. A change request is a form that documents a change that occurred to the project and how much that change will affect the total cost of completion. If the owner approves the alteration and cost of the change then it will be added or subtracted from the cost of completion. The PM can also use the provided contingency amount for changes that occur to the

project, however it will not affect the total cost of the project up to the total amount of the contingency. Change requests are typically used for changes that occurred outside of the original scope of work for the project that could be due to incomplete or incorrect drawings. The contingency is typically used for changes that occurred within the original scope of work. The contract changes due to approved changes through the project can be seen in **Figure 8**. The change in contingency amount can be reviewed in **Figure 9**.

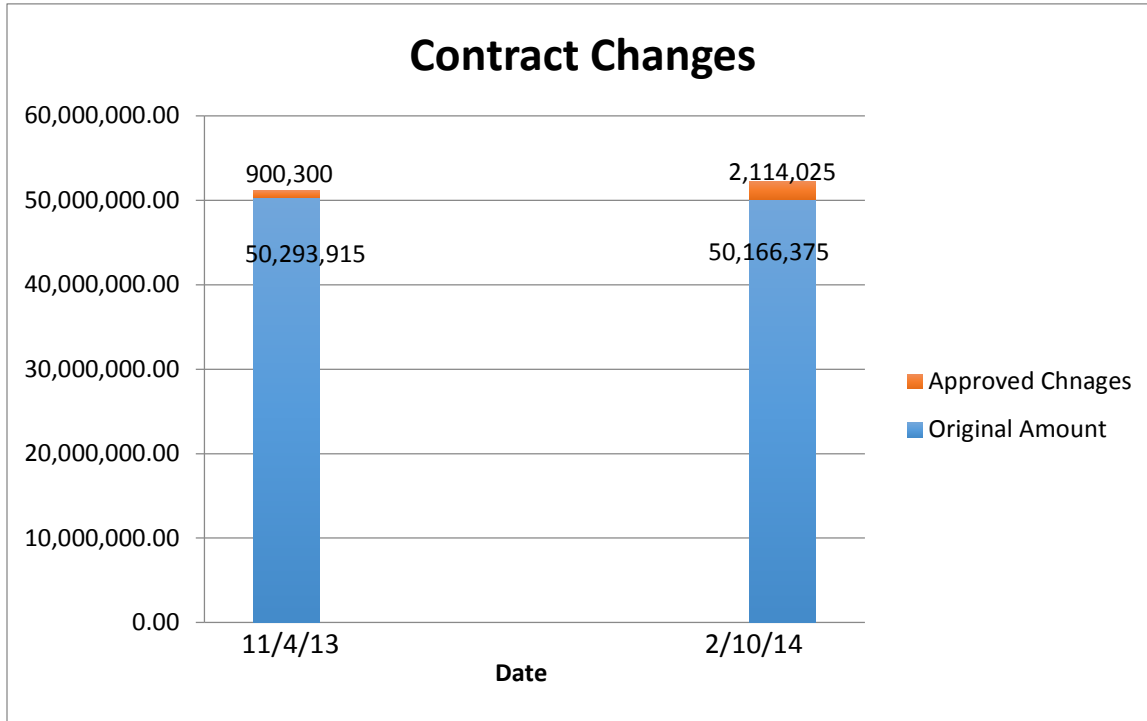


Figure 8: Contract Changes

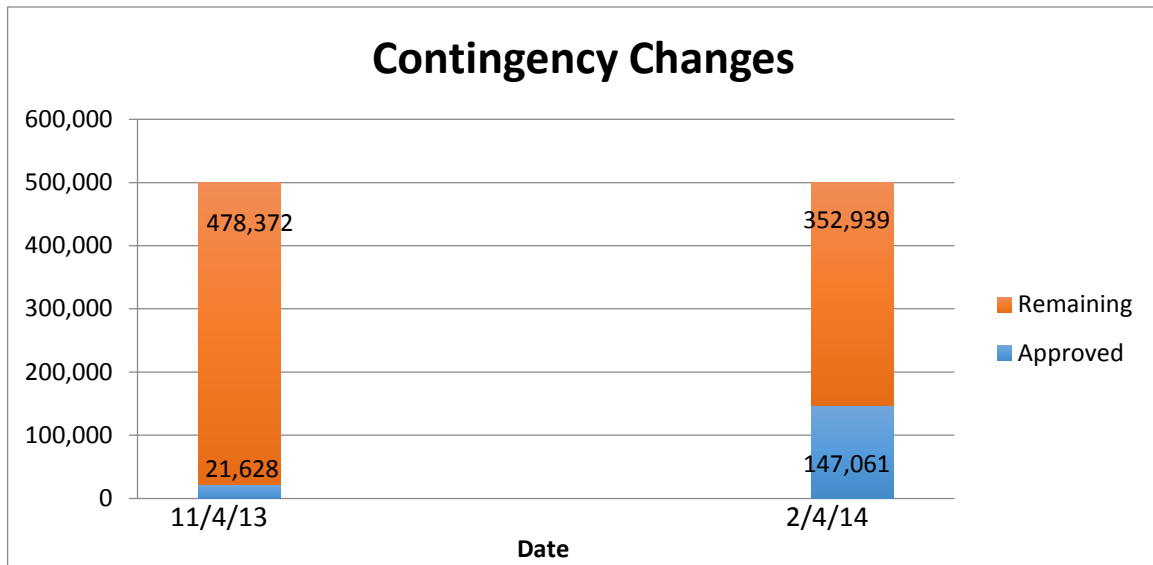


Figure 9: Contingency Changes

From a project management evaluation perspective these changes are good for the PM. Since the amount for approved contingency is low compared the allowable remaining this means that there have been minimal changes to the project within the scope of work. This is a good indication that the PM had a good understanding of what the scope of work was for the project provided a good cost estimation for the project. At the end of the job the reaming contingency will go back to the owner, therefore it creating a good partnership and a good track record for future work.

3.3 Schedule

In order to effectively compare Consigli’s updated as-built schedule (**Appendix E**) to their original baseline schedule (**Appendix D**), the baseline schedule was first recreated using the Primavera P6 Project Management software. The original baseline schedule obtained from Consigli included all the activities from the Sheehan Hall project. However, the intent of this project is to determine the effects of an alternative floor system design on the cost and schedule.

For this reason, the baseline schedule that was recreated in Primavera was reduced to only the foundations, the steel structure, and the pre-cast floor slabs, since these are the only components that would be effected by a new floor system design. **Figures 10** and **11** show the recreated baseline schedule, complete with the list of activities on the left, and a bar chart showing the activities and their relationships on a timeline on the right.

In recreating the baseline schedule, each of the activities were first entered into Primavera, along with its original duration and expected start and finish dates. Once all the activities were entered, the relationships between the activities were determined and assigned in order to create a network and from it generate the bar chart. A majority of the activities have a “Finish to Start” relationship, meaning that an activity would be started only when its predecessors are finished. However, some activities have a “Start to Start” relationship with a time lag, meaning that an activity would be started a certain number of days (equal to the lag time) after its predecessor has been started. A “Start to Start” relationship saves time compared to a “Finish to Start” relationship because activities with the former kind of relationship can be worked on simultaneously but those with the latter kind cannot. Upon running the schedule after the relationships have been established, Primavera automatically identifies the critical activities (those whose combined duration determine the completion date of the project) and highlights them in red on the bar chart. Once the schedule had been recreated, it was possible to obtain the slated start and finish dates: excavation for footings and foundations would start on 20th March 2013, and the structure would be complete by 8th October, 2013. This baseline schedule was compared to the as-built schedule updated by Consigli on 6th November 2013, in order to determine how well they adhered to their baseline schedule.

To compare the as-built schedule to the baseline, the actual start and finish dates for each activity were entered into the baseline schedule created in Primavera. After doing so, Primavera automatically updates the schedule and shows the remaining duration for each activity, based on the percent completion of the activity. The activities that have been completed have a remaining duration of zero, while those that have not yet been started have a remaining duration equal to the original duration. Once the as-built schedule was complete, it was then possible to determine how different it is from the baseline schedule. According to the as-built schedule, the structure would not be complete until 3rd December 2013. This is 40 working days behind the baseline structure completion date of 8th October 2013. **Figure 12** shows the remaining activities as of 6th November 2013 (blue vertical line), according to the as-built schedule. These activities are shown in red on the bar chart because they have become critical activities, since their durations dictate the completion date of the structure.

In order to take a better look at which activities took longer to complete than expected, **Table 1** was prepared. It compares the original duration with the actual duration, as well as the planned start date with the actual start date of the activities that took longer (Please see **Appendix Y** for a detailed comparison of all the activities). All the activities whose actual duration exceeded its original duration by 10 days are highlighted in red. However, it is important to note that not all the activities listed are critical activities, thus not all of them contribute to the delay. The only ones that would contribute to the delay are those that are critical and took longer than expected, and those that are not initially critical but took longer by a number of days greater than their total float.

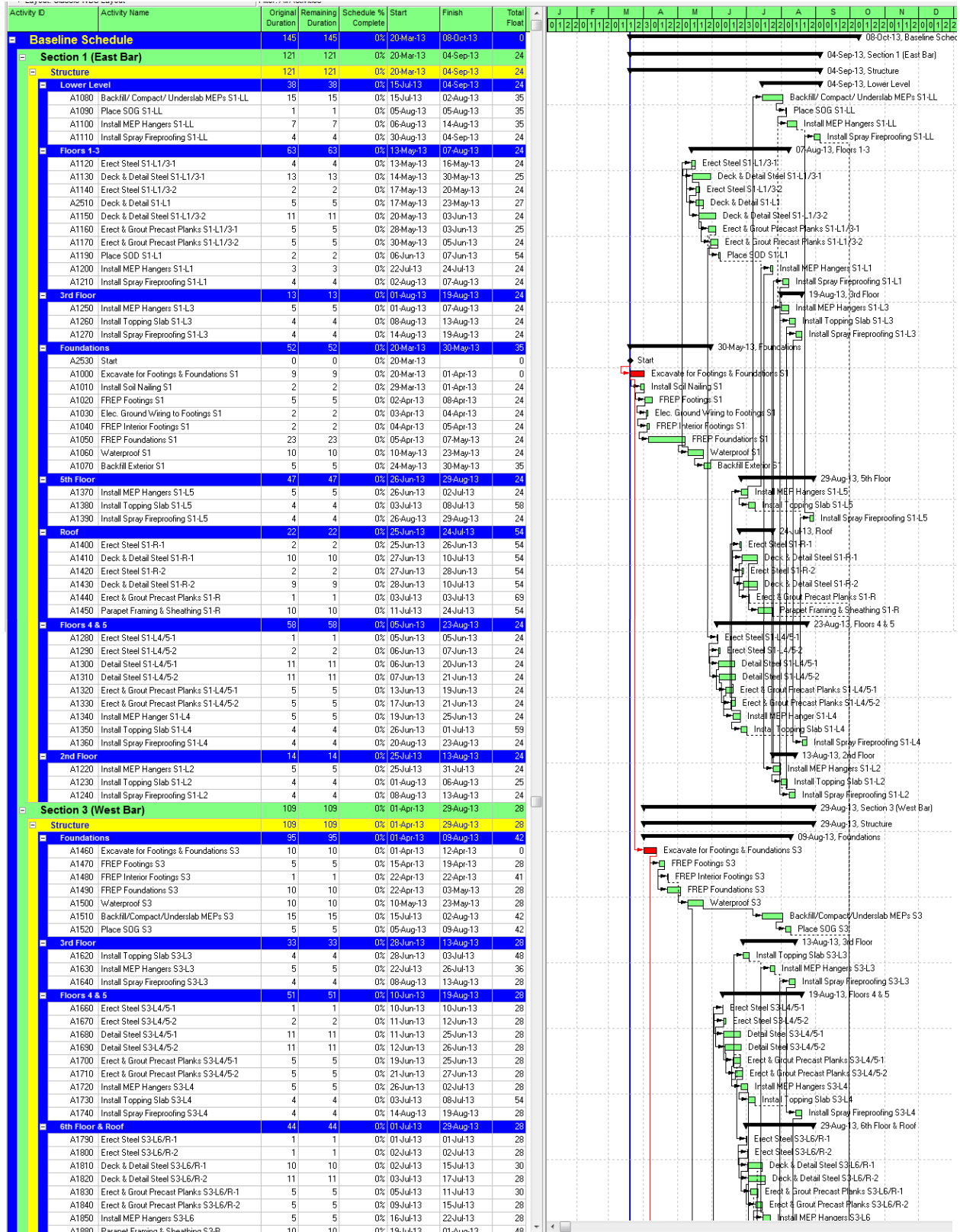


Figure 10: Baseline Schedule

A1880	Parapet Framing & Sheathing S3-R	10	10	0%	19-Jul-13	01-Aug-13	48
A1860	Install Topping Slab S3-L6	4	4	0%	23-Jul-13	26-Jul-13	48
A1870	Install Spray Fireproofing S3-L6	4	4	0%	26-Aug-13	29-Aug-13	28
Floors 1-3							
A1530	Erect Steel S3-L1/3-1	2	2	0%	22-May-13	23-May-13	28
A1540	Deck & Detail Steel S3-L1/3-1	11	11	0%	23-May-13	06-Jun-13	28
A1550	Erect Steel S3-L1/3-2	1	1	0%	24-May-13	24-May-13	28
A1560	Deck & Detail Steel S3-L1/3-2	10	10	0%	28-May-13	10-Jun-13	28
A1570	Erect & Grout Precast Planks S3-L1/3-1	5	5	0%	03-Jun-13	07-Jun-13	28
A1580	Erect & Grout Precast Planks S3-L1/3-2	5	5	0%	04-Jun-13	10-Jun-13	28
A1590	Install SDD S3-L1/2	5	5	0%	11-Jun-13	17-Jun-13	52
A1600	Install MEP Hangers S3-L1/2	5	5	0%	22-Jul-13	26-Jul-13	28
A1610	Install Spray Fireproofing S3-L1/2	4	4	0%	02-Aug-13	07-Aug-13	28
5th Floor							
A1750	Install MEP Hangers S3-L5	5	5	0%	11-Jul-13	17-Jul-13	28
A1760	Install Topping Slab S3-L5	4	4	0%	19-Jul-13	24-Jul-13	46
A1770	Install Spray Fireproofing S3-L5	4	4	0%	20-Aug-13	23-Aug-13	28
Section 2 (Center Bar)							
Structure							
Foundations							
A1890	Excavate for Footings & Foundations S2	10	10	0%	15-Apr-13	02-Aug-13	14
A1900	FREP Footings S2	4	4	0%	29-Apr-13	02-May-13	0
A1910	FREP Interior Footing S2	1	1	0%	03-May-13	03-May-13	19
A1920	FREP Foundations S2	19	19	0%	03-May-13	29-May-13	0
A1930	Waterproof S2	15	15	0%	31-May-13	20-Jun-13	0
A1940	FREP Elevator Pits S2	10	10	0%	18-Jun-13	01-Jul-13	0
A1950	Waterproof Elevator Pits S2	10	10	0%	02-Jul-13	15-Jul-13	0
A1960	Backfill/Compact/Underlab MEPs S2	15	15	0%	15-Jul-13	02-Aug-13	14
Floors 1-3							
A1970	Erect Columns & Steel S2-L1/3-1	3	3	0%	11-Jul-13	15-Jul-13	0
A1980	Deck & Detail Steel S2-L1/3-1	13	13	0%	12-Jul-13	30-Jul-13	0
A1990	Erect Steel S2-L1/3-2	3	3	0%	16-Jul-13	18-Jul-13	0
A2000	Deck & Detail Steel S2-L1/3-2	12	12	0%	17-Jul-13	01-Aug-13	0
A2010	Erect & Grout Precast Planks S2-L1/3-1	5	5	0%	25-Jul-13	31-Jul-13	0
A2020	Erect & Grout Precast Planks S2-L1/3-2	5	5	0%	26-Jul-13	01-Aug-13	0
A2030	Place SOG S2-L1	5	5	0%	05-Aug-13	09-Aug-13	14
A2040	Install MEP Hangers S2-L1	6	6	0%	12-Aug-13	19-Aug-13	14
A2050	Install Spray Fireproofing S2-L1	4	4	0%	15-Aug-13	20-Aug-13	14
A2060	Place SDD S2-L1	2	2	0%	21-Aug-13	22-Aug-13	33
3rd Floor							
A2100	Install Topping Slab S2-L3	4	4	0%	08-Aug-13	30-Aug-13	14
A2110	Install MEP Hangers S2-L3	1	1	0%	14-Aug-13	14-Aug-13	22
A2120	Install Spray Fireproofing S2-L3	4	4	0%	27-Aug-13	30-Aug-13	14
Floors 4 & 5							
A2130	Erect Steel S2-L4/5-1	1	1	0%	29-Jul-13	29-Jul-13	0
A2140	Erect Steel S2-L4/5-2	1	1	0%	30-Jul-13	30-Jul-13	0
A2150	Detail Steel S2-L4/5-1	10	10	0%	30-Jul-13	12-Aug-13	0
A2160	Detail Steel S2-L4/5-2	10	10	0%	31-Jul-13	13-Aug-13	0
A2170	Erect & Grout Precast Planks S2-L4/5-1	5	5	0%	06-Aug-13	12-Aug-13	0
A2180	Erect & Grout Precast Planks S2-L4/5-2	5	5	0%	08-Aug-13	14-Aug-13	0
A2190	Install Topping Slab S2-L4	4	4	0%	15-Aug-13	20-Aug-13	16
A2200	Install MEP Hangers S2-L4	5	5	0%	21-Aug-13	27-Aug-13	16
A2210	Install Spray Fireproofing S2-L4	4	4	0%	03-Sep-13	06-Sep-13	14
5th Floor							
A2220	Install Topping Slab S2-L5	4	4	0%	21-Aug-13	26-Aug-13	17
A2230	Install MEP Hangers S2-L5	5	5	0%	28-Aug-13	03-Sep-13	16
A2240	Install Spray Fireproofing S2-L5	4	4	0%	09-Sep-13	12-Sep-13	14
Floors 6, Roof, PH							
A2250	Erect Steel S2-L6/PH-1	1	1	0%	09-Aug-13	09-Aug-13	0
A2260	Erect Steel S2-L6/PH-2	1	1	0%	12-Aug-13	12-Aug-13	0
A2270	Detail Steel S2-L6/PH-1	10	10	0%	12-Aug-13	23-Aug-13	0
A2280	Detail Steel S2-L6/PH-2	10	10	0%	13-Aug-13	26-Aug-13	0
A2290	Erect & Grout Precast Planks S2-L6/PH-1	5	5	0%	19-Aug-13	23-Aug-13	0
A2300	Erect & Grout Precast Planks S2-L6/PH-2	5	5	0%	21-Aug-13	27-Aug-13	0
A2310	Install Topping Slab S2-L6	4	4	0%	28-Aug-13	02-Sep-13	17
A2340	Parapet Framing & Sheathing S2-R	10	10	0%	28-Aug-13	10-Sep-13	20
A2320	Install MEP Hangers S2-L6	5	5	0%	04-Sep-13	10-Sep-13	16
A2330	Install Spray Fireproofing S2-L6	4	4	0%	13-Sep-13	18-Sep-13	14
2nd Floor							
A2070	Install Topping Slab S2-L2	4	4	0%	02-Aug-13	07-Aug-13	17
A2080	Install MEP Hangers S2-L2	1	1	0%	08-Aug-13	08-Aug-13	22
A2090	Install Spray Fireproofing S2-L2	4	4	0%	21-Aug-13	26-Aug-13	14
Section 4 (Cafeteria)							
Structure							
Foundations							
A2350	FREP Footings S4	3	3	0%	03-May-13	03-Jun-13	75
A2360	FREP Interior Footing S4	1	1	0%	08-May-13	08-May-13	58
A2370	FREP Foundations S4	14	14	0%	10-May-13	29-May-13	58
A2380	Waterproof Foundations S4	1	1	0%	31-May-13	31-May-13	58
A2390	Backfill S4	1	1	0%	03-Jun-13	03-Jun-13	75
Lower Level							
A2400	Place SOG S4-LL	5	5	0%	17-Sep-13	23-Sep-13	0
A2410	Install MEP Hangers S4-LL	6	6	0%	24-Sep-13	01-Oct-13	0
A2420	Install Spray Fireproofing S4-LL	4	4	0%	27-Sep-13	02-Oct-13	0
1st Floor							
A2430	Erect Columns & Steel S4-L1	3	3	0%	22-Aug-13	26-Aug-13	0
A2440	Deck & Detail Steel S4-L1	13	13	0%	26-Aug-13	11-Sep-13	0
A2450	SDD S4-L1	2	2	0%	13-Sep-13	16-Sep-13	0
A2460	Install MEP Hangers S4-L1	6	6	0%	24-Sep-13	01-Oct-13	0
A2470	Install Spray Fireproofing S4-L1	4	4	0%	03-Oct-13	08-Oct-13	0
Roof							
A2480	Erect Steel S4-R	2	2	0%	27-Aug-13	28-Aug-13	3
A2490	Deck & Detail Steel S4-R	13	13	0%	29-Aug-13	16-Sep-13	3
A2500	Parapet Framing & Sheathing S4-R	10	10	0%	20-Sep-13	03-Oct-13	3
A2520	Structure Complete	0	0	0%	08-Oct-13	08-Oct-13	0

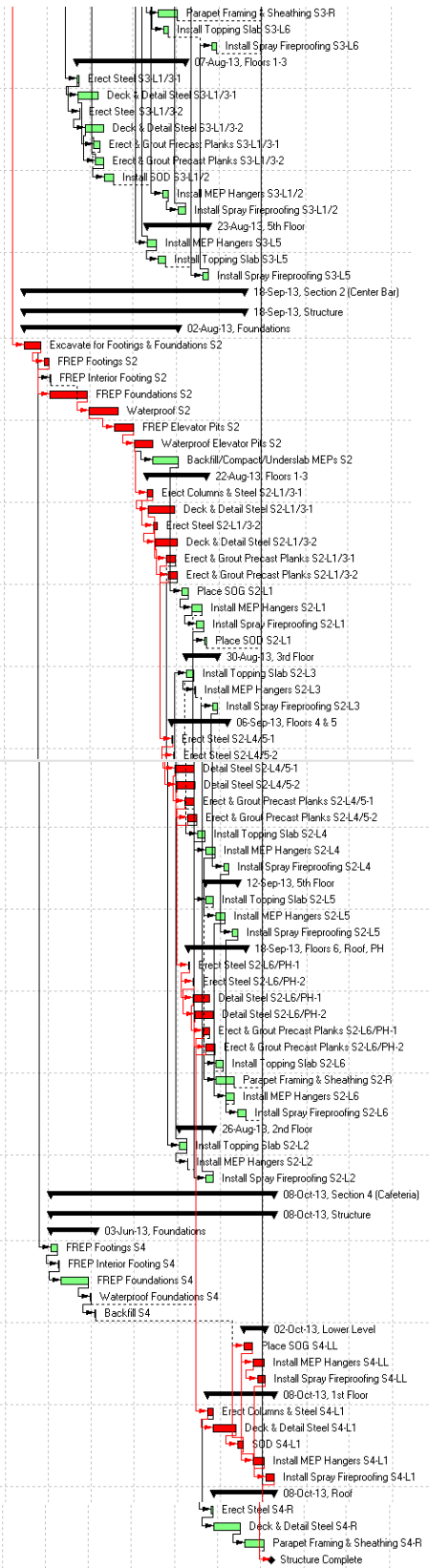


Figure 11: Baseline Schedule (Continued)

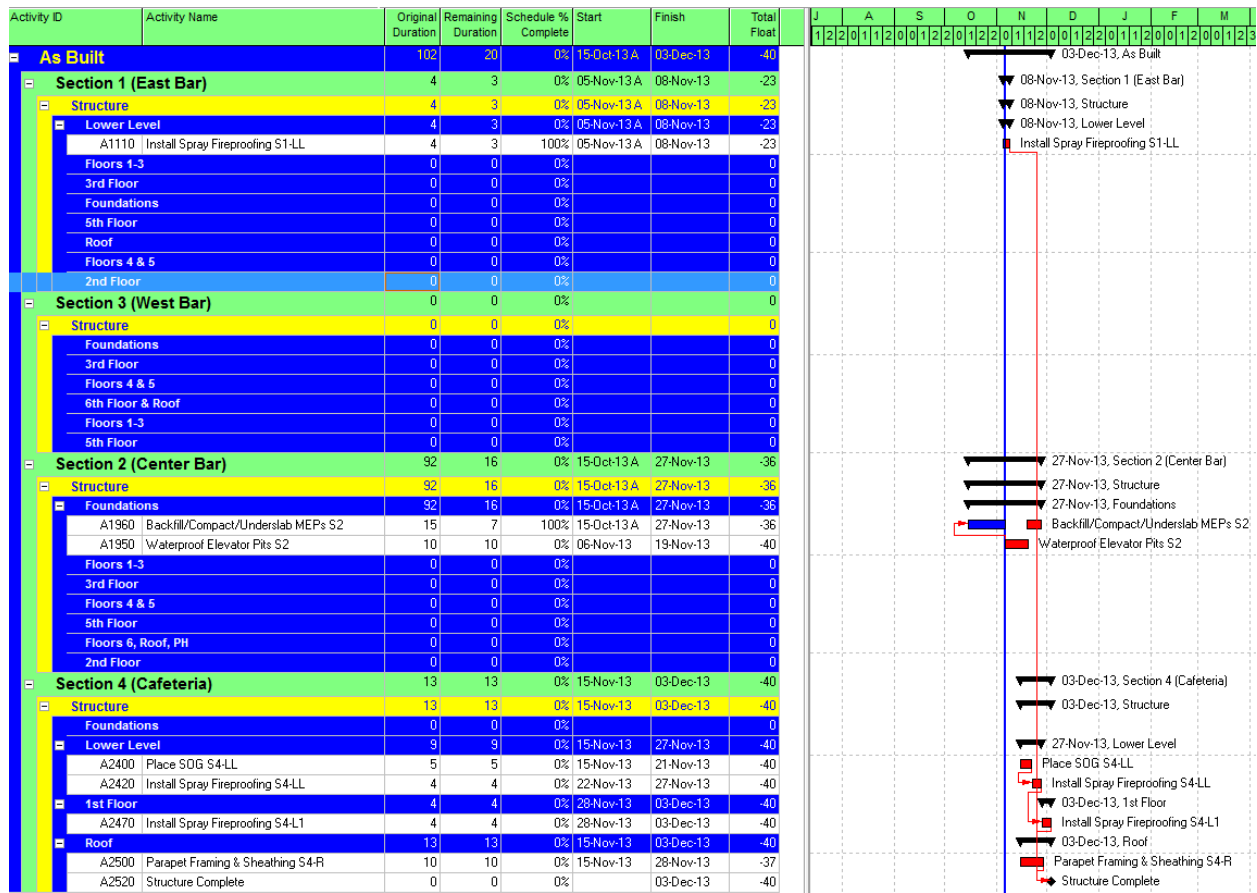


Figure 12: As-Built Remaining Critical Activities

The reason for the two backfilling activities taking much longer than expected can be attributed to the fact that the design drawings for Sheehan Hall were delayed by a month. This resulted in the electrical subcontractor not being able to plan ahead and understand the scope of the work as well as they could have. The delay in the design drawings also impacted the erection of steel for the roof of the cafeteria (Section 4), since the design for that particular roof was significantly different from all the other roofs (Consigli, 2013). The excavation for the footings and foundations of the west building (Section 3) taking longer than expected is not surprising because any type of geotechnical or site work can be highly variable depending on the conditions of the ground.

Table 1: Baseline vs. As-built

Baseline vs. As Built				
Activity Name	Original Duration	Actual Duration	Planned Start	Actual Start
FREP Interior Footings S1	2	7	4/4/2013	4/12/2013
Backfill/ Compact/ Underslab MEPs S1-LL	15	65	7/15/2013	7/2/2013
Place SOG S1-LL	1	5	8/5/2013	10/16/2013
Erect Steel S1-L1/3-2	2	4	5/17/2013	5/20/2013
Erect & Grout Precast Planks S1-L1/3-1	5	10	5/28/2013	5/29/2013
Erect & Grout Precast Planks S1-L1/3-2	5	9	5/30/2013	6/3/2013
Install Spray Fireproofing S1-L3	4	5	8/14/2013	8/13/2013
Erect Steel S1-L4/5-2	2	3	6/6/2013	6/11/2013
Erect Steel S1-R-2	2	3	6/27/2013	6/24/2013
Erect & Grout Precast Planks S1-R	1	6	7/3/2013	7/8/2013
Excavate for Footings & Foundations S3	10	20	4/1/2013	4/1/2013
FREP Footings S3	5	8	4/15/2013	4/16/2013
FREP Interior Footings S3	1	4	4/22/2013	5/20/2013
FREP Foundations S3	10	15	4/22/2013	4/29/2013
Backfill/Compact/Underslab MEPs S3	15	35	7/15/2013	7/15/2013
Erect Steel S3-L1/3-2	1	2	5/24/2013	5/24/2013
Erect Steel S3-L4/5-2	2	4	6/11/2013	6/14/2013
Parapet Framing & Sheathing S3-R	10	13	7/19/2013	8/19/2013
Excavate for Footings & Foundations S2	10	11	4/15/2013	4/16/2013
FREP Footings S2	4	10	4/29/2013	5/3/2013
Waterproof S2	15	52	5/31/2013	5/20/2013
Erect Steel S2-L1/3-2	3	4	7/16/2013	7/16/2013
Erect & Grout Precast Planks S2-L1/3-1	5	7	7/25/2013	7/24/2013
Erect & Grout Precast Planks S2-L1/3-2	5	8	7/26/2013	7/25/2013
Install Spray Fireproofing S2-L4	4	6	9/3/2013	9/5/2013
Erect & Grout Precast Planks S2-L6/PH-2	5	7	8/21/2013	8/26/2013
Parapet Framing & Sheathing S2-R	10	19	8/28/2013	9/16/2013
FREP Footings S4	3	4	5/3/2013	6/17/2013
FREP Interior Footing S4	1	3	5/8/2013	7/2/2013
Erect Columns & Steel S4-L1	3	10	8/22/2013	9/2/2013
Erect Steel S4-R	2	20	8/27/2013	9/2/2013

3.4 Uses of BIM

One of the main purposes of BIM for this study is to provide a visual comparison of Consigli's baseline and as-built schedules, in the form of an animation of the construction. In order to do so, a 3D Revit model of the structure obtained from Consigli was initially supposed to be integrated with the schedule and cost to create a 5D model. However, due to a delay in obtaining the structural Revit model, Consigli's original BIM model that was used for coordination purposes was integrated with the schedule and cost instead. This BIM model was obtained from Consigli's BIM expert, Jack Moran, in the form of an Autodesk Navisworks file. The 3D model that had already been imported into the Navisworks file was in the form of a Tekla model (Tekla is a BIM modeling tool used primarily for steel and concrete detailing and fabrication). Since the Tekla model was a steel fabrication model, it contained numerous details such connections and bolts, as well as detailing for the steel. Being a BIM model that was used for coordination, the model also included the architectural components, as well as all the other building systems such as HVAC, MEP, plumbing, etc. For the purposes of this study, the model was limited to show just the foundations and the structure by hiding all the other systems and components.

Once all the other systems and components were hidden in the model, the next step was to import the baseline schedule that had already been updated with the as-built dates in Primavera. Although Navisworks directly supports Primavera files to be imported, a software add-on that allows Navisworks to access the Primavera online database needs to be installed locally on the computer to be used. Due to the fact that students do not have the permission to install software applications on school computers at WPI, an alternate format was considered to import the schedule into Navisworks. The schedule from Primavera was first exported as a

Microsoft Project (.mpx) file, which could then be imported into the TimeLiner function of Navisworks with the click of a button (TimeLiner is the function in Navisworks that allows a schedule to be imported and integrated with the 3D model).

After the schedule had been successfully imported, the next step was to divide the objects from the 3D structural model into sets. These sets had to correspond to the activities and their sequencing on the schedule because the sets will be attached to the activities in order to create an animation of the construction of the building. Consigli, for the purpose of construction, had decided to divide the entire building into four different sections, three large ones and one small one. Each of the three large building sections were erected two floors at a time, in two separate phases. For example, half of the bays of the first two floors were first erected, followed by the other half of the bays of both floors. Then, the next two floors would be erected, also half at a time. The sets created had to match this construction sequencing so that the animation of the construction of the building will match the way the building was actually built on site. Once the entire structural model had been divided into sets, the model and schedule can then be integrated to obtain a 4D model.

In order to integrate the model with the construction schedule, each of the sets that had been created were assigned to individual activities on the schedule in TimeLiner. **Figure 13** shows the baseline versus as-built schedule in TimeLiner, with the sets (in blue text), attached to some of the activities, on the right. The reason for not all the activities being attached to sets is that the level of detail contained in the schedule is much higher than the level of detail of the 3D model. For example, the 3D model does not contain objects that correspond to activities such as backfilling, waterproofing, installation of fireproofing, etc. As a result, of the 128 total activities on the schedule, only 49 had sets attached to them.

Active	Name	Status	Planned Start	Planned End	Actual Start	Actual End	Task Type	Attached	Total Cost
<input checked="" type="checkbox"/>	FREP Footings S3		4/15/2013	4/19/2013	4/16/2013	4/26/2013	Construct	Sets->S3 Foundations->S3 Footings	152,689.79
<input checked="" type="checkbox"/>	FREP Interior Footings S3		4/22/2013	4/22/2013	5/20/2013	5/24/2013			
<input checked="" type="checkbox"/>	FREP Foundations S3		4/22/2013	5/3/2013	4/29/2013	5/20/2013	Construct	Sets->S3 Foundations->S3 FREP Foundations	145,716.39
<input checked="" type="checkbox"/>	Waterproof S3		5/10/2013	5/23/2013	5/13/2013	5/24/2013			
<input checked="" type="checkbox"/>	Backfill/Compact/Underslab MEPs S3		7/15/2013	8/2/2013	7/15/2013	9/2/2013			
<input checked="" type="checkbox"/>	Place SOG S3		8/5/2013	8/9/2013	9/18/2013	9/25/2013	Construct	Sets->S3 Foundations->S3 SOG	112,750.28
<input checked="" type="checkbox"/>	3rd Floor		N/A	N/A	N/A	N/A			
<input checked="" type="checkbox"/>	Install Topping Slab S3-L3		6/28/2013	7/3/2013	7/10/2013	7/12/2013			
<input checked="" type="checkbox"/>	Install Spray Fireproofing S3-L3		8/8/2013	8/13/2013	8/22/2013	8/23/2013			
<input checked="" type="checkbox"/>	Floors 4 & 5		N/A	N/A	N/A	N/A			
<input checked="" type="checkbox"/>	Erect Steel S3-L4/5-1		6/10/2013	6/10/2013	6/13/2013	6/14/2013	Construct	Sets->S3 Steel->S3 L4/5-1	160,915.11
<input checked="" type="checkbox"/>	Erect Steel S3-L4/5-2		6/11/2013	6/12/2013	6/14/2013	6/20/2013	Construct	Sets->S3 Steel->S3 L4/5-2	95,214.85
<input checked="" type="checkbox"/>	Detail Steel S3-L4/5-1		6/11/2013	6/25/2013	6/17/2013	7/1/2013			
<input checked="" type="checkbox"/>	Detail Steel S3-L4/5-2		6/12/2013	6/26/2013	6/18/2013	7/2/2013			
<input checked="" type="checkbox"/>	Erect & Grout Precast Planks S3-L4/5-1		6/19/2013	6/25/2013	6/27/2013	7/3/2013	Construct	Sets->S3 Precast->S3 L4/5-1	82,531.25
<input checked="" type="checkbox"/>	Erect & Grout Precast Planks S3-L4/5-2		6/21/2013	6/27/2013	6/28/2013	7/3/2013	Construct	Sets->S3 Precast->S3 L4/5-2	95,500.44

Figure 13: Baseline vs. As-built TimeLiner Schedule

Once all the sets had been attached to activities, the “Task Type” for each activity that has a set attached to it was set to “Construct.” This tells TimeLiner to animate the construction of the activity and the set attached to it on the 4D model. The next step before running the animation was to assign costs to the activities that will be constructed. In order to do this, a quantity take-off was performed for each set using Navisworks’ Quantification function. Once the quantity and volume of all the objects in each set was known, the cost for each set was calculated based on the per unit prices of materials calculated from Consigli’s cost packages. The costs can then be assigned to their corresponding activities and sets in TimeLiner to complete the 5D model.

The last step of creating the visual schedule comparison was to configure the animation in such a way that objects that were constructed as planned appear in green, those that were constructed earlier than planned appear in yellow, and those that were constructed later than planned appear in red. However, for this to work, the view in the Animation Settings of TimeLiner must be set to “Planned against Actual” so that the animation shows a comparison of the baseline and as-built schedules. Such color coding makes it easier to visually identify any

variances between the two schedules. At this point, the animation was ready to be run, and an animation file can be exported from Navisworks in the form of a Windows AVI file. **Figures 14** through **18** below show screenshots of the construction of Sheehan Hall along the timeline of the 5D model. Information such as the day, date, and total costs up to that date are shown in the upper left corner of the animation.

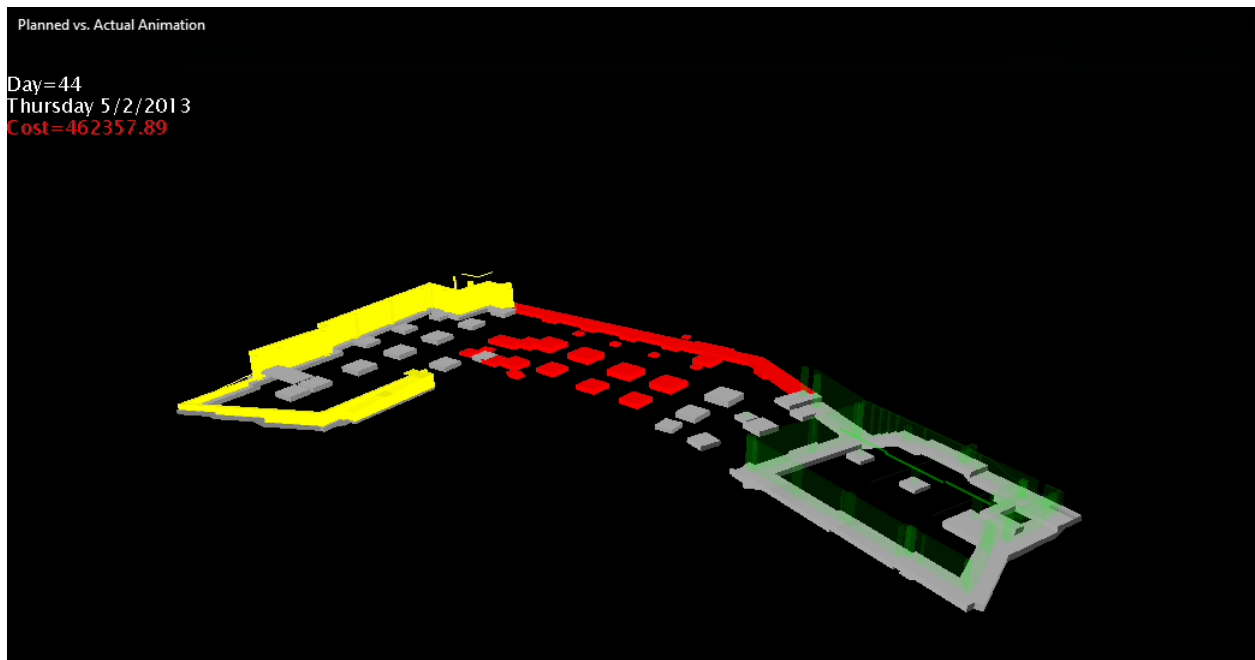


Figure 14: Baseline vs. As-Built Animation 1

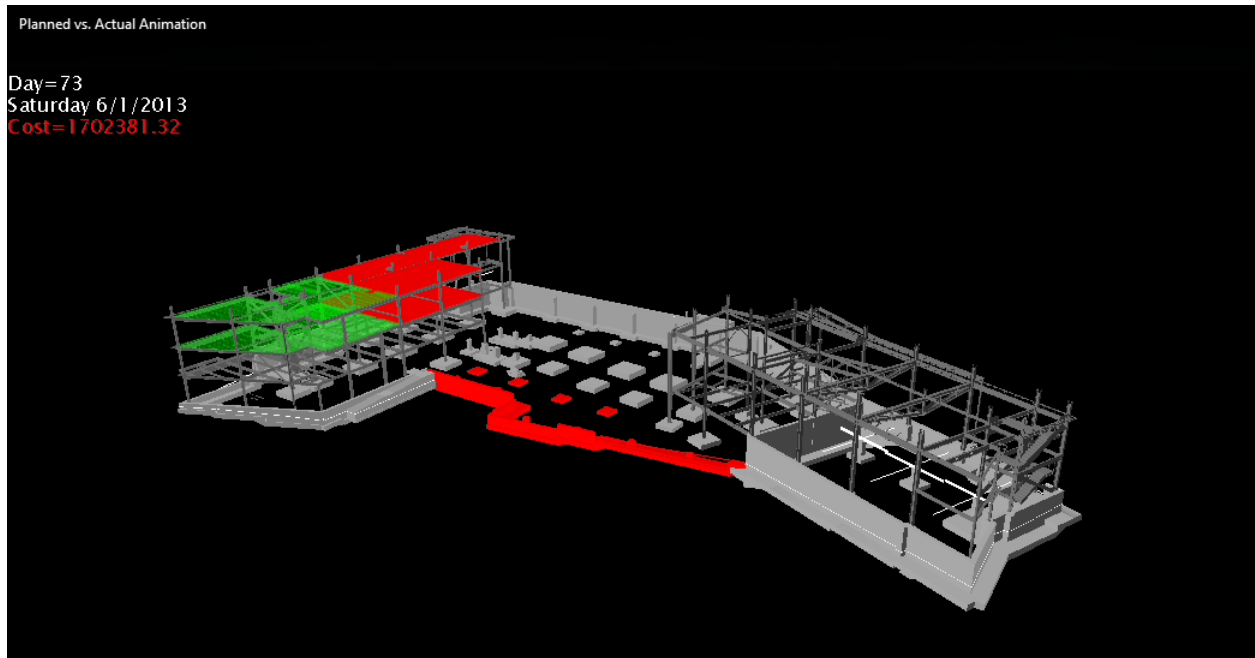


Figure 15: Baseline vs. As-Built Animation 2

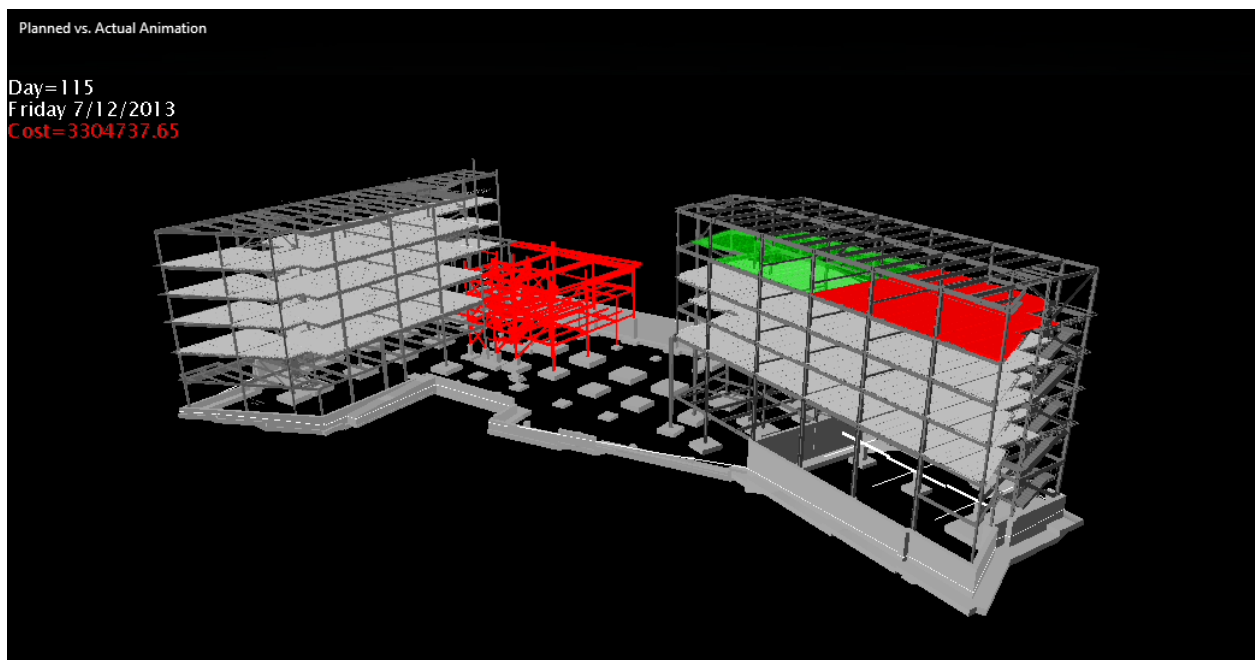


Figure 16: Baseline vs. As-Built Animation 3

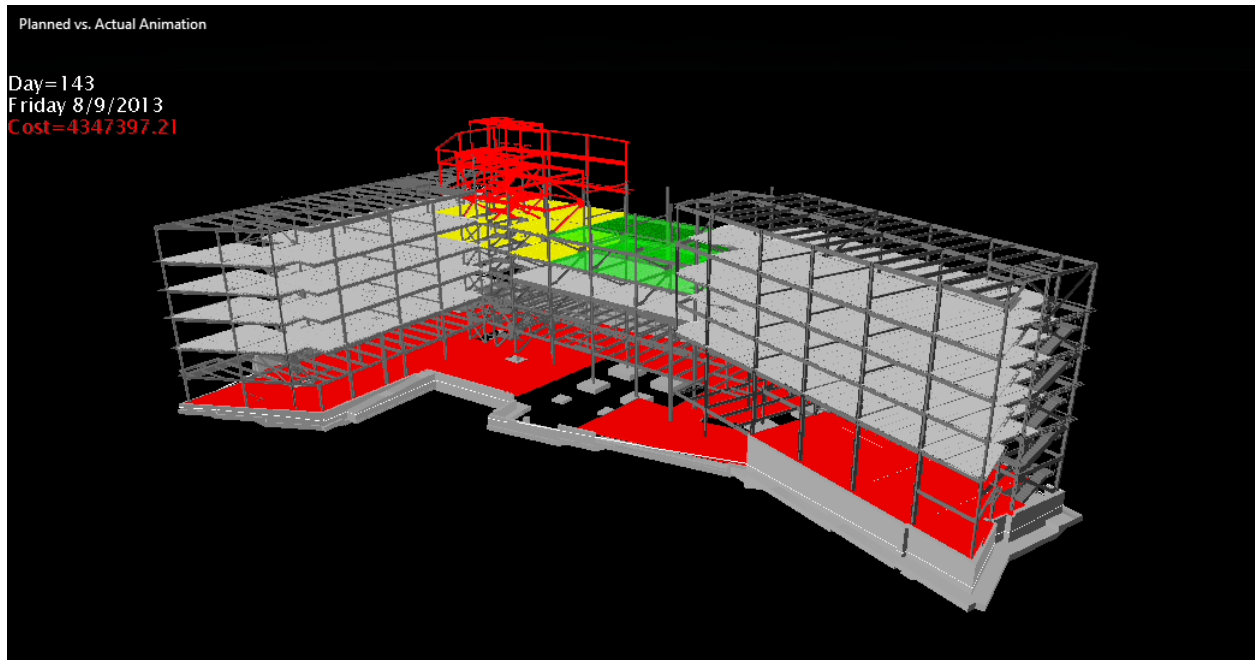


Figure 17: Baseline vs. As-Built Animation 4

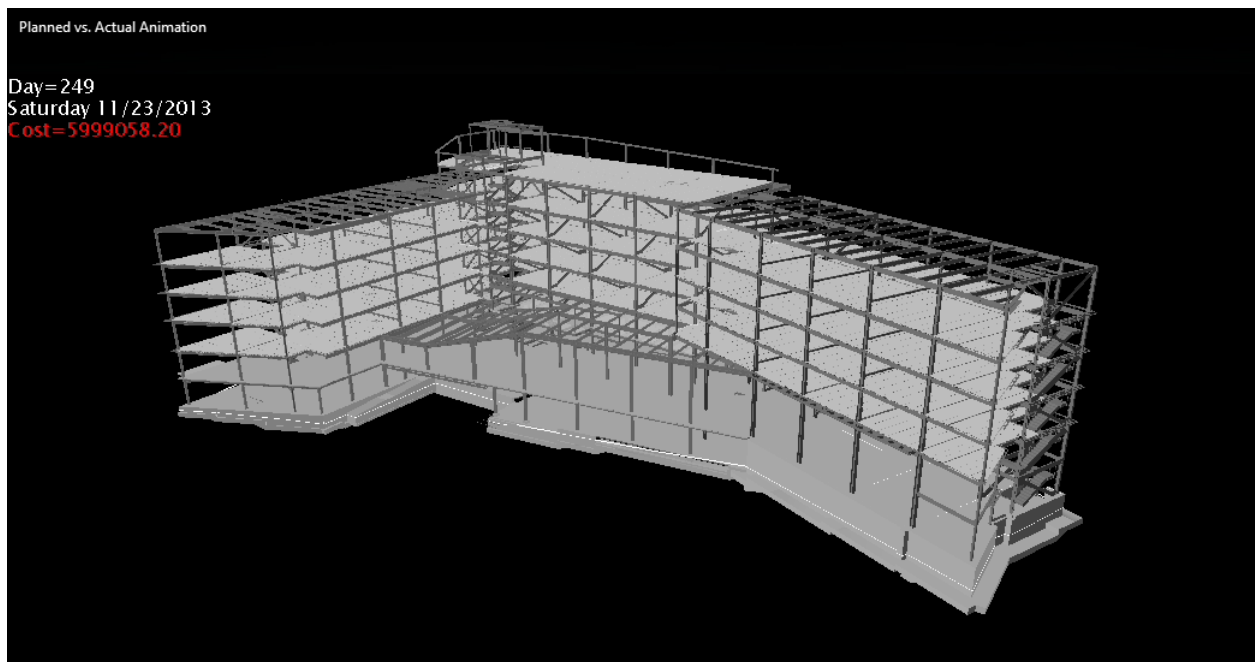


Figure 18: Baseline vs. As-Built Animation 5

4.0 Structural Design Overview

For this project, we examined the differences between two types of structural floor system designs pertaining to the development and construction of multi-story residential buildings. Also through our examination we have identified the construction and design applications of these two different but related floor systems. Our project dealt with evaluating WSU's new residential building "Sheehan Hall" and its modernized construction and design process and comparing it to a more traditional method of construction. The most significant difference between both types of design, of which will be explained in greater detail in this section, is the transformation of building materials from the modern usage of hollow-cored precast concrete planks partnered with dissymmetric open-web steel beams to the most commonly seen and traditional design of a reinforced cast-in-place concrete slab on metal decking. Throughout this project we have outlined the significance of changing a design's construction materials and its impacts to the overall design, construction, cost and scheduling process. For our design, instead of using a girder-slab system with the existing steel frame setup, we were tasked with modifying the existing frame into more of a skeletal steel frame. Skeletal steel frames are designed so that all structural steel can transmit all of the dead loads and live loads from the roof down through the steel beam and column framework and into the foundations (Construction Field, 2011). This type of framework is commonly used today in many commercial and industrial builds as well as for buildings with two or more stories (Construction Field, 2011). Below is a list of many of the advantages as well as the disadvantages for using steel frame construction (Construction Field, 2011):

Advantages

- Consistent material quality
- Light weight and very strong
- Non-Combustible material
- Dimensionally stable in any climate
- Insect resistance very good and steel will not rot
- Can be used to build very tall and wide structures (used in some of the tallest buildings in the world)
- Prefabricated- allows for quick assembly
- Precise and predictable with excellent quality control

Disadvantages

- Steel is an expensive material (more costly than masonry or concrete)
- Frames can become unstable without proper bracing
- Need for fire protection

The next major component of our design is reinforced concrete (C.I.P.) floor slabs. This building material is almost always used with steel-framed buildings (McCormac & Csernak, 2012). The reinforced concrete offers exceptional strength, as well as great fire ratings; furthermore, concrete is noncombustible and provides an insulated barrier between building floors. Consequently, concrete floors are heavy, they require reinforcement to boost its strength properties, and they can be difficult to make waterproof. Below is a list of commonly used concrete floor systems supported by steel frames:

- Concrete slabs supported with open-web steel joists.
- One-way and two-way reinforced concrete slabs supported on steel beams.
- Concrete slab and steel beam composite floors.
- Concrete-pan floors.
- Steel-decking floors.
- Flat slab floors.
- Precast concrete slab floors.

When selecting a concrete floor system there are many factors that must be considered such as loads, fire rating code regulations, sound and heat transmission code regulations, ceiling types, MEP concealment, time restraints for construction, etc. To begin the design process, a floor system is chosen in the architectural design that most adequately meets the project building's requirements in the most economical manner (i.e. the architect selects the best suited floor system at the cheapest construction cost). For WSU's dormitory project "Sheehan Hall" a Precast Concrete Girder-Slab system was chosen as it offers speedy construction, a lighter structure and a more efficient use of construction materials as well as labor. There are many alternatives to this system that may offer other advantages, but at the same time it is important to note that this can substantially affect the project's total cost of construction or even impact the project's schedule. The goal of an architect is to design a building that meets the specified requirements of the owner, while choosing from a multitude of designs. For our Project (as previously indicated above), we proposed an alternative floor system using reinforced CIP concrete on 20 gage metal decking. Additionally, our alternative design adds K-series open-web steel joists on top of our altered steel frame meant for supporting the extra-anticipated weight of our slab on deck system.

4.1 Structural Design Criteria and Baseline Loads

The design loads are critical to the structural analysis of a building. The design criteria for the analysis were provided by the structural engineer and are outlined for the entire building in the structural general notes and schedules (S0.00 in plan set). The applicable code identified is the Massachusetts State building code – 8th edition. The performance requirements for Sheehan Hall are outlined in the project specifications (Structural section). For this project, all structural design calculations will follow the allowable stress design (ASD) requirements and standards. The service-load levels to be used are identified in the structural plans. For a conservative analysis of the floor and roof systems, wind loads were not included in the analysis, as they generally provide uplift to the roof system, which counteracts the gravity loads. Other loads that were neglected for this design were seismic loads, as their major effect is on the frame of a building. The design live loads that were used for analysis can be seen below in Table 2.

Table 2: Design Live Loads

Occupancy	First Floor	Second Floor	Residences	Mechanical Roof
Loads				
Live Load	100	100	40	100

The next step was to determine the dead load for the floor system of the building. To do this, the Vulcraft catalog was used for obtaining the dead load of a 3 3/4 inch normal weight concrete slab on 1 1/2 inch 20-gage steel decking carrying a dead load of 63 psf as indicated by the design load table for 150 Pcf concrete ("Vulcraft steel deck," 2008). The current floor system

on floors two through six consists of normal weight concrete on steel decking. Each of the floors have been subdivided into 3 groups based on their anticipated loads as well as for their expected occupancy/layouts. These groups are used throughout the entire design process as they help to reduce the overall difficulty of the design and the time needed for manual calculations, which can be referenced throughout the appendices section. The groups are as follows: floors one to three, floors four to six and both the lower roof and upper level mechanical penthouse roof.

For our roofs there is a 1 1/2 inch 20 gage steel decking without concrete. The weight of the steel used for each roof dead load was 2.14 psf, which was based on the weight of the steel decking only. The insulation on top of the steel deck is neglected, as the weight of the insulation was not significant. Snow loads were provided by the design criteria located in the structural drawings (S0.00); however the snowdrift calculations were not included resulting in the need of a manual calculation. After our snowdrift load was calculated at maximum intensity using information provided by the Massachusetts State Building Code on snow loads ("Structural Loads,"2001), the maximum drift was then applied to the entire roof system to ensure that the design satisfies the worst possible snow conditions (see **Appendix J**). The maximum load was found to be for leeward drift, which was 151.56 psf, which is a very substantial live load for the design of a roof system. This load would be applied to the entire roof area as a snow load and additionally, the snowdrift calculations need to be included for review as specified in ASCE 7 (Minimum Design Loads for Steel Buildings), a standard code for the design of snow loads.

After determining all the service loads for the floors and roof systems, the ASD load combinations provided in the IBC (International Building Code) section 1605.3.1 were then used for determining our factored combined loads. After obtaining our combined factored loads for

all levels, we were then able to begin the structural design process. Below is a 3-Dimensional representation (**Figure 19**), created using Revit 2014, of our complete alternative design which includes the following features: Our CIP concrete slabs on metal deck, our modified skeletal steel framing system and our new addition of the K-series open-web steel joists.

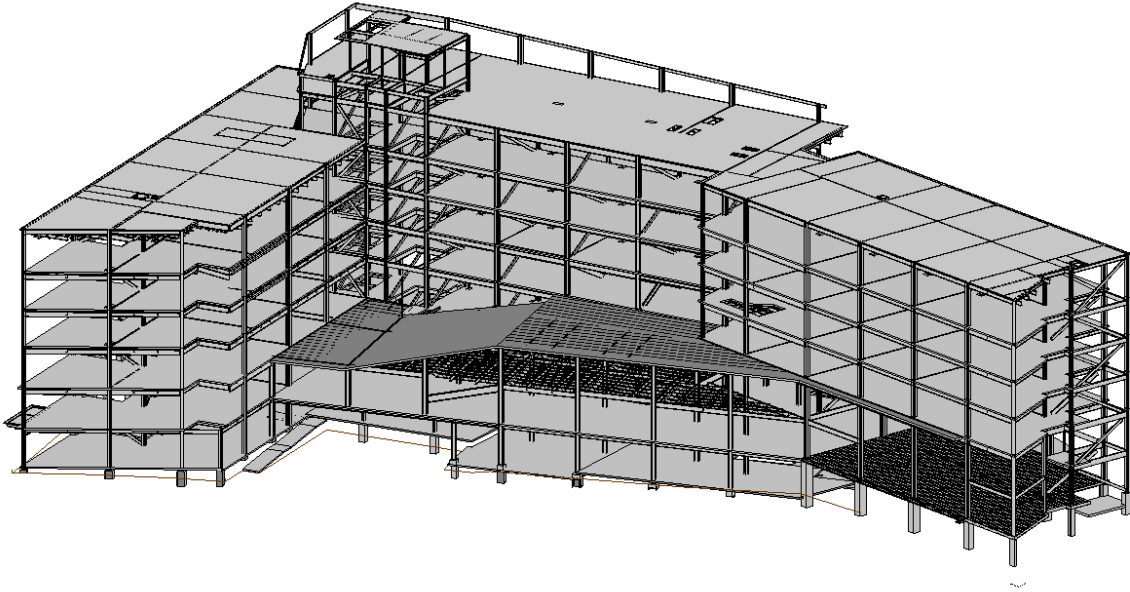


Figure 19: Complete Revit Structural Model

Design Load Criteria				
(D) total design dead load (Psi)	47.14	124	81.1	37.1
E	0	0	0	0
F	0	0	0	0
L (Psi)	0	0	0	0
Lr	0	100	0	0
S	151.56	0	151.56	151.56
W(Neglected)	0	0	0	0
R (normal load due to rainwater or Ice)	0	0	0	0
DL + LL	198.7	224	232.66	188.66

Table 3 Design Load Criteria

Floors 1-3: (30'x30') ASD Load Combinations- Baseline		
Equations	Office	Residence
D	76	76
D + L	176	114
D + (Lr or S or R)	76	74
D + .75(L) + .75(Lr or S or R)	151	104
D + (.6W or .7*E)	76	74
D + .75(.6W or .7E) + .75(L) + .75(Lr or S or R)	151	104
.6(D) + .6(W)	45.6	44.4
.6(D) + .7(E)	45.6	44.4
Governing Load	176	114

Table 4: Factored Loads for Levels 1 - 3

Our design loads were provided by the structural engineer and shown in the structural detail sheet (S0.00). The tables above are used to illustrate the most critical loading scenario for levels one through three (Table 2) and also illustrates the design criteria for all load types used for obtaining our factored load (Table 3). This table only refers to the first three levels because they are assumed to experience similar types of loads as a result of their architectural design along with their expected usage and occupancy. After we calculated our critical loading case, which resulted from our combined dead and live loads, we concluded that the office areas were our critical loading area. Finally, after finding where our most critical loads would be located, we designed each of the first three levels to withstand our factored load of 176 psf through both the steel frame and joists design process.

For this project it was important that we checked the capacity of all existing steel members because our alternative design was expected to be slightly heavier as a result of our floor's material change from hollow-cored precast planks to a CIP reinforced concrete slab. Therefore, we assumed that our alternative design would receive higher gravity loads to be supported by the frame, which would ultimately make the existing frame design less desirable while also creating the possibility for failure to occur. To avoid any type of failure from occurring, we conservatively addressed this

concern by reselecting new steel members with higher capacities and replaced nearly all of the existing steel with them, ultimately resulting in a partially new frame design.

Our dead load used during the design of the first three levels consisted of the following components: the weight of our reinforced concrete slab and the 20 gage steel decking; furthermore we tried to enhance the precision of our design by factoring an additional 25 psf to account for other attributes of the building such as ceilings, mechanicals, ECT. This same approach would be taken for the design of the next 3 levels with the only difference being in the amount of loading, which can be seen in the corresponding load table in **Appendix J**.

Table 5: Factored Loads for levels 4 - 6

Floors 4-6: (30'x30') ASD Load Combinations- Baseline		
Equations	Office	Residence
D	76	76
D + L	176	116
D + (Lr or S or R)	76	76
D + .75(L) + .75(Lr or S or R)	151	106
D + (.6W or .7*E)	76	76
D + .75(.6W or .7E) + .75(L) + .75(Lr or S or R)	151	106
.6(D) + .6(W)	45.6	45.6
.6(D) + .7(E)	45.6	45.6
Governing Load	176	116

4.2 Structural Steel Frame

Table 6 Steel Frame Design Levels 1 - 3

Floors 1-3 Frame Design													
Bay Size	Trib. Spacing (ft)	Span Length L (ft)	W_n (klf)	M_{max} (k-ft)	$M_n = (F_y * Z_x)$ (ksi)	I_x (in ⁴)	$Z_x, Actual$ (in ³)	$Z_{x,Min} = (1.67 * M_{max}) / F_y$ (in ³)	$M_{allow} = (M_n / 1.67)$ (Kips)	Deflection Check:1 =D ≤ (L/240)	Deflection Check:2 =D ≤ (L/360)	Beam Selection & (#) of members	Total Wt of Steel (Lbs)
28'-9" x 29'-2 1/4"	7.30	28.75	1.54	158.745	277.083	510.0	66.50	63.62	165.918	0.08148	0.01504	(4) W18x35	4025
28'-9" x 22'-7 3/4"	7.55	22.65	1.54	98.561	277.083	510.0	66.50	39.50	165.918	0.02979	0.00579	(3) W18x35	2378.25
29'-2 1/4" x 24'-0"	7.30	24.00	1.46	105.467	197.083	291.0	47.30	42.27	118.014	0.06433	0.01280	(4) W14x30	2880
22'-7 3/4" x 24'-0"	8.00	22.65	1.63	104.816	197.083	291.0	47.30	42.01	118.014	0.05251	0.01015	(3) W14x30	2038.5
24'-0" x 20'-7 5/8"	8.00	20.64	1.61	85.969	179.583	238.0	43.10	34.46	107.535	0.04341	0.00856	(3) W12x30	1857.6
29'-6" x 22'-7 3/4"	7.40	22.65	1.47	94.449	179.583	238.0	43.10	37.86	107.535	0.06158	0.01241	(4) W12x30	2718
29'-6" x 20'-7 5/8"	7.40	20.64	1.46	77.626	179.583	238.0	43.10	31.11	107.535	0.04177	0.00856	(4) W12x30	2476.8
28'-9" x 23'-11"	7.20	23.92	1.45	103.450	197.083	291.0	47.30	41.46	118.014	0.06358	0.01262	(4) W14x30	2869.92
13'-9" x 20'-7 5/8"	6.90	13.75	1.32	31.097	122.083	156.0	29.30	12.46	73.104	0.01190	0.00257	(3) W12x22	907.5
13'-9" x 8'-6 5/8"	4.60	8.55	0.87	7.974	122.083	156.0	29.30	3.20	73.104	0.00176	0.00038	(3) W12x22	564.3
9'-3" x 22'-7 3/4"	7.55	9.25	1.40	14.937	122.083	156.0	29.30	5.99	73.104	0.00232	0.00053	(3) W12x22	610.5
9'-3" x 29'-2 1/4"	7.30	9.25	1.34	14.286	122.083	156.0	29.30	5.73	73.104	0.00228	0.00053	(4) W12x22	814

The design of the Structural steel frame on the Roof level was designed to accommodate the critical loading case being the "Green Roof" as it is exposed to the highest total load of the three types of roofs listed above and also, that its design loads will be used throughout all roof calculations for the entirety of the roof level (with the exception of the Mechanical Penthouse Roof). The reason for not including the Mech. Penthouse roof in the table above is mainly due to the types of loading that each of the different roofs would need to be designed to withstand. As it can be seen in the above tables entitled, "30'x30' ASD Load Combinations Baseline", the only roof not experiencing a Snow load of 151.56 psf is the Mechanical Penthouse Roof. Additionally, it can also be seen that the mechanical penthouse roof is again the only roof experiencing a roof live load of 100 psf. Each roof is exposed to a dead load that varies from 47.14 psf to 124 psf, this variation corresponds to the type of roof and its required building materials and components. Each of the design loads discussed above were used throughout all steel frame calculations on the roof level.

4.2.1 Structural K-Series Open-Web Joist

The use of open-web steel joists are a very common practice for steel-frame buildings as they allow for easier installation of metal decking and a stronger supporting surface for concrete slabs to be placed on (McCormac & Csernak, 2012). These k-series joists consist of small parallel chord trusses that are made up of members of bar, small angles, or other rolled steel shapes, as displayed in **Figure 20** below. Steel decking is then typically attached to the joists through a welded or self-drilled/self-tapped screw connection. The use of steel joists is a very economical and lightweight type of concrete floor system. Additionally, Open-web steel joists are ideal for relatively light loads and structures that do not have much vibration (McCormac & Csernak, 2012). They are well suited for low-level buildings, but they can be used in tall building constructions as well. The bar joists must be braced laterally to prevent twisting and buckling, using either horizontal rods fastened to the top and bottom chords of the joists or diagonal cross bracing.

The use of open-web steel joists are a very common practice for steel-frame buildings as they allow for easier installation of metal decking and a stronger supporting surface for concrete slabs to be placed on (McCormac & Csernak, 2012). These k-series joists consist of small parallel chord trusses that are made up of members of bar, small angles, or other rolled steel shapes, as displayed in Figure -. Steel decking is then typically attached to the joists through a welded or self-drilled/self-tapped screw connection. The use of steel joists is a very economical and lightweight type of concrete floor system. Additionally, Open-web steel joists are ideal for relatively light loads and structures that do not have much vibration (McCormac & Csernak, 2012). They are well suited for low-level buildings, but they can be used in tall building constructions as well. The bar joists must be braced laterally to prevent twisting and buckling,

using either horizontal rods fastened to the top and bottom chords of the joists or diagonal cross bracing.

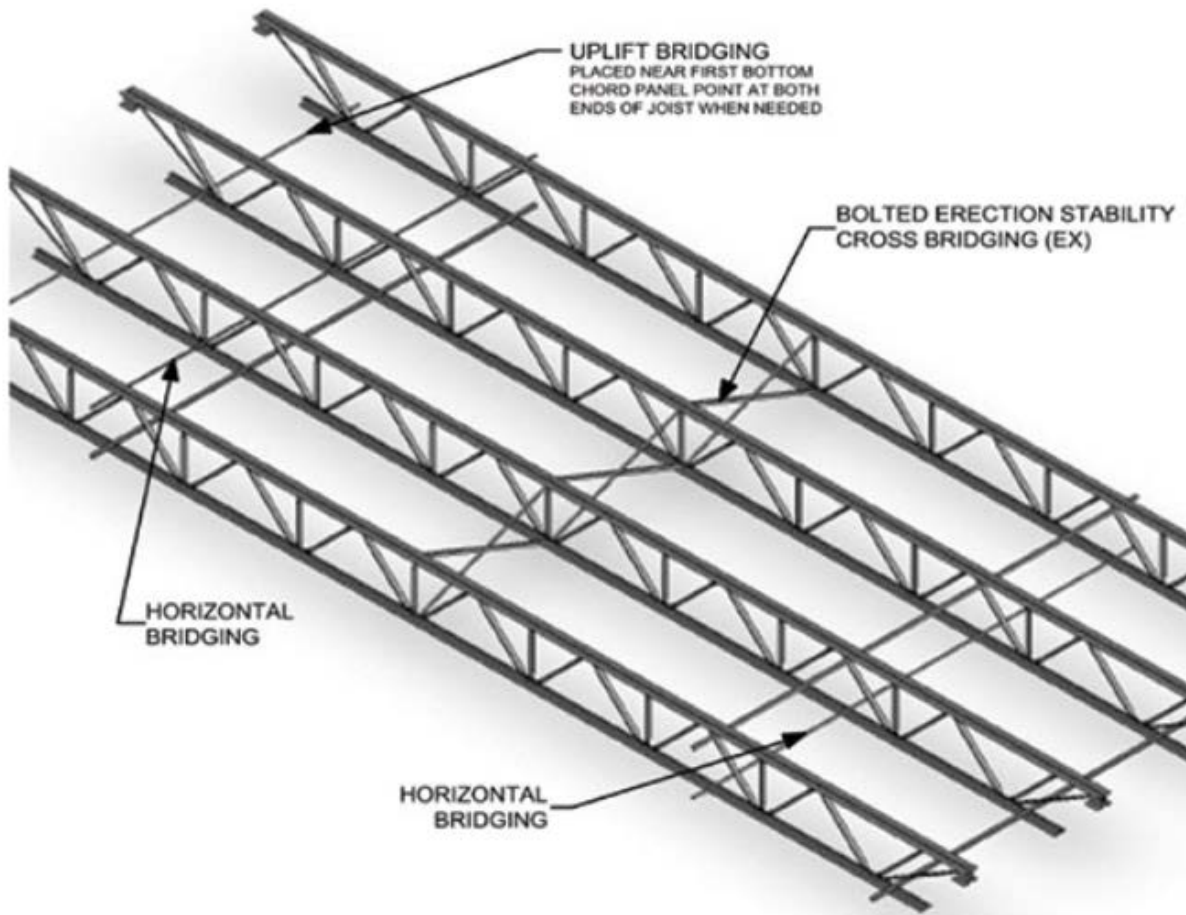


Figure 20: K-Series Open-Web Joist Diagram

Open-web joists are very quick to erect and easy to handle. Furthermore, they provide open spaces in the web that can be used to conceal MEP (McCormac & Csernak, 2012). They also offer the ability to accommodate a variety of geometric configurations that a typical steel beam cannot. The open-web bar joists offer advantages that an I-beam or other typically used steel beam cannot. However, there are some disadvantages to open-web bar joists. For instance, they need to be pre-manufactured for the job, and may not offer the same desirable strength

capacity as an I-beam. It is very important to account for all possible design approaches along with any advantages/disadvantages they could have before choosing any kind of steel to support a floor system.

4.2.2 Joist Design

Open-web bar joists are designed by a number of different joist manufacturers. The Steel Joist Institute (SJI) is a United States based nonprofit organization of active joist manufacturers who address the lack of uniform joist standards for the industry (Steel Joist Institute, 2010). SJI also offers seminars along with a multitude of training and research aids. After looking into the information offered by the SJI a manufacturer of the steel needed to be chosen, as the bar joists are unique to their manufacturer. Some of the major manufacturers that are recognized by the SJI are Nucor Vulcraft, Canam Steel Corp, and SMI Joist Company (Delhi University). For this project our group chose to reference Nucor Vulcraft Group information in the design of the joist floor system as they have a large number of catalogues available online.

A major advantage of using Vulcraft was that they offer multiple catalogues for each of the different types of steel products that they produce. Three main catalogues that we used were the Composite and Non-composite Joists, Steel Joists and Girders, and Steel Roof and Floor Deck. Using the steel deck catalogue a dead weight for the 20 gage steel and normal weight concrete slab was found to be 63 pounds square foot ("Vulcraft steel deck," 2008). This number would be carried throughout the design of the floor systems. The catalogue was also used to determine the weight of the steel deck for the purpose of calculating the roof dead load. The other design loads to be used in the selection of the open-web bar joists would be the building's service live loads, snow loads, and snowdrift loads discussed previously. All other loads were neglected, as they do not have a major effect on the design and performance of the floor systems.

The design of the floor system consisted of K series standard joists that range from 16 inches to 22 inches in depth due to the floor-ceiling clear height that maintained as much as possible instead of choosing the lightest selection. The first step in the design of the joists is determining what size joist to use. To do so the spacing between the joists is determined so that the joists will satisfy the magnitude of the floor load over a given span. The spacing can range from 2 feet to 10 feet (Ching, 2008). Furthermore, the joists spans are limited to 24 times their depth. The spacing of the joists used in our design was the maximum possible spacing for a joist spanning a given distance in order to minimize the number of joists. **Figure 21** below shows K series joists and the different types of bridging.

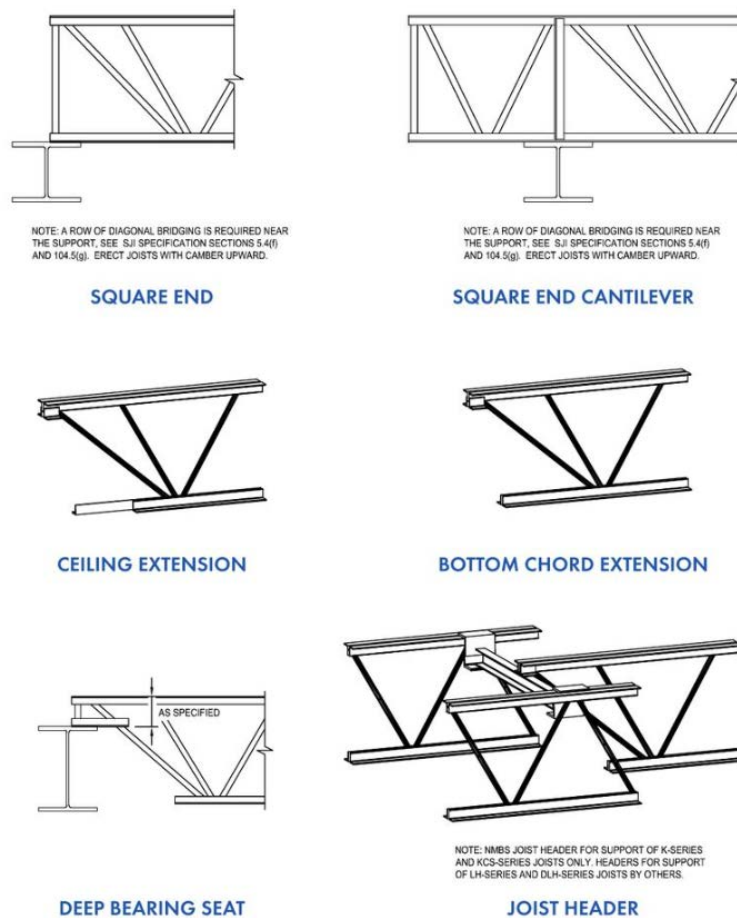


Figure 21: Types of Steel Joist Bridging

It is important to verify that the steel decking is able to support the newly designed joist spacing for both the decking and the slab. As specified in the Vulcraft Steel Deck catalogue, it provides alternative live load information for reinforced concrete slabs. It can be seen in the catalogue that for the 20 gage galvanized steel decking the capacity increases with a decrease in the clear span between beams. We confirmed that the steel decking was sufficient for up to a 12-foot clear span in our revit model; therefore our decking would then be sufficient for our smaller designed spacing. The spacing of the joists range from 2 ½ feet to about 3 feet, so the 20 gage steel would easily be able to maintain its structural integrity, also suggesting that the use of a lighter steel decking could possibly be used of support the loads at a 3 foot span. However, for the design of this system the 20 gage steel decking will be used in the structural analysis.

After determining the spacing of the joist the next step was selecting the correct joist to be used. This was done by listing the possible joists that met the load criteria at a proposed spacing and choosing the option with the smallest depth due to the architectural restriction mentioned above. The Vulcraft catalogue was used to find the possible joists over a given span. For a given joist size, the catalogue would display a maximum total allowable load for the corresponding joist (listed in black) and also the maximum allowable live load (listed in red), as a function of the span which can be seen below in **Figure 22**.

For rectangular bays to achieve the design's maximum strength, the joists were oriented to span in the short direction and are to be spaced along the long dimension (Lecture 18-open web). The design of a 30-foot by 30-foot bay was used to review this guideline as it would ensure that our 28' 9" by 29' 2 1/4 " would be more than adequate. This was chosen, as the width and length of the bays are fairly similar. After checking manually, it was confirmed that the joists

are more efficient when orientated in the short spanning direction. An important note is that when there is a greater difference in length and width, that it will only be magnified. For all other designs the spanning direction was carried out in each bay design, and a bar joist was chosen for each bay as seen in **Figures 23, 24, 25, 26, 27, 28, 29, and 30. Table 7** below is a summary of all of the open-web joist designs for the bay systems of the building. The column outside the figure shows the reference page in **Appendix I** for review of the supporting calculations.

ASD

STANDARD LOAD TABLE FOR OPEN WEB STEEL JOISTS, K-SERIES
Based on a 50 ksi Maximum Yield Strength - Loads Shown in Pounds Per Linear Foot (plf)

Joist Designation	18K3	18K4	18K5	18K6	18K7	18K9	18K10	20K3	20K4	20K5	20K6	20K7	20K9	20K10	22K4	22K5	22K6	22K7	22K9	22K10	22K11
Depth (In.)	18	18	18	18	18	18	18	20	20	20	20	20	20	20	22	22	22	22	22	22	22
Approx. Wt. (lbs./ft.)	6.4	7.2	7.7	8.4	8.9	10.1	11.6	6.5	7.2	7.7	8.4	8.9	10.1	11.6	7.3	7.7	8.5	9.0	10.2	11.7	11.9
Span (ft.)																					
↓																					
18	550 550	550 550	550 550	550 550	550 550	550 550	550 550														
19	514 494	550 523	550 523	550 523	550 523	550 523	550 523	550 550	550 550	550 550	550 550	550 550	550 550	550 550							
20	463 423	550 490	550 490	550 490	550 490	550 490	550 490	517 517	550 550	550 550	550 550	550 550	550 550	550 550							
21	420 364	506 426	550 460	550 460	550 460	550 460	550 460	468 453	550 520	550 520	550 520	550 520	550 520	550 520	550 550	550 550	550 550	550 550	550 550	550 550	550 550
22	382 316	460 370	518 414	550 438	550 438	550 438	550 438	426 393	514 461	550 490	550 490	550 490	550 490	550 490	550 548	550 548	550 548	550 548	550 548	550 548	550 548
23	349 276	420 323	473 362	516 393	550 418	550 418	550 418	389 344	469 402	529 451	550 468	550 468	550 468	550 468	518 491	550 518	550 518	550 518	550 518	550 518	550 518
24	320 242	385 284	434 318	473 345	526 382	550 396	550 396	357 302	430 353	485 396	528 430	550 448	550 448	550 448	475 431	536 483	550 495	550 495	550 495	550 495	550 495
25	294 214	355 250	400 281	435 305	485 337	550 377	550 377	329 266	396 312	446 350	486 380	541 421	550 426	550 426	438 381	493 427	537 464	550 474	550 474	550 474	550 474
26	272 190	328 222	369 249	402 271	448 299	538 354	550 361	304 236	366 277	412 310	449 337	500 373	550 405	550 405	404 338	455 379	496 411	550 454	550 454	550 454	550 454
27	252 169	303 198	342 222	372 241	415 267	498 315	550 347	281 211	339 247	382 277	416 301	463 333	550 389	550 389	374 301	422 337	459 367	512 406	550 432	550 432	550 432
28	234 151	282 177	318 199	346 216	385 239	463 282	548 331	261 189	315 221	355 248	386 269	430 298	517 353	550 375	348 270	392 302	427 328	475 364	550 413	550 413	550 413
29	218 136	263 159	296 179	322 194	359 215	431 254	511 298	243 170	293 199	330 223	360 242	401 268	482 317	550 359	324 242	365 272	398 295	443 327	532 387	550 399	550 399
30	203 123	245 144	276 161	301 175	335 194	402 229	477 269	227 153	274 179	308 201	336 218	374 242	450 286	533 336	302 219	341 245	371 266	413 295	497 349	550 385	550 385

Figure 22: Standard ASD Steel Joist Design Load Table

Table 7: Proposed Open-Web Bar Joists

Proposed Open-Web Bar Joists									
	Bay Size	Live Load	Proposed Selection	Wt / Ft (lb/ft)	Span Length	Spacing	# of Joists	Total (Lbs) of Joists	
Roof	24'-0" x 22'-7 3/4"	188.78	16K9	10	24	2.517	9	2160	
	13'-9" x 29'-2 1/4"	171.90	20K10	11.6	29.19	2.292	6	2031.624	
	10'-3" x 30'-0 3/8"	192.23	22K10	11.7	30.0313	2.563	4	1405.4648	
	18'-11 1/4" x 30'-0 3/8"	202.95	22K10	11.7	30.0313	2.706	7	2459.5635	
	24'-0" x 29'-2 1/4"	200.03	22K10	11.7	29.19	2.667	9	9221.121	
	28'-9" x 22'-7 3/4"	188.78	20K10	11.6	28.75	2.517	9	3001.5	
	28'-9" x 29'-2 1/4"	196.05	20K10	11.6	29.19	2.614	11	3724.644	
	9'-3" x 22'-7 3/4"	173.48	16K7	8.6	22.65	2.313	4	779.16	
	9'-3" x 29'-2 1/4"	173.48	20K10	11.6	29.19	2.313	4	1354.416	
	29'-6" x 29'-2 1/4"	199.05	22K10	11.7	29.5	2.654	11	3796.65	
Mechanical Penthouse Roof	10'-3" x 10'- 0"	200.00	16K9	10	10.25	2	5	512.5	
	13'-9" x 12'-7 3/4"	180.70	20K10	11.6	13.75	1.807	7	1116.5	
	10'-3" x 6'-1 1/4"	152.50	22K10	11.7	10.25	1.525	4	479.7	
	10'-3" x 12'-0"	170.80	22K10	11.7	12	1.708	6	842.4	
	13'-9" x 10'-0"	200.00	22K10	11.7	13.75	2	5	804.375	
	9'-3 7/16" x 10'-0"	185.80	18K10	11.6	10	1.858	5	580	
	11'-3 7/16" x 12'-7 3/4"	188.30	20K10	11.6	12.65	1.883	6	880.44	
	28'-9" x 29'-2 1/4"	292	20K10	11.6	29.19	2.92	10	3386.04	
	28'-9" x 22'-7 3/4"	251.7	20K10	11.6	28.75	2.517	9	3001.5	
	29'-2 1/4" x 24'-0"	300	22K10	11.7	29.19	3	8	8196.552	
Floors 4-6	22'-7 3/4" x 24'-0"	283	16K9	10	24	2.83	8	1920	
	24'-0" x 20'-7 5/8"	300	16K9	10	24	3	8	1920	
	29'-6" x 22'-7 3/4"	283	20K10	11.6	29.5	2.83	8	2737.6	
	29'-6" x 20'-7 5/8"	294	20K10	11.6	29.5	2.94	7	2395.4	
	28'-9" x 23'-11"	266	18K10	11.6	28.75	2.66	9	3001.5	
	13'-9" x 20'-7 5/8"	294	16K6	8.1	20.635	2.94	7	1170.0045	
	13'-9" x 8'-6 5/8"	275	12K1	5	13.75	2.75	5	343.75	
	9'-3" x 22'-7 3/4"	283	16K7	8.6	22.65	2.83	8	1558.32	
	9'-3" x 29'-2 1/4"	292	20K10	11.6	29.19	2.92	10	3386.04	
	28'-9" x 29'-2 1/4"	292	20K10	11.6	29.19	2.92	10	3386.04	
	28'-9" x 22'-7 3/4"	251.7	20K10	11.6	28.75	2.517	9	3001.5	
	29'-2 1/4" x 24'-0"	300	22K10	11.7	29.19	3	8	8196.552	
	22'-7 3/4" x 24'-0"	283	16K9	10	24	2.83	8	1920	
	24'-0" x 20'-7 5/8"	300	16K9	10	24	3	8	1920	
	29'-6" x 22'-7 3/4"	283	20K10	11.6	29.5	2.83	8	2737.6	
29'-6" x 20'-7 5/8"	294	20K10	11.6	29.5	2.94	7	2395.4		
Floors 1-3	28'-9" x 23'-11"	266	18K10	11.6	28.75	2.66	9	3001.5	
	13'-9" x 20'-7 5/8"	294	16K6	8.1	20.635	2.94	7	1170.0045	
	13'-9" x 8'-6 5/8"	275	12K1	5	13.75	2.75	5	343.75	
	9'-3" x 22'-7 3/4"	283	16K7	8.6	22.65	2.83	8	1558.32	
	9'-3" x 29'-2 1/4"	292	20K10	11.6	29.19	2.92	10	3386.04	
	Total Wt of Open-Web Bar Joists (lbs) -Roof							33013.94331	
	Total Wt of Open-Web Bar Joists (lbs) -Mech. Roof							5215.915	
	Total Wt of Open-Web Bar Joists (lbs) -Floors 1 - 3							33016.7065	
	Total Wt of Open-Web Bar Joists (lbs) -Floors 4 - 6							33016.7065	
	Total Wt of Proposed Open-Web Bar Joists (lbs)							104263.2713	



Figure 23: Level One Floor Plan

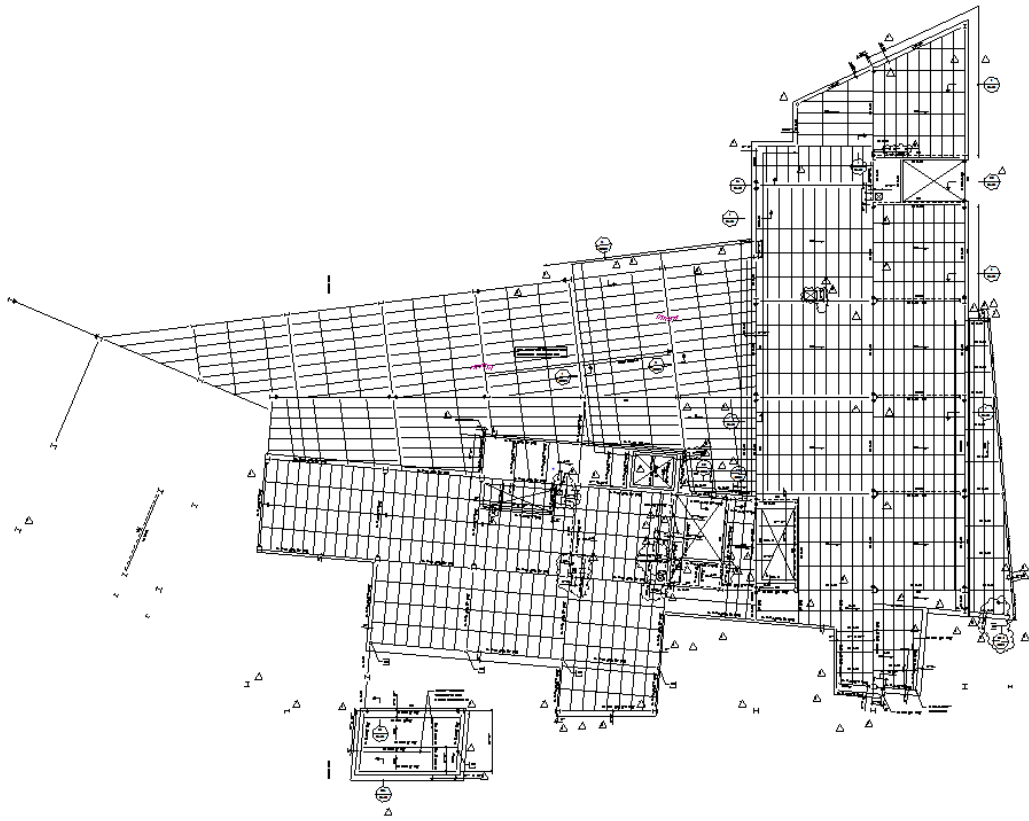


Figure 24: Level Two Floor Plan

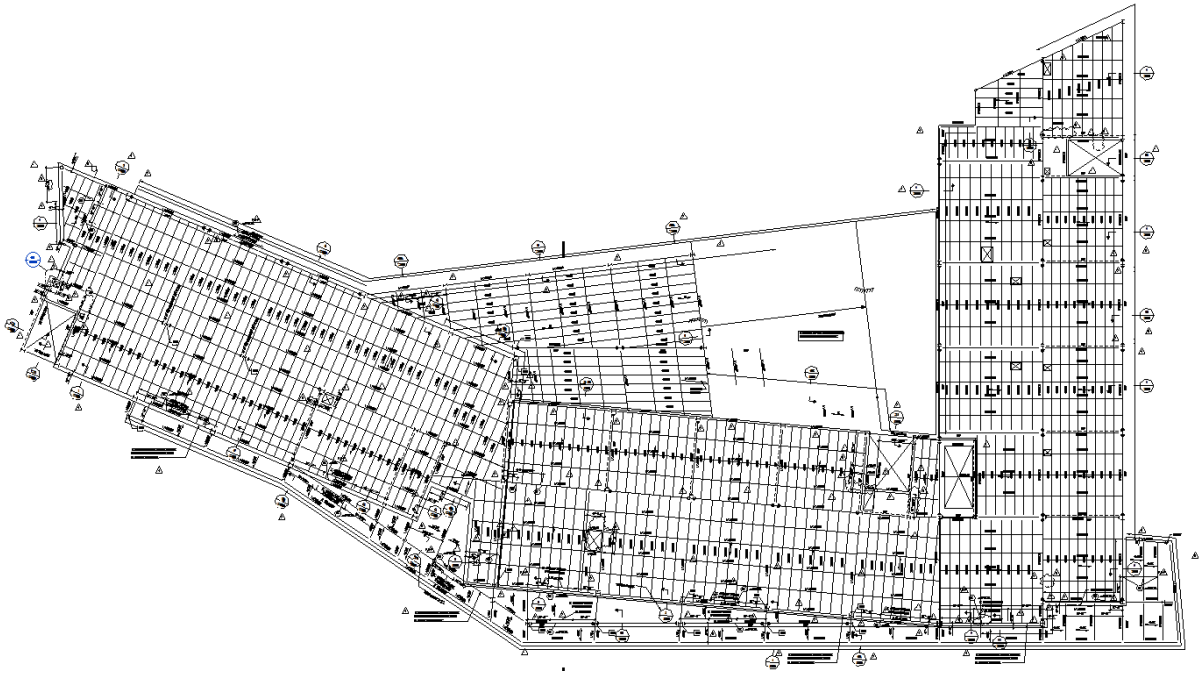


Figure 25: Level Three Floor Plan

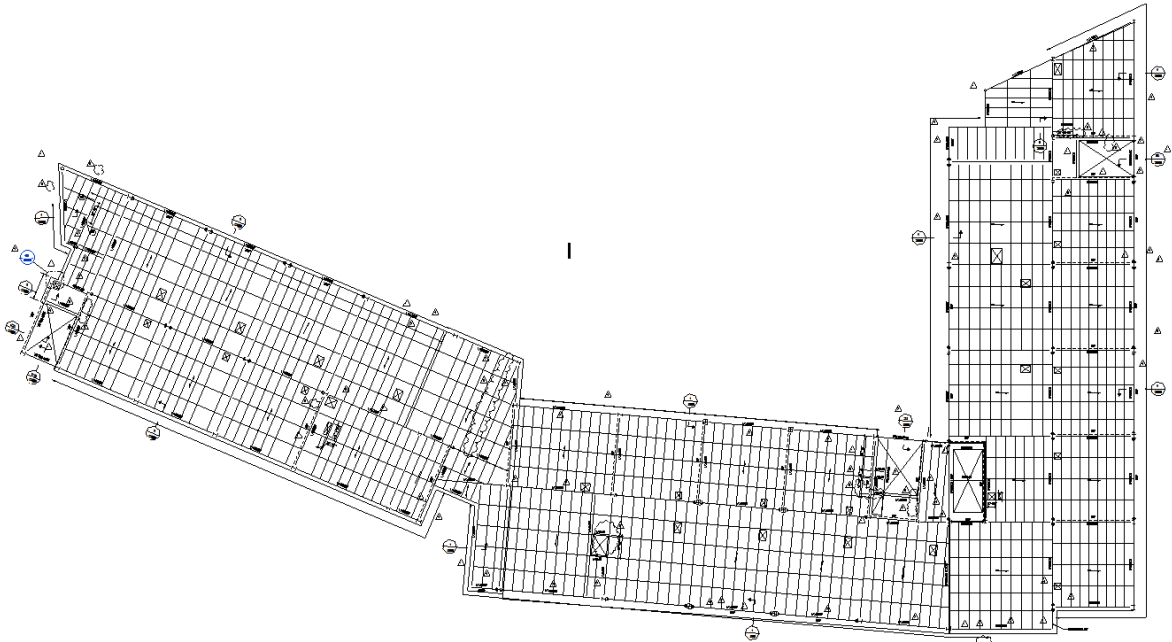


Figure 26: Level Four Floor Plan

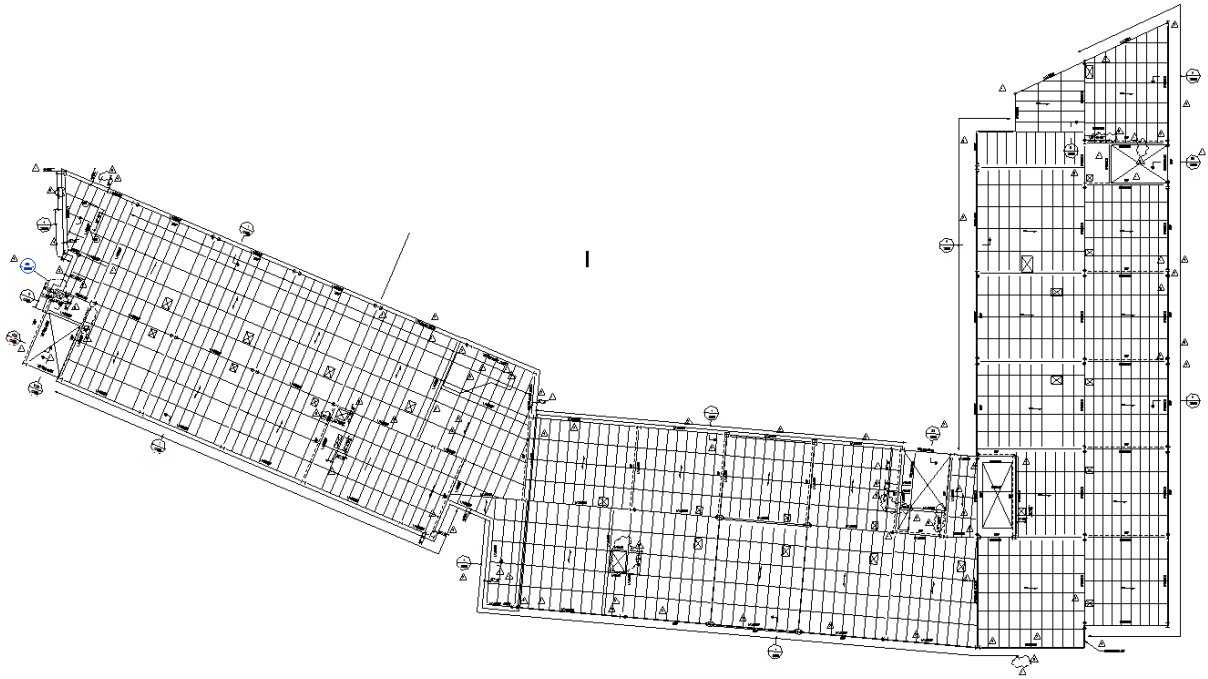


Figure 27: Level Five Floor Plan

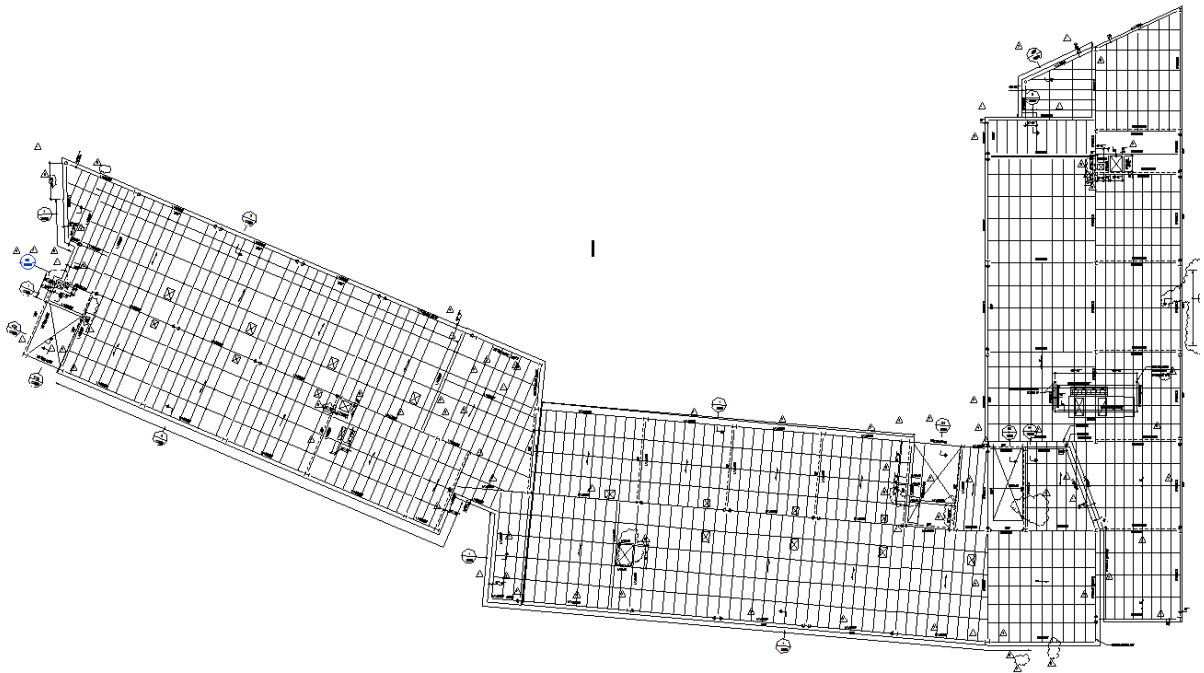


Figure 28: Level Six Floor Plan

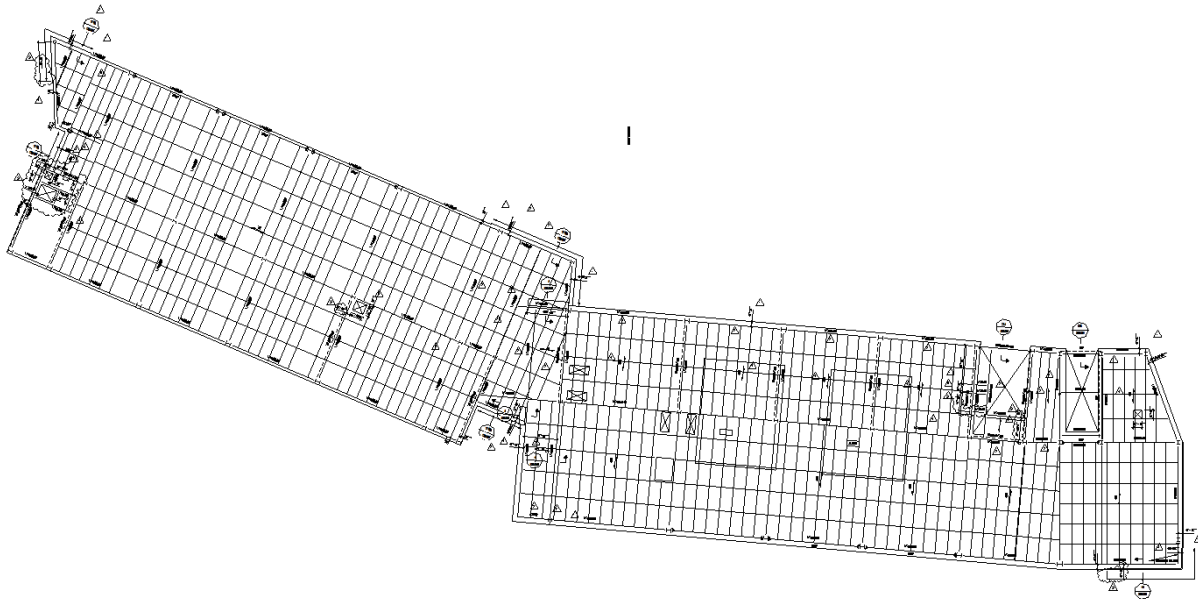


Figure 29: Level Seven Floor Plan

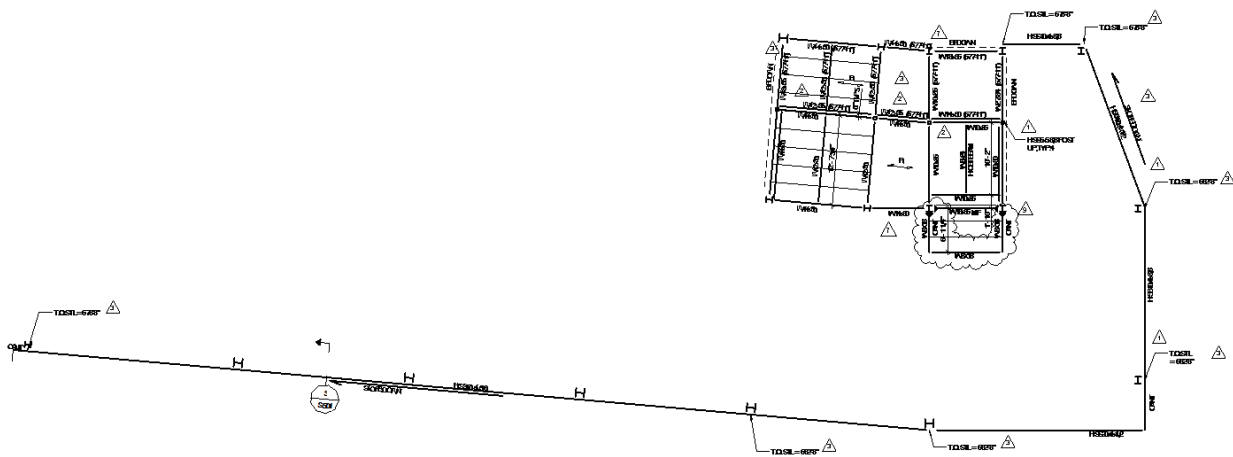


Figure 30: Level 7 Mechanical Roof Plan

It can be seen in the figures above and also for the design of the open web joist system that some bays do not have open-web bar joists. Not all bays in the building are rectangular; therefore the baseline I-beam system was kept intact to ensure it maintained is structural

integrity. There are many openings throughout each of the floor systems which were meant to accommodate for ductwork, elevators, stairs, etc. which provided complications to our design design and also to our revit model. Also, the building has many edges with awkward geometry that proved to even further complicate the design process for our open web steel joists. To reduce the complexity of our design these bays with special geometry were also not altered, but the original beam designs were instead just replicated. The entire design of the floor system was changed in order to incorporate the open web steel joists into our alternative floor system for our building. The intent of this project is to evaluate the differences between the concrete slab on deck with open-web joists and the girder-slab system with hollow-core precast planks using D-beams using as much of the original design as possible to help magnify the effects of changing the concrete floor material to the rest of the project.

Another important factor for the design of open-web bar joists is horizontal or diagonal bridging to prevent any lateral movement of joist chords (Ching, 2008). Bridging can be seen above in **Figure 21**. Bridging was designed similarly to the design of joists with the use of Vulcraft catalogues that were able to specify all necessary bridging requirements. This was not completed for all of the bays however, as it is not necessary in the scope of our group's structural analysis along with our Revit model. It is just important to note that in an actual joist design, bridging in open-web bar joist systems should be properly specified to stay in accordance with the SJI and also the ASD regulations.

4.3 Structural Columns

In order to analyze the load on each column and determine the required strength a spreadsheet was created. The complete spreadsheet can be found in **Appendix J**. The first step in the analysis was to organize the columns by, locations, original sizes and their lengths.

Next Additional Tabs were created based on their similarity of the calculations and also by bay size. When using revit to evaluate the 3-D structure, all the columns are modeled and placed on intersecting grid lines where their location can then be recorded. After identifying all the critical bay sizes by wing or by typical bay sizes, we then began to gather necessary information (like column sizes, lengths, ect.) needed in order to be able to analyze each of the columns' required capacities for our alternative design. The analysis was first completed on the first level of the building because the columns on the first floor are the most critical since they are responsible for supporting the entire weight of all the floors above. For our project we were looking only to increase column capacities where it was necessary without having to change locations or creating an alternative column design; additionally, this part of our structural analysis was only needed due to our anticipation of our design yielding higher gravity loads and thereby yielding an overall heavier structure that is located within the column spreadsheet are many tabs of calculations pertaining to different critical loading areas where we have determined our alternative design will be impacted the most. It can also be seen that inside the spreadsheet that not all columns were included in calculations which was purposely done to help expedite this part of the analysis as there are many repeating sections of the design. The first table called load calculations (**Table 8**) sums the loads of the concrete slab, metal decking and all surrounding steel beams, girders and joists.

Table 8: Load Calculations

Load Calculations			
Concrete	Thickness (ft)	Weight (lb/cu ft)	Load (psf)
Slab	0.4375	145	63.438
Steel	Weight (lb/ft)	Length (ft)	Weight (lb)
Girder Below	30	11.32	339.6
Girder Above	30	11.32	339.6
Girder Left	30	12	360
Girder Right	30	12	360
Extra Beams	30	6	1080
Joists	22.2	12.9599	287.70978
Total			2766.9098
All Floors			16601.459
Column	170	23.33	11898.3

The first section calculates the expected weight of the cast-in-place concrete slab. This calculation is based on our slab being 3 ¼ inches and the weight of our reinforced concrete being 150 pounds per cubic foot. We obtained a total load for our concrete slab to be 40.6 psf. The next section totals the beams, girders, and joists whose weight is being carried by the column around it. Below in Figure 12, we provided an example of this calculation process by using column BB-12 and all the surround steel members.

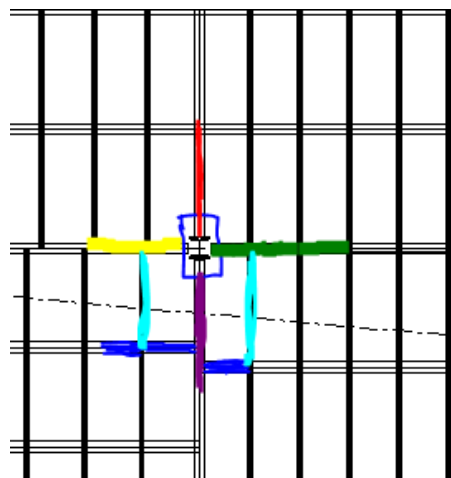


Figure 31: Example Column BB-13

The Blue box identifies Column BB-13. The Green Girder is what is referred to as girder right and the yellow girders are is referred to as girder left. The Purple and Red Girders are respectfully referred to as girder below and girder above. The blue beams are referred to as large and small beams. Finally, all members that are similar to those highlighted in Cyan are referred to as joists. This table contains the weight (plf) of the girders and beams, the length of the members being supported by column BB-13. From these two properties we can calculate the total weight in pounds that the column must be able to support. For girders, the column will carry half of the length because these members are connected between two columns by which each half carries loads to its nearest column. For each Surround beam located in the frame this figure (the column's tributary area in both the vertical and horizontal directions) the columns is responsible for carrying a quarter of the member's length because these beams are connecting two girders which distributes its weight between four columns. For joists, their total weight being carried by column BB-13 is similar to the beams in that of the total length of each joist located inside the tributary area only transfers a portion of its weight to the nearest column. Located near the bottom of **Table 8** is the total weight in pounds of the beams, girders and joists and then multiplies this by the number of floors above which is 6 due to there being 6 levels including one roof above column BB-13. Lastly, the bottom row contains the weight of the column, the height of the column above the first floor, and therefore calculates the weight of the column that the first floor column BB-13 must be able to support.

The next table called Variables involves the length of the column and the areas of the floor that the column supports. An example of this can be seen below with BB-13's calculation (**Table 9**). The first row demonstrates the length of the column that is on the first floor. The next four rows are used to sum up the lengths in the x and y direction of the floor that column BB-13

supports. In order to find the total Column tributary area, which represents the area for which the column is designed to support, it is typically half of the length between each column both vertically and horizontally. Also, this tributary area can be visualized above in figure 12 by assuming the exterior edge of the image on all sides is the combined tributary area of column BB-13 as it roughly half way between all surrounding columns. To calculate the total tributary area we multiplied both combined lengths in the y and x directions and obtained an area of 565.056 square feet.

Table 9: Column Tributary Area

Variables	Feet
Height	21.1133
Total Tributary Width (x)	21.8
Tributary Width (y)	25.92
Tributary Area	565.056

This area is then used in determining the total dead load in pounds that column BB-13 contains, an example of this is shown below in **Table 10**. The snow load and roof loads were both given by the Structural Engineers except for snow drift calculations as mentioned previously in design load section 4.2. The concrete load is based on the concrete slab that was calculated in **T**. The metal deck, ceilings, insulation, and MEP loads are typically used in most Structural analysis (McCormac, 41 -42). The row referred to as dead load totals the concrete slab, metal deck, ceilings, insulation and MEP. The row called Largest Lr, S, or R refers to the largest roof load, which changes depending on the location of the column. The last few rows of this table refer to the live loads of the building. For typical residential buildings these loads are about 100 pounds per square foot for common areas/office areas and about 40 pounds per foot for living areas and these values are usually used throughout the building in their respective areas

(McCormac, 43). The third column of the table multiplies the pounds per square feet of the second column by the tributary area the column supports in order to convert the total weight into pounds that the column carries. It is important to note that the tributary area for each column varies and must be recalculated for each column that is checked. Lastly, the fourth column multiplies the weight per foot by the number of floors above it to give the total load being supported by the first level.

Table 10: Total Design Load Criteria for Columns

Loads	psf	lbs	Upper Floors
Snow Load	151.56	85639.88736	
Roof Load	47.14	26636.73984	
Concrete	65.438	36976.13453	221856.81
Metal Deck	2.14	1209.21984	7255.319
Ceiling	1.5	847.584	5085.504
Insulation	1.5	847.584	5085.504
MEP	5	2825.28	16951.68
Total Dead Load	75.578	71205.56105	427233.37
Largest Lr, S, or R	151.56	85639.88736	
Live Load 1	100	56505.6	339033.6

The last table used for the column analysis (**Table 11**) sums up all the calculation results by providing the column's required capacity (P_u) or ultimate load in the first row. The second row of the table contains the design load (ΩP_n), which is the strength of the selected column. The column that is selected is displayed in the last row just to help identify and conclude each column's capacity check/ analysis. These strengths can be referenced in the American Institute of Steel Construction's (AISC) Steel construction manual part 4. This section of the manual contains a variety of tables that display all the design loads, ultimate loads/story height for each

column type; However for this project, we mainly used tables 4-3 and 4-4 for selecting our desired columns.

Table 11: Final Load and Column Selection

Final Load	
Required Load (Pu) Kips	767.1492036
Design Load (ϕP_n) Kips	960
Column Size	(3)W10x77

4.4 Structural Foundations

The foundation is the part of a structure that is placed below the ground level and is responsible for transmitting the loads from the structure above to the underlying soil (Nilson, 2009). For our project with an increase in weight from the original design to our alternative design, adds more bearing loads to the soil that must be distributed evenly without causing the soil experience failure for the life of our building. To accommodate this increase in load being transmitted to our spread footings, we needed to address the size of our footings and modify them as needed to again satisfy the demand of our structure. In order to design the size of a footing, the actual stress being applied for our column loads must be less than or equal to the allowable pressure (equation used to describe this scenario is listed below).

$$\frac{P}{Q} = A_{\text{req.}}$$

Q = Soil Bearing Pressure (pounds per square foot)

P = Acting Loads (pounds)

$A_{\text{req.}}$ = Required area for bottom of footing (square foot)

The variables in the equation above were used to obtain the required area needed for each footing in order to maintain the existing spread design. The loads variable referred to as (P) is based on the calculations obtained from our structural analysis of the columns described in the previous section of the report. The loads used to obtain all of the required areas relate to each of the selected columns' locations and were provided in kip units where they were eventually converted to pounds for the purpose of this part of our analysis. Below in **Table 12** is the recommended spread footing modifications. Also, this table includes all of the footings for the columns that were evaluated based on their locations.

Table 12: Footing Modifications

Column Location	Previous Column Type	Column Type	Original Footing	total load on Footing (P)-lbs	Allowable Pressure/Soil Bearing Capacity (6 ksf for all) (Pa) - psf	Proposed Footing	Previous Footing size Ft ³	Required Area (Ar)= P/pa
B-1	14x61 14x53	W14x61	F8	354,506	6000	Same	64	59.084
D-1	14x61 14x53	W14x90	F9	477,412	6000	Same	162	79.569
D-3	W14x74 W14x48	W14x61	F10	938,233	6000	Same	225	156.372
D-4.5'	W14x61	14x90	F6	407,601	6000	F'6	85.75	67.934
B-3	W10x49 W10x45 W10x33	W10x60	F1	262,562	6000	Same	96.5	43.76
B'-5	W10x100 W10x77 W10x39	W10x77	F13	150,763	6000	Same	429	25.127
A.1'-4	W14x61 (2)	HSS7x7x3/8	F4	324,167	6000	F'4	63	54.028
D-2	W10x77 W10x39	W10x49	F11	672,842	6000	Same	302.5	112.14
B'- 8	W10x100 W10x68 W10x39	W14x61	F12	635,693	6000	Same	396	105.949
BB-13	W10x49 W10x33	W10x77	F7	827,077	6000	F'7	162	137.846
BB-14	W10x100 W10x60 W10x33	W10x77	F2	850,874	6000	Same	687.375	141.812

In the table above for all areas highlighted in green, it was determined that the existing footing size was sufficient in supporting our newly selected columns at the location specified in the first column on the left. For the areas highlighted in red, it was determined that the previous footing size was not able to support our newly chosen columns. For column BB-14 for example,

it can be seen that the required area needed for the footing is 137.846 square feet and the previous footing size was 73.5 square feet. To fix this problem without having to redesign the entire footing, which would be outside the scope of this project, we used adjusted the width, height and thickness of the footing (previously 7' - 0" x 7' - 0" x 1' - 6") to match the size of footing F9 which has a total area of 162 square feet. In changing the size of footing F7 to match F9, the same concept would apply to the reinforcement which was previously 8-#7 EW BOTT and would now become 10-#7 EW BOTT to satisfy the original design for F9. Lastly, to address the remaining footings highlighted in red above, the same process for modifying footing F7 was followed. Below in **Table 13** is the updated footing schedule based on the calculations made in **Table 12**.

Table 13 Revised Footing Schedule

Revised Footing Schedule		
F1	18'-3"x 9'-0"x 2'-0"	#8 @ 12 EW BOTT
F2	23'-5"x 13'-0"x 2'-3"	#8 @ 9 EW BOTT
F3	3'-0" x 3'-0" x 1'-3"	3-#5 EW BOTT
F4	6'-0"x6'-0"x1'-9"	5-#5 EW BOTT
F5	5'-0"x5'-0"x1'-3"	5-#5 EW BOTT
F6	7'-0"x7'-0"x1'-9"	6-#8 EW BOTT
F7	9'-0"x9'-0"x2'-0"	8-#7 EW BOTT
F8	8'-0"x8'-0"x1'-9"	8-#7 EW BOTT
F9	9'-0"x9'-0"x2'-0"	10-#7 EW BOTT
F10	10'-0"x10'-0"x2'-3"	10-#8 EW BOTT
F11	11'-0"x11'-0"x2'-6"	12-#8 EW BOTT
F12	12'-0"x12'-0"x2'-9"	14-#8 EW BOTT
F13	13'-0"x12'-0"x2'-9"	16-#8 EW BOTT

From the calculations that were conducted to determine the newly required area for our proposed spread footings schedule, our alternative design has been concluded. Our group has

observed, based on the procedure we used to conduct our structural analysis, how increasing the gravity loads for an entire building by simply changing a floor system's building material and also the addition of new features (steel joists) can have an adverse effect on the design of a foundation.

4.5 Impacts of Alternative Floor System Design on Cost

Once the alternative design Revit model of our building was created a cost comparison to the existing model was created. This entailed comparing the quantities of materials used in each design to determine which design is more cost efficient. The first step was to determine the amount of material used in the existing design. This was performed using the existing design in Revit. Material Takeoff schedules were created in Revit for each of the structural components of the project that included structural framing, columns, foundations, walls and floors. A full list of the material takeoff schedules for the existing design can be viewed in **Appendix F**. Once the total amount of steel tonnage and concrete volume were determined using the material takeoff schedules the total cost of steel and concrete were calculated. Consigli provided the MQP group with the cost of each package so the total cost of steel per ton and concrete per cubic yard were calculated. **Table 14** below displays the total cost of steel per ton for the project, and **Table 15** displays the total cost of concrete per cubic yard for the existing design.

Table 14: Cost of Steel

Cost of Steel Package:	\$3,265,000.00
Total Volume of Steel	2719.94
Total Volume of Steel	490lbs/ft ³
Total Tons of Steel	666.3853
Steel cost per ton	\$4,899.57

Table 15: Cost of Concrete

Total Cost of Concrete	\$1,692,118.00
Total Volume of Concrete (CF)	95954.98
Cost per CF	\$17.63
Total Volume of Concrete (CY)	3553.9
Cost per CY	\$476.13

Using these numbers as a basis we were able to calculate the cost of the alternative design based on the cost of the packages for the existing design. To determine the amount of material that was used in the alternative design material takeoff schedules were used for the alternative design. In order to complete a correct 5D BIM model it was necessary to perform the material takeoff schedules for each of the pre-determined phases of the alternative design. The design was split up into 10 sections or phases and the cost of each phase was calculated. Using the cost per

ton of steel and cost per cubic yard for the existing design the cost per ton of steel and cubic yard of concrete were calculated for each phase of the alternative design. **Table 16** below displays the total cost for each phase of the alternative design. The calculations for each phase can be reviewed in **Appendix G**.

Table 16: Alternative Design Phase Costs

Phase	Cost of Concrete	Cost of Steel	Total Cost
Building 1 Phase 1	\$114,842.53	0	\$114,842.53
Building 1 Phase 2	\$236,674.17	\$557,706.91	\$794,381.07
Building 1 Phase 3	\$177,782.14	\$838,931.08	\$1,016,713.22
Building 2 Phase 1	0	0	0
Building 2 Phase 2	\$310,889.01	\$621,444.04	\$932,333.05
Building 2 Phase 3	\$167,531.06	\$1,104,734.72	\$1,272,265.78
Building 3 Phase 1	\$261,990.47	0	\$261,990.47
Building 3 Phase 2	\$248,254.12	\$825,114.91	\$1,073,369.03
Building 3 Phase 3	\$122,703.43	\$678,066.63	\$800,770.06
Building 4 Phase 1	\$455,442.04	0	\$455,442.04
Building 4 Phase 2	0	\$311,682.33	\$311,682.33
Total Alternative Design Cost			\$7,033,789.58

Summing the total cost of each phase it was determined that the total cost of the alternative design was \$7,033,789.58. Using the total cost of the packages provided by Consigli the cost of the existing design was \$6,027,760. The cost of the alternative design came out to be \$1,006,029.58 more than the existing design. According to these calculations the existing design was more cost effective than the alternative design for the building.

4.6 Impacts of Alternative Floor System Design on Schedule

In order to determine the impacts the alternative floor system design has on the construction schedule, a new schedule that includes the activities required for the installation of the cast-in-place concrete slabs needed to be created. To create this alternative design schedule, appropriate changes and adjustments were first made to the original baseline schedule that was created in Primavera. Changing the floor system from pre-cast planks to cast-in-place (CIP) slabs required two major changes to be made in terms of the schedule. The first of these was to add floor joists and decking in order to properly support the new CIP slabs, and the second was to actually replace the pre-cast planks with CIP ones.

In order to address the addition of the floor joists, some of the existing activities of the baseline schedule were edited. When using pre-cast planks, the floors required neither joists nor decking to support the slabs. The steel only needed to be detailed after it was erected, before the pre-cast planks could be installed. Therefore, the corresponding activities on the baseline schedule were named “Detail Steel”, and each of these activities required between 10 to 13 days to complete, depending on the specific floor and section of the building. For these activities to represent the installation of the joists and the decking to support the CIP slabs, their names were changed to “Deck & Detail Steel.” After consulting with Jody Staruk, the Consigli project manager for Sheehan Hall, it was determined that adding floor joists and decking would take

approximately two additional working days for each floor. Therefore, the durations of the activities were also increased by two days.

Originally, the next activity after “Detail Steel” was to “Erect & Grout Precast Planks”, and these activities were connected via a start-to-start relationship with a time lag. The time lag ensured that the steel would be detailed and ready by the time the pre-cast planks are to be installed. Since the addition of the floor joists and decking added two days to the duration of the activity, the original time lags were also increased by two days to still ensure that the steel would be ready for the new CIP slabs. The next step in creating the new alternative design schedule was to address the replacement of the pre-cast planks with the CIP ones in the schedule. Originally in the baseline schedule, the activities that represented the installation of the pre-cast planks were named “Erect & Grout Precast Planks”, and each activity took five working days to complete. These activities were renamed “Place SOD” (slab on deck) in order to represent the pouring and setting of the new CIP slabs. The durations for the activities were left at five days because it was determined that that was approximately the amount of time it would take for each slab to be poured and set.

Once these adjustments had been made, the new schedule for the alternative floor system design was ready. The original sequencing of the activities was left unchanged because it was determined that erecting each of the four sections of the building, two levels at a time would still work with the new CIP slabs and was the most efficient way to work around waiting for the slabs to set after being poured (Consigli). It should also be noted that, by erecting the structure two levels at a time, Consigli will comply with the regulations set by the Occupational Safety & Health Administration (OSHA) that a steel structure cannot be taller than three levels or 30 feet, whichever comes first, without having installed a floor or deck below. In other words, at least

one floor must be installed for every 30 feet, or three levels, the steel rises. This regulation primarily serves to protect steel workers, so that in the unfortunate event that someone should fall, he/she will not fall more than 30 feet, or three levels.

Table 17 below shows a comparison of the critical activities of the alternative design schedule with those of the baseline schedule. Only the critical activities were compared because they are the only activities that had an impact on the date of completion of the structure. The activities that are highlighted in red are those to which changes were made. The activities highlighted in yellow are those that were originally not on the critical path but became critical as a result of the changes made to the activities highlighted in red. In total, the new schedule for the alternative floor system design incurred eight additional calendar days, and as a result, the completion date for the structure was pushed back from October 8th 2013 to October 16th 2013.

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Alternative Design Critical Activities			
Activity Name	Original Duration	Early Start	Early Finish
Start	0	3/20/13	
Excavate for Footings & Foundations S1	9	3/20/13	4/1/13
Excavate for Footings & Foundations S3	10	4/1/13	4/12/13
Excavate for Footings & Foundations S2	10	4/15/13	4/26/13
FREP Footings S2	4	4/29/13	5/2/13
FREP Foundations S2	19	5/3/13	5/29/13
Waterproof S2	15	5/31/13	6/20/13
FREP Elevator Pits S2	10	6/18/13	7/1/13
Waterproof Elevator Pits S2	10	7/2/13	7/15/13
Erect Columns & Steel S2-L1/3-1	3	7/11/13	7/15/13
Deck & Detail Steel S2-L1/3-1	15	7/12/13	8/1/13
Erect Steel S2-L1/3-2	3	7/16/13	7/18/13
Deck & Detail Steel S2-L1/3-2	14	7/17/13	8/5/13
Place SOD S2-L2/3-1	5	7/29/13	8/2/13
Place SOD S2-L2/3-2	5	7/30/13	8/5/13
Erect Steel S2-L4/5-1	1	7/31/13	7/31/13
Erect Steel S2-L4/5-2	1	8/1/13	8/1/13
Deck & Detail Steel S2-L4/5-1	12	8/1/13	8/16/13
Deck & Detail Steel S2-L4/5-2	12	8/2/13	8/19/13
Place SOD S2-L4/5-1	5	8/12/13	8/16/13
Place SOD S2-L4/5-2	5	8/13/13	8/19/13
Erect Steel S2-L6/PH-1	1	8/15/13	8/15/13

Baseline Critical Activities			
Activity Name	Original Duration	Early Start	Early Finish
Start	0	3/20/2013	
Excavate for Footings & Foundations S1	9	3/20/2013	4/1/2013
Excavate for Footings & Foundations S3	10	4/1/2013	4/12/2013
Excavate for Footings & Foundations S2	10	4/15/2013	4/26/2013
FREP Footings S2	4	4/29/2013	5/2/2013
FREP Foundations S2	19	5/3/2013	5/29/2013
Waterproof S2	15	5/31/2013	6/20/2013
FREP Elevator Pits S2	10	6/18/2013	7/1/2013
Waterproof Elevator Pits S2	10	7/2/2013	7/15/2013
Erect Columns & Steel S2-L1/3-1	3	7/11/2013	7/15/2013
Deck & Detail Steel S2-L1/3-1	13	7/12/2013	7/30/2013
Erect Steel S2-L1/3-2	3	7/16/2013	7/18/2013
Deck & Detail Steel S2-L1/3-2	12	7/17/2013	8/1/2013
Erect & Grout Precast Planks S2-L1/3-1	5	7/25/2013	7/31/2013
Erect & Grout Precast Planks S2-L1/3-2	5	7/26/2013	8/1/2013
Erect Steel S2-L4/5-1	1	7/29/2013	7/29/2013
Erect Steel S2-L4/5-2	1	7/30/2013	7/30/2013
Detail Steel S2-L4/5-1	10	7/30/2013	8/12/2013
Detail Steel S2-L4/5-2	10	7/31/2013	8/13/2013
Erect & Grout Precast Planks S2-L4/5-1	5	8/6/2013	8/12/2013
Erect & Grout Precast Planks S2-L4/5-2	5	8/8/2013	8/14/2013
Erect Steel S2-L6/PH-1	1	8/9/2013	8/9/2013

Erect Steel S2-L6/PH-2	1	8/16/13	8/16/13
Deck & Detail Steel S2-L6/PH-1	12	8/16/13	9/2/13
Deck & Detail Steel S2-L6/PH-2	12	8/19/13	9/3/13
Place SOD S2-L6/PH-1	5	8/27/13	9/2/13
Place SOD S2-L6/PH-2	5	8/28/13	9/3/13
Place SOG S4-LL	5	9/25/13	10/1/13
Install MEP Hangers S4-LL	6	10/2/13	10/9/13
Install Spray Fireproofing S4-LL	4	10/7/13	10/10/13
Erect Columns & Steel S4-L1	3	8/30/13	9/3/13
Deck & Detail Steel S4-L1	13	9/3/13	9/19/13
SOD S4-L1	2	9/23/13	9/24/13
Install MEP Hangers S4-L1	6	10/2/13	10/9/13
Install Spray Fireproofing S4-L1	4	10/11/13	10/16/13
Erect Steel S4-R	2	9/4/13	9/5/13
Deck & Detail Steel S4-R	13	9/6/13	9/24/13
Parapet Framing & Sheathing S4-R	10	9/30/13	10/11/13
Structure Complete	0		10/16/13
Incurred 8 additional days			

Erect Steel S2-L6/PH-2	1	8/12/2013	8/12/2013
Detail Steel S2-L6/PH-1	10	8/12/2013	8/23/2013
Detail Steel S2-L6/PH-2	10	8/13/2013	8/26/2013
Erect & Grout Precast Planks S2-L6/PH-1	5	8/19/2013	8/23/2013
Erect & Grout Precast Planks S2-L6/PH-2	5	8/21/2013	8/27/2013
Place SOG S4-LL	5	9/17/2013	9/23/2013
Install MEP Hangers S4-LL	6	9/24/2013	10/1/2013
Install Spray Fireproofing S4-LL	4	9/27/2013	10/2/2013
Erect Columns & Steel S4-L1	3	8/22/2013	8/26/2013
Deck & Detail Steel S4-L1	13	8/26/2013	9/11/2013
SOD S4-L1	2	9/13/2013	9/16/2013
Install MEP Hangers S4-L1	6	9/24/2013	10/1/2013
Install Spray Fireproofing S4-L1	4	10/3/2013	10/8/2013
Structure Complete	0		10/8/2013

Table 17: Baseline vs. Alternative Design Critical Activities

4.7 BIM for Alternative Floor System Design

A 5D BIM model was created for the alternative floor system design, in order to provide a visualization of the impacts that the new design had on the cost and construction schedule of the building. The first step in creating the alternative design model was to make the necessary changes to the structure of the building in the original 3D Revit structural model obtained from Consigli. These changes included updating the original beam, column, and girder sizes to the revised sizes calculated in the structural analysis, replacing the hollow-core pre-cast planks with cast-in-place (CIP) slabs, and adding floor joists to the beams and girders.

Once the necessary changes had been made in the 3D Revit model, the next step was to break down the model into phases that correspond to the way the building would be actually built on site. This can be done in Revit by creating new phases and selecting and adding the desired objects to each of the phases. Once the different phases were created, it was possible to perform a quantity take-off for each phase; the quantity information obtained from the quantification process was used to calculate the cost for each phase, as well as the cost for the entire alternative design. Then, the next step was to export the updated 3D Revit model in the format of a Navisworks file. Thanks to the great interoperability of Revit with other BIM applications, this export can be done with the click of a button in Revit.

At this point, the updated 3D model was ready to be imported into Navisworks, in order to integrate it with the cost and schedule. Upon opening the file in Navisworks, the 3D model was automatically imported, along with the phases created in Revit: the phases can be accessed from the Selection Tree in Navisworks. Next, the updated schedule for the alternative floor system design in Primavera was exported in the format of a Microsoft Project file. This file can then be easily imported into the TimeLiner function of Navisworks. Once the schedule had been

imported into TimeLiner, the phases can be attached to the corresponding activities on the schedule. Once again, as with the baseline versus as-built BIM model, there were far more activities than there were phases, resulting in only some of the activities being attached to phases. Next, the calculated costs for each of the phases were attached to the activities, and the “Task Type” for these activities was set to “Construct” in TimeLiner. Once these steps were finished, the 5D model for the alternative floor system design was complete and the animation was ready to be run. **Figures 32** through **36** below show the progress of construction along the timeline of the 5D BIM model. The upper left corner of the animation displays information such as the day, date, and costs as the construction of the building progresses throughout the timeline. Objects are shown in green while they are being constructed, and become gray when completed.

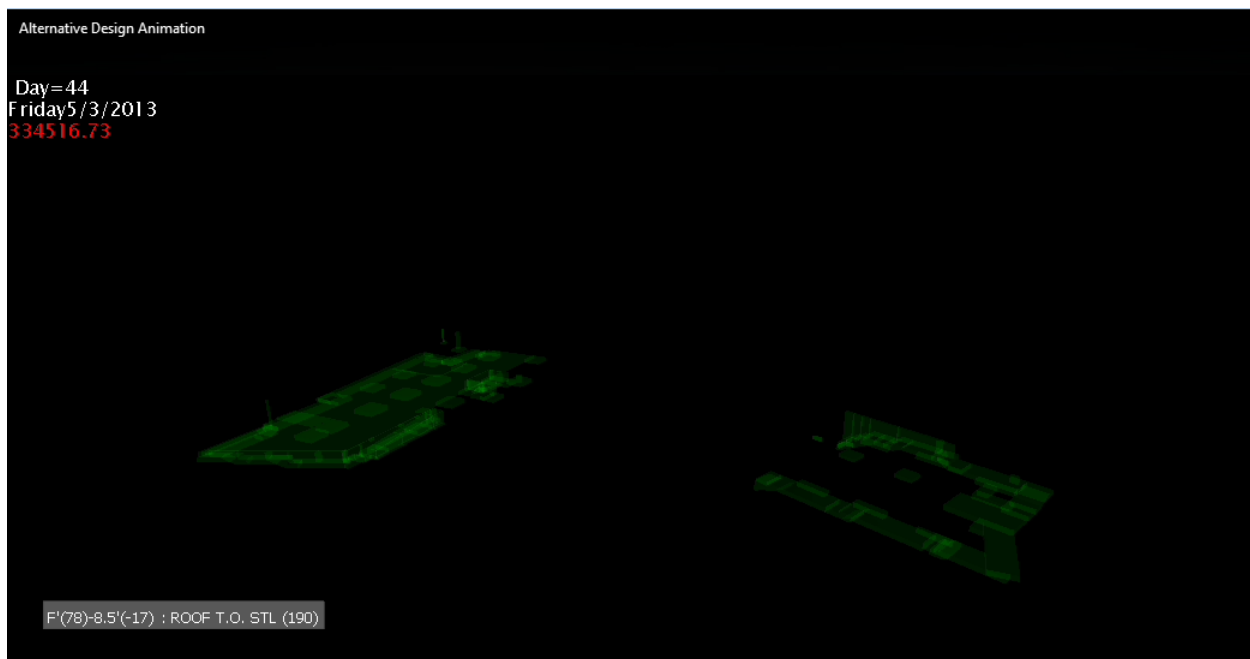


Figure 32: Alternative Design Animation 1

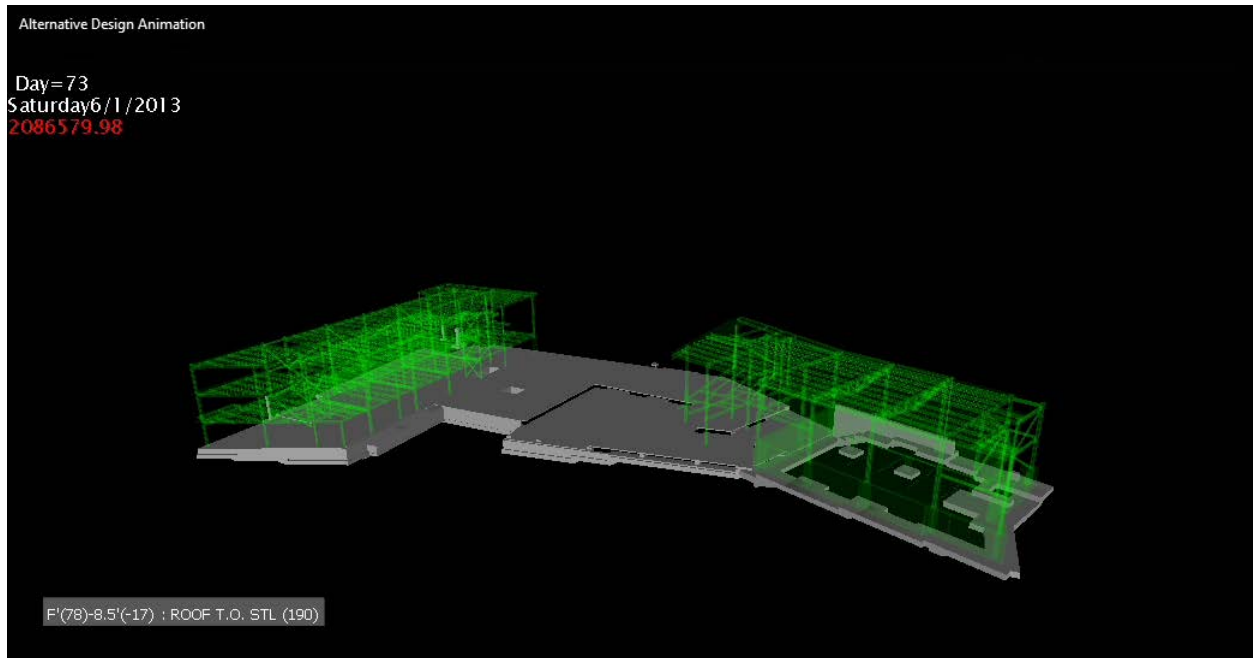


Figure 33: Alternative Design Animation 2

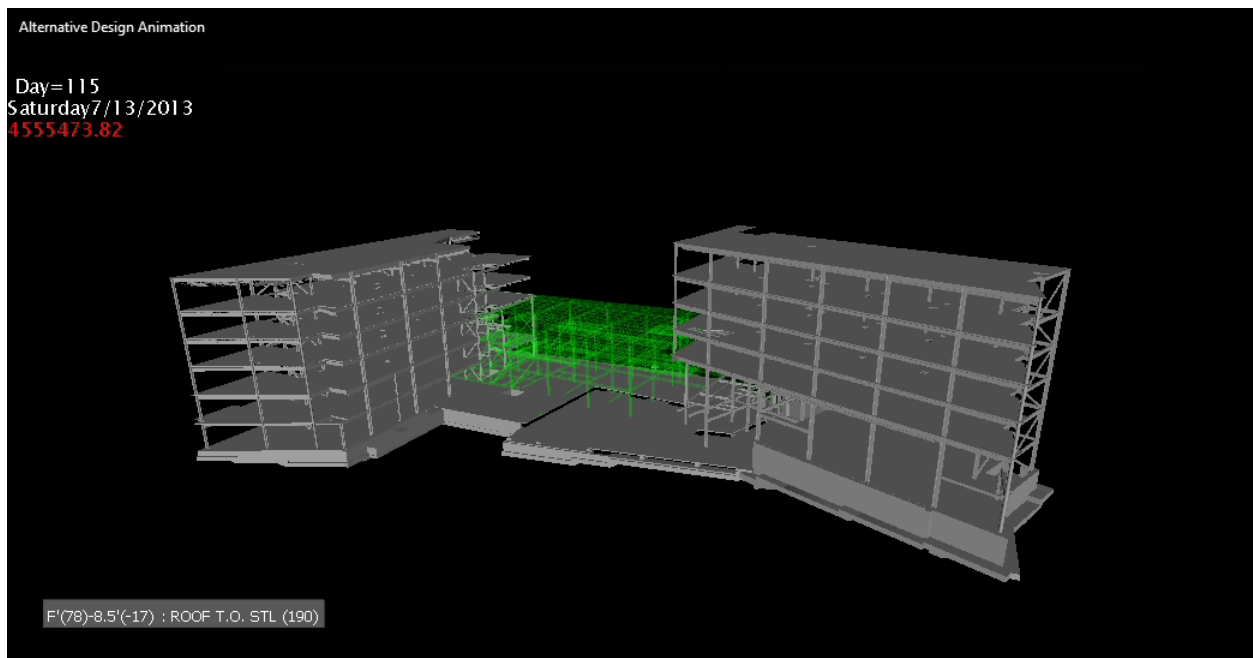


Figure 34: Alternative Design Animation 3

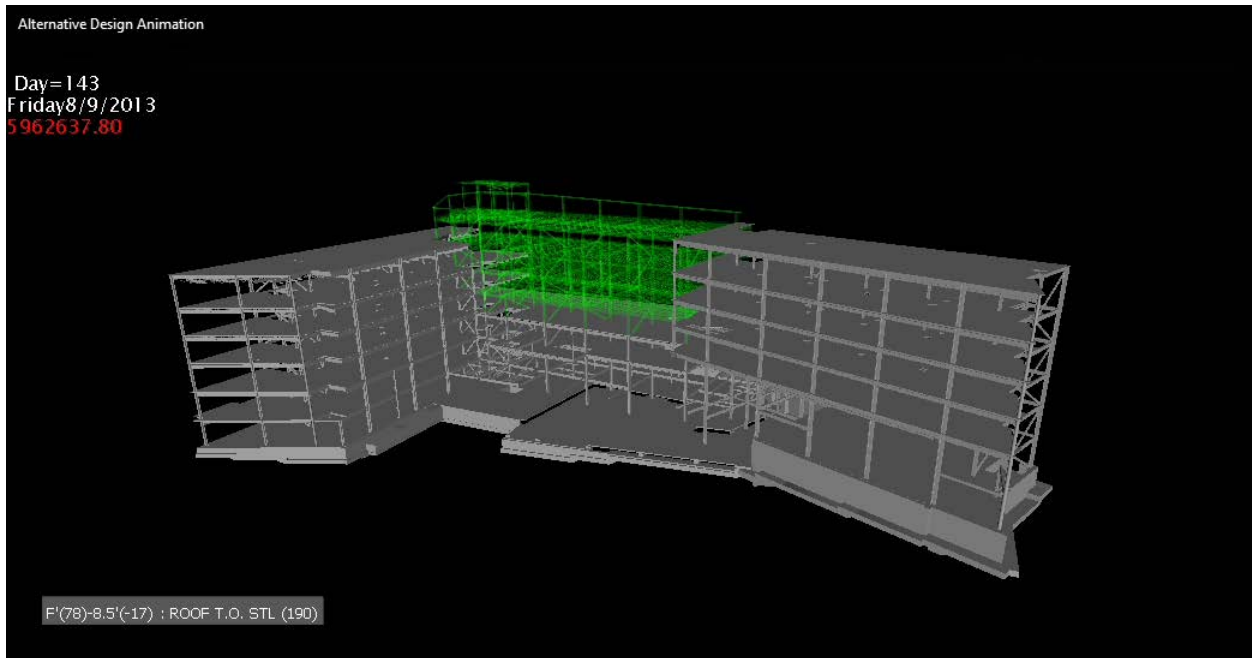


Figure 35: Alternative Design Animation 4

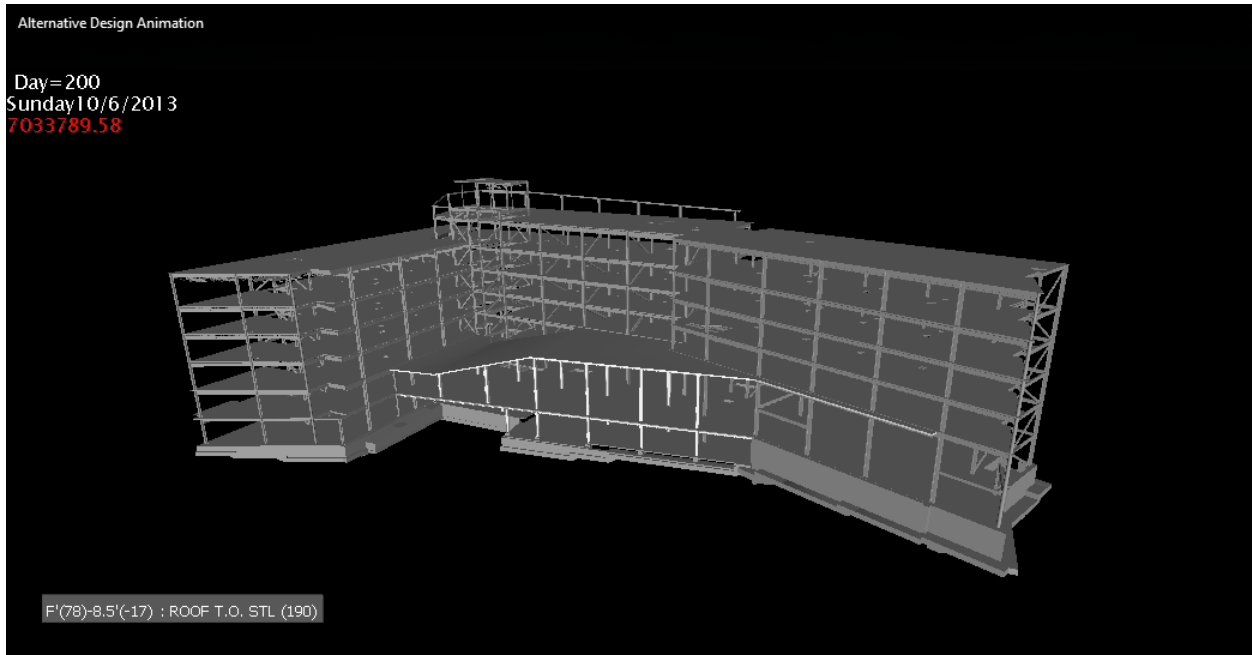


Figure 36: Alternative Design Animation 5

5.0 Site Logistics & Development

This section of the report is a generalized recap of the pre-construction, site work or land development stage of Sheehan hall. This section will discuss the design goals for the site as well as providing project site development photos to help visualize the process of prepping a site for construction. For WSU's "Sheehan Hall" before any construction could begin, the first step that was taken was to evaluate the site's existing conditions and to establish the project boundaries. The photo below shows the site during this evaluation period.

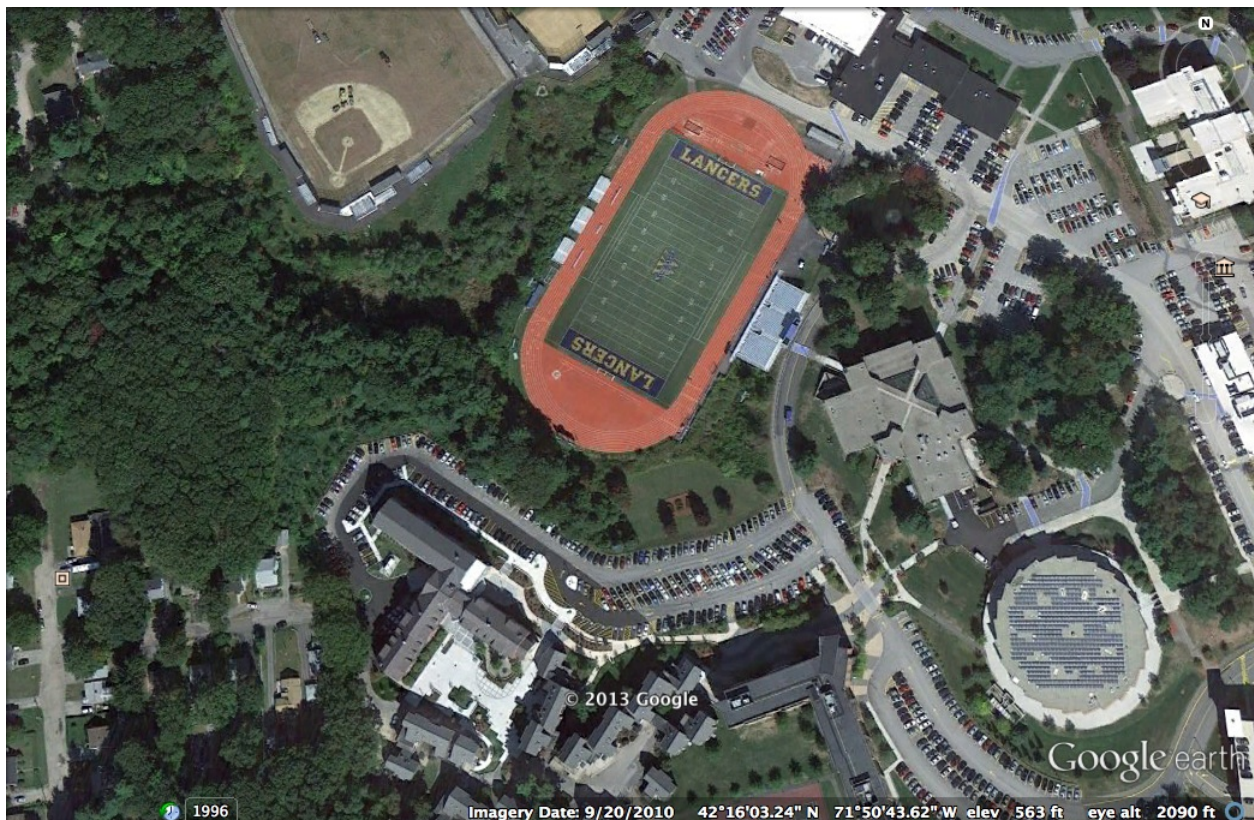


Figure 37: Existing Site (Pre-Development)

Now that the site has gone through its initial evaluation period, the designers are now able to sub-divide the project area and begin the site design process. To help understand the main focus areas or design objectives for the development of Sheehan Hall's new construction site our group met with one of Consigli's project engineers Paul Galligan. In our interview, we asked for

Paul to identify the site designer's main focus areas during the site development phase of the project and the advantages of choosing their design approach in comparison to other possible approaches, He said "I think the differentiator for the preliminary site work for this project was the modular bi-level retaining wall that was put in the winter before we began work here.

Originally, the site was a parking lot and a relatively steep hill down to the wetlands/football field. Using a modular approach (i.e. the wall is built out of precast concrete blocks instead of a cast-in-place wall) we were able to bring up the level of the blocks and the elevation of the grade in the same day. This allowed for us to avoid waiting for cure times and also having to consistently monitor the temperature of newly casted concrete over the course of the winter." He continued further by describing another objective for the site development "As with any building, we focused on hitting the low utilities on the project first while keeping contact with the school to make sure we were not impacting their operations."

The Proposed Site Development plans taken from the official plan set for Sheehan Hall can be viewed in **Appendix H**. Additionally, Consigli had provided the team with photos representing each accomplishment/milestone that was achieved throughout the construction process. Below is a list of project photos dating back to November 2012 and leading up to February 2014.



Figure 38: Site - November 2012



Figure 39: Retaining Wall Construction - December 2012



Figure 40: Retaining Wall - January 2013



Figure 41: Site Excavation - February 2013



Figure 42: Site Excavation - March 2013



Figure 43: Foundation Walls - April 2013



Figure 44: Steel Framing - June 2013

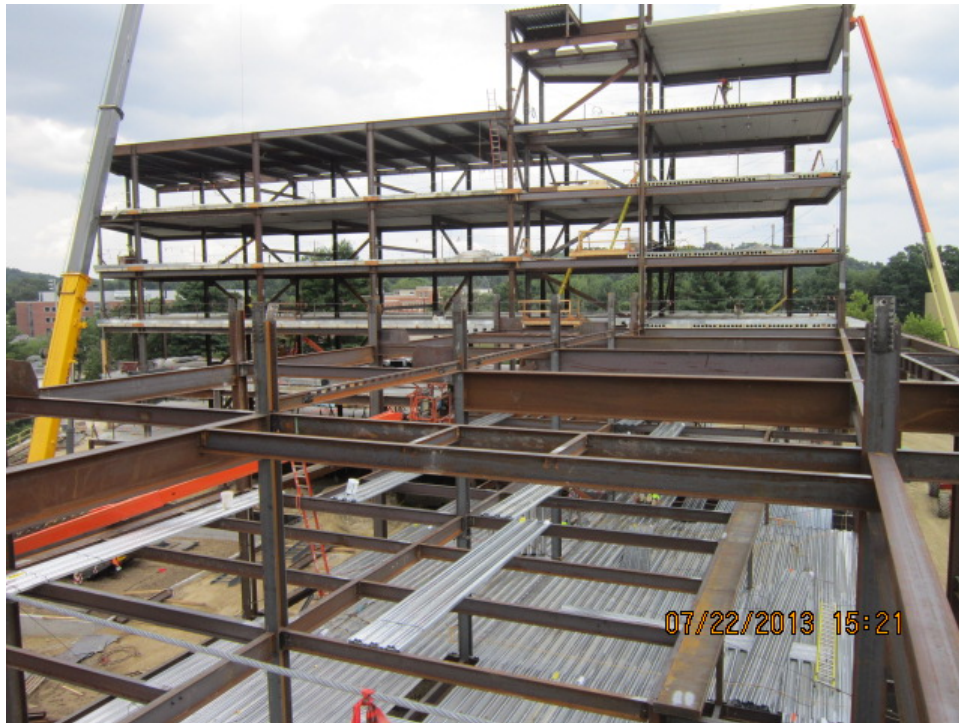


Figure 45: Hollow-cored Precast Planks - July 2013



Figure 46: Site Overview - August 2013



Figure 47: Curtain Walls - September 2013



Figure 48: Site Overview - October 2013



Figure 49: Masonry (Wing 1) - November 2013



Figure 50: Site Overview - December 2013



Figure 51: Interior Finishes - January 2014



Figure 52: Site Overview - February 2014

6.0 Conclusions

This MQP for Sheehan Hall dormitory on the campus of Worcester State University contains the design of an alternative floor system and an analysis of the alternative design's effects on the project's cost and schedule. The existing design includes the use of precast planks in the floor system while the alternative design uses only cast in place concrete. The dormitory hall will provide the campus with extended on campus housing to meet the demands of the growing student population. The MQP group used Autodesk *Revit*, and *Primavera* software to determine the most effective design with regards to cost and scheduling. The alternative design meets the desired loading for the project, this being verified through the structural hand calculations.

The best way to compare the existing design to the alternative design is by comparing the respective cost and schedules. The alternative design resulted in an increase in cost of \$1,006,029.58. There was also an increase in the project duration, which resulted in an additional 8 days. The alternative design examined did not provide a favorable result with regards to the cost and schedule, compared to the existing design. A 5-D model of the existing design was created to provide a visualization of the construction of the project and see the relation to cost and schedule. A 5-D model was also created for the alternative design to provide a visual comparison of the two designs. The 5-D models combine the Revit models with the cost estimations and Primavera schedules in Autodesk Navisworks to display the advancement of the project. Using a 5-D model aids the construction process as the progression of the project can be seen and the two models can be visually compared.

Through this MQP an alternative floor system was designed that could meet the structural requirements, however after analyzing the impacts of cost and schedule to the existing design the MQP team has recommended the use of the existing design for the project.

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Appendix A: Consigli Interview

This appendix contains the interview of the Consigli project manager and engineer who are working on Sheehan Hall.

Consigli Interview 2013

1. Is the project currently on schedule? Are there any outstanding issues that might potentially delay the project?
 - Progress is good
 - Financial incentive for finishing project by June 13th
 - Possible problem areas: 25th bulletin, Cogen coordination, MEP delivery
 - Windows coming in the 21st, won't affect the end date...Contractual document that if it had impacted end date, claim to transfer risk to subcontractor
2. What are the challenges of being on a fast track schedule?
 - CPM not only critical path...anything with a 10 day float
 - Tougher to make up time on a fast track schedule
3. What was the impact of the design drawings being delayed by a month?
 - GC as well as subs are impacted because without drawings not able to plan and understand scope as well as they could have
 - Architect left MEP drawings 80% complete...Electrical bid date pushed back 2 weeks
4. Were there any impacts to the project due to sub-contractors being brought on board late?
 - Windows impacted due to subs being brought on late
 - Focus more on what needs to be done momentarily, not able to look at the future
 - Not Proactive...more reactive
5. Does having to work on an occupied campus affect the schedule?
 - Conscientious about parking and students
 - Delivery is hardest part
6. How is the practice of “lean construction” affecting the schedule of the project?
 - Forces foreman to look at drawings and figure out possible problems beforehand
 - Trades understand scope of work better
 - Tighten up schedule
 - Anticipated readiness vs. guaranteed
 - Exterior faster, but does not really affect the schedule of the project
 - Constraint Log - Pre deficiency log - look at problems 6 weeks in advance
 - Practice on site...working in smaller spaces as opposed to whole floors
7. Why was cast in place used only on the first floor?
 - Considered a public space...a lot of people at one location at a time
 - Main reasons
 - Loading
 - Anticipated deflection and movement
 - Piping fed into the floor easier for equipment

- Building 4 loading dock needs to be heated
8. Are there any significant benefits from using the current concrete method?
 - Faster to install and cheaper
 - Curing time 3-5 days for cast in place
 - Using crane for 2 activities at the same time
 9. Why was a GMP bid contract chosen for this project?
 - MSCBA prefers it...competitive bid.... any money not spent goes back to the GMP
 - GMP...open estimate...open book can be seen the whole time
 10. How will winter affect the construction schedule, has the schedule been altered so that exterior work will be complete by the time that winter comes?
 - There is no way around doing masonry work during the winter
 11. How is the relationship between the Owner/Arch,Eng/GC, have any communication problems occurred?
 - Relationship is good
 - When there is a problem MSCBA likes to hear the problem and the solution
 - Only real problem was with national grid feeding power to site through manholes...problem with the manhole covers
 - Communication is good...need to understand the personalities that you are dealing with
 12. What steps were taken to determine the current foundation plan, and what factors were considered in its design process?
 - For the development of the current foundation, a Geotechnical soil report was done to determine if soil is adequate for designed structure.
 1. Soil Report
 1. Analysis of Existing Site Conditions
 2. Groundwater and Soil Observations
 3. Particle size analysis
 13. Were there any issues during the development of the foundation?
 - Found that for a few boring locations along the east and western sides of the site that there was greater fill thicknesses of approximately 6.5 ft. (boring HA12-8), 7.5 ft. (HA12-9), and 12.3 ft. (HA12-12).
 - Determined that since the basement floor will be constructed below groundwater levels, a foundation perimeter drain and under-drain system should be constructed to provide permanent groundwater control around foundation walls and beneath slabs.

Appendix B: Consultants

This appendix provides a list of all the consultants involved in the Sheehan Hall project.

LANDSCAPE ARCHITECTS: Brown, Richardson and Rowe, Inc

CIVIL ENGINEERING: Nitsch Engineering

GEOTECHNICAL: Haley & Aldrich, Inc

STRUCTURAL ENGINEERING: RSE Associates

DOOR HARDWARE CONSULTANT: SMOOT Associates

ENERGY MODELING: Andeiman Lilek

FOOD SERVICE CONSULTANT: Vision Builders

ELEVATOR CONSULTANT: VERTRAN Enterprises

MECHANICAL, ELECTRICAL, PLUMBING, AND FIRE PROTECTION ENGINEERING

TEL-DATA & AV: AKF Engineering

LIGHTING DESIGN CONSULTANT: Ateller Ten

CODE CONSULTANT: Rolf Jensen and Associates

SUSTAINABILITY: Soden Sustainability Consulting.

SPECIFICATIONS CONSULTANT: Wil-Spec LLC

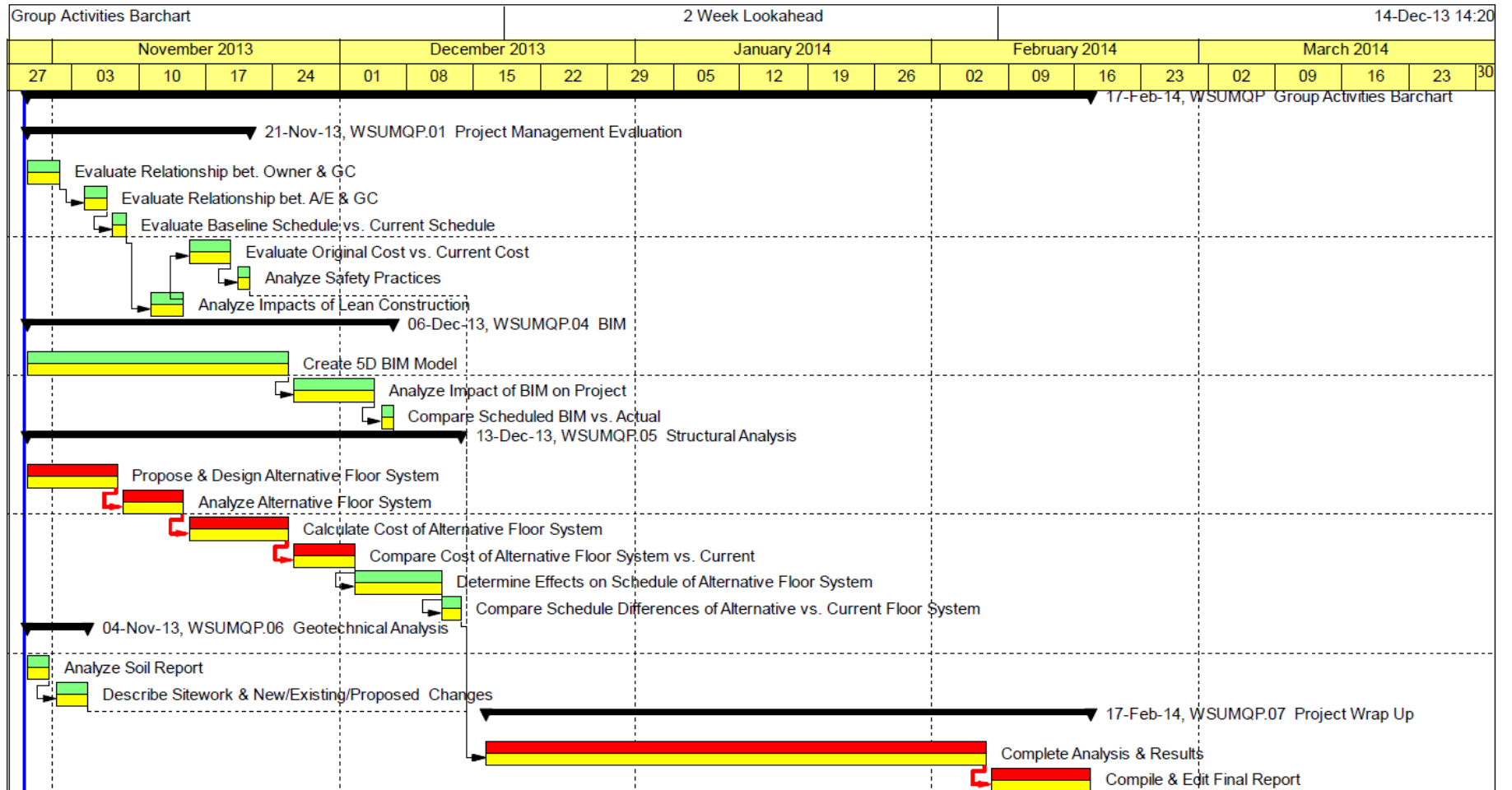
Appendix C: Group Activities List, Bar Chart and Organizational Breakdown Structure (OBS)

This appendix includes a list of activities to be completed by the project group, along with a bar chart and a breakdown of the tasks among the group members.

1

Activity ID	Activity Name	Original Duration	Planned Start	Planned Finish
WSUMQP G roup Activities Barchart		80	29-Oct-13	17-Feb-14
WSUMQP.0 1 Project Management Evaluation		18	29-Oct-13	21-Nov-13
A1000	Evaluate Relationship bet. Owner & GC	4	29-Oct-13	01-Nov-13
A1010	Evaluate Relationship bet. A/E & GC	3	04-Nov-13	06-Nov-13
A1060	Evaluate Baseline Schedule vs. Current Schedule	2	07-Nov-13	08-Nov-13
A1070	Evaluate Original Cost vs. Current Cost	3	15-Nov-13	19-Nov-13
A1080	Analyze Safety Practices	2	20-Nov-13	21-Nov-13
A1250	Analyze Impacts of Lean Construction	4	11-Nov-13	14-Nov-13
WSUMQP.0 4 BIM		29	29-Oct-13	06-Dec-13
A1130	Create 5D BIM Model	20	29-Oct-13	25-Nov-13
A1140	Analyze Impact of BIM on Project	4	26-Nov-13	04-Dec-13
A1150	Compare Scheduled BIM vs. Actual	2	05-Dec-13	06-Dec-13
WSUMQP.0 5 Structural Analysis		34	29-Oct-13	13-Dec-13
A1200	Propose & Design Alternative Floor System	8	29-Oct-13	07-Nov-13
A1210	Analyze Alternative Floor System	5	08-Nov-13	14-Nov-13
A1230	Calculate Cost of Alternative Floor System	7	15-Nov-13	25-Nov-13
A1240	Compare Cost of Alternative Floor System vs. Current	2	26-Nov-13	02-Dec-13
A1260	Determine Effects on Schedule of Alternative Floor System	7	02-Dec-13	11-Dec-13
A1270	Compare Schedule Differences of Alternative vs. Current Floor Sys	2	11-Dec-13	13-Dec-13
WSUMQP.0 6 Geotechnical Analysis		5	29-Oct-13	04-Nov-13
A1160	Analyze Soil Report	3	29-Oct-13	31-Oct-13
A1220	Describe Sitework & New/Existing/Proposed Changes	2	01-Nov-13	04-Nov-13
WSUMQP.0 7 Project Wrap Up		46	16-Dec-13	17-Feb-14
A1170	Complete Analysis & Results	20	16-Dec-13	06-Feb-14
A1180	Compile & Edit Final Report	7	07-Feb-14	17-Feb-14

¹ Note: The schedule is based on the WPI Undergraduate Calendar for the Academic year 2013-2014.

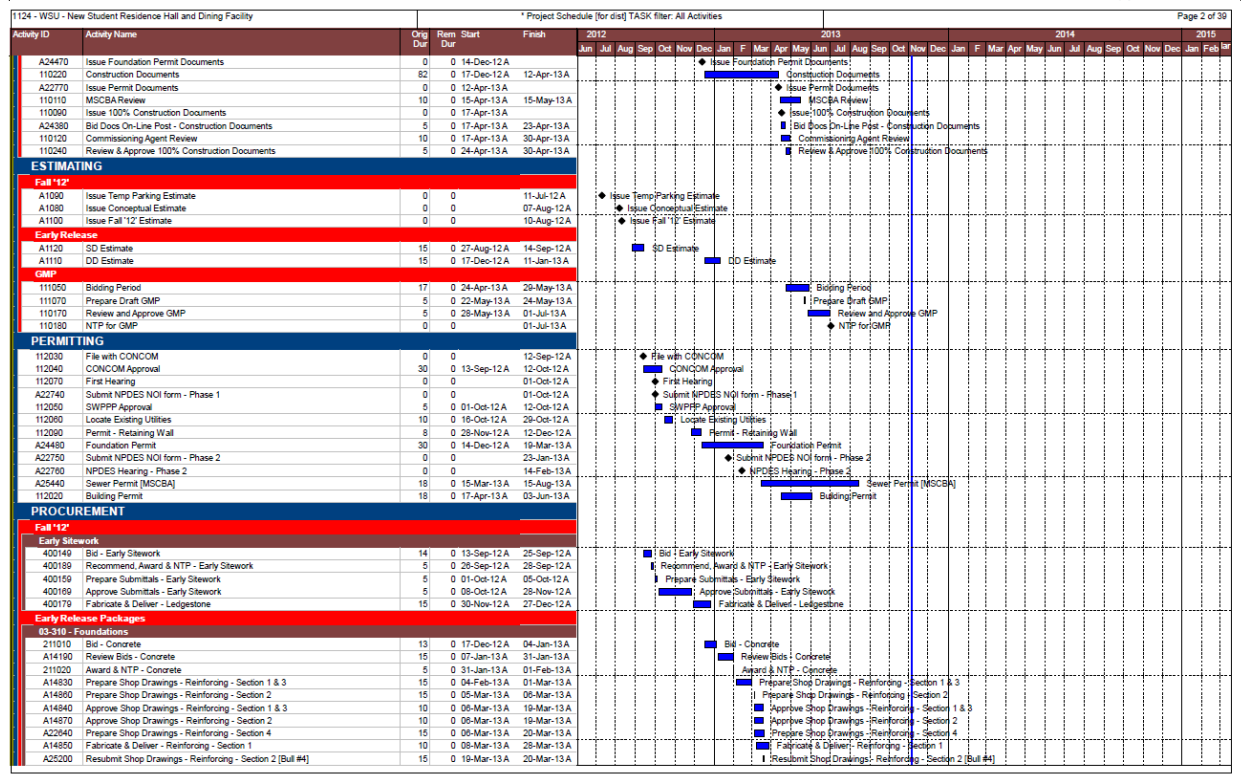
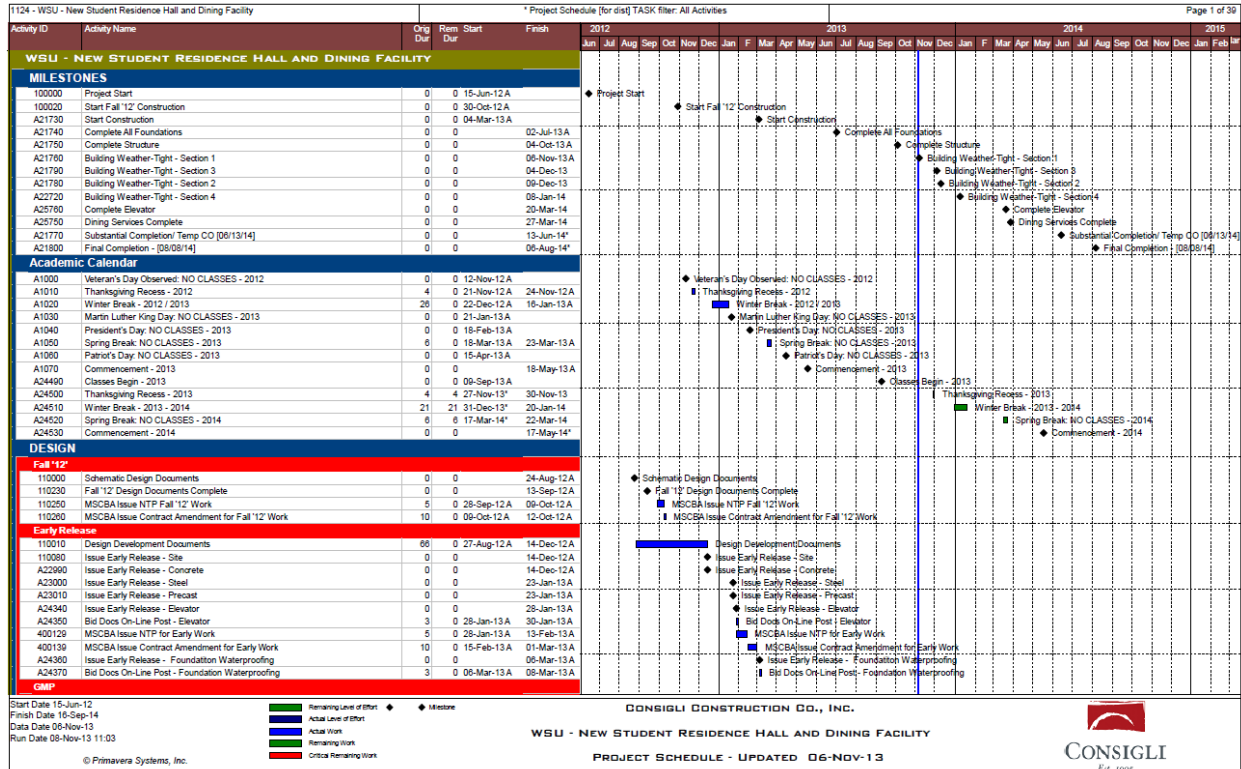


Group Organizational Breakdown Structure

Activity	Team Member Assignment
Project Management Evaluation	Ben, Matt
BIM	Ben, Matt
Structural Analysis	Tom
Geotechnical Review	Tom
Project Wrap Up	Ben, Matt, Tom

Appendix E: Consigli's As-Built Schedule

This appendix contains the as-built schedule from Consigli for the construction of Sheehan Hall, updated on November 6th, 2013.



Appendix F: Existing Design Material Take-off

This appendix contains the schedules for quantity take-off, for different categories of building components.

Wall Material Take-off

Wall Material Takeoff			
Family and Type	Material: Volume	Material: Area	Structural Material
Basic Wall: 20 RC WALL	4267.92 CF	2562 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 20 RC WALL	149.96 CF	91 SF	Concrete - Cast-in-Place Concrete
Basic Wall: D - Exterior Wall	90.21 CF	54 SF	Concrete 02
Basic Wall: D - Exterior Wall	13.53 CF	54 SF	Concrete 02
Basic Wall: 17.5" RC WALL	470.34 CF	323 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	101.13 CF	70 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	57.79 CF	40 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	1448.28 CF	994 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	135.86 CF	94 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	404.62 CF	279 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	71.95 CF	51 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	420.63 CF	297 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	185.63 CF	127 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	326.74 CF	224 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	219.48 CF	154 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	85.22 CF	58 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	110.30 CF	76 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	748.06 CF	513 SF	Concrete - Cast-in-Place Concrete
Basic Wall: Concrete - 24"	46.67 CF	23 SF	Concrete - Cast-in-Place Concrete
Basic Wall: Concrete - 24"	46.66 CF	23 SF	Concrete - Cast-in-Place Concrete
Basic Wall: Concrete - 24"	46.66 CF	23 SF	Concrete - Cast-in-Place Concrete
Basic Wall: Concrete - 24"	46.67 CF	23 SF	Concrete - Cast-in-Place Concrete
Basic Wall: Concrete - 24"	47.29 CF	24 SF	Concrete - Cast-in-Place Concrete
Basic Wall: Concrete - 24"	47.29 CF	24 SF	Concrete - Cast-in-Place Concrete
Basic Wall: Concrete - 24"	38.14 CF	20 SF	Concrete - Cast-in-Place Concrete
Basic Wall: Concrete - 24"	10.14 CF	6 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 12" CONC	112.00 CF	112 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 12" CONC	112.00 CF	112 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 12" CONC	60.50 CF	61 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	48.39 CF	39 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 12" CONC	34.33 CF	34 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 12" CONC	42.67 CF	43 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 12" CONC	38.67 CF	39 SF	Concrete - Cast-in-Place Concrete

Basic Wall: 12" CONC	38.33 CF	38 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 12" CONC	48.50 CF	49 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	193.78 CF	133 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	107.25 CF	75 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	187.22 CF	130 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	391.02 CF	268 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	120.31 CF	83 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	45.94 CF	32 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	741.65 CF	510 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	75.83 CF	52 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	64.17 CF	44 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	52.50 CF	36 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	129.70 CF	89 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	690.36 CF	473 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 16 RC WALL	977.03 CF	734 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 24 RC WALL	2.02 CF	1 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 24 RC WALL	1.50 CF	1 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 24 RC WALL	10.32 CF	5 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 24 RC WALL	6.84 CF	3 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 24 RC WALL	61.75 CF	31 SF	Concrete - Cast-in-Place Concrete
Basic Wall: Concrete - 24"	54.53 CF	28 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	59.24 CF	44 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	116.31 CF	80 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	23.47 CF	16 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 24 RC WALL	44.69 CF	22 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 12" CONC	75.43 CF	75 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 12" CONC	104.65 CF	105 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	224.55 CF	154 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	368.67 CF	253 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	236.47 CF	162 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	152.36 CF	104 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	84.80 CF	58 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	9.80 CF	7 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	922.84 CF	647 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	2778.23 CF	1920 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	105.75 CF	73 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	106.65 CF	73 SF	Concrete - Cast-in-Place Concrete
Basic Wall: Concrete - 24"	28.00 CF	14 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 12" CONC	3.00 CF	3 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 12" CONC	4.00 CF	4 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 12" CONC	3.00 CF	3 SF	Concrete - Cast-in-Place Concrete

Basic Wall: 12" CONC	2.00 CF	2 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 12" CONC	4.79 CF	6 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 12" CONC	3.00 CF	3 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 12" CONC	3.00 CF	3 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 12" CONC	2.27 CF	4 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 24 RC WALL	231.80 CF	116 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	350.03 CF	241 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	139.49 CF	96 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	28.48 CF	20 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	34.76 CF	24 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 8" CONC	26.83 CF	40 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 8" CONC	68.94 CF	103 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 8" CONC	9.51 CF	14 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 8" CONC	39.98 CF	60 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	10.27 CF	7 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	13.22 CF	9 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	48.59 CF	33 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	12.61 CF	9 SF	Concrete - Cast-in-Place Concrete
Basic Wall: 17.5" RC WALL	73.24 CF	50 SF	Concrete - Cast-in-Place Concrete
Grand total: 93	20641.03 CF	14139 SF	

Structural Foundation Material Takeoff

Family and Type	Material: Area	Material: Volume	Structural Material
Footing-Rectangular: F9	234 SF	162.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F10	290 SF	225.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F10	290 SF	225.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F10	290 SF	225.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F9	234 SF	162.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F11	352 SF	302.50 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F11	352 SF	302.50 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F10	290 SF	225.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F7	140 SF	73.50 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F11	352 SF	302.50 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F7	140 SF	73.50 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F9	234 SF	162.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F9	234 SF	162.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F9	234 SF	162.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F9	234 SF	162.00 CF	Concrete - Cast-in-Place Concrete

Footing-Rectangular: F10	290 SF	225.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F7	140 SF	73.50 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F10	290 SF	225.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F12	420 SF	396.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F12	420 SF	396.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F13	450 SF	429.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F13	450 SF	429.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F12	420 SF	396.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F4	52 SF	20.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F4	52 SF	20.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F4	52 SF	20.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F6	102 SF	45.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F6	102 SF	45.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F6	102 SF	45.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F6	102 SF	45.00 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 9'x2' (2'-0 1/2" FND WALL)	1815 SF	1400.76 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 9'x2' (2'-0 1/2" FND WALL)	351 SF	257.64 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 5'x2'	333 SF	223.83 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 7'x2' loading dock	934 SF	687.49 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 9'x2' (2'-0 1/2" FND WALL)	400 SF	275.42 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 9'x2' (2'-0 1/2" FND WALL)	695 SF	536.56 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 9'x2' (2'-0 1/2" FND WALL)	196 SF	112.50 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 9'x2' (2'-0 1/2" FND WALL)	178 SF	103.63 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 9'x2' (2'-0 1/2" FND WALL)	671 SF	518.87 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F10	290 SF	225.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F9	196 SF	122.75 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F10	290 SF	225.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F7	140 SF	73.50 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F4	52 SF	20.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F10	290 SF	225.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F10	290 SF	225.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F8	184 SF	112.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F8 (2' Thick)	192 SF	128.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F10	290 SF	225.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F10	290 SF	225.00 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 9'x2' (2'-0 1/2" FND WALL)	118 SF	45.22 CF	Concrete - Cast-in-Place Concrete
Foundation Slab: 12" Foundation Slab	117 SF	116.89 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 9'x2' (2'-0 1/2" FND WALL)	803 SF	627.19 CF	Concrete - Cast-in-Place Concrete
Foundation Slab: 12" Foundation Slab	173 SF	173.36 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 5'-9"x2'	440 SF	299.86 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 5'-9"x2'	53 SF	8.78 CF	Concrete - Cast-in-Place Concrete

Wall Foundation: 5'x2'	318 SF	203.38 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 5'-9"x2'	159 SF	77.77 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 5'-9"x2'	708 SF	497.61 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 9'x2' (2'-0 1/2" FND WALL)	811 SF	553.24 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 9'x2' (2'-9 1/2" FNDN WALL)	569 SF	410.90 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 9'x2' (2'-0 1/2" FND WALL)	370 SF	233.97 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 6.5'x2'	143 SF	56.22 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 6.5'x2'	571 SF	371.46 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 9'x2' (2'-9 1/2" FNDN WALL)	727 SF	461.86 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 5'x2'	184 SF	107.45 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F7	140 SF	73.50 CF	Concrete - Cast-in-Place Concrete
Structural Foundations 1: Structural Foundations 11	304 SF	206.68 CF	
Structural Foundations 7: Structural Foundations 11	230 SF	157.89 CF	
Structural Foundations 8: Structural Foundations 11	206 SF	133.46 CF	
Structural Foundations 10: Structural Foundations 11	165 SF	97.30 CF	
Wall Foundation: 9'x2' (2'-0 1/2" FND WALL)	91 SF	46.02 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 9'x2' (2'-0 1/2" FND WALL)	188 SF	124.55 CF	Concrete - Cast-in-Place Concrete
Structural Foundations 12: Structural Foundations 11	297 SF	206.57 CF	
Structural Foundations 13: Structural Foundations 11	266 SF	206.19 CF	
Wall Foundation: 9'x2' (2'-9 1/2" FNDN WALL)	599 SF	456.81 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 9'x2' (2'-0 1/2" FND WALL)	526 SF	385.57 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 9'x2' (2'-9 1/2" FNDN WALL)	324 SF	223.22 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 9'x2' (2'-9 1/2" FNDN WALL)	190 SF	119.23 CF	Concrete - Cast-in-Place Concrete
Structural Foundations 14: Structural Foundations 11	372 SF	294.30 CF	
Structural Foundations 6: Structural Foundations 11	185 SF	135.16 CF	
Wall Foundation: 9'x2' (2'-0 1/2" FND WALL)	3139 SF	2345.39 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 5'x2'	310 SF	206.08 CF	Concrete - Cast-in-Place Concrete
Structural Foundations 23: Structural Foundations 11	152 SF	99.78 CF	
Wall Foundation: 9'x2' (2'-9 1/2" FNDN WALL)	135 SF	81.04 CF	Concrete - Cast-in-Place Concrete
Structural Foundations 4: Structural Foundations 11	557 SF	524.38 CF	
Structural Foundations 21: Structural Foundations 11	235 SF	171.90 CF	
Structural Foundations 5: Structural Foundations 11	179 SF	122.85 CF	
Wall Foundation: 5'x2'	208 SF	108.12 CF	Concrete - Cast-in-Place Concrete
Structural Foundations 9: Structural Foundations 11	193 SF	124.61 CF	
Structural Foundations 26: Structural Foundations 11	131 SF	80.94 CF	
Foundation Slab: 12" Foundation Slab	25 SF	25.00 CF	Concrete - Cast-in-Place Concrete
Structural Foundations 25: Structural Foundations 11	46 SF	15.00 CF	
Foundation Slab: 12" Foundation Slab	25 SF	25.00 CF	Concrete - Cast-in-Place Concrete
Structural Foundations 28: Structural Foundations 11	46 SF	15.00 CF	
Wall Foundation: 2'-8"x1'	40 SF	12.44 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 2'-8"x1'	35 SF	10.78 CF	Concrete - Cast-in-Place Concrete

Wall Foundation: 2'-8"x1'	126 SF	42.34 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F3	33 SF	11.25 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 3'x2'	172 SF	103.67 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 3'x2'	113 SF	64.50 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F9	234 SF	162.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F9	234 SF	162.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F9	234 SF	162.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F9	234 SF	162.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F9	234 SF	162.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F9	234 SF	162.00 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F9	234 SF	162.00 CF	Concrete - Cast-in-Place Concrete
Foundation Slab: 2' Foundation Slab	36 SF	72.00 CF	Concrete - Cast-in-Place Concrete
Foundation Slab: 2' Foundation Slab	36 SF	72.00 CF	Concrete - Cast-in-Place Concrete
Foundation Slab: 2' Foundation Slab	36 SF	72.00 CF	Concrete - Cast-in-Place Concrete
Foundation Slab: 2' Foundation Slab	36 SF	72.00 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 1'-4"x1'	77 SF	21.33 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 1'-4"x1'	101 SF	28.00 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 1'-4"x1'	34 SF	9.06 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 1'-4"x1'	20 SF	4.40 CF	Concrete - Cast-in-Place Concrete
Footing-Rectangular: F3	33 SF	11.25 CF	Concrete - Cast-in-Place Concrete
Foundation Slab: 2' Foundation Slab	108 SF	215.18 CF	Concrete - Cast-in-Place Concrete
Foundation Slab: 2' Foundation Slab	140 SF	280.16 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 9'x2' (2'-0 1/2" FND WALL)	48 SF	9.75 CF	Concrete - Cast-in-Place Concrete
Foundation Slab: 2' Foundation Slab	170 SF	332.84 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 9'x2' (2'-0 1/2" FND WALL)	81 SF	28.87 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 9'x2' (2'-9 1/2" FNDN WALL)	688 SF	474.78 CF	Concrete - Cast-in-Place Concrete
Structural Foundations 3: Structural Foundations 11	190 SF	105.21 CF	
Footing-Rectangular: F10	290 SF	225.00 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 9'x2' (2'-0 1/2" FND WALL)	112 SF	62.25 CF	Concrete - Cast-in-Place Concrete
Wall Foundation: 9'x2' (2'-0 1/2" FND WALL)	119 SF	68.16 CF	Concrete - Cast-in-Place Concrete
Foundation Slab: 2' Foundation Slab	146 SF	292.83 CF	Concrete - Cast-in-Place Concrete
Foundation Slab: 2' Foundation Slab	18 SF	36.00 CF	Concrete - Cast-in-Place Concrete
Foundation Slab: 2' Foundation Slab	46 SF	91.00 CF	Concrete - Cast-in-Place Concrete
Foundation Slab: 2'3" Foundation Slab	306 SF	687.38 CF	Concrete - Cast-in-Place Concrete
Grand total: 131	36173 SF	27366.07 CF	

Floor Material Takeoff

Family and Type	Material: Volume	Material: Area	Structural Material
Floor: 2" Conc Topping Slab	3624.70 CF	21748 SF	Concrete - Cast-in-Place Concrete
Floor: 2" Conc Topping Slab	3582.26 CF	21494 SF	Concrete - Cast-in-Place Concrete
Floor: 2" Conc Topping Slab	1103.91 CF	6623 SF	Concrete - Cast-in-Place Concrete
Floor: 2" Conc Topping Slab	2476.07 CF	14856 SF	Concrete - Cast-in-Place Concrete
Floor: 2" Conc Topping Slab	3643.32 CF	21860 SF	Concrete - Cast-in-Place Concrete
Floor: 2" Conc Topping Slab	519.78 CF	3119 SF	Concrete - Cast-in-Place Concrete
Floor: 5 1/4" Composite Deck_8880	2449.83 CF	5600 SF	Concrete - Cast-in-Place Concrete
Floor: 5" Concrete Slab	7898.39 CF	18956 SF	Concrete - Cast-in-Place Concrete
Floor: 5" Concrete Slab	1579.68 CF	18956 SF	Concrete - Cast-in-Place Concrete
Floor: 5" RC SOG	564.88 CF	1356 SF	Concrete - Cast-in-Place Concrete
Floor: 5" RC SOG	105.32 CF	253 SF	Concrete - Cast-in-Place Concrete
Floor: 5" RC SOG	1912.71 CF	4591 SF	Concrete - Cast-in-Place Concrete
Floor: 5" RC SOG	28.97 CF	70 SF	Concrete - Cast-in-Place Concrete
Floor: 5" RC SOG	11.02 CF	26 SF	Concrete - Cast-in-Place Concrete
Floor: 5" RC SOG	400.84 CF	962 SF	Concrete - Cast-in-Place Concrete
Floor: 5-1/4" Slab on Deck	2597.59 CF	5937 SF	Concrete - Cast-in-Place Concrete
Floor: 5-1/4" Slab on Deck	0.00 CF	5937 SF	Concrete - Cast-in-Place Concrete
Floor: 5-1/4" Slab on Deck	11168.46 CF	25528 SF	Concrete - Cast-in-Place Concrete
Floor: 5-1/4" Slab on Deck	0.00 CF	25528 SF	Concrete - Cast-in-Place Concrete
Floor: 5-1/4" Slab on Deck	38.25 CF	87 SF	Concrete - Cast-in-Place Concrete
Floor: 5-1/4" Slab on Deck	0.00 CF	87 SF	Concrete - Cast-in-Place Concrete
Floor: 8" Concrete Slab_8880	108.37 CF	163 SF	Concrete - Cast-in-Place Concrete
Floor: 11" RC SOG	4133.56 CF	4509 SF	Concrete - Cast-in-Place Concrete
Grand total: 28	47947.88	223135 SF	

Structural Framing Material Takeoff

Family and Type	Structural Material	Material: Volume (CF)
C-Channel: C6X10.5	Metal - Steel - ASTM A36	0.37
C-Channel: C10x15.3	Metal - Steel - ASTM A36	2.96
C-Channel: C15x33.9	Metal - Steel - ASTM A36	4.43
DBeamRevit: DB9x46	Metal - Steel - ASTM A992	108.69
HSS-Hollow Structural Section: HSS8x8x1/2	Metal - Steel - ASTM A500 - Grade B - Rectangular and Square	0.35
HSS-Hollow Structural Section: HSS10x4x1/2	Metal - Steel - ASTM A500 - Grade B - Rectangular and Square	1.92
HSS-Hollow Structural Section: HSS10x4x3/8	Metal - Steel - ASTM A500 - Grade B - Rectangular and Square	10.62

HSS-Hollow Structural Section: HSS10x5x3/8	Metal - Steel - ASTM A500 - Grade B - Rectangular and Square	77.19
HSS-Hollow Structural Section: HSS10x6x1/2	Metal - Steel - ASTM A500 - Grade B - Rectangular and Square	77.82
HSS-Hollow Structural Section: HSS10x10x3/8	Metal - Steel - ASTM A500 - Grade B - Rectangular and Square	8.17
HSS-Hollow Structural Section: HSS12x6x1/2	Metal - Steel - ASTM A500 - Grade B - Rectangular and Square	23.12
HSS-Hollow Structural Section: HSS14x6x1/2	Metal - Steel - ASTM A500 - Grade B - Rectangular and Square	4.91
HSS-Hollow Structural Section: HSS16x4x3/8	Metal - Steel - ASTM A500 - Grade B - Rectangular and Square	14.07
HSS-Hollow Structural Section: HSS16x8x1/2	Metal - Steel - ASTM A500 - Grade B - Rectangular and Square	32.12
HSS-Hollow Structural Section: HSS16x16x1/2	Metal - Steel - ASTM A500 - Grade B - Rectangular and Square	10.23
HSS-Hollow Structural Section: HSS20x4x1/2	Metal - Steel - ASTM A500 - Grade B - Rectangular and Square	6.17
HSS-Hollow Structural Section: HSS20x4x3/8	Metal - Steel - ASTM A500 - Grade B - Rectangular and Square	2.26
HSS-Hollow Structural Section: HSS20x4x5/16	Metal - Steel - ASTM A500 - Grade B - Rectangular and Square	3.93
W-Wide Flange: W8x10	Metal - Steel - ASTM A992	3.5
W-Wide Flange: W8x15	Metal - Steel - ASTM A992	7.7
W-Wide Flange: W8X18	Metal - Steel - ASTM A992	0.8
W-Wide Flange: W8X24	Metal - Steel - ASTM A992	0.85
W-Wide Flange: W10x26	Metal - Steel - ASTM A992	2.19
W-Wide Flange: W10x30	Metal - Steel - ASTM A992	0.71
W-Wide Flange: W12x16	Metal - Steel - ASTM A992	74.85
W-Wide Flange: W12x19	Metal - Steel - ASTM A992	43.06
W-Wide Flange: W12x22	Metal - Steel - ASTM A992	5.27
W-Wide Flange: W12x26	Metal - Steel - ASTM A992	2.9
W-Wide Flange: W12x30	Metal - Steel - ASTM A992	4.21
W-Wide Flange: W12x35	Metal - Steel - ASTM A992	1.71
W-Wide Flange: W12x65	Metal - Steel - ASTM A992	3.61
W-Wide Flange: W14x22	Metal - Steel - ASTM A992	104.1
W-Wide Flange: W14x26	Metal - Steel - ASTM A992	44
W-Wide Flange: W14x30	Metal - Steel - ASTM A992	23.99
W-Wide Flange: W14x34	Metal - Steel - ASTM A992	2.1
W-Wide Flange: W14x38	Metal - Steel - ASTM A992	9.91
W-Wide Flange: W14x43	Metal - Steel - ASTM A992	2.55
W-Wide Flange: W14x48	Metal - Steel - ASTM A992	2.82
W-Wide Flange: W14x53	Metal - Steel - ASTM A992	3.05
W-Wide Flange: W14x61	Metal - Steel - ASTM A992	1.66
W-Wide Flange: W14x74	Metal - Steel - ASTM A992	4.41
W-Wide Flange: W14x99	Metal - Steel - ASTM A992	6.98
W-Wide Flange: W14x109	Metal - Steel - ASTM A992	6.68
W-Wide Flange: W14x132	Metal - Steel - ASTM A992	23.46
W-Wide Flange: W14x145	Metal - Steel - ASTM A992	108.69
W-Wide Flange: W14x159	Metal - Steel - ASTM A992	33.41
W-Wide Flange: W14x176	Metal - Steel - ASTM A992	38.29

W-Wide Flange: W16x26	Metal - Steel - ASTM A992	130.86
W-Wide Flange: W16x31	Metal - Steel - ASTM A992	65.7
W-Wide Flange: W16x36	Metal - Steel - ASTM A992	24.11
W-Wide Flange: W16x40	Metal - Steel - ASTM A992	90.53
W-Wide Flange: W16x45	Metal - Steel - ASTM A992	12.63
W-Wide Flange: W16x50	Metal - Steel - ASTM A992	92.41
W-Wide Flange: W16x57	Metal - Steel - ASTM A992	20.88
W-Wide Flange: W16x67	Metal - Steel - ASTM A992	107.52
W-Wide Flange: W16x77	Metal - Steel - ASTM A992	25.79
W-Wide Flange: W16x89	Metal - Steel - ASTM A992	34.72
W-Wide Flange: W16x100	Metal - Steel - ASTM A992	1.25
W-Wide Flange: W18x35	Metal - Steel - ASTM A992	42.19
W-Wide Flange: W18x40	Metal - Steel - ASTM A992	21.66
W-Wide Flange: W18x50	Metal - Steel - ASTM A992	8.93
W-Wide Flange: W18x55	Metal - Steel - ASTM A992	2.77
W-Wide Flange: W18x65	Metal - Steel - ASTM A992	42.78
W-Wide Flange: W18x71	Metal - Steel - ASTM A992	4.06
W-Wide Flange: W18x86	Metal - Steel - ASTM A992	41.74
W-Wide Flange: W18x143	Metal - Steel - ASTM A992	8.44
W-Wide Flange: W21x44	Metal - Steel - ASTM A992	15.91
W-Wide Flange: W21x48	Metal - Steel - ASTM A992	11.19
W-Wide Flange: W21x50	Metal - Steel - ASTM A992	26.9
W-Wide Flange: W24x55	Metal - Steel - ASTM A992	35.85
W-Wide Flange: W24x62	Metal - Steel - ASTM A992	27.55
W-Wide Flange: W24x68	Metal - Steel - ASTM A992	24.24
W-Wide Flange: W24x76	Metal - Steel - ASTM A992	7.25
W-Wide Flange: W24x84	Metal - Steel - ASTM A992	4.96
W-Wide Flange: W24x117	Metal - Steel - ASTM A992	16.27
W-Wide Flange: W27x84	Metal - Steel - ASTM A992	39.9
W-Wide Flange: W30x90	Metal - Steel - ASTM A992	5.86
W-Wide Flange: W30x124	Metal - Steel - ASTM A992	8.79
W-Wide Flange: W36x150	Metal - Steel - ASTM A992	11.82
W-Wide Flange: W36x160	Metal - Steel - ASTM A992	10.8
W-Wide Flange: W40x199	Metal - Steel - ASTM A992	14.7
Grand total: 1297		1997.75

Structural Column Material Takeoff

Family and Type	Structural Material	Material: Volume (CF)
HSS-Hollow Structural Section-Column: HSS4X4X5/16	Metal - Steel - ASTM A992	1.48
HSS-Hollow Structural Section-Column: HSS5X5X3/8	Metal - Steel - ASTM A992	4.03
HSS-Hollow Structural Section-Column: HSS6x6x1/2	Metal - Steel - ASTM A992	7.57

HSS-Hollow Structural Section-Column: HSS6X6X3/8	Metal - Steel - ASTM A992	4.09
HSS-Hollow Structural Section-Column: HSS10X5X3/8	Metal - Steel - ASTM A992	1.6
HSS-Round Hollow Structural Section-Column: HSS10X0.250	Metal - Steel - ASTM A53	4.29
HSS-Round Hollow Structural Section-Column: HSS10X0.312	Metal - Steel - ASTM A53	2.1
HSS-Round Hollow Structural Section-Column: HSS10X0.500	Metal - Steel - ASTM A53	2.48
W-Wide Flange-Column: W8X35	Metal - Steel - ASTM A992	1.89
W-Wide Flange-Column: W10X33	Metal - Steel - ASTM A992	38.96
W-Wide Flange-Column: W10X39	Metal - Steel - ASTM A992	32.86
W-Wide Flange-Column: W10X45	Metal - Steel - ASTM A992	17.23
W-Wide Flange-Column: W10X49	Metal - Steel - ASTM A992	32.89
W-Wide Flange-Column: W10X54	Metal - Steel - ASTM A992	19.24
W-Wide Flange-Column: W10X60	Metal - Steel - ASTM A992	13.11
W-Wide Flange-Column: W10X68	Metal - Steel - ASTM A992	20.01
W-Wide Flange-Column: W10X77	Metal - Steel - ASTM A992	55.99
W-Wide Flange-Column: W10X88	Metal - Steel - ASTM A992	6.84
W-Wide Flange-Column: W10X100	Metal - Steel - ASTM A992	52.8
W-Wide Flange-Column: W12X40	Metal - Steel - ASTM A992	1.1
W-Wide Flange-Column: W12X72	Metal - Steel - ASTM A992	5.07
W-Wide Flange-Column: W12X87	Metal - Steel - ASTM A992	6.77
W-Wide Flange-Column: W14X43	Metal - Steel - ASTM A992	78.54
W-Wide Flange-Column: W14X48	Metal - Steel - ASTM A992	3.59
W-Wide Flange-Column: W14X53	Metal - Steel - ASTM A992	16.28
W-Wide Flange-Column: W14X61	Metal - Steel - ASTM A992	98.49
W-Wide Flange-Column: W14X68	Metal - Steel - ASTM A992	19.71
W-Wide Flange-Column: W14X74	Metal - Steel - ASTM A992	11.59
W-Wide Flange-Column: W14X82	Metal - Steel - ASTM A992	31.09
W-Wide Flange-Column: W14X90	Metal - Steel - ASTM A992	92.01
W-Wide Flange-Column: W14X109	Metal - Steel - ASTM A992	5.65
W-Wide Flange-Column: W14X145	Metal - Steel - ASTM A992	5.49
W-Wide Flange-Column: W14x193	Metal - Steel - ASTM A992	27.35
Grand total: 351		722.19

Appendix G: Alternative Design Phase Costs

This appendix contains the cost packages for each phase of the construction process for the alternative floor system design.

Building 1 Phase 1 Cost

	Concrete Volume (CF)	Concrete Volume (CY)	Cost Per CY	Total
Wall Total	536.16	19.86	\$476.13	\$9,455.94
Foundation Total	5976.14	221.34	\$476.13	\$105,386.59
			Concrete Total Cost	\$114,842.53
			Total Phase Cost	\$114,842.53

Building 1 Phase 2 Cost

	Concrete Volume (CF)	Concrete Volume (CY)	Concrete Cost per CY	Total Concrete
Floor Total	8244.06	305.34	\$476.13	\$145,381.50
Column Total	575.87	21.349	\$476.13	\$10,164.90
Framing Total				
Wall Total	4600.49	170.39	\$476.13	\$81,127.77
			Total Concrete Cost	\$236,674.17
	Steel Volume (CF)	Steel Tonnage	Steel Cost per Ton	Total Steel
Floor Total				
Column Total	196.36	48.5991	\$4,899.57	\$238,114.69
Framing Total	263.55	65.228625	\$4,899.57	\$319,592.21
Wall Total				
			Total Steel Cost	\$557,706.91
			Total Phase Cost	\$794,381.07

Building 1 Phase 3 Cost

	Concrete Volume (CF)	Concrete Volume (CY)	Cost per CY	Total Concrete
Floor Total	10081.4	373.39	\$476.13	\$177,782.14
Column Total				
Framing Total				
			Total Concrete Cost	\$177,782.14
	Steel Volume (CF)	Steel Tonnage	Cost per Ton	Total Steel
Floor Total				
Column Total	167.34	41.41665	\$4,899.57	\$202,923.78
Framing Total	524.48	129.8088	\$4,899.57	\$636,007.30
			Total Steel Cost	\$838,931.08
			Total Phase Cost	\$1,016,713.22

Building 2 Phase 2 Cost

	Concrete Volume (CF)	Concrete Volume (CY)	Cost per CY	Total Concrete
Floor Total	13064.06	483.85	\$476.13	\$230,375.45
Column Total				
Framing Total				
Wall Total	4565.63	169.1	\$476.13	\$80,513.56
			Total Cost Concrete	\$310,889.01
	Steel Volume (CF)	Steel Tonnage	Cost per Ton	Total Steel
Floor Total				
Column Total	187.74	45.9963	\$4,899.57	\$225,361.99
Framing Total	329.96	80.8402	\$4,899.57	\$396,082.05
Wall Total				
			Total Cost Steel	\$621,444.04
			Total Phase Cost	\$932,333.05

Building 2 Phase 3 Cost

	Concrete Volume (CF)	Concrete Volume (CY)	Cost per CY	Total Cost
Total Floor	9500.31	351.86	\$476.13	\$167,531.06
Total Column				
Total Framing				
			Total Cost Concrete	\$167,531.06
	Steel Volume (CF)	Steel Tonnage	Cost per Ton	Total Steel
Total Floor				
Total Column	233.24	57.1438	\$4,899.57	\$279,979.93
Total Framing	687.07	168.33215	\$4,899.57	\$824,754.79
			Total Cost Steel	\$1,104,734.72
			Total Phase Cost	\$1,272,265.78

Building 3 Phase 1 Cost

	Concrete Volume (CF)	Concrete Volume (CY)	Cost per CY	Total Cost
Floor Total	4045.1	149.82	\$476.13	\$71,333.78
Column Total	247.39	9.16	\$476.13	\$4,361.35
Foundation Total	9104.53	337.2	\$476.13	\$160,551.00
Wall Total	1459.8	54.07	\$476.13	\$25,744.34
			Concrete Total Cost	\$261,990.47
			Total Phase Cost	\$261,990.47

Building 3 Phase 2 Cost

	Concrete Volume (CF)	Concrete Volume (CY)	Cost per CY	Total Concrete
Floor Total	10328.38	382.53	\$476.13	\$182,133.97
Column Total	285.18	10.56	\$476.13	\$5,027.93
Framing Total				
Wall Total	3464.36	128.31	\$476.13	\$61,092.23

			Total Cost Concrete	\$248,254.12
	Steel Volume	Steel Tonnage	Cost per Ton	Total Steel
Floor Total				
Column Total	197.8	48.461	\$4,899.57	\$237,437.96
Framing Total	489.57	119.94465	\$4,899.57	\$587,676.95
Wall Total				
			Total Cost Steel	\$825,114.91
			Total Phase Cost	\$1,073,369.03

Building 3 Phase 3 Cost

Column1	Concrete Volume (CF)	Concrete Volume (CY)	Cost per CY	Concrete Total
Floor Total	6958.08	257.71	\$476.13	\$122,703.43
Column Total				
Framing Total				
			Concrete Total Cost	\$122,703.43
	Steel Volume (CF)	Steel Tonnage	Cost per Ton	Steel Total
Floor Total				
Column Total	114.6	28.077	\$4,899.57	\$137,565.17
Framing Total	450.27	110.31615	\$4,899.57	\$540,501.46
			Steel Total Cost	\$678,066.63
			Total Phase Cost	\$800,770.06

Building 4 Phase 1 Cost

	Concrete Volume (CF)	Concrete Volume (CY)	Cost per CY	Total Concrete
Floor Total	7665.37	283.9	\$476.13	\$135,173.27
Column Total	139.63	5.17	\$476.13	\$2,461.59
Foundation Total	12444.19	460.9	\$476.13	\$219,448.26
Floor Total	5577.75	206.58	\$476.13	\$98,358.91

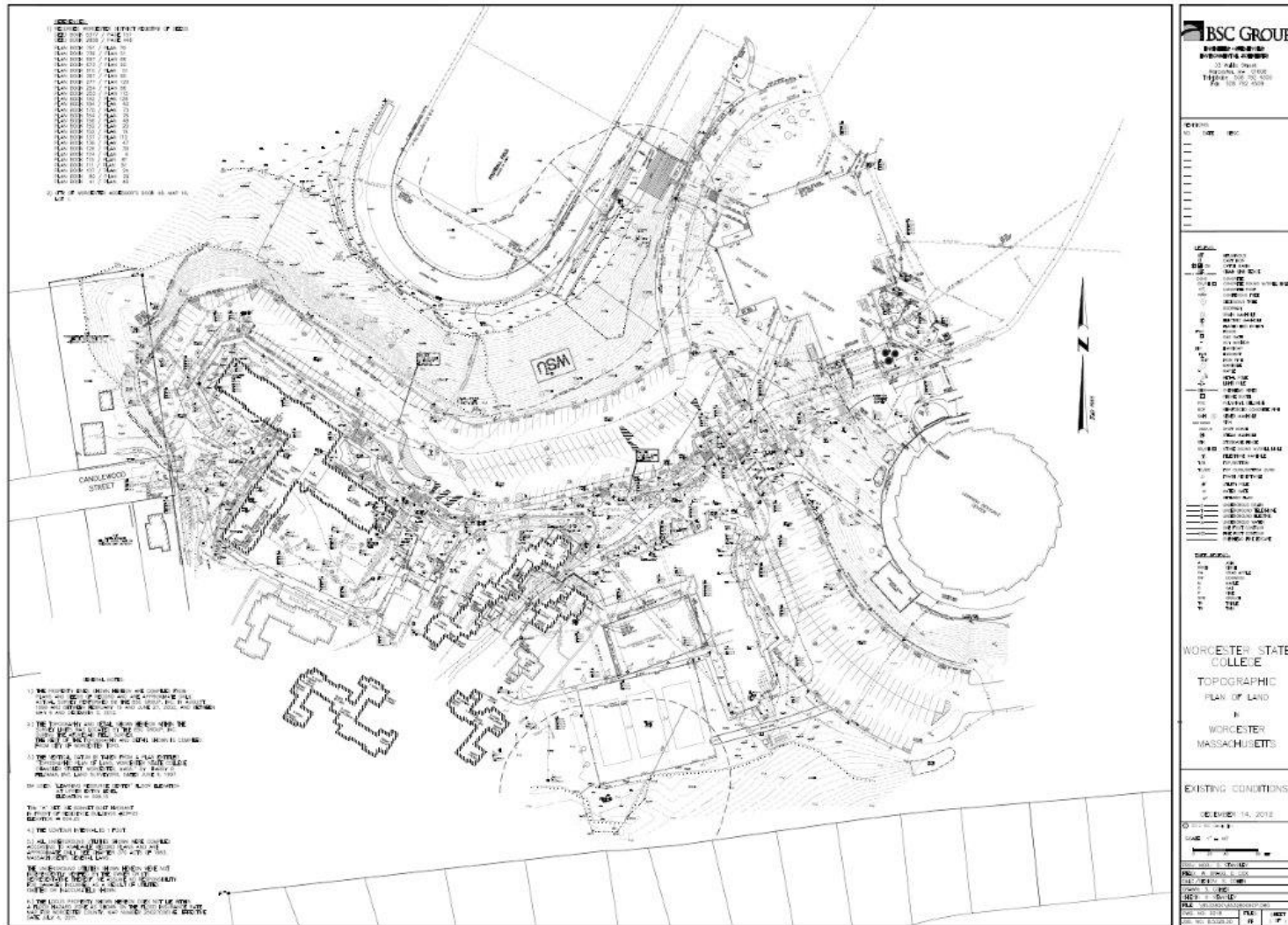
			Total Cost Concrete	\$455,442.04
			Total Phase Cost	\$455,442.04

Building 4 Phase 2 Cost

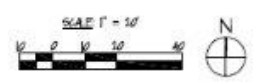
	Steel Volume (CF)	Steel Tonnage	Cost per Ton	Steel Total
Column Total	75.21	18.42645	\$4,899.57	\$90,281.64
Framing Total	184.44	45.1878	\$4,899.57	\$221,400.69
			Steel Total Cost	\$311,682.33
			Total Phase Cost	\$311,682.33

Appendix H: Proposed Site Development Plans

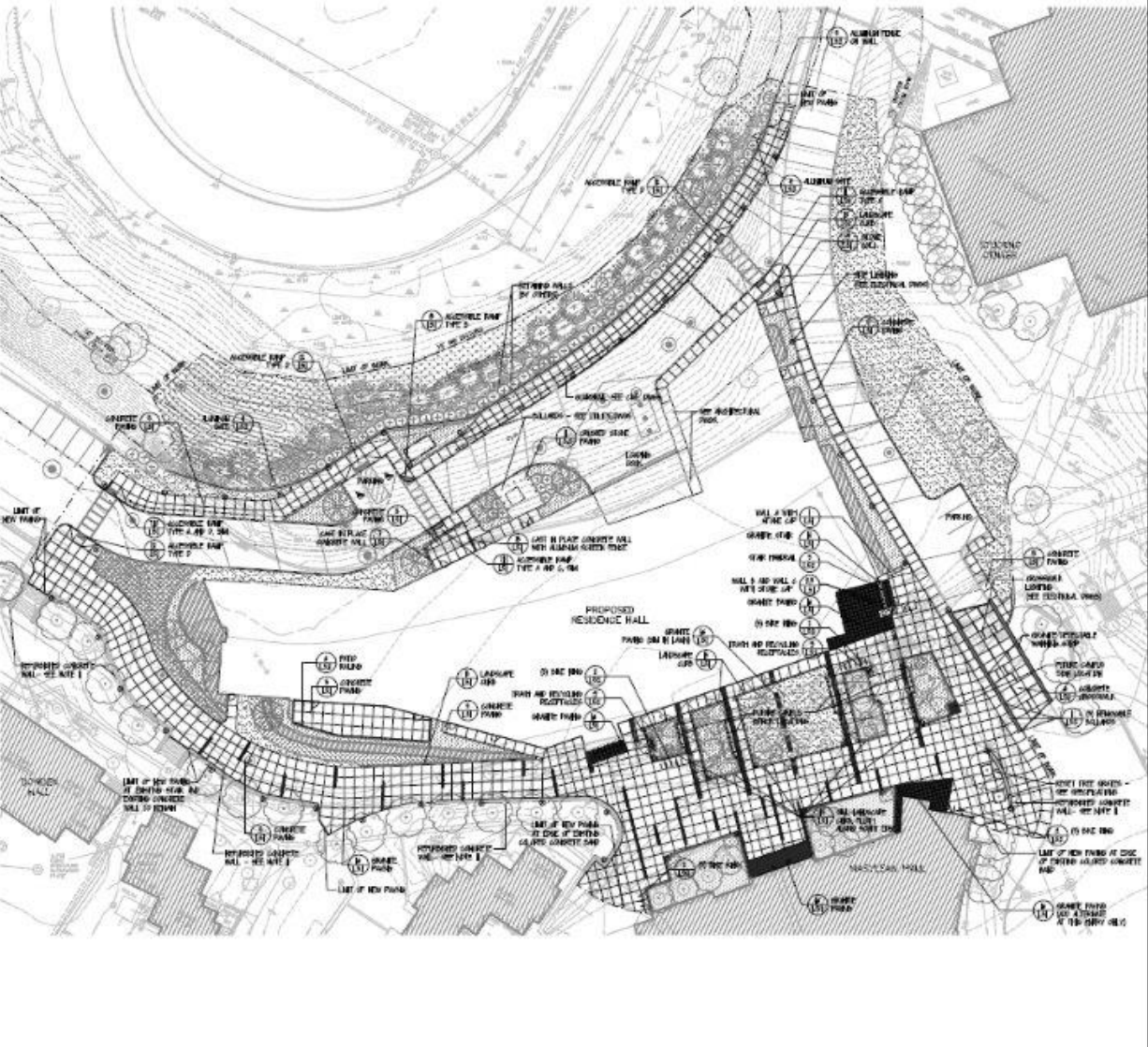
This appendix contains Consigli's proposed site development plans: Landscape Improvement plan (L1.0), Landscape Grading plan (L2.0), Landscape Layout plan (L3.0), Landscape Planting plan (L4.0), Landscape Planting Details (L5.0), Landscape Site Details (L5.1), Landscape Site Details (L5.2).



- LEGEND**
- | | | | |
|--|--------------------------------|--|--------------------------|
| | PROPOSED PAVED AREA | | PROPOSED GREEN |
| | PROPOSED 2" x 4" PAVERS | | PROPOSED INTERLOCKING |
| | PROPOSED ASPHALT PAVE. 4" x 6" | | PROPOSED LAWN |
| | PROPOSED AGGREGATE PAVE. | | PROPOSED PERMEABLE PAVE. |
| | PROPOSED LIMESTONE PAVE. | | PROPOSED SEEDED MEADOW |
| | PROPOSED LIMESTONE PAVE. | | PROPOSED STONE |
| | PROPOSED LIMESTONE PAVE. | | PROPOSED TREE |
| | PROPOSED LIMESTONE PAVE. | | PROPOSED TREE |



- NOTES**
- ALL DIMENSION LINES ARE EITHER PARALLEL TO OR PERPENDICULAR TO LINES FROM WHICH THEY ARE DRAWN UNLESS OTHERWISE INDICATED BY THE DRAWING.
 - WHERE CONSTRUCTION METHODS ARE NOT SPECIFIED, ALL LOCATIONS SHALL BE SHOWN IN THE FIELD, MATERIALS SPECIFIED AND VERIFIED BY THE CONTRACTOR AND APPROVED BY THE LANDSCAPE ARCHITECT. ADJUSTMENTS SHALL BE MADE AT NO ADDITIONAL COST TO THE OWNER.
 - SEE SPECIFICATIONS FOR MATERIALS AND INSTALLATION REQUIREMENTS.
 - BEFORE TO COMMENCE ANY EXCAVATION WORK, THE CONTRACTOR SHALL NOTIFY ALL UTILITY COMPANIES IN ACCORDANCE WITH THE "811 CALL" NOTIFICATION PROCEDURES PROVIDED BY INDUSTRY UTILITY COMPANIES.
 - WHERE PROPOSED TREES EXISTING PREVIOUSLY, THE OWNER'S PAYMENT SHALL BE SUFFICIENT TO OBTAIN A GUARANTEE BOND.
 - BEFORE TO PROCEED WITH THE WORK, LIMITS OF EXISTING UTILITIES SHALL BE LOCATED AND MARKED BY THE CONTRACTOR AT HIS/HER OWNERS RISK, SUBJECT TO THE APPROVAL OF THE LANDSCAPE ARCHITECT.
 - SEE DIMENSIONING REQUIREMENTS FOR ALL PROPOSED LIGHTING, DRAINAGE, POWER, GAS, AND ELECTRICAL UTILITY LINES AND STRUCTURES.
 - WHERE NEW PAVED AREAS EXISTING PREVIOUSLY, MEET LINE AND GRADE OF EXISTING WITH NEW PAVED.
 - EXCAVATION REQUIRED WITHIN THE SETBACK LINE OF EXISTING TREES AND UP TO 10' ABOVE THE GROUND LINE SHALL BE DONE BY HAND. CONTRACTOR SHALL TAKE EXTENSIVE CARE AND SOIL TO MAINTAIN THE ROOT SYSTEMS OF EXISTING TREES.
 - ALL PROVISIONS CONTAINED SHALL COMPLY WITH THE APPLICABLE PROVISIONS OF THE LOCAL ORDINANCES.
 - SEE SPECIFICATIONS FOR SLOPE AND AND ALL APPLICABLE WORK.
 - LANDSCAPE AND TREES SHALL COMPLY WITH THE LANDSCAPE GRADING PLAN.
 - UTILITIES DIMENSIONING SHALL COMPLY WITH THE APPLICABLE REQUIREMENTS OF THE ORDINANCES.
 - UTILITIES 5/8" DIMENSION LINE AT ALL VERTICAL STRUCTURE NECESSARY AND 5/8" DIMENSION LINE AT ALL PAVED AREAS.
 - REFER TO THE LANDSCAPE LAYOUT PLAN FOR ADDITIONAL INFORMATION.



Appendix I: Structural Design Loads & Beam Selection Hand-Calculations

of 14 Design Loads

Loads Provided by Goady & Clancy (\$0.00)

Dead Load: (Vulcraft Catalog Pg. 54) (slab wt)
 - Normal wt. Concrete = 150 pcf
 - Total slab depth = 5'-1/4"
 - Decking (DL) = 55 pcf (for all 20 gage steel & 2.5 MW concrete) and 3/4" and 2" concrete

Roof Dead Load = 1 1/2" 20 gage galv steel (Type 42) = 2.14 pcf ✓
 or 1 1/2" 19 gage " " = 2.84 pcf

Design Live Loads

* Note Load Corr. correspond to Exell file

- Residences - 40
- First Floor - 100 pcf
- Second Floor - 160 pcf

Snow Load (Ce=1, Cs=1, Z=1) Total Snow Load = 109.56 pcf + 42
 => **151.56 pcf**

$P_g = 55 \text{ pcf}$
 $P_f = 42 \text{ pcf}$ or $50 \times (2) \times (20) \times (1)$

Wind Load: IRC 1609.2

$P_{net} = P_g \times K_z \times C_{net}$

$P_g = 0.00256 V^2 = 25.6 \text{ pcf}$ ($K_z=1.0, I=1.15$)
 1100 MPH
 MWFRS Design Load = 32 pcf

Seismic Loading (site class: C)

$V = C_s W = 50_s \cdot W / (1.4)$; $C_s = 0.032; R = 3$
 $\rightarrow F_a = 1.19$

$S_s/S_1 = 0.24/0.067 \Rightarrow S_{ms} = 2.85$

$S_{0.5}/S_0 = 0.19/0.076$

Design Base shear => **V = 508 k** → given in Drawing (56.05)

Green roof	Mechanical roof	Roof
$D = 21.14 \text{ psf}$	$D = 13.0 \text{ psf}$	$D = 17.14$

ASD (spread sheet used)

$\frac{G_R}{M_R} \cdot R$ governing load eq. $D + H + F + 1.75(L) + 1.75(S) \Rightarrow 137.64 + \text{Drift}$
 $D + H + F + (0.95L) + 1.75(S) \Rightarrow 226.65 + \text{Drift}$
 $D + H + F + S \Rightarrow 87.14 + \text{Drift}$
 $\frac{h_c}{h_u} > 0.2 \Rightarrow \text{Snow drift required.}$

Drift Load: Max Intensity

$\gamma = 0.13 \times 55 \text{ psf} + 14 \Rightarrow 21.15 \text{ psf} = \text{unit weight of snow.}$

$h_u = \frac{r_f}{\gamma} = \frac{42}{21.15} = 1.99 \text{ feet}$

$h_c = [h_u - \text{height difference between units}] = 1.99 - 61.41 = -62.42 \text{ or } -749.08$
Difference of Top roof & Whisk roof

Raised Area: $137.53 \times 51.73 = 7129.64 \text{ sqft}$
 $167.07 \times 51.73 = 8639.76 \text{ sqft}$
 $170.50 \times 51.73 = 8819.57 \text{ sqft}$
 Total Raised Area = 22632.92
 Wing 1 foot = 4701
 $\frac{1}{2}(167.22 \times 97.50)$
 $\} = 26734.92 \text{ sqft}$

$\frac{h_c}{h_u} = \frac{-62.42}{1.99} = -31.37 > 0.2$ Snow drift required.

$L_u = \frac{\text{roof (level 3) top roof}}{167.07} \Rightarrow h_d = 0.43 \sqrt{L_u} \sqrt{V_g + 10} - 1.5 = (0.43)(54.717)(2.9394) - 1.5$
 $\text{Level 3} \Rightarrow \boxed{H_d = 5.18 \text{ ft}}$

$P_d = h_d \times \gamma = 5.18' \times 21.15 \text{ psf} = 109.56 \text{ psf}$
 $\frac{0.43 \sqrt{167.07} \sqrt{55+10} - 1.5 = 79.88 \text{ ft}}$

$P_{\text{total}} = P_d + P_f = 109.56 + 42 \Rightarrow \boxed{P_{\text{total}} = 151.56 \text{ psf}}$

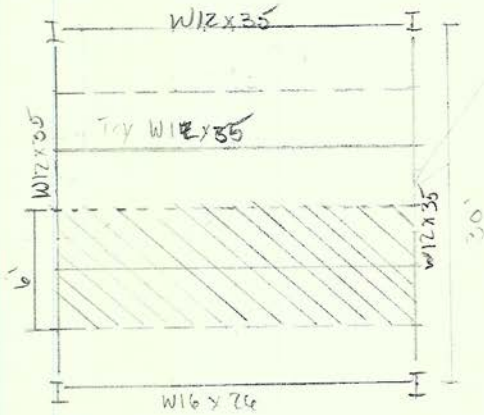
ASD Load combinations (worst case)

Green roof	$D + H + F + S =$
Mechanical roof	$D + F + S =$
Roof	$D + S =$

Width of Drift: $W = 4h_d = 4(5.18) \Rightarrow \boxed{20.72 \text{ ft}}$

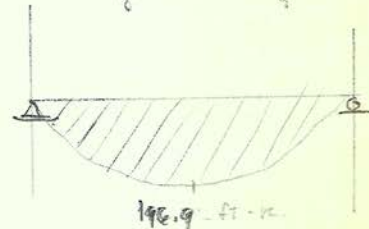
Max Bending Moment

Floor: 1 (30' x 30') Layout (worst case)



$$W_u = (DL + LL) \times S = (175) \times 6 = 1050 \text{ lbf}$$

$$M_{max} = \frac{W_u L^2}{8} = \frac{1050 \times 30^2}{8} = 196.9 \text{ ft-k}$$



- Nominal capacity $M_n = M_p = F_y Z_x \Rightarrow 50 \times (33.2 \text{ in}^3) = 1660 \text{ in-kip}$
 $= 138.3 \text{ ft-k}$

- Allowable Moment Capacity

$$M_{allow} = \frac{M_n}{\Omega} = \frac{M_n}{1.67} = \frac{F_y Z_x}{1.67} \Rightarrow \frac{138.3}{1.67} = 82.8 \text{ ft-k} < 196.9 \text{ ft-k}$$

∴ New Beam should be used.

If

$$\frac{F_y Z_x}{1.67} \geq M_{max} = \frac{W L^2}{8}$$

capacity Target

For minimum beam size to satisfy Target solve for (Z_x)

$$Z_x \geq \frac{\Omega}{F_y} \left(\frac{W L^2}{8} \right) = \frac{1.67}{50} (1050 \times 30^2 / 8) \approx 1352.64 \text{ in}^3$$

$$Z_x \geq 45.7 \text{ in}^3$$

∴ Choose W12x35 - $Z_x = 57.2$

Check:

$$M_{allow} = \frac{F_y Z_x}{1.67} > 1352 \text{ in-kip} \Rightarrow \frac{50(57.2)}{1.67} = 1532.9 \text{ in-kip} > 1352.64 \text{ in-kip} \checkmark$$

New $M_{req} = 236.25 \text{ ft-kips} \leq M_{allow} = \frac{50 \times 57.2}{1.67} = 1532.9 \text{ in-kip}$ X Try 6" Beam Spacing

Floor 1 30'x30' Bay w/ 6' Slab

$$W_u = 1050 + \left(\frac{35 \times 30}{12} \right) = 1095 \text{ k-ft} \approx 1,095 \text{ k-ft (with beam wt.)}$$

$$\Rightarrow M_{max} = \frac{1,095 (30^2)}{8} = 122.1 \text{ ft-kip Target}$$

Check: W12x35 capacity

$$M_{allow} = \frac{F_y Z_x \left(\frac{R_h}{1.67} \right)}{1.67} = \frac{50 \times 51.2 \left(\frac{1}{1.67} \right)}{1.67} = 127.74 \text{ ft-kip} > 122.1 \text{ ft-kip} \checkmark$$

\(\therefore\) For 30'x30' bays W12x35 beams are sufficient.

\(\Rightarrow\) All W14x22 beams are switched to W12x35 steel members.

Deflection check.

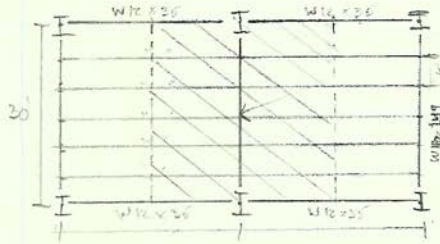
$$a) \frac{5(W_{DL} + \frac{1}{2}W_U)L^4}{384 E I_x} \leq \frac{L}{240} \Rightarrow \frac{5(110 + 50)(30^4)}{384 (29000)(225)}$$

$$\Rightarrow \dots \boxed{.000255} < .13 \checkmark$$

$$b) \frac{5(\frac{1}{2}W_U)(L^4)}{384 E I_x} \leq \frac{L}{360} \Rightarrow \frac{5(25)(30^4)}{384 \times 29000(225)} = \boxed{.03} \leq .08 \checkmark$$

\(\therefore\) W12x35 Beam is sufficient for all bays of (30'x30') and smaller.

Floor: 1 (30'x30') Moment Connection



$$W_u = 75 + 100 \times 30 = 5750 \text{ PLF} \approx 5,750 \text{ K/ft}$$

Trying Existing (W14x35; $Z_x = 66.5, I_x = 510$)
 since $L_T > 400 \text{ feet}$, must apply reduction factor.

$$LLRF = .6 \text{ for } (30' \times 30') \text{ slabs}$$

$$LL_{\text{reduced}} = 100 \times 30 \times .60 \Rightarrow 1800 \text{ PLF}$$

$$DL = 2250 + \left(\frac{35 \times 60}{30} \right) = 2320 \text{ PLF} \text{ (Beam wt included)}$$

$$W_u = 2320 \text{ PLF} + 1800 \text{ PLF} \Rightarrow 4120 \text{ PLF} \approx 4,120 \text{ K/ft}$$

Target: Max Bending Moment

$$M_{\text{max}} = \frac{(4,120)(30^2)}{8} = 4635 \text{ K-ft} \leq \text{Allowable}$$

Maximum Allowable Moment Capacity

$$M_{\text{allowable}} = \frac{Z_x F_y (1/6)}{1.67} = \frac{66.5 \times 50 \times 1/2}{1.67} = 165.92 \text{ K-ft} \times 28 = 4635 \text{ K-ft} \times$$

∴ New Beam is to be Selected.

$$\text{Solve for } Z_x \geq \frac{1.67 (M_{\text{max}} \times 12^{3/4})}{50} = \frac{1.67 (4635 \times (2^{3/4}))}{50} = 185.77 \text{ in}^3 = \text{Min. } Z_x$$

∴ Try W24x76 ($Z_x = 200, I_x = 2100$)

$$(w/\text{beam wt}) W_u = 2250 + \left(\frac{76 \times 60}{30} \right) + 1800 \text{ PLF} = 4202 \text{ PLF} \approx 4,202 \text{ K/ft}$$

$$M_{\text{max}} = \frac{(4,202)(30^2)}{8} = 472.73 \text{ K-ft} \leq (M_{\text{allowable}} = \frac{200 \times 50 \times 1/2}{1.67}) = 499.05 \text{ K-ft} \checkmark$$

∴ W24x76 Satisfies Max allowable Moment Resistance, ∴ check Deflection

Deflection

$$a) \frac{5(2402 + 400)(30^4)}{384(29000)(2100)} = 1.57 < 1.3 \times \left(\frac{1}{400} \right) \times \checkmark \quad \therefore \text{Select New beam with } (Z_x \geq 200 \text{ \& } I_x \geq 2100)$$

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Moment Connection

Floor: 2 (30' x 30') Moment Connection

∴ Try W27 x 84 ($Z_x = 244$, $I_x = 2850$)

$$\begin{aligned} \text{(Including)} \\ \text{Kerenski)} \quad w_u &= 2250 + \left(\frac{24 \times 60}{20} \right) + 1800 \text{ plf} = 4218 \quad \text{plf} \approx \frac{4.218 \text{ k/ft}}{\text{ft}} \\ &= 2250 + 168.0 + 1800 \end{aligned}$$

$$M_{\max} = \frac{(4.218)(30^2)}{8} = 474.5 \text{ k-ft} \leq \text{Allowable}$$

$$M_{\text{allowable}} = \frac{244 \times 50 \times \frac{1}{2}}{1.67} = 608.7 \geq 474.5 \text{ k-ft} \quad \checkmark$$

Deflection

$$a) \quad \frac{5(2418 + 900)(30^4)}{384(29000)(2850)} = .14 \leq .13 \quad \text{close!} \quad (4200)$$

∴ Select New beam with ($Z_x \geq 244$, $I_x \geq 2850$)Try W27 x 119 ($Z_x = 315$, $I_x = 4760$)

$$\text{(w/Beam)} \quad w_u = 2250 + \left(\frac{24 \times 60}{20} \right) + 1800 = 4308 \quad \text{plf} \approx \frac{4.308 \text{ k/ft}}{\text{ft}}$$

$$M_{\max} = \frac{(4.308)(30^2)}{8} = 484.65 \leq \text{Allowable} \quad (\text{k-ft})$$

$$M_{\text{allowable}} = \frac{315 \times 50 \times \frac{1}{2}}{1.67} = 985.5 \geq 484.65 \text{ k-ft} \quad \checkmark$$

Deflection

$$a) \quad \frac{5(2508 + 900)(30^4)}{384(29000)(4760)} = .26 \leq .13 \quad \times$$

∴ Select Lighter W-shape with
($Z_x > 315$; $I_x > 4760$)

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Floor 1 Moment
connecting.

Try W 36 x 150 ($Z_y = 581, I_x = 9040$)

$$W_u = 2250 + \left(\frac{150 \times 160}{30} \right) + 1900 = 2550 + 1000 = 4350 \text{ PLF} \approx 4.350 \text{ k/ft}$$

$$M_{max} = \frac{(4.350)(30^2)}{8} = 489.38 \text{ k-ft} \leq \text{Allowable}$$

$$\text{Allowable} = \frac{581 \times 50 \times \frac{1}{12}}{1.67} = 1449.60 \text{ k-ft} \geq 489.38 \text{ k-ft} \checkmark$$

Deflection

$$\frac{5(2550 + 900)(30^4)}{384 \times 29000 \times 9040} = .14 \leq .13 \text{ X closer!}$$

Try W 40 x 149 ($Z_y = 598, I_x = 9900$)

$$W_u = 2250 + \left(\frac{149 \times 160}{30} \right) + 1900 = 2548 + 1900 = 4348 \text{ PLF} = 4.348 \text{ k/ft}$$

$$M_{max} = \frac{4.348 \times (30^2)}{8} = 489.15 \text{ k-ft} \leq \text{Allowable}$$

$$\text{Allowable} = \frac{598 \times 50 \times \frac{1}{12}}{1.67} = 1449.6 \text{ k-ft} \geq 489.15 \text{ k-ft} \checkmark$$

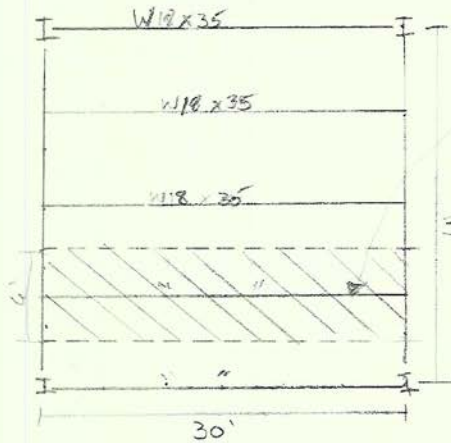
Deflection

$$a) \frac{5(2548 + 900)(30^4)}{384 \times 29000 \times 9900} = .13 \leq .13 \checkmark \quad (4/240)$$

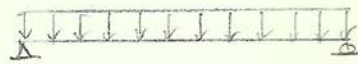
$$b) \frac{5(900)(30^4)}{384 \times 29000 \times 9900} = .03 \leq .08 \quad (4/360)$$

∴ for 30' x 30' bay W 40 x 149 satisfies all Moment & Deflection checks

Floor: 2 (35' x 20') Layout



$$w_u = (DL + LL) \times S = [(75 + 26) + (120)] \times 6 = 1326.0 \text{ PLF}$$



$$M_{max} = \frac{w_u L^2}{8} = \frac{1326 \text{ PLF} \times (30')^2}{8}$$

$$M_{max} = 149.1 \text{ ft-kips Target}$$

$$\text{Nominal capacity } (M_n) = M_f = F_y \times Z_x = 50(44.6) = 2230 \text{ in-kip} \approx 185.8 \text{ ft-k}$$

$$M_{allow} = \frac{M_n}{1.67} = \frac{185.8 \text{ ft-k}}{1.67} = 111.29 \text{ ft-kips} < 149.19 \text{ ft-kips} \quad \times$$

∴ Select New beam size: solve for (Z_x)

$$Z_x \geq \frac{(1.67)(149.19 \times 12'')}{50} = 59.79 \text{ in} \quad \therefore \text{Try } W18 \times 35 (Z_x = 66.5, I_x = 510)$$

$$w_u = ((75 + 26) + (120)) \times 6 = 1326 \text{ PLF} \approx 138 \text{ kips/ft (with beam wt)}$$

$$M_{max} = \frac{(138)(30')^2}{8} = 155.25 \text{ ft-kips}$$

$$\text{Nominal Moment capacity} = F_y \times Z_x = 3325.0 \text{ in-kips} \approx 277.09 \text{ ft-kips}$$

$$M_{allowable} = \frac{277.09 \text{ ft-kips}}{1.67} = 165.9 > 155.25 \quad \checkmark$$

∴ Satisfies Allowable moment

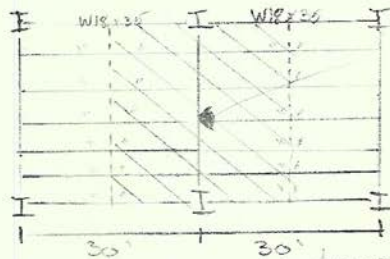
Deflection checks (2)

$$a) \frac{5(116 + 60)(30'')^4}{384(29000)(510)} = .12 \text{ in} < .13 \text{ in} \quad \checkmark$$

$$b) \frac{5(12 \times 170)(30'')^4}{384(29000)(510)} = .09 \text{ in} \leq .08 \quad \checkmark$$

∴ The W18 x 35 steel beams provide sufficient Moment Capacity and Allowable Levels of deflections, ∴ all previous W16 x 26 & W18 x 24 can be replaced with tested beam.

(30' x 30') Vertical column supporting members



$$W_u = (110 + 120) \times 30' = 6900 \text{ lbf} = 6.9 \text{ k/ft}$$

$$M_{max} = \frac{6.9 \times (30')^2}{2} = 776.3 \text{ kft-ft} \quad \text{Target}$$

$$\text{Solve for: } (Z_x) \geq \frac{1.67(776.3 \times 12)}{50} = 311.12 \text{ in}$$

(Horizontal axis)

∴ Try W14 x 176 ($Z_x = 320, I_x = 244$)

$$W_u = 6900 + \left(\frac{176 \times 60}{30}\right) = 7252 \text{ lbf} = 7.252 \text{ k/ft} \quad M_{max} = \frac{(7.252)(30')^2}{8} = 815.75 \text{ ft-kips}$$

$$M_{min} = \frac{M_u}{1.67} = F_y \times Z_x = 50 \times 320 = 9580.8 \text{ in-kips} = 798.40 \text{ ft-kips} > 1370.25 \text{ ft-kips} \quad \times$$

* IF $A_T > 400 \text{ sqft}$, calc Live Load reduction factor *

$$\text{Influence Area} = A_T = 2A_T = 2 \times 900 = 1800 \text{ sqft}$$

$$\Rightarrow \text{LL reduction factor} = 0.25 + \left[\frac{15}{\sqrt{1800}} \right] = 1.60$$

$$\text{New LL} = 120 \text{ lbf} \times 30' \times 1.60 = 2160 \text{ lbf} = 2.160 \text{ ft-kip}$$

$$W_u = 3652 + 2160 = 5812 \text{ lbf} = 5.812 \text{ ft-kip}$$

$$M_{max} = \frac{(5.812)(30')^2}{8} = 653.25 < 798.40$$

∴ w/ LL-reduction, W14 x 176 satisfies Max Moment Capacity

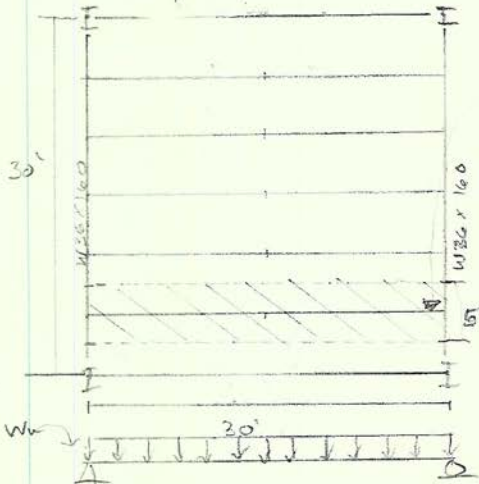
Check Deflection:

$$a) \frac{5(346)(30')^4}{324(29000)(2140)} = .06 < \left[\frac{L}{240} = .13 \right] \quad \checkmark$$

$$b) \frac{5(220)(30')^4}{324(29000)(2140)} = .01 < \left[.08 = \frac{L}{360} \right] \quad \checkmark$$

∴ For Floor: 2 (30' x 30') bays W18 x 35 (Horizontal) & W14 x 176 (Vertical) beams satisfy The Max Moment Capacity as well as providing allowable levels of Deflection and Existing Beams do NOT!

Floor: 3 (30' x 30') Layout (5' spacing)



$$w_u = (DL + LL) \times S = \left[(25 \times 6) \times 1.90 \frac{1}{ft} \right] + 100 \frac{1}{ft}$$

(with beam) $w_u = 594 \text{ PLF} + 600 \text{ PLF} = 1194 \text{ PLF} = 1194 \frac{1}{ft}$

$$M_{max} = \frac{(1194 \frac{1}{ft}) (30^2)}{8} = 134.3 \text{ k-ft (target)}$$

If $Z_x = (294 \text{ in}^3)$ ($I_x = 2250 \text{ in}^4$) for W27 x 84

$$\text{Then } \frac{M_n}{1.67} = \frac{244 \times 50}{1.67} = 608.7 \text{ PLF} > 134.3 \text{ k-ft} \quad \checkmark$$

∴ Capacity is too high, can be reduced

∴ Select New Trial W-shape, solve (Z_x)

$$Z_x \geq \frac{1.67 (M_{max} \times 12)}{50}$$

calc M_{max} (w/out beam wt) = $w_u = 1110 \text{ PLF} = 1.110 \frac{1}{ft}$

$$M_{max} = \frac{1.110 \frac{1}{ft} (30^2)}{8} = 124.9 \text{ k-ft (target)}$$

$$Z_x \geq \frac{1.67 (124.9 \times 12)}{50}$$

$$Z_x \geq 53.8$$

Beam Selection Criteria

∴ Select: W12 x 40 ($Z_x = 57.0 \text{ in}^3$, $I_x = 307$)

$$M_n = M_r = Z_x \times F_y = 57.0 \times 50 = 2850 \text{ in-kip} = 237.22 \text{ ft-kip}$$

$$M_{allowable} = \frac{237.22}{1.67} = 142.22 > 124.9 \text{ k-ft} \quad \checkmark$$

Check Allow include Beam wt

(w/ beam wt) $w_u = 1110 \text{ PLF} + 40 \frac{1}{ft} = 1150 \text{ PLF} = 1.15 \frac{1}{ft} \Rightarrow M_{max} = \frac{(1.15)(30^2)}{8} = 129.375 \leq 142.22$

∴ W12x40 Beam Passes Check for sufficient Moment Resistance.

Deflection Check

$$a) \frac{5(550 + \frac{1}{2}(600))(30^4)}{394(24000)(307)} = 1.01 \leq 1.3 \quad \checkmark$$

∴ Choose Larger beam w/ higher (I_x Value).

From before: $Z_x \geq 53.8$ & $I_x \geq 304 \Rightarrow$ Try W18x25 ($Z_x = 66.5$ & $I_x = 510 \text{ in}^4$)

$$W_u = 600 + 510 + 35 = 1145 \text{ PLF} = 1.145 \text{ k/ft}$$

$$M_{max} = \frac{(1.145)(30^2)}{8} = 128.91 \text{ k-ft} \leq \text{Allowable}$$

$$\text{Allowable} = \frac{(50 \times 66.5)(1/2)}{1.67} = 165.92 \text{ k-ft} \geq 101.81 \text{ k-ft} \checkmark$$

Re-check Deflection

a) $\frac{5(545+300)(30^4)}{384(29000)(510)} = .66 \leq .13 \times$ I_x for beam ≥ 570

\therefore Try W24x68 ($Z_x = 177 \text{ in}^3$, $I_x = 1830$)

$$W_u = DL = (510 + 68) + (600) = 1178 \text{ PLF} = 1.178 \text{ k/ft}$$

$$\text{Target } M_{max} = \frac{(1.178)(30^2)}{8} = 132.5 \text{ k-ft} \leq \text{Allowable (over design)}$$

$$\text{Allowable} = \frac{177 \times 50 \times 1/2}{1.67} = 441.62 \geq 132.53 \text{ k-ft} \checkmark$$

Deflection

a) $\frac{5(578+300)(30^4)}{384(29000)(1830)} = .17 \leq .13 \times$ close!

\therefore Test W24x69 capacity w/50ksi

$$W_u = (75 \times 5^3) + 68 + (100 \times 5^3) = 493 + 500 = 993 \text{ PLF} = .993 \text{ k/ft}$$

$$\text{Target } M_{max} = \frac{(993)(30^2)}{8} = 111.71 \text{ k-ft} \leq \text{Allowable}$$

$$\text{Allowable} = 441.62 \text{ FT-kip} \geq 111.71 \text{ k-ft} \checkmark$$

Deflection:

a) $\frac{5(493+250)(30^4)}{384(29000)(1830)} = .15 \leq .13 \times$

b) $\frac{5(250)(30^4)}{384(29000)(1830)} = .05 \leq .8 \checkmark$

\therefore For Horizontal supporting members, W24x69 satisfies the allowable moment resistance, but NOT the Deflection checks. \Rightarrow Choose another W-shape w/ ($Z_x \geq 177 \text{ in}^3$, $I_x \geq 1830 \text{ in}^4$)

Since: $Z_y \geq 177$ & $I_x \geq 1830 \text{ in}^4$

Try W24 x 76 ($Z_y = 200$, $I_x = 2100 \text{ in}^4$)

With Beam wt $W_u = 425 \text{ PLF} + 76 \frac{1}{8} \text{ PLF} + 500 \text{ PLF} \Rightarrow 1001.0 \text{ PLF} = \frac{1.001 \text{ k/ft}}$

$$M_{max} = \frac{(1.001)(30^2)}{8} = 112.61 \text{ k-ft} \leq \text{Allowable}$$

$$\text{Allowable} = \frac{200 \times 50 \times \frac{1}{2}}{1.67} = 499.39 \text{ k-ft} \geq 112.61 \text{ k-ft} \quad \checkmark$$

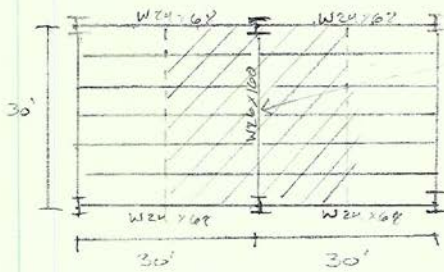
Check Deflection

a) $\frac{5(501+250)(30^4)}{384(29000)(2100)} = .13 \leq .13 \quad \checkmark$ (4/20)

b) $\frac{5(250)(30^4)}{384(29000)(2100)} = .04 \leq .08 \quad \checkmark$ (4/20)

∴ W24 x 76 satisfies both Moment Resisting & Deflection checks and can adequately replace all existing W27 x 84.
Spacing is to be no less than 5'

Floor 2 Vertical Supporting Members (30' x 30')
(Moment connection)



$$W_{un} = (15 + 100) \times 20 = 5550 \text{ PLF} \rightarrow \frac{160 \text{ PLF} \times 16}{30}$$

$$= 5870 \text{ PLF (with beam)}$$

$$\approx 5.870 \text{ K/FT}$$

$$M_{max} = \frac{(5.870)(30^2)}{8} = 660.4 \text{ K/FT} \leq \text{Allowable}$$

For W36 x 160 ($E_s = 29,000$, $I_s = 97600$)

$$M_{allowable} = \frac{624 \times 50 \times 1/2}{1.67} = 1536.89 \geq 660.4 \text{ K/FT} \checkmark$$

∴ over design ⇒ reduce if
excess deflection

Deflection

$$a) \frac{5 \left(\frac{2350 + 320}{29000} \right) (30^4)}{384 (29000) (97600)} = .16 \leq .13 \quad (1/240)$$

∴ Beam didn't satisfy Deflection Before LL-reduction ⇒ Calc LL-reduction.

LL-reduction = .60 for (30' x 30' bays)

$$(LL = 30000)(.60) = 19000 \text{ PLF}$$

re-check deflection

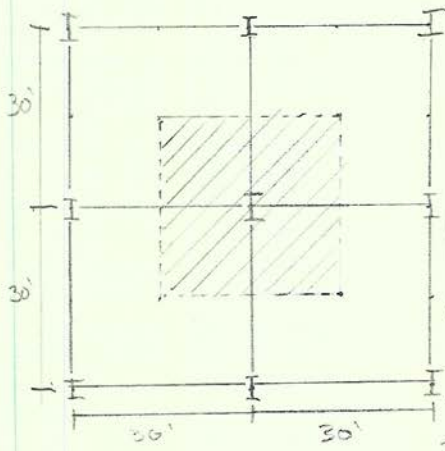
$$a) \frac{5 (2970 + 900) (30^4)}{384 \times (29000) \times 97600} = .00069 \leq .13 \quad (1/240) \checkmark$$

∴ W36 x 160 (Existing)
Satisfies both allowable
Moment Resistance & Deflection
Checks.

$$b) \frac{5 (900) (30^4)}{384 (29000) (97600)} = .03 \leq .08 \quad (1/600) \checkmark$$

Roof Column B'-7

Axioms



Tributary Area

$$A_T = 30 \times 30 = 900 \text{ sq. ft.}$$

Roof Design Loads

(green roof) $D = 21.14$ $S = 1667.7$
 (roof) $D = 47.14$ $S = 1667.7$

CR

$$D = 21.14 \times 900 = 19026 \text{ psf} \quad / \quad S = 1500930 \text{ k-ft}$$

R

$$D = 47.14 \times 900 = 42426 \text{ psf} \quad / \quad S = 1500930 \text{ k-ft}$$

CR(w)

$$W = 1573956 \text{ k-ft}$$

R(w)

$$W = 1543366 \text{ k-ft}$$

For Column B'-7

Influence Area:

$$A_I = 4 A_T = 3600 \text{ sq. ft.}$$

LL-reduction

$$0.25 + \frac{15}{A_I} = .50$$

New Snow Load

$$S = 1667.7 \times 900 \times .50$$

$$S = 750465 \text{ psf} \approx \underline{750.5 \text{ kips}}$$

$$D + S = 42426 \text{ kips} + 750.5$$

$$\Rightarrow \boxed{792.93 \text{ kips}}$$

Appendix J: Structural Design Loads & Beam Design Hand-Calculations

1 of 11

JOIST DESIGN

29'-9" x 29'-2 1/4"

DL: 25 psf L: 100 psf
 Span: 29'-2 1/4" Spacing = 2.92' spacing (15ft)
 DL: 2.92 (75) + 10 = 277.25 PLF
 LL: 2.92 (100) = 292 PLF
 Total Load => 570.25 PLF ∴ use 20K10
 load conn.

Actual
 DL: 230 psf
 LL: 292 psf = 326 ✓
 Total Load: 522 psf = 533 ✓

NOTES
 * Joists are to be oriented Perpendicular to Shortest Span Length of Bay for Maximum Strength
 * Joist selection is governed by selecting the lightest joist possible within the small depth.
 * (Vulcraft 5-7): Joists are to be spaced so that the load on any single joist does not exceed the design load (ASD)

Other Possible Joist Selections (p. 53-56)

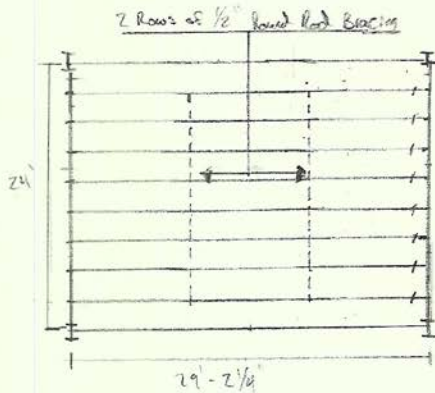
Type	WT (lb)	Depth (in)	Live Load (psf)
20K10	11.6	20"	396
22K10	11.7	22"	385
22K11	11.9	-	-
24K9	10.8	24"	419
24K10	11.7	-	422
24K12	13.5	-	-
26K9	9.7	26"	457
26K9	10.4	-	459
26K10	11.8	-	-

← Best choice for size restricted design ∴ use (10) 20K10's (11 spans)
 ← Best choice for lightest possible selection

Bridging Requirements for 20K10 (p. 36 Vulcraft Steel Joist Catalog)
 section: 10 } (2) Rows of 1/2" Round Rod Horizontal Bracing.
 Spacing: 2.92 } ∴ Good for 12'-6" (horiz.) & 16'-7" (diag)

∴ (10) 20K10 Joists to be used @ 2.92' spacing (11 spans) w/ (2) Rows of 1/2" Round Rod Horizontal Bracing.

Total WT of steel = (no. of Joists x span length) x (joist self wt.)
 Joists
 => (10 Joist x 29.1875') x (11.6 lb/ft) = 3385.75 lbs of steel Joists



Total Load = 175 PLF LL = 100 PLF
DL = 75 PLF
 Assuming Joists + Accessory Weights = 117 PLF
 Span = 29'-2 1/4" ∴ Try Smallest Beam (22K10)
 3' spacing w/ 7 spaces
 $D = 3(75 PLF) + 117 PLF \Rightarrow D = 2367 PLF$
 $LL = 3(100) \Rightarrow LL = 300 PLF$

Governing Load Combination

$D + H + F + L + T = 2367 + 300 \Rightarrow 5367 PLF$

∴ For span less than 20', Try 22K10 = 550 max Load
 385 max Live Load

⇒ for 3' spacing, 22K10 steel joists meet required loadings

Total Load = 5367 PLF ≤ 550 + 385 > 300 ✓

Possible Bar Joist Selection:

Type	WT (k/ft)	Depth (in)	Live Load (PLF)
22K10	11.3	22"	385
22K11	11.9	-	-
24K9	10.3	24"	419
24K10	11.2	-	422
24K12	13.5	-	-
26K8	9.7	26"	452
26K9	10.4	-	459
26K10	11.2	-	-

Best choice for AHJ design (Smallest Depth)

Typical choice (Lightest Possible)

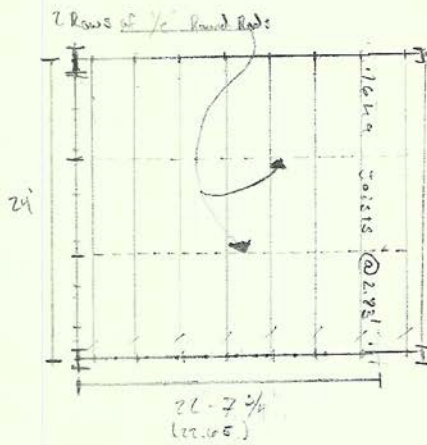
Bridging Requirements for 22K10 @ 3' spacing & spanning 29'-2 1/4"

⇒ Vulcraft (p.36) Sections 10 } 1/2" diameter
 spacing: 3'

(2K x 7/32) Bridging 1/2" Round Rod good for 12'-0" & 16'-7"
 (Horizontal) (Diagonal)
 * TWO rows of bracing are need for span of 29'-2 1/4"

∴ (8) 22K10 Joists to be used @ 3' spacing with two rows of 1/2" Round Rod K-series Bracing.

Total wt of Steel = 2731.95 lbs
 Joists



DL = 75 psf ; LL = 100 psf

Assuming Joists ; Accessories = 10 psf.

Span = 24'-0"

D: 2.22'(75) + 10 = 222.25 PLF.

L: 2.22(100) = 222 PLF < 363 ✓

Total load = 535 PLF. < 550 ✓ (for 16k9)

Governing Load Combination

D + H + F + L + T = 235 + 200 = 535 PLF

Possible Joist selection (Manual X - Series Load Table: p. 85-86)

Type	WT (k/ft)	depth (in)	Live Load (PLF)
16k9	10	16"	363
18k7	9.9	18"	418
14k9	10.1	-	-
18k10	11.6	-	-
20k6	9.4	20"	469
20k7	9.9	-	-
20k9	10.1	-	-
20k10	11.6	-	-
20k5	7.7	27"	519

← choose for "governed by member dimensions."

← Choose for design governed by WT (lightest selection)

∴ chose 16k9 joist due to its small depth and also because Alt. Design is restricted to Architectural Design for floor-ceiling heights.

Bridging Requirements for 16k9

Section #: 9 } 2 Rows of 1/2" Round Rod will be used.
desired spacing: 2.22'

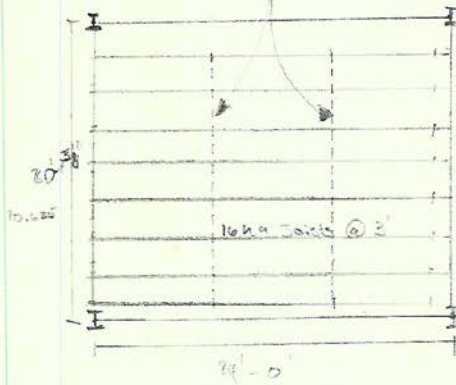
∴ good up to length of 12'-6" (Horizontal) & 16'-7" (Diagonal)

∴ (7) 16k9 Joists to be used @ 2.22' spacing & 9 spaces (7 equal w/ 2 small spaces)
⇒ 2 Rows of 1/2" Round Rod to be used for horizontal X-Series Joist Bracing

Total wt of steel Joists = [(# of Joists) × (Span length)] × Self wt of Joist
⇒ Total wt = (8 × 24') × 10 k/ft ⇒ 1,920 lbs of Steel Joists

2 Rows of 1/2" Round Rod Bracing

DL 75 psf & LL 100 psf



Span = 24'

Assume Joist & Accessories = 10 psf

∴ Try 3.0' spacing = 8' Joists

DL: $3.0 \times (7.5) + 10 = 235 \text{ PLF}$

LL: $3.0 \times 100 = 300 \text{ PLF} < 396 \checkmark$

Total load = 535 PLF ∴ Try 16k9

Governing Load combination

$D + H + F + L + T = 235 + 300 = 535 < 550 \checkmark$

Possible Joist Selection

Type	WT (lb)	Depth (in)	Clearance
16k9	10.5	12"	24 1/2"
18k9	10.1	13"	23 1/2"
12k10	11.6	-	-
10k7	8.9	20"	21 1/2"
10k9	10.1	-	-
10k10	11.6	-	-
17k5	7.7	22"	4 1/2"
17k6	8.5	-	4 1/2"
17k7	9.0	-	-
17k9	10.2	-	-
17k10	11.7	-	-
17k11	11.9	-	-

← Choose for design governed by member dimension.

← Choose for design governed by WT (Lightest)

∴ Choose 16k9 Joists for smallest Depth and Average WT.

Bridging Requirements for 16k9

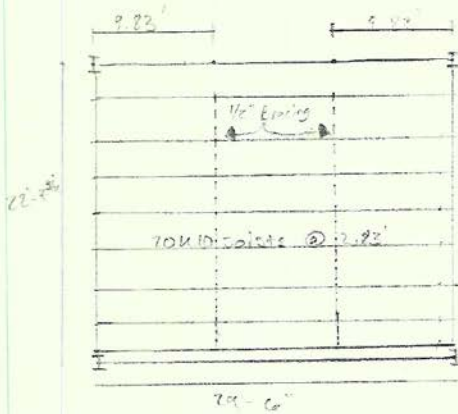
Span: 24' } 2 Rows of 1/2" Diameter Round Rods will be used.
 Spacing: 3' } ∴ good w/ the lengths of 12'-6" & 16'-7" (Horizontal) (Diagonal)

∴ (8) 16k9 Joists to be used @ 3' spacing w/ 9 braces & 2 Rows of 1/2" Horizontal Round Rod Bracing.

Total WT of Steel ∴ 1920 lbs of steel
 Joist

Joist Design

29'-6" x 22'-7 3/4"



DL = 25 PLF W_f = 100
 Spans: 20'-6" Try 2'-8 1/2" spacing = 8 Joists.
 U.L: 2'-8 1/2" (75) + 10 = 272.25 PLF
 L.L: 2'-8 1/2" (100) = 223.0 PLF
 Total Load = 505.25 PLF
 Governing Load Combination (D+L+T+L+T)
 (w/ joist wt) ∴ Use 20K16
 Actual DL = 272.25 PLF 506.15 < 533 ✓
 Actual LL = 223.0 PLF 423.6 < 336 ✓

Other Possible Joist Selections

Type	WT (lb)	W _f (in)	(PLF) Live Load
20K10	11.6	20"	236
22K10	11.7	22"	325
22K11	11.9	-	-
24K9	10.3	24"	419
24K10	11.7	-	422
24K12	13.5	-	422
26K8	9.7	26"	457
26K9	10.4	-	459
26K10	11.2	-	-
26K12	13.7	-	-

← Best choice for size restricted design

← Best choice for wt. restricted design

∴ Choose 20K10 Joist due to Architectural size limitations.

Bridging Requirements for 20K10

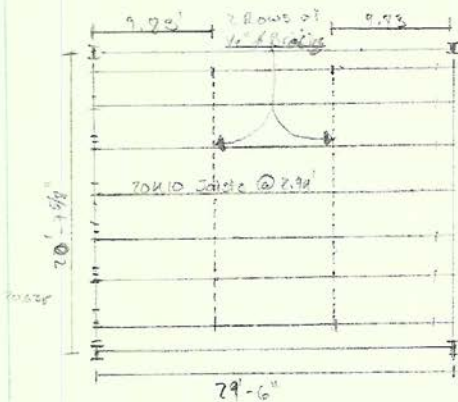
Section #: 10 2 Rows of 1/2" ∅ Horizontal Round Rod Bracing
 Spacing: 2'-8 1/2" * Good up to 12'-6" (Horizontal) & 12'-7" (Diagonal)

(8) 20K10 Joists to be used @ 2'-8 1/2" spacing (8 spans) w/ 2 Rows of 1/2" ∅ Round Rod Horizontal Bracing.

Total WT of Steel = 2,937.6 lbs.
 Joists

Joist Design

29'-6" x 70'-7 1/2"



DL = 75 psf ; LL = 100 psf
 Span: 29'-6" for 2'-9 1/2" spacing = 7 Joists
 $DL = 2.94 (75) + 10 = 230.5 \text{ PLF}$
 $LL = 2.94 (100) = 294 \text{ PLF}$
 Total Load
 Governing Load Comb. = 524.5 PLF
 ∴ Use 20x10

Actual DL = 232.10 PLF
 LL = 294.0 PLF < 326 ✓
 Total load = 526.1 < 522 ✓

Other Possible Joist Selections

Type	WT (lb)	Span (ft)	(DL+LL) Dead Load
20x10	11.6	25'	336
22x10	11.7	22'	325
22x11	11.9	-	-
24x9	10.3	24'	414
24x10	11.2	-	427
24x12	13.5	-	-
26x10	9.7	26'	457
26x11	10.4	-	459
26x12	11.2	-	-
28x12	13.7	-	-

← Best choice for size restricted design.
 ← Best choice for WT. Restricted design.
 ∴ Choose 20x10 Due to Architectural Size Limitations.

Bridging Requirements for 20x10

Section #: 10 } 2 Rows of 1/2" Round Rod Horizontal Bracing
 Spacing: 2'-9 1/2" }
 * good at 12'-6" (Horizontal) & 16'-7" (Diagonal)

∴ (7) 20x10 Joists to LR used @ 2'-9 1/2" Spacing (2 Rows) w/ 2 Rows of 1/2" Round Rod Horizontal Bracing.

Total WT of Steel = 2395.4 lbs
 Joists

DL = 75 psf LL = 100 psf.

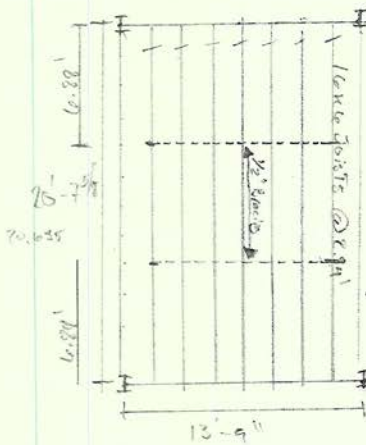
Span: 20'-7 1/2" Try 2.94' Spacing = 7 Joints

DL: 7.94(75) + 10 psf = 230.5 PLF

LL: 2.94(100) = 294 PLF

Total Load = 524.5 PLF

∴ Use 16K6



Actual

DL = 228.6 PLF

LL = 294 psf < 405 ✓

Total Load = 522.6 PLF < 549 ✓

Other possible JOINT selections:

Type	WT (lb/ft)	Depth (in)	Self-Weight (psf)
16K6	8.1	16"	405
16K7	8.6	-	406
16K9	10.0	-	-
18K5	7.7	18"	460
19K6	8.4	-	-
19K7	8.9	-	-
18K9	10.1	-	-
19K10	11.6	-	-
20K4	7.2	20"	576
20K5	7.7	-	-
20K6	8.4	-	-

← Best choice for size restricted design.

∴ Choose 16K6 due to Architectural size limitations.

← Best choice for Lightest possible selection.

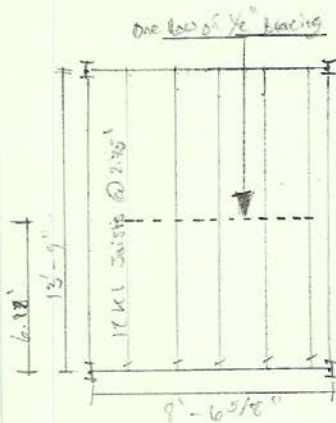
Bridging Requirements for 16K6

Section #16 } 2 rows of 1/2" Round Rod Horizontal Bracing
Spacing: 2.94

* good when 12'-6" & 16'-7" (Horizontal) (Diagonal)

(7) 16K6 Joints to be used @ 2.94' spacing (8 spaces) w/ 2 rows of 1/2" Round Rod Horizontal bracing.

Total WT of steel = 1170.03 lbs. Joints



DL: 75 R/F LL: 100 R/F

Span: 12'-9" Try 2.75' spacing = 5 JOISTS 6 JOISTS

DL: 270(75) + 100 R/F = 214.25 R/F

LL: 2.75(100) = 275 R/F

Total Load = 491.25 R/F USE 12K1
Load comb.

Actual

DL: 211.25 R/F

LL: 275 R/F < 425 ✓

Total Load: 486.25 < 500 ✓

Other possible joist selections

Type	WT (lb/ft)	Span (ft)	Live Load (R/F)
12K1	5.0	12"	425
12K3	5.2	-	425
12K5	2.1	-	-
14K1	5.2	14"	550
14K3	6.0	-	-
14K4	6.7	-	-
14K6	7.7	-	-

← Best choice for size restricted design & lightest possible selection.

∴ Choose 12K1 (5)

Bridging Requirements for 12K1

Section #1:1 } one row of 1/2" Round Rod Horizontal Bracing.
Spacing: 2.75'

* good for up to 12'-6" (horiz.) & 16'-7" (diag.)

∴ (5) 12K1 Joists to be used @ 2.75' spacing (6 spans) w/ one row of 1/2" Round Rod Horizontal Bracing.

Total Wt. of steel = 343 lbs
Joists

Joint Design

9'-3" / 22'-7 3/8"



DL: 75 psf LL: 100 psf

Span: 22'-7 3/8" Try 2.73' spacing (9 joists)

DL: $2.73(75) + 10 \text{ psf} = 222.25 \text{ PLF}$

LL: $2.73(100) = 273 \text{ PLF}$

Total Load = 505 PLF Use: 16K7

Load comb.

Actual

DL: 270.85 PLF

LL: 283 PLF < 339 ✓

Total Load: 503.85 PLF < 507 ✓

Other possible joint solutions

Type	WT (lb/ft)	Area (sq ft)	Live load (psf)
16K7	8.6	16"	339
16K9	10.0	-	363
18K6	9.4	18"	393
18K7	9.9	-	412
18K9	10.1	-	-
18K10	11.6	-	-
20K5	7.7	20"	451
20K6	8.1	-	462
20K7	8.9	-	-
20K9	10.1	-	-

← Best choice for size restricted design

← Best choice for lightest possible selection

∴ choose 16K7 (8) 9 joists

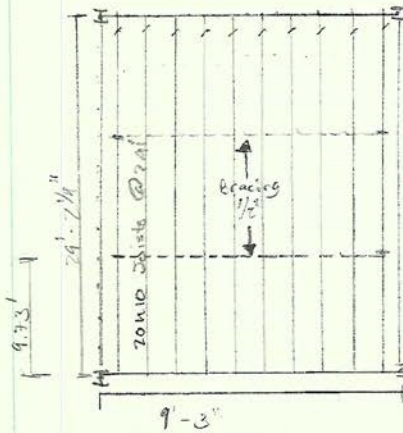
Bridging Requirements for 16K7

Section #: 7 } one 1/2" Round Rod Horizontal Bracing.
Spacing: 2.73'

* good for up to 12'-6" (width) & 16'-7" (diag)

∴ (8) 16K7 Joists to be used @ 2.73' spacing (9 joists) w/ one row of 1/2" Round Rod Horizontal Bracing.

Total WT of Steel = 1558.02 lbs.
Joists



DL: 75 psf LL: 100 psf

Span: 29'-2 1/4" Try 2.92' Spacing = 10 Joists

$DL = 2.92(75) + 10 = 229 \text{ PLF}$

$LL = 2.92(100) = 292 \text{ PLF}$

Total Load = 520.5 PLF

∴ use 20x10

Load combination

Actual

$DL = 236 \text{ PLF}$

$LL = 292 \text{ PLF } 326 \checkmark$

Total Load = 522.6 PLF 523 \checkmark

Other possible Joist selections

Type	WT (lb/ft)	Depth (in)	Live Load (psf)
20x10	11.6	20"	326
22x16	11.7	22"	395
22x11	11.9	-	-
24x9	10.3	24"	419
24x16	11.7	-	402
24x12	13.5	-	-
26x8	9.7	26"	457
26x9	10.4	-	459
26x10	11.8	-	-

Best choice for size restricted Design

∴ use (10) 20x10's (11 spans)

Best choice for Lightest possible selection.

Bridging Requirements for 20x10

Section 4:10 } 2 Rows of 1/2" Round Rod Horizontal Bridging.
 Spacing: 2.92' } * good for 16'-6" & 16'-7" (Horiz) (Vert)

∴ (10) 20x10 Joists to be used @ 2.92' spacing (11 spans) w/ (2) Rows of 1/2" Round Rod Horizontal Bridging.

Total WT. of steel = 3385.75 lbs Joists

Appendix K: Electronic Files Directory

The following is a list of the contents of Appendix K:

- 1. Consigli's Baseline Schedule (PDF)**
- 2. Consigli's Updated As-Built Schedule (PDF)**
- 3. Consigli's 3D Model (RVT)**
- 4. Consigli's BIM Model (NWD)**
- 5. Baseline Schedule (XER)**
- 6. As-Built Schedule (XER)**
- 7. Alternative Design Schedule (XER)**
- 8. Baseline vs. As-Built Schedule Comparison (XLS)**
- 9. Alternative Design 3D (RVT)**
- 10. Planned vs. As-Built 5D (NWF)**
- 11. Alternative Design 5D (NWF)**
- 12. Planned vs. As-Built 5D Animation (AVI)**
- 13. Alternative Design 5D Animation (AVI)**
- 14. WSU Site Photos (ZIP)**
- 15. Structural Load Calculation, Frame and Joist Design Level 1-Roof (XLS)**
- 16. Structural Column Capacity & Footing Dimension Calculations (XLS)**



Worcester State University Sheehan Hall MQP:
Project Management and Alternative Design

A Major Qualifying Project Proposal

Submitted to the faculty of
WORCESTER POLYTECHNIC INSTITUTE
In partial fulfillment of the requirements for the
Degree of Bachelor of Science

Submitted on:

12/19/2013

Submitted by:

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Submitted to:

Project Advisor: Guillermo F. Salazar

Abstract

This project proposes an alternative design for Worcester State University's Sheehan Hall residence dormitory and compares it to the existing design in terms of scheduling and costs. It also reviews project management services including scheduling, cost and lean construction. Building Information Modeling will be used to visualize the impacts of the alternative design and create a 5D model, which will be used to compare the planned cost and schedule to the actual cost and schedule of the project.

Capstone Design Statement

1.0 Introduction

Educational institutions all over the world are drawing in more and more students each year, partly owing to the fact that an increasing number of people are realizing the value and importance of higher education nowadays (*Admission Statistics*, 2013). This may be beneficial for universities and colleges in the sense that they are educating an increasing number of the population while generating greater revenue and growing in size, but an increasing student population also demands more on-campus facilities such as dormitories, cafeterias, etc. Many universities and colleges have very limited on-campus accommodation, meaning that a large number of students must live elsewhere and commute to campus, which is not ideal. Due to this increasing demand for construction within the education sector, the construction industry is witnessing a growing number of projects for educational buildings (*Construction Market Research*, 2013).

Such is the case of Worcester State University (WSU). Located in a residential neighborhood on the west of Worcester, MA, WSU is a commuter-heavy university that is currently facing the same problem of not being able to provide enough housing for its current student population. In an effort to address this problem and keep more students on campus, WSU is currently constructing a new facility, namely Sheehan Hall (Kotsopoulos, 2012). It is imperative that this new facility is completed on time and within budget because it needs to be ready for move-in by fall 2014. When completed, Sheehan Hall will rise six stories beside the football field and house 400 beds. In addition, the facility will also feature amenities such as a cafeteria capable of seating 575 people, a large community room, and offices for the residential

and health services. The total budget for the design and construction of the project is \$60 million.

The goal of this study is focused on exploring the impacts of an alternative floor system design on the total project duration and cost. It will also include a thorough analysis and evaluation of the construction management services, in which the planned schedule and costs will be compared with the actual construction schedule and costs.

The current structural design of the facility is comprised of a steel frame, with cast-in-place concrete slabs for the first floor, and pre-cast concrete slabs for floors two through six. This study will propose an alternative floor system design, in which the pre-cast slabs on floors two through six will be entirely replaced with cast-in-place slabs. Although pre-cast slabs could potentially speed up the construction process, they are slightly more costly due to the fact that they have to be transported to the site, and the installation process requires the use of cranes. Also, any deviations in measurement from the design specifications are relatively harder to fix due to the fact that the slabs have already been cast (Consigli, 2013). By changing them to cast-in-place, the study will examine the effects on the cost and schedule, and determine which method will be more beneficial for the project. The alternative design will first be visualized through a 3D model which will be created in *Autodesk Revit*. Next, the model will be imported into *Autodesk Robot* where it will undergo structural analysis to ensure that the structure is sound. The impacts that this new design will have on the project will then be analyzed in terms of cost and time by preparing a cost estimate and a schedule of activities, using *Primavera*. This schedule and cost data will then be incorporated with the 3D structural model using *Autodesk Navisworks* to create a 5D Building Information Model (BIM). The BIM will serve as a complete

visual overview of the project and aid in better understanding the alternative design, including its time and cost implications on the project.

The study will also consist of an analysis of the overall project management for the actual construction of the project, which will entail evaluations of the relationships between different parties involved in the project, cost and schedule, safety practices, and the use of lean construction. A visual comparison of the baseline cost and schedule to the as-built cost and schedule will be presented in the form of a 5D BIM model. The 5D model will be created through the integration of the *Primavera* schedules with the *Revit* model in *Navisworks*. Lastly, the study will involve a geotechnical review section which will provide a description of the existing site work and layout.

2.0 Background

This chapter discusses the planning and need of a new dormitory on the campus of Worcester State. The section will start with an overview of the project as well as some information about Worcester State. Construction project management practices such as cost, schedule, and Building Information Modeling (BIM) will be overviewed for the construction of Sheehan Hall. Structural and Geotechnical analyses will also be discussed.

2.1 Worcester State's Plan

More students are attending colleges now more than ever. From 2000 to 2010 there has been an increase in enrollment in degree-granting institutions by 37%. Worcester State University (WSU) has been planning on adding more on-campus housing for their students to address this increase in students and students who live on campus. Sheehan Hall, the new dormitory on the campus will help this cause for the college. In the Worcester State University Master Plan from 2007 it was estimated that 700 new beds would be needed by 2014 (Sieniewicz, C. K, 2007). Sheehan Hall helps the university meet the needs of a growing student population. The college has many commuter students and the addition of this residence hall will help the process to have more students that stay and live on campus. Worcester State's President Maloney stated that "When Sheehan Hall is completed in August 2014, two out of every five of our students will be housed here on campus-and we know that residential students will both add vitality to our campus community and positively affect our retention and completion rates" (Reis, J, 2012). In the "Phase 3: beyond the framework horizon" section of the Campus Framework Plan it was described that a new residence hall would be implemented on the hillside of the sports field and six years later that plan was put into place. With this new residence hall the opportunity presented itself to enhance the "main street" of the campus (Sieniewicz, C. K, 2007).

The college campus lacks a clean pedestrian path or circulation pattern but this new building will add to the circulation pattern. The reason that the college wants a more prominent pedestrian path is to try to connect all campus buildings in one path, and this hall will fit into that path.

Figure 1 displays where the new residence hall will be located on the campus.

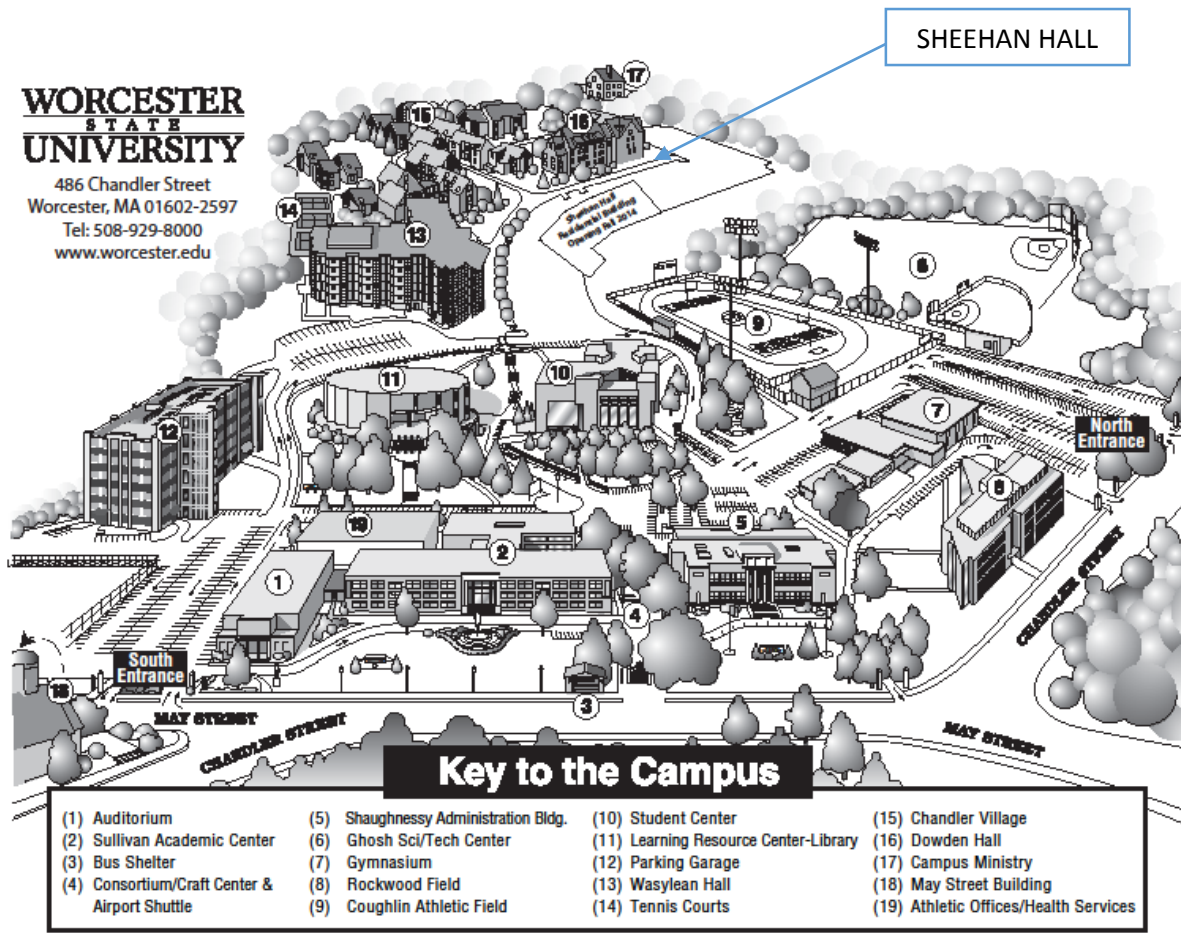


Figure 1: Campus Map of Worcester State University

2.2 Sheehan Hall

Worcester State University's new residence hall construction officially began in spring 2013, and has an expected completion date slated for fall 2014. The new facility is designed to accommodate 400 students and also includes the following features such as a large community

room, a dining hall with two-story windows capable of seating 575 students, faculty and staff, as well as additional outdoor seating overlooking the John F. Coughlin Field. This new residence hall will add approximately 10 percent to the University's on-campus housing capacity. Sheehan Hall will be named after Lt. Col. James F. Sheehan USMC (ret.) who graduated from the college in 1955. Over the years Lt. Sheehan has provided \$3.6 million in support for the college. Lt. Sheehan's support has gone towards scholarships, academic excellence and international study support. Massachusetts Higher Education Commissioner Dr. Richard M. Freeland stated that the support from Sheehan and the naming of the building was "truly a magnificent achievement for Worcester State and ... as a testament to his loyalty and gratitude towards the college" (Herrin, C). Sheehan Hall will now become the fourth residential complex among those currently part of campus such as Wasylean and Dowden Halls, and the Chandler Village. Positioned on the hillside above the Coughlin Athletic Field, the new residential facility will serve as a clear anchor to the residential area of the campus, offering a panoramic view of the university grounds as well as creating a pedestrian core that integrates all residential life on campus.

Sheehan Hall received an allocation of a budget of \$60 million for design and construction, the bulk of which is financed through the Massachusetts State College Building Authority (MSCBA). The MSCBA is responsible for the financing, designing, constructing and also the management of all revenue-funded projects including housing, dining, athletics, parking and other student recreational facilities with the goal to support the academic mission of the nine Massachusetts state universities. The Authority receives no appropriation from the Commonwealth. All revenues to support facility design, construction and operation are derived from the rents and fees paid by students for the use of these facilities and services (MSCBA, 2013).

2.3 Construction Project Management (CPM) Overview

Construction project management is the planning and execution of a project. There are many different components that are critical to completing the project on time and within budget. The CPM overview section explains the main components of the CPM methods that were used for this project. This section includes the contract type that was used for this project, the organization breakdown structure of the people and companies that are working on this project, the CPM practices that were used for cost estimating and scheduling, how Building Information Modeling (BIM) is used in project management and the concept of Lean Construction and how it was used in this project.

2.3.1 Organizational Breakdown Structure (OBS)

Construction Management at Risk is the contract type for this project. For this contract type the Owner chooses an architectural firm along with an engineering firm to design the project for the owner. Firms that offer construction management practices then bid on the project and owner then chooses the best contractor to complete the project based on variables such as bid cost, projected schedule, contractor qualifications and familiarity with the contractor. The work is being done for Worcester State University, which is a state school so the owner is the Massachusetts State College Building Authority (MSCBA, 2013). The MSCBA finances, helps design and oversees the residence halls and student activity facilities on the nine State University campuses in Massachusetts (MSCBA, 2013). The Authority uses all revenues to support facility design, construction, and operation are derived from the rents and fees paid by students for the use of these facilities and services (MSCBA, 2013). The MSCBA chose Goody Clancy and Associates from Boston, MA as the architectural firm for this project. The general contractor that was chosen for this project is Consigli Construction Co. based out of Milford, MA. Consigli is a Construction Manager and General Contractor that also has offices in Williamstown, MA,

Portland, ME and Hartford, CT and Boston, MA as well as having affiliates in NY. Once Consigli was awarded the project they began hiring the subcontractors for the job. There are also many engineering design consultants hired by the MSCBA who are involved with many different trades on the project. A list of all of these consultants can be seen in Appendix C. Figure 2 displays the organization breakdown for this project.

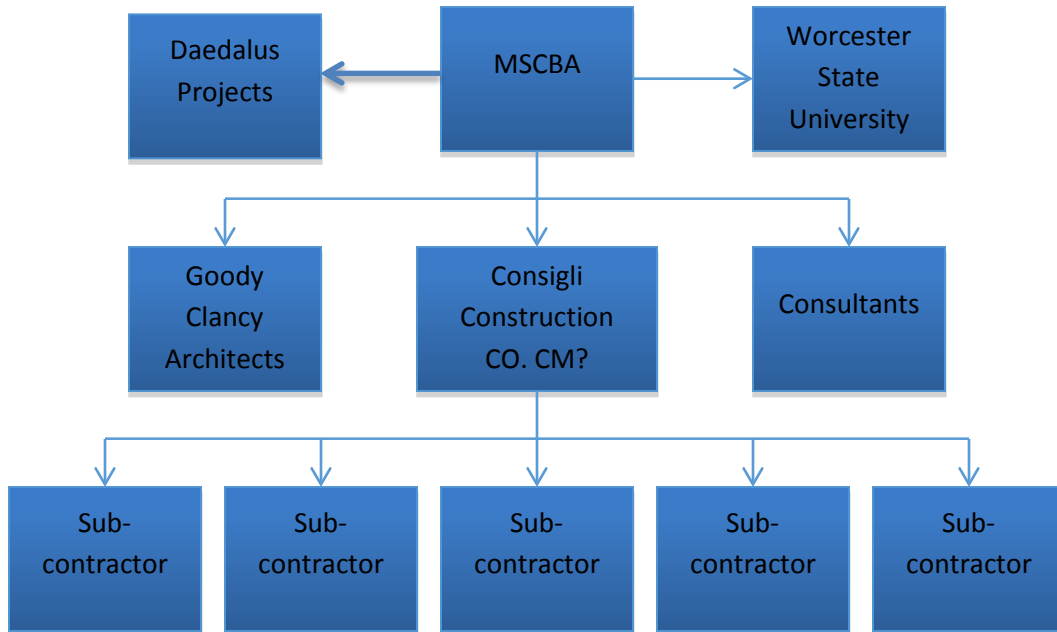


Figure 2: Organizational Breakdown Structure of Sheehan Hall

2.3.2 CPM Contract

The contract type for this project is Construction Management (CM) at risk with a Guaranteed Maximum Price (GMP). Construction management at risk is the financial agreement between the CM and the owner with the contract total being the GMP. The GMP is the summation of the cost for general conditions construction, the CM's fee, the CM's contingency, the subcontractor's work and an estimate for any work not yet approved. The general conditions construction includes the cost for any site work that has to be done to the site before construction. The CM's fee is the cost that the owner pays the CM for the construction project

management services they are providing. The main difference between a GMP and a Lump Sum contract type (Lump Sum is the other typical contract type) is the CM's contingency. The contingency is a portion of money in the contract that is used for unforeseen changes that occur to the project due to lack of scope, incomplete drawings or specifications or to cover unforeseen costs to a project. If a change has to be made to the project that is not specified through the scope of work then money from the contingency can be used for this change and it will not change the overall cost of the project. It is called a Guaranteed Max Price because of the contingency aspect so the max price does not change. However the GMP can be subjected to change if the owner or the Architect/Engineer makes a change to the scope of work. The CM is at risk in this contract because after the money from the contingency is used the CM has to pay for unexpected costs that come up on a project, other than owner approved scope changes. One of the main reasons that a GMP contract was chosen for this project is because the MSCBA likes using this contract type because they receive the remaining amount of contingency back once the project is done (Consigli, 2013). The initial GMP bid for this project was \$50,262,375 (Consigli, 2013). This cost to complete bid will change through the project based on changes and unforeseen expenditures.

2.3.3 Scheduling

Scheduling is one of the most important tasks involved in construction project management. A carefully planned and well-defined schedule, endorsed by all parties involved, is a necessary component of any project in order to ensure that the project gets completed within the specified time and budget. Construction projects involve a myriad of activities that need to be completed by many different subcontractors and professional teams in order to properly finish the project. A well-coordinated schedule not only helps in identifying all the activities in the project as well as the sequence in which the activities are to be performed, but it is also necessary

for determining the critical activities of the project, determining the overall project duration, and the order and timing in which each subcontractor is expected to complete the tasks. A schedule can also be used to gauge the progress of the entire project by comparing the activities planned on the schedule with the activities that have been completed. If an activity falls behind schedule and could potentially delay the completion of the project, it is the job of the project management team to manipulate the schedule and reallocate resources in order to finish on time. In the case of Sheehan Hall, finishing on time is essential because WSU needs to have the building ready for move-in by fall 2014.

The Sheehan Hall project was started in November of 2012 and is expected for completion in July of 2014, with a total project duration of 20 months (Consigli). The project is on a fast-track schedule, meaning that the design and construction phases are overlapped in order to compress the total duration of the project. For example, the construction can begin as soon as the structural design is complete, while the rest of the details and designs can be finalized as the project moves along. This enables the project management to significantly expedite the construction process since they don't have to wait for the all designs to be established to commence construction, but it demands greater coordination and communication between the designers and the project management team.

In any schedule, it is important to identify the critical activities whose completion is absolutely necessary in order for the project to move along. The Critical Path Method (CPM) is commonly used in construction schedules to identify the tasks that are critical to the project, and based off these tasks, the total project duration. In the CPM, all activities that have a total float of zero are considered critical while activities whose total floats are greater than zero are considered non-critical. The path with the longest total duration along these critical activities is known as the

critical path and the duration of the critical path determines the duration of the entire project. Total float is the leeway between the earliest date at which an activity can start and the latest date it can start without resulting in a delay for the entire project (Halpin & Senior, 2011). Therefore, delaying a critical activity (zero total float) will result in the total duration being extended as well. On the other hand, non-critical activities (total float greater than zero) can be delayed by up to a number of days equal to its total float without impacting the total duration of the project. The CPM is a very useful tool for the project management team in planning and controlling a project from start to finish: critical activities indicate which tasks require immediate attention and resources, and shortening the duration of the critical path can shorten the total duration of the entire project.

The larger the project, the greater the number of activities involved in the schedule of the project. Large construction projects involve tens of thousands of individual activities and scheduling all these activities can be very complex and time-consuming. For this reason, various computer software exist that make scheduling a project fast, simple, and manageable. Programs such as *Primavera Project Manager* are very capable of organizing and performing calculations on many information, and can handle various tasks, from planning and generating a simple timeline for all the activities of a project, to evaluating entire projects and portfolios (Primavera Works, 2013). *Primavera* is widely used by many construction and contracting firms to create schedules for projects because the program is also capable of tracking many important aspects of a project such as costs, duration of individual activities, and the relationship between activities. It can even be used to manage risks, keep track of all the contracts, documents, and change orders pertaining to the project, and monitor Requests for Information? (RFIs) and unresolved issues (Oracle, 2013). A part of the *Primavera* baseline schedule developed by Consigli for the Sheehan

Hall project can be seen in **Figure 2**, with the list of activities on the left and a bar chart showing the relationships between the activities and the schedule on the right (please refer to **Appendix E** for a complete display of the baseline schedule).

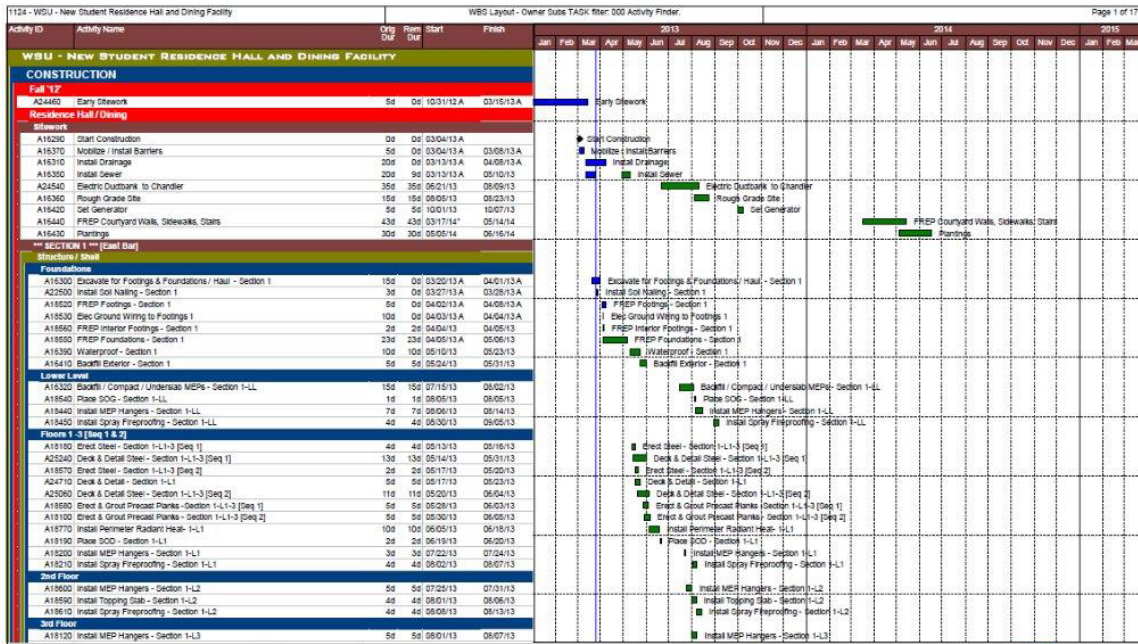


Figure 3: Consigli's Baseline Schedule

2.3.4 Building Information Modeling in Project Management

Building Information Modeling (BIM) is an emerging tool in the construction industry that is being adopted by an increasing number of construction firms. BIM enables firms to virtually construct a structure or facility before the actual construction occurs, thus minimizing the chances for error and clashes that would likely occur during construction (Consigli, 2013). BIM is mainly based on a 3D model, to which large amounts of information and other models can be added as desired. The BIM of a construction project usually incorporates into a single model all the information from different aspects of the project such as the architectural details, the structural design, the HVAC and MEP designs, as well as geotechnical information. Different

parts of this complete model can then be exported into special application software, such as *Autodesk Robot*, to be analyzed. This means that the relationships between the different aspects can be determined and any potential clashes identified and addressed, thus enabling the project management team to reduce the duration and cost of the project. Consigli's main uses of BIM in the Sheehan Hall project are to ensure proper sequencing of the steel and concrete, and to identify potential problems before they can actually occur on-site (Consigli, 2013).

In addition to being capable of providing a complete 3D model of a facility, BIM can also incorporate other information such as the schedule of the project and the costs associated with the construction of the building into the same model. A BIM model with incorporated cost and schedule data is known as a 5D model. BIM models are great tools for project management because they enable the project management team to simulate the actual construction process and prepare cost estimates along different project phases (Autodesk, 2013).

BIM is a great way of communicating various aspects and objectives of a project with everyone involved, from the owner to the field workers, because it provides a visual model with integrated time and cost data. The complexity of these models enables information from all the different trades of the project to be stored in a single file, from which data can be pulled as necessary and each individual component of the project can be analyzed. BIM has dramatically enhanced the construction industry with its versatility, and is becoming increasingly popular.

This study will incorporate the use of BIM for two purposes; to compare the baseline schedule for the actual construction of the structure to the as-built schedule, and to help with the visualization of the alternative floor system design and its impacts on the schedule and cost. Thus, the 3D model, which will be drawn in *Autodesk Revit*, will be focused on and limited to

the structural design only. This 3D model will then be imported into *Autodesk Navisworks*, where it will be incorporated with the baseline and as-built *Primavera* schedules that contain cost information to create a visual 5D comparison. The structural model will then be updated in *Revit* with the proposed alternative design, and analyzed for structural soundness in *Autodesk Robot*. The alternative model will then be integrated with an updated schedule in *Navisworks* to provide a visual representation of the effects of the alternative design on the project.

2.3.5 Lean Construction

Lean Construction is an increasingly popular method of managing construction that is being employed by many construction firms nowadays (Consigli, 2013). In lean construction, a production management-based approach is used to help streamline the process of designing and building new facilities, in order to minimize the waste of materials, time and effort, and maximize value (Lean Construction Institute, 2013). Lean construction is especially useful for projects that are complex, uncertain and quick because the techniques used in lean construction call for enhanced collaboration among the different parties involved, reduced waste and redundancy, and improved efficiency and project outcome (Turner Construction, 2013).

Consigli also decided to adopt lean construction practices in the WSU New Dormitory and Cafeteria project in order to make the construction process more efficient and to tighten up the schedule (Consigli, 2013). In order to implement lean construction practices in a project, each work area is sub-divided into smaller sections, in which a single trade focuses on the work they need to complete before the next trade takes over the section. This method of dividing up the work areas into sections and having trades work in these smaller sections over a certain period of time creates a production-line type of effect and increases efficiency, as opposed to giving the work area to just a single trade at a time. This is true due to the fact that each trade is

under the responsibility of completing their work properly and on time so that the next trade can move in and begin their work as scheduled. The added benefit of having multiple trades working simultaneously on different sections of a work area is that there is increased communication among the trades.

The practice of lean construction can also be applied to equipment and resources in order to ensure a better flow of work among the trades and to reduce costs; this is achieved through careful scheduling and allocation of the equipment and resources among the various trades involved in the project. It allows the project management to reduce the planning, coordination, and clutter that would otherwise be involved with moving the equipment frequently from place to place on site among different trades. There are many benefits to incorporating the principles of lean construction in a project. Lean construction achieves better efficiency in the use of materials, time, and effort by streamlining the traditional construction process and making it more like an assembly-line of a manufacturing plant.

2.4 Structural Components Overview

This project as it pertains to the structural components of WSU's new dormitory building, Sheehan Hall, is based on proposing an alternative design for the current concrete floor system. With our alternative design, we will be looking to impact the overall construction period as well as the building's total cost of construction. To achieve this, we will be changing the current floor system's concrete method from a girder-slab system utilizing Hollow Core Precast Planks with Dissymmetric open-web steel beams to a cast-in-place concrete slab on metal deck system. In our new system the primary elements will be the addition of a composite-acting slab on deck as well as the use of floor joists for additional support on the girders. The goal for this

MQP is to compare both methods (precast planks versus composite slab) and determine the advantages and limitations for each system in terms of the project's constructability and cost.

2.4.1 Precast Concrete vs. Cast in Place

Precast Concrete is a type of construction material that is typically used for both architectural and structural applications on a variety of buildings (PCI, 2013). This material is commonly used as the primary structural system for many high rise or multi story buildings because of its ability to transfer roof, floor, and lateral loads while also reducing the overall weight of the entire system (PCI, 2013). The use of Precasted Hollow Core Planks allows for designers to integrate both the architectural and structural systems while reducing the total amount of materials, detailing, costs and also construction complexity (PCI, 2013). Precast is also valued for its high versatility, because it can serve many needs for the structure of a building and most importantly, in terms of its growing popularity, Precast is more than just a very good building material because it can take almost any form and shape. Other beneficial traits for precast concrete is that there are different types of precast materials such as prestressed concrete- which is a type of structural member that is known for its exceptional load-carrying capacity. Due to having such high load-carrying capacity, this typically results in the use of smaller sections, longer spans, or even both when compared to other structural systems (ACP Co., 2013).

What makes this building material so advantageous to use during construction is its ability to be transported to a construction site where it can then be lifted and set into place all in the same day. During the production of precast concrete, the controlled environment it is mixed in is typically referred to as a precast plant. At this plant, the production process is done on ground level, which has been proven to help with production safety (ACP Co., 2013). Also this provides a greater ability to control the quality of materials being added to the mixture while also affecting the workmanship in a precast plant versus being on a congested construction site (ACP

Co., 2013). After the mixture has been poured and shaped, it begins the curing process where it is closely monitored to reduce the possibility of deformities from being created within the structure that would typically be caused by unnecessary exposure to inclement weather or other disturbances found on any construction site.

This type of concrete is widely being used for construction projects today because it offers numerous positive advantages during construction scheduling and also requires less coordinating by CPM's during its installation process. This can be seen in the construction of WSU's new residence hall as the use of precast planks contributed to an accelerated and simplified installation process for all six stories. While spectating the installation of the first level, the use of these hollow-cored planks allowed for them to just lift the material to its desired location and set them in place on the D-beams all in the same day. An important installation technique that was used in this project was the way in which each of the floors were turned into a composite system. To establish composite action between the planks and D-beams a process called grouting was used. Grouting ("Grout" also known as super-strength concrete) is the process of filling the hollow cores with this high-strength concrete, and it was done by passing the grout through the open web of the D-beams and into the cores. As it cures, this will essentially connect the two materials together making it possible for the floor system to successfully transfer loads throughout each of the precast planks, to their supporting steel members, down through the system's columns and into the buildings foundation and soil. This Grouting technique uses similar steps to the use of ordinary site-casted concrete but for this project specifically, the system's design using open-web D-beams and hollow core planks allowed the grouting process to be done quickly and efficiently; alternatively with the use of site-

casted concrete, additional time is needed before pouring to lay down steel decking where as in this project the amount of CIP was very limited due to very strict schedule deadlines.

Lastly, from more of a financial standpoint, the prep work needed for the use of precast concrete members is very small and consists of the following: the excavation (if needed and is typically done for foundations and footings) of soil for pre determinedly sized members to be placed in, and the use of a boom lift or crane to lift the members off the delivery truck and lowered into place, like what was seen for WSU's Sheehan Hall and their use of prefabricated HC Planks. Precast concrete can be used to expedite a significant portion of the construction process and listed below is a summarization of all the main points previously mentioned in this section (ACP Co., 2013):

- Precast concrete structures are made easily available by a variety of precast suppliers.
- Can be manufactured to accommodate almost ever construction project need.
- Because of the controlled environment it is made in, inclement weather is not a factor in the planning process, which will help to avoid any unnecessary delays due to unworkable conditions.

Cast-in-Place concrete is brought onsite in its un-hardened liquid state where upon arrival it is poured into site-specific forms (typically "molds") and cured on site. This material is typically manufactured in a factory or batching plant (according to a set recipe), and is then delivered to a site by a truck mounted in-transit mixers. The result from a precise batch provides the ability to create special concrete mixtures and with the convenience of making other alterations to the mix and implemented on a construction site.

Cast-in-place (also known as ready mix concrete) is the material of choice for slab-on-ground and foundations as well as on steel or metal decking because of the material's long-term durability as well as its structural support.

CIP concrete can serve many needs for a variety of different types of buildings, some of the common many applications of CIP consist of beams, columns, floors, walls and roofs.

Additionally, widely used building material has been shown to have environmental attributes during construction and have also been known for being present during the structure's life span.

These environmental benefits during construction are as follows:

- There is very little wasting of material due to the specific state that the material is in during construction applications, it can really only be used and placed on an as-needed basis. This material can't be left around on-site as it will begin to harden unless continuously stirred or mixed.
- Additionally, this material is very easily recycled and used for the creation of other structures like jersey barriers or retaining wall blocks (Mineral Industry, 2011).

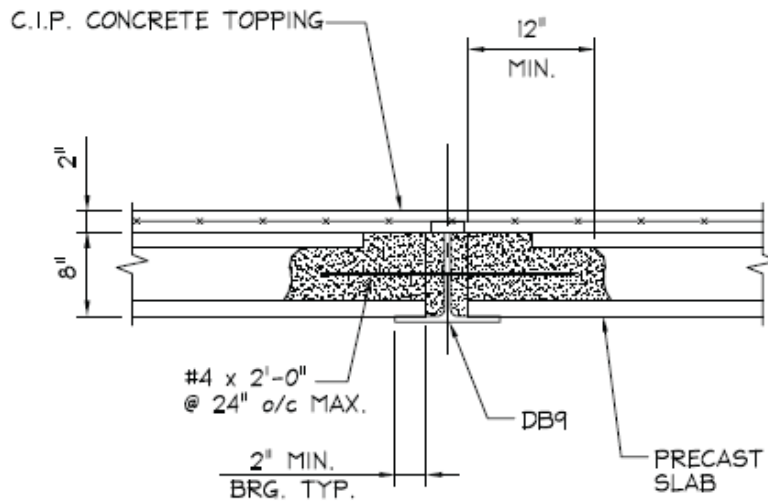
Some projects actually prefer the use of cast-in-place over the use of precast members because of the precision of the mixture and also due to its reduced work site confusion. The use of a predetermined concrete mixture (typically associated with concrete suppliers) helps to reduce any inconsistencies as well as the flexibility of both the supply chain and the actual concrete components. Ready mix concrete (Cast-In-Place) is known for its customizability in the type of concrete product being produced for commercial as well as private purposes. Also, ready mix concrete companies typically offer different variations of concrete according to the user's mix design or industrial standard. Each of the variations of RMC can be manufactured to meet

the demand specified for each new delivery or project. Some disadvantages from using RMC are (Mineral Industry):

- The materials are batched at a central plant where the concrete is mixed before being shipped to the site. This poses a critical time period beginning from when supply truck leaves the plant and ending once the supply truck reaches its destination. This critical time period becomes increasingly difficult to manage over longer distances. This is the reason for supply trucks to be built not only to ensure a quick and safe delivery but also to prevent the concrete from losing its ideal pouring state through means of installing a continuously rotating holding tank.
- The travel route taken by the supplier, as high levels of road traffic can become an issue for not only the supplier but can also add delays to construction where deadlines are not met due to late arrivals. Additionally Site access for supply trucks is an unavoidable issue for construction projects, Amongst being a contributing factor in a projects site development plan, access roads must be provided and able to support workers, emergency vehicles as well as large and heavy supply trucks; However this not usually an issue and can be avoided by utilizing what's called a "mini-mix company"- a company that deals with using smaller 4m³ capacity mixers that have the ability to reach more-restricted construction sites.

Cast-In-Place Slab on Steel Deck versus Precast (HC Planks) Girder-Slab Floor Systems

A Precast Girder-Slab floor systems consists of interior girders (also known as an open-web-dissymmetric beam or D-beam) and prestressed hollow-core slabs that are connected using cementitious grout. The use of a Girder-Slab system allows for the concrete slabs, being supported by the steel frame, to resist all gravity and lateral loads. Once the hollow core slabs are place on the D-beams, the process of creating composite action is done by grouting through the web openings and into the hollow slab cores and is completed once the grout has been cured properly. Similar to the floor system chosen for WSU's new Sheehan Hall, a Girder-Slab system is typically used for mid to high-rise residential structures such as hotels, apartments and condominiums. There are two basic D-beam girder sections available for use with an 8" thick precast slab (generally spanning as long as 28 ft.) and they are a DB-8 and DB-9. The DB-8 provides an 8" thick slab assembly, while the DB-9 is designed to be installed with a 2" concrete topping layer resulting in a 10" total slab thickness. A Girder-Slab system is constructed in accordance with the "Underwriters Laboratories Inc. Floor-Ceiling Design K912". The reason why this system is so highly valued is because it has been shown to greatly improve a projects construction operations as well as a project's ability to stay on schedule and meet critical deadlines. An example of the Floor System used for WSU's new dormitory building can see below in **Figure 4**.



**TYPICAL SECTION: REINFORCED CORE
WITH 2" CONCRETE TOPPING**

(DETAILS S4, S5, S6, S7, S8, S9, S10 & S14 ARE SIMILAR FOR DB-9)

GROUT TO ATTAIN
4000 PSI PRIOR
TO TOPPING

Figure 4: Typical Girder-Slab Section Detail- Reinforced Core with 2-inch Concrete Topping

The use of a pre-topped system allows for faster construction at a slightly more reduced cost than with field-topped systems (Cudney, 1998). However, field topped systems offer less floor vibration, positive drainage (easier to achieve), and also a lower maintenance cost for joint sealants.

A cast-in-place, post-tensioned concrete system is typically constructed by pouring concrete into temporary forms (typically either plywood or steel) that are made on site. This system utilizes a one-way, post-tensioned slab that is supported by long spanned, post-tensioned beams (Cudney, 1998). These beams are typically located at the column line and are about 14 to 18 inches wide by about 32 to 36 inches deep (Cudney, 1998). The advantages and disadvantages

for the use of each type of system are listed below in table-3. When properly designed, detailed, constructed and maintained, the durability of the CIP, post-tensioned and precast systems are very similar. Both systems include elements such as expansion joints, joint sealants, and exposed painted metal connections as well as railings that will require preventative maintenance, and even reparations; however, because of the increased number of sealant joints, the precast system would require more maintenance than would a CIP system. Both structural concrete systems are cost effective and durable, but the decision on which structural system to select comes down to the following points (Cudney, 1998):

- The Owner's preference
- Requirements of the structural component's-lateral load system, foundation, flexibility of the framing, ramping, expansion joints, site dimensions, etc.
- Maintenance considerations
- Aesthetics, facade treatment
- Openness, visibility and lighting
- Economics, including first cost and life cycle maintenance costs.
- Construction schedule
- Ability to utilize local labor
- Availability of competitive contractors

Precast

Cast-In-Place

Advantages	Disadvantages	Advantages	Disadvantages
<ul style="list-style-type: none"> - Slightly shorter on-site construction period. - Potential for a slightly lower initial cost, especially for the pre-topped system (if standard sizes and repetition of structural and architectural components are used). - Long-span construction with typical column spacing of 30 ft. - More adaptable to winter construction. - Potentially better concrete quality control in plant conditions. 	<ul style="list-style-type: none"> - Higher Maintenance costs; precast systems require a caulk/sealant joint between double tees to prevent water leakage. These joints (typically located at 10' to 12' o.c.) create a greater potential for leakage and are usually replaced every 8 to 10 years. - Perceived lower headroom, less desirable distribution of lighting and signage visibility due to the depth and spacing of the double tee stem. - Wind and seismic loads are resisted by shear walls or shear frames. The shear walls/frames are typically located on the exterior faces (affecting architectural appearance), or at the interior (reducing visibility and openness). - Drainage profiles for non-ramping floors are typically flatter than those found in a CIP structure, due to limits on the amount of warping of the precast without excessive cracking of the tee flanges. - Longer shop drawing review and fabrication schedule. - Many cities don't have local precast concrete subcontractors. 	<ul style="list-style-type: none"> - Monolithic construction, resulting in fewer joints. -Easier to achieve positive drainage. - Post-tensioning compressive force reduces cracking in slabs. - Flexible framing layout to fit the site with typical column spacing of about 20 to 24ft. -Wind and Seismic lateral loads are resisted by frame action and distributed into the foundations through the columns, eliminating the need for shear walls. -The perception of higher ceiling and more openness. -Better lighting distribution and visibility of signage due to fewer beam soffit members. -Lower maintenance cost. - The construction is typically performed by local subcontractors using other local laborers and material suppliers. 	<ul style="list-style-type: none"> - Slightly longer on-site construction period. - Less adaptable to winter construction in northern regions. - Construction quality is typically more difficult to achieve.

Table 1: Advantages/Disadvantages for Precast or Cast-In-Place Concrete Floor Systems

Cost advantages

Among the many differences found in each type of concrete construction (production and distribution methods for example) the most important difference is the cost of the material. For many contractors and project managers there is a big difference between Price and Cost. Price only happens to be one element of cost; it is the initial and the easier of the two to understand along with being the most visible. Focusing on price is not a preferred strategy in any business,

especially when it comes to a material's quality, and the reliability of manufactured goods.

Instead, the prime focus should be set on the "Total Cost of Ownership". TCO is equal to the sum of the four cost components: quality, service, delivery, and price (NPCA, 2010).

In terms of cost elements, a clear advantage of using precast concrete over cast-in-place (CIP) is the speed of its delivery and also its ease of installation, or service (NPCA, 2010). These collectively contribute to a lower TCO. Precast concrete, especially when produced in controlled plants, boasts the additional benefit of higher quality. The controlled batch proportions placed under uniform conditions consistently creates a better product than can be cast in place (NPCA, 2010).

On any construction site, scheduling is an important but unpredictable and expensive risk. Nature stacks the cost odds against CIP concrete because it is much easier to order precast concrete structures (assembled ahead of time) and have them delivered and installed the same day than it is to have to excavate, form, pour, and strip, the CIP concrete which is then followed by having to cure it, damp proof and backfill each structure. Depending on the type of project and the different constraints present, research shows that on average "the use of precast concrete structures over cast-in place structures can save roughly 5-6 days in construction scheduling" (NPCA, 2010). CIP requires three separate days to pour the base, walls and top of each structure; additionally, curing and stripping adds one day to the CIP process, totaling seven working days of open-hole time. The TCO of precast is a fixed cost; however the TCO of CIP just begins at an initial cost of the product itself (does not include its delivery and installation costs, etc.) which makes the choice of using precast actually cheaper even though its fixed cost can be higher than CIP's initial cost. It is this concept of TCO that our group plans to implement in our alternative floor system design of Sheehan Hall.

3.0 Methodology

The methodology section presents the proposed activities to be performed in this MQP and describes how the activities will be executed. This section will discuss how the project management of the Sheehan Hall Project will be evaluated with regards to criteria such as relationships between different parties involved, schedule, cost, and safety, and how an alternative floor system design will be proposed and analyzed in terms of structural soundness and impacts on the cost and schedule. It will also include a description of how a BIM model will be developed and utilized to aid with the visualization of the alternative design and its effects, as well as how a geotechnical analysis of the site will be conducted. The execution of some of the activities mentioned in this section will require the use of software such as *Primavera*, *Autodesk Revit*, *Autodesk Robot*, and *Autodesk Navisworks*. For a breakdown of how and when the group will be performing the above-mentioned activities, please refer to **Appendix C**.

3.1 Project Management Evaluation

- Evaluate the relationship between Owner and GC
 - Attend Owner Meetings
 - Observe Communication and Relationship
- Evaluate the relationship between Architect and GC
 - Record RFI response time
 - Attend Owner Meetings
 - Observe Communication and Relationship
- Evaluate Schedule
 - Analyze and compare actual schedule to baseline schedule
 - Analyze problems in scheduling and their impact on the schedule

- Examine the changes made and determine their impact on the schedule
- Analyze the impacts of using lean construction and a fast track schedule
- Evaluate Cost
 - Analyze the current cost of the project
 - Compare the planned costs of construction to the actual costs of construction
- Analyzing Safety
 - Record and track the safety scores received by this site

3.2 Structural Analysis

- Propose alternative design
 - Identify and analyze critical bays
 - Change precast floor system to cast-in place floor system
 - Implement changes in Revit model
- Analyze the alternative design for structural soundness
 - Perform hand calculations
 - Perform structural analysis in Robot
- Analyze the impacts on the schedule of using precast concrete versus cast in place concrete
 - Analyze the amount of time required for each method
 - Determine and compare how the critical path is affected
 - Analyze the scheduling effects on equipment and labor
- Compare cost of existing vs. alternative design
 - Calculate the cost of alternative design using

- Calculate the cost of labor for alternative design
- Calculate the cost of materials for alternative design
- Calculate the cost of equipment needed for installation of alternative design
- Compare the total cost of current design vs. total cost of alternative design

3.3 BIM

- Analyze the use of BIM and its impact on the entire project
- Create a 5D model of the structural components in *Autodesk Navisworks* by integrating a 3D *Revit* structure model with a *Primavera* time and cost schedule
- Provide a visual comparison of the planned and actual schedules using animation in *Navisworks*
- Perform structural analysis on the alternative model using *Autodesk Robot* as the primary software
- Provide a visual representation of the impacts of the alternative design on the cost and schedule of the project

3.4 Geotechnical Review

- Provide a description of the existing site-work, layout, and conditions