LDA – 1605

Academic Library Design

A Case Study of the George C. Gordon Library

A Major Qualifying Project Report submitted to the faculty of Worcester Polytechnic Institute in partial fulfillment of the requirements for the Degree of Bachelor of Science in Civil Engineering

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March 4th 2016

WPI

Abstract

This project created a modern academic library design that accommodates a variety of learning styles and balances social and communal spaces. The Gordon Library at Worcester Polytechnic Institute was benchmarked and structural alternatives were developed using reinforced concrete and structural steel. The recommendation is to use reinforced concrete in combination with an architectural layout that provides a comfortable environment to facilitate learning through the use of a modern facade, an atrium, natural lighting, mixed-use and open space.

Authorship

Rania Attala contributed to the design of the foundations, structural steel, reinforced concrete, and cost estimating calculations.

Johnpatrick Connors was primarily responsible for structural steel and reinforced concrete design, cost estimating, formatting, and contributed to all sections of the report.

Acknowledgements

This project began in August 2015 with the help of several individuals to whom we express our sincerest gratitude. First, we feel so fortunate to have been mentored by Professor Albano. He has been remarkably helpful in guiding our research, critiquing our calculations, and providing feedback throughout the entire project. Professor Albano pushed us to use critical thinking to explore and create the most interesting ideas. He also infused a spirit of constant questioning and passion for excellence into our work ethic. For that we are forever thankful.

We would also like to thank Yvette Rutledge and Nick Palumbo from the WPI Facilities Department for providing us with plans of the existing Gordon Library. Finally, we would like to extend a special thanks to Michael Kemezis from the WPI Archives and Special Collections Department. Michael located and retrieved crucial documents in the university archives that helped shape our understanding of the existing library.

Capstone Design Statement

The Accreditation Board for Engineering and Technology (ABET) requires undergraduate engineering education to culminate in a design project that fulfills a series of conditions outlined in its accreditation criteria. This Major Qualifying Project (MQP) team addressed those conditions by utilizing a wide breadth of knowledge gained throughout our undergraduate civil engineering career.

The design problem addressed in this project was to benchmark the Gordon Library at Worcester Polytechnic Institute (WPI) and identify alternative architectural layouts that are better suited to meet the needs of twenty-first century library users. Structural alternatives in reinforced concrete and structural steel were developed along with foundations to support the alternative layouts. Cost estimates were also prepared to compare the alternatives. As documented in this report, engineering standards were used to solve a complex problem involving economic, social, ethical, health and safety, constructability, and sustainability constraints.

Economic

The cost of the structural alternatives was one of the major factors that impacted the final recommendations. *RS Means Square Foot Costs* was utilized to perform cost analyses of the structural steel and reinforced concrete alternatives, including allowances for non-structural components such as interiors, electrical, plumbing, and fire protection services.

Social

The layout of a library facility and the services it provides has a direct impact on the quality of education and academic culture that students experience. Learning outcomes are highly dependent on the ability of a space to provide a comfortable environment that facilitates knowledge sharing and creation. Consequently, the architectural layout was tailored to meet the needs and changing study habits of students. Spaces were designed to increase social interaction and collaboration and a variety of novel facilities including cafés, an art gallery, and a large group work room were incorporated into the design.

Ethical

The American Society of Civil Engineering (ASCE) code of ethics served as the guiding ethical principles used to execute this project. At the outset of this project, a non-disclosure and confidentiality agreement was made between the WPI Facilities Department and this project group. This agreement stipulated that the architectural drawings of the existing Gordon Library could not be published in the final report. In addition, as stated in Canon 1, the health and safety of occupants was held paramount throughout the design process.

Health & Safety

The structural design meets the minimum requirements of the *2010 AISC Specification, ACI 318-11, and 780 CMR*. In addition, the floor plan is designed in a way that facilitates efficient egress in the event of a fire event and allows people with disabilities to use the space in a safe and comfortable manner.

Constructability

Implementing solutions that enhance constructability was an important consideration. For example, a key component of the column grid design was reducing the number of columns used in the structure in order to create large open floor spaces. Repetition of standard sections, member sizes, orientations, and dimensions was encouraged throughout the project in order to promote an economy of scale and to control formwork costs for the reinforced concrete alternative.

Sustainability

The proposed architectural layout incorporates a number of sustainable features and offers unique spaces that ultimately give rise to a healthier, more flexible interior environment. The use of large, open structural bays provides building occupants freedom to repurpose the space as their needs continue to evolve. Another notable feature is the extensive use of daylighting throughout the building which improves productivity and contributes to a healthier interior environment.

Professional Licensure Statement

Civil engineers design, investigate, and rehabilitate structures that have a direct impact on the safety and well-being of the public. The public entrusts civil engineers to perform engineering services in an ethical and competent manner. In order to assure their competence, state and local governments require civil engineers who prepare and seal engineering plans and drawings to be professionally licensed.

Graduation from an accredited undergraduate institution is just the first step on the path to becoming a licensed professional engineer. Prior to graduation, or shortly thereafter, aspiring civil engineers must pass the Fundamentals of Engineering (FE) Exam. This exam is eight hours long and tests students on their competence with math, science, and civil engineering principles. A student who passes the FE exam is designated as an engineer in training (EIT) and is eligible to work as a civil engineer under the supervision of a licensed professional engineer (PE). Although requirements vary by state, EITs must typically gain a minimum of four years of work experience under a licensed professional engineer to apply for professional licensure. It should be noted that the work experience requirement can often be lessened by one year if students attain a graduate degree. Graduate degrees provide students with more technical knowledge and opportunities for professional advancement and are increasingly required for entry-level engineering positions.

After successfully completing the work experience requirement, engineers are eligible to take the Principles and Practice of Engineering (PE) Exam. This exam is also eight hours long and tests students on their breadth and depth of civil engineering knowledge. Individuals who pass this exam are eligible to apply for a professional engineering license in each state they practice. In order to maintain their license, professional engineers must fulfill continuing education requirements, which vary by state.

Professional engineering licensure is important for the civil engineering profession, civil engineers themselves, and for the public. Just like the requirements for becoming a licensed medical doctor or lawyer, licensure requirements establish civil engineering as a field composed of professionals with a high level of dedication and technical competence. As stewards of the built environment, civil engineers should take pride in knowing the quality of their work is taken for granted by the public. Furthermore, civil engineers who follow the path to attaining professional licensure will gain better technical skills, more self-confidence, more responsibility, and will move up the corporate ladder more rapidly. The safety and wellbeing of the public is also greatly enhanced by professional engineering licensure requirements. Civil engineers design dams, roads, bridges, buildings, water and wastewater treatment facilities. All of these infrastructure components are of vital importance to the functioning of modern society, and licensure requirements establish that infrastructure is designed to a high level of performance that ensures the safety and well-being of the public.

As a final note, engineers should balance their technical knowhow with an external awareness for the needs of society. Engaging in critical thinking and awareness of all facets of human society will enable civil engineers to have transformative impacts on the people they serve and will allow them to remain at the forefront of their craft.

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Chapter 1: Introduction

Academic libraries have traditionally formed an integral part of the intellectual and social fabric of universities. Yet, in recent years, their relevance has been put into question by those who view the library solely as a repository for information. The purpose of this project was to research how universities across the country are reshaping the academic library with new spaces and architectural features. Emerging trends in library design were studied and aspects of the physical form and architectural quality of library facilities that establish the library as a place for student-centered learning and balance library users' multiplicity of needs were highlighted.

A list of evaluation criteria was developed as a result of the research, and the Gordon Library at Worcester Polytechnic Institute (WPI) was benchmarked against these criteria. Results from the benchmarking activity revealed the functional limitations with the existing building. This activity also helped identify an alternative layout as well as structural and building envelope designs that may be better suited to meet the needs of twenty-first century students.

1.1 Problem Statement

As times change, libraries must adapt to host new types of media and activities necessary to meet the changing size, work habits, and needs of university communities. As such, the level of thought given to library layouts and their compatibility with structural systems should be commensurate with the importance of libraries, or they risk becoming obsolete. The Gordon Library was used as a case study for evaluating the performance of academic libraries constructed in an era separated from the present, not only by time, but by great advances in building and information technology. Following the research, it was found that the needs and work habits of the WPI community have changed significantly since the Gordon Library was constructed in 1967. These changes are significant enough to explore the use of alternative layouts, structural systems, and building envelopes that may be better suited to meet the current and future needs of the WPI community.

1.2 Scope of Work

In order to address the problem outlined above, the following five objectives were established:

1
- Research changing resource types, study habits, desired library services, and amenities.

2
- Benchmark the Gordon Library using criteria developed from Objective 1 and identify ways to reduce the demarcation between the interior and exterior environment, improve lighting, group study spaces, and aesthetics.

3
- Investigate new layouts and structural configurations in response to the research and benchmarking activity

4
- Develop the foundations, reinforced concrete, and structural steel alternatives by performing engineering calculations to specify the configuration, quantity, and material properties of the structural members that will support the proposed layout.

5
- Perform a cost analysis of the structural alternatives in order to perform a comparison between them.

1.3 Report Outline

The following chapters of this report provide background information relevant to understand and develop the salient features of the work as well as chapters covering architecture and layout design, structural steel design, reinforced concrete design, foundation design, and cost analysis. Finally, the report wraps up with a summary of the findings and recommendations.

Chapter 2: Background

In this chapter, a discussion of the background information necessary to understand the underlying historical, social, and technological concepts of the work is presented. In order to demonstrate why a redesign of the library may be appropriate, it is necessary to situate the reader in the era in which the present structure was designed. Such a process will reveal the social and technological conventions that informed the current structure's design. A discussion of emerging technologies and the changing role of the library will follow to demonstrate how a new design can better meet the needs of twenty-first century students. The chapter will conclude with sections that provide a base for developing the alternative designs.

2.1: The Gordon Library at Worcester Polytechnic Institute

WPI has a long history of growth and has enjoyed a distinctive record of achievement in the sciences and engineering. By 1963, a pivotal year in the university's history, enrollment had reached 1,142 undergraduates, an increase of 44 percent in the last seven years [Worcester Polytechnic Institute (1963), 1]. Meanwhile, the launch of Sputnik in 1957 and the intensification of the Cold War arms race created a significant impetus to improve science and engineering education across the United States.

As one of the premier technical universities on the East Coast, WPI was looking to further increase enrollment and continue to produce engineers of the highest caliber during this period. However, in order to produce a quality engineering curriculum at the graduate and undergraduate levels, WPI needed to provide students with access to science and technology information.

At the time, the university lacked a centralized library. A general library located in Boynton Hall contained a wide variety of volumes in literature, economics, history, and art [Coombs (N.D.), 2]. The remaining academic resources were dispersed across the university; each academic department had its own library.

With a desire to expand its collection of books, centralize its resources, provide students with a quiet study environment, and expand into emerging audio-visual and microfilm technologies, the university sought to construct a new library facility.

Constructing a new library was a bold endeavor and required significant capital investment. Fortunately, George C. Gordon, a distinguished alumnus who graduated in 1895, left a bequest of $5,000,000 to the university. [Worcester Polytechnic Institute (1967), 1]. This donation enabled WPI to commission the design and construction of a modern library facility with a capacity for 600 students and 200,000 volumes. The interior design included individual reading tables for concentration, group study rooms, smoking rooms, music rooms, and lounges on each floor. The library cost $2,053,133 [Worcester Polytechnic Institute (1967), 1] and was officially dedicated on October 28, 1967.

Today, the Gordon Library holds over 270,000 volumes of books, more than 4,000 volumes of archival materials and rare books, and provides students access to more than 70,000 electronic journals, books, and databases. The facility has undergone several renovations over the years and now contains computer labs and a library café.

The building is a four-story, reinforced concrete structure with a brick and precast concrete panel facade; a rendering of the architect's design is shown below in Figure 1. WPI engaged O.E. Nault & Sons of Worcester, Massachusetts as the architect while Harvey and

Tracey Consulting Engineers served as the structural engineer of record. The structural system is comprised of two-way waffle slabs on each floor, which transmit gravity loads to concrete columns that vary in size and reinforcement patterns along the building's elevation.

The current interior layout, although modified to accommodate increased use of technology and group work, is still influenced significantly by the twentieth century specifications from which the building was tailored. Smoking rooms, music rooms, and the need to store information in the printed medium dominated the building's original design. Aesthetically, the Gordon Library resembles more of a bunker than a library, and there is some perception that it exudes an unwelcoming and cold feeling as a result.

The library has one entrance from campus to the third floor of the building. This entry floor currently features a large open space for computer use and group work along with conference rooms equipped with computers and flat screen TVs called "tech suites" as well as a café for students and faculty. Above the main floor is additional flexible space for group work, tech suites, a lounge containing newspapers and periodicals, quiet study areas, and book stacks. The second floor of the library is primarily comprised of additional quiet study areas, tech suites, and book stacks. Finally, the ground floor of the library contains a much smaller assortment of compact shelving, group study areas, and the recently renovated university archives and special collections department.

Figure 1: Gordon Library Rendering. Taken from [Worcester Polytechnic Institute (1967). Unpublished Rendering]

2.2: The Future of Libraries

There have been remarkable advances in knowledge sharing and research methods since the 1960s. Today, information is more accessible because of the emergence of the Internet and the prevalence of smartphones and tablet devices. The Internet not only reflects a change in the way researchers access information but also poses a significant challenge to libraries, which must continue to be relevant in an age when information is so readily accessible. Not surprisingly, the proliferation of technology is having tangible effects on university libraries across the country – there has been a sharp decline in the circulation of print sources, a reduction in the use of reference services, and falling gate counts [Gayton (2008), 60].

At the same time that advances in technology are threatening the existence of libraries as physical spaces, the traditional notion that libraries are "communal" spaces strictly to support quiet studious activities is also being called into question. One of the driving forces behind this reimagining of the library is a major shift in thinking about learning at the undergraduate level. The classical learning model is one-size fits all. It assumes that students learn best from a teacher and develop and internalize that knowledge independently, in a highly structured environment. Learning is now embraced as a highly individualized and complex process that depends on and is adaptable to the cognitive abilities and learning styles of each student.

While some students thrive in an environment where information is presented by a professor and studied in a quiet, focused environment, other students enjoy informal learning – they learn from friends, Khan Academy, Youtube videos, and other non-traditional methods. Learning also occurs in different environments – some students learn best in noisy environments like cafés, some learn outside, and others prefer communal environments such as the traditional library [Matthews and Walton (2013), 145].

The type of work students are assigned is also changing. Collaborative group work is playing a much bigger role in undergraduate curricula, particularly in response to the need to develop team players capable of working in a fast-paced, global economy.

In short, there has been a paradigm shift in the way colleges think about learning, and while the communal model still has a place, learning increasingly "involves a variety of active, problem-solving experiences that engage the learner in the 'social', rather than the 'individual', development of knowledge" [Matthews and Walton (2013), 144].

These changes in thinking about learning and the increased incorporation of group work into undergraduate curricula are leading to the development of library spaces with a wide variety of environments that support the collaboration between students and faculty in their endeavors to learn and to create new knowledge. One of the primary ways designers have supported these new activities is with the addition of creative commons or social spaces such as group study facilities, information commons, cafés, and art galleries [Gayton (2008), 60].

However, at the same time that many academics are excited by the incorporation of social spaces which support collaborative group work and a multiplicity of learning styles, others fear that

> the social model undermines something that is highly valued in academic libraries: the communal nature of quiet, serious study. Communal activity in academic libraries is a solitary activity: it is studious, contemplative, and quiet. Social activity is a group activity: it is sometimes studious, not always contemplative, and certainly not quiet [Gayton (2008), 60].

This view of the social space as a threat to the communal space makes apparent the need to isolate these very different environments.

The library of the future should also be an inviting and friendly space on the bright side of the line between hip and intimidating. Due to the prevalence of electronic resources and remote access, libraries need to remarket themselves as places where students want to study and create new knowledge. One way to accomplish this goal is to design libraries that are aesthetically appealing – libraries should look more like Apple stores and less like bunkers to attract visitors who would otherwise be satisfied accessing the same information from the comfort of their dormitory.

In summary, future libraries need to address the entire range of learning styles and student needs by incorporating both social and communal spaces. Both environments play a role in supporting learning and the development of knowledge but the design of library spaces must take into account the need to keep them separate from one another. Library spaces should also utilize bold, comfortable designs that motivate students to study at the library.

2.3: Structural elements of Library Facilities

Structures are designed to resist vertical and horizontal forces. Vertical forces include dead loads such as the self-weight of a structure and the weight of permanent, non-structural elements like roofing, flooring, and elevators. Live loads from building occupants, furniture, books, and the environment are another class of vertical loads that structural engineers design for. Horizontal forces, on the other hand, include forces from wind and earthquakes. These forces are "put into the special category of lateral live loads due to the severity of their action upon a building and their potential to cause failure" [Peting, D., and Luebkeman, C.H. (1996)]. The structural elements that resist these forces, including slabs, columns, and lateral force resisting structures, are described in the following sub-sections.

2.3.1: Floor Slabs

Floor slabs are structural elements that resist vertically applied forces and provide occupants with a usable surface to carry out the activities for which a structure was designed to house. Slabs receive and transmit load to other elements in the structural system such as beams, girders, and columns. The simplest type of slab is primarily supported on two opposite sides. In this configuration, the structural action of the slab is one-way. When a load is applied to a one-way slab, a single strip of slab transmits load perpendicularly to the supporting the beams, which in turn, transmit load to columns [MacGregor and Wight (2005), 608]. A slab supported on all four sides is considered to have two-way structural action. In this configuration, one strip of slab transmits load perpendicular to one set of beams, and another strip of slab transmits load perpendicular to another set of beams. Since the slab must transmit load in two directions, it must be reinforced in both directions and is referred to as a two-way slab. It should be noted that a slab supported on all four sides still utilizes one-way structural action if the ratio of length to width of one slab panel is greater than two [Nilson, Darwin, and Dolan (2009), 424].

There are several types of two-way slabs used for different span lengths. For relatively small spans between fifteen and twenty feet, flat plate slabs are used. A flat plate slab is a slab of uniform thickness supported only by columns. For larger spans from twenty five to forty feet, the thickness needed to transmit applied loads to columns exceeds the thickness needed to resist bending moments [MacGregor and Wight (2005), 608]. In such a case, the material of the slab at mid-span is not used efficiently and can be removed to save material and reduce slab moments. This system is referred to as a waffle slab because ribs intersect the areas of removed material creating a waffle-like pattern on the underside of the slab, which is shown below in Figure 2. It should also be noted that the full depth of the slab is maintained in the regions surrounding the columns, a feature called shear head, which allows load to be transmitted from the slab to the columns.

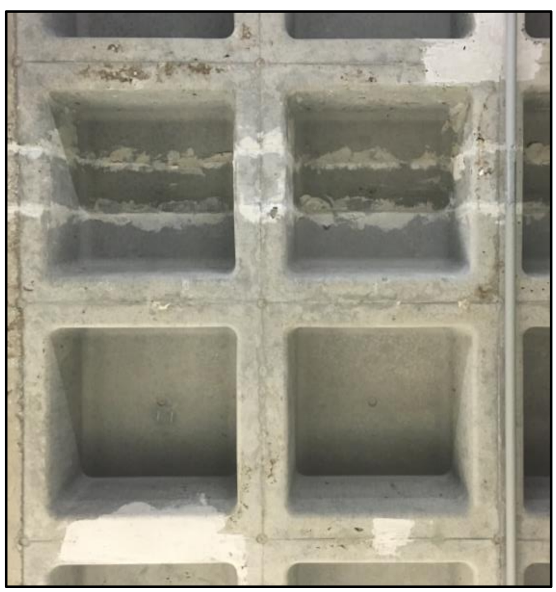

Figure 2: Underside of Waffle Slab on the Ground Floor of the Gordon Library

2.3.2: Columns

Columns are vertical structural members that support axial compressive loads and transmit those loads to a structure's foundation. In a concrete structure, columns are reinforced with longitudinal and transverse reinforcing steel, which vary in configuration depending on the application and loads applied to the column. Longitudinal reinforcing extends from one column into the overlying column where it is lap-spliced with that column's reinforcing. Transverse reinforcing either consists of ties or a spiral. The most common type of column used in non-seismically active regions is the tied column. A tied column consists of longitudinal (vertical) reinforcing bars that are braced with smaller bars along the length of the column. When high strength or high ductility performance is required, the longitudinal reinforcement is arranged in a circle, and a helical or spiral-shaped piece of rebar is wrapped around the longitudinal reinforcing. Under compressive forces, the column tends to expand laterally, and the spiral reinforcement provides confinement to the concrete and enhances its capacity [MacGregor and Wight (2005), 477]. An alternative column type is the composite compression member in which a concrete member is reinforced by a structural steel shape, pipe, or tubing. This column type is becoming increasingly popular, especially in high rise construction, due to its ability to resist very high loads in a small footprint [Denavit, *et.al*. (2008)]

2.3.3: Lateral Force Resisting Systems

A lateral force resisting system (LFRS) is a system of horizontal and vertical structural elements that work integrally to resist wind or earthquake loads. Diaphragms make up the horizontal component of the LFRS while shear walls, moment-resisting frames, or a combination of the two can comprise the vertical component. A model building that resists lateral loads with diaphragms, moment-resisting frames, and shear walls is shown in Figure 5.

Diaphragms are the basis for lateral load resisting systems. They most often serve as the floors and roof of a building and as such, they are also responsible for resisting gravity loads. Diaphragms are responsible for conjoining the vertical elements of the LFRS and transmitting lateral inertial forces to those vertical elements. Diaphragms also provide resistance to out-of-plane forces that develop from wind loads acting on exterior walls and resist thrust from inclined columns [Hooper, *et.al*. (2010), 2]. Diaphragms can transfer lateral forces to interior shear walls, exterior shear walls, or moment-resisting frames [Killian, D.M., and Lee, K.S. (2012), 2] and are required for buildings constructed in Seismic Design Category B, C, D, E, or F. The major components of a diaphragm system include the diaphragm slab, chords, collectors, and connections to the vertical elements of the structure, which are shown below in Figures 3 and 4.

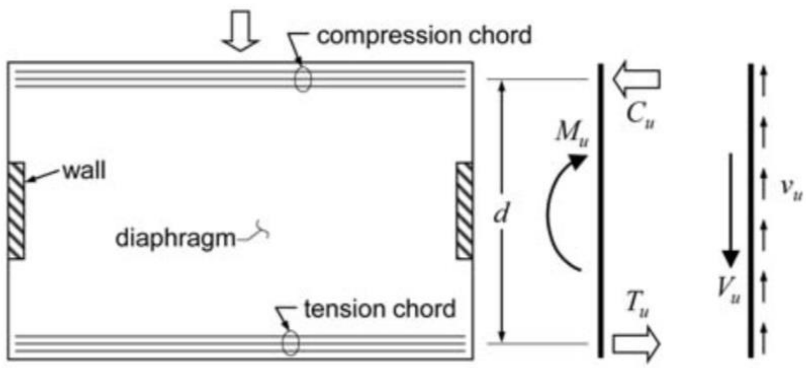

Figure 3: Solid rectangular diaphragm spanning between two end walls, with lateral inertial loading. Taken from [Hooper, *et.al.* (2010), 3]

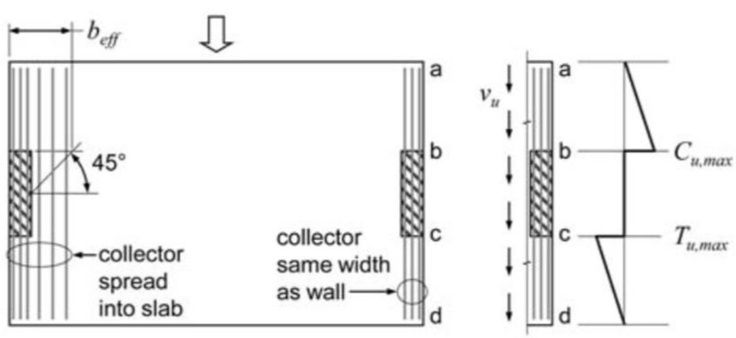

Figure 4: Collectors and Collector Actions. Taken from [Hooper, *et.al.* (2010), 3]

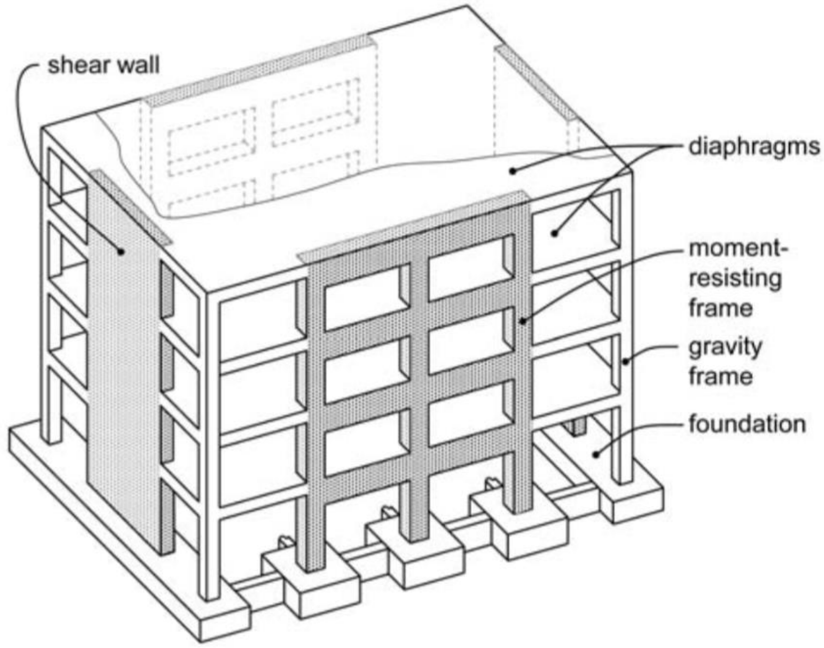

Figure 5: Isometric View of Structural System. Taken from [Hooper, *et.al.* (2010), 1]

Diaphragms work integrally with either shear walls or moment-resisting frames to resist lateral forces from wind and earthquakes.

A moment-resisting frame is composed of interconnected beams and columns that are rigidly connected at their ends to prohibit rotations between the attached members. While the joints of a moment-resisting frame may rotate as a unit, rigid frame members are essentially considered to be continuous through the joints and do not rotate with respect to each other [Schodek (2013), 350]. The advantage to this is that rigid connections restrain columns from freely rotating under laterally applied forces, which could cause a major structural failure.

Shear walls, also known as structural walls, are another example of vertical elements that resist lateral forces applied to a structure. They are primarily responsible for resisting in-plane loads applied along the height of a building. In a reinforced concrete building, shear walls are typically composed of cast-in-place concrete and deformed steel reinforcement [Fields, *et.al*. (2012), 1], but precast concrete can also be used as a shear wall. There are several types of shear walls: the most basic shear wall is designed to resist combinations of shears, moments, and axial forces while shear walls designed for buildings located in Seismic Design Categories D, E, or F are referred to as special structural walls and must conform to the requirements listed in Chapter 21 of *ACI 318* [Fields, *et.al*. (2012), 2]. The placement of shear walls is also very important. Not only located at the building exterior, shear walls are commonly found on the interior as elevator or stairway cores where they serve a dual purpose of enclosing a space and resisting axial and lateral forces. Shear walls are typically the most cost effective for low to mid-rise buildings where floor-to-floor heights are typically minimized and the added depth required for moment frame members would translate into higher construction costs.

2.3.4: Foundations

Foundations transfer load from the superstructure to the underlying soil or rock. Factors that influence foundation design include the load to be transferred from the building, the behavior of soils under load and their resistance to load, the building code requirements, the geological conditions of the soil, and the depth of frost in colder climates [Das, B. (2011), 1]. There are two main classes of foundations: shallow foundations and deep foundations. Shallow foundations are typically embedded to a depth of three to four times the width of the foundation or less and include spread footings, wall footings, and mat foundations. Drilled shaft and piles make up the second class of foundations and are used in cases where the top layers of the soil have insufficient load bearing capacity.

2.4: Building Codes

A building code is a legal document created to ensure that structures are designed to a standard level of performance, which protects public safety, health, and welfare. Building codes provide minimum strengths of materials, maximum occupancies, and design loads for structures of all kinds. These criteria may be defined in the building code or established by reference to industry standards, such as AISC and ACI specifications and ASCE, NFPA, and ASHRAE standards.

If a new library were being constructed in Worcester, Massachusetts, in the present day, it would have to comply with the *Eighth Edition, Massachusetts Building Code* (780 CMR). This building code is based off the 2009 *International Building Code* (IBC) produced by the International Code Council (ICC). The *IBC* is a model building code adopted by most localities in the United States and amended through the publication of building codes at the state level. The first edition of the Massachusetts building code was published in 1974. In years prior, the city of Worcester promulgated its own building code, which was used in the design and construction of the Gordon Library.

The current *Massachusetts Building Code*, 780 CMR, varies drastically from the 1965 *Worcester Building Code* which was used to design the Gordon Library. Significant technical advances in fire protection engineering, and earthquake, wind, and snow modeling have changed the way engineers think about designing structures and these changes are reflected in the building code.

Another facet of the building code is industry standards. The American Institute of Steel Construction (AISC) and the American Concrete Institute (ACI) publish design requirements for steel and concrete structural members, respectively. These requirements are referenced by the *IBC* and must be followed by designers to ensure public safety. Since structural steel shapes produced today vary significantly from those used in the Gordon Library, the *AISC Rehabilitation and Retrofit Guide (2002)* was obtained for the benchmarking process.

2.5: Structural Design and Evaluation

Design by analysis and an economic evaluation of the alternatives was used to facilitate the design and comparison of the structural steel and reinforced concrete alternatives.

In order to automate and mitigate the complexity of the LFRS design process, finite element models of the rigid frames were prepared and analyzed. A finite element model is a computer assembly of building elements modeled using their physical and engineering properties and arranged in their desired configurations. Once the structure is modeled, loads are applied to its columns, girders, and floors, and the analysis software automatically calculates the resulting stresses and bending moments. *RISA 2D* which is an industry standard finite element analysis program was used. The code check feature of *RISA 2D* was included in the analytical approach to verify that the structural members satisfy the requirements of the *2010 AISC Specification* and *ACI 318-11*.

A key component of developing the highest quality and best value solution involved estimating the cost of the structural steel and reinforced concrete alternatives. Material takeoffs were performed for the structural framing alternatives and *RS Means* construction cost data was used to determine the cost of the alternatives. *RS Means* is a reliable source of construction costs based on U.S. national averages. The cost of standard building elements such as electrical, mechanical, and interior finish work was also evaluated using *RS Means*. Ancillary costs such as fireproofing for the steel alternatives and formwork for the reinforced concrete alternative were factored into the cost analysis and influenced the decision making process when selecting the best value solution.

Chapter 3: Architecture

This chapter begins with results from the Gordon Library benchmarking activity and proceeds to discuss the broad spatial layout and key features of the new design. The aesthetics and context of the design as well as various interpretations of the utility the building will provide to its users is a main focus of the chapter.

3.1: Benchmarking Results and Implications for New Design

The main focus of the benchmarking activity was to quantify the amount of daylighting and study space in the existing Gordon Library. A summary of the daylighting assessment is presented below in Table 1 and Table 2. To evaluate daylighting, the number of windows on each floor of the library was tallied and the total window area was calculated.

Table 1: Window Count for Each Floor of the Gordon Library

Level	Number of windows facing East	Number of windows facing North	Number of windows facing South	Number of windows facing West
Ground floor	11	0	0	0
First floor	11	4	0	0
Second floor	11	2	2	0
Third floor	11	5	6	3

Table 2: Window Area for Each Floor of the Gordon Library

Level	Total Number of Windows	Total Window Area (ft^2)	Percent of Perimeter Area
Ground floor	11	616	13
First floor	15	840	17
Second floor	15	840	12
Third floor	25	1400	19

The desire to increase the amount of daylighting in the space was derived from the rather minimal percentage of facade perimeter area composed of windows as evidenced by Table 2.

The amount of study space in the existing facility was also benchmarked by counting the total number of tech suites in the space and performing area measurements of the study spaces throughout the library. There are 11 tech suites in the existing facility and the study space area measurements are shown below in Table 3.

Table 3: Approximate Study Space Area for Each Floor of the Gordon Library Including Tech Suites and Communal Study Space

Level	Approximate study space area (ft^2)
Ground floor	4418
First floor	1238
Second floor	5380
Third floor	2138

These values contributed to the overall objective of increasing the total number of tech suites and study space with the new design.

Finally, a comparison of the design live loads prescribed by the 1965 Worcester Building Code and the *Massachusetts Building Code* was performed to get a sense for the loads that the existing structure was designed for. Results from this exercise are shown below in Table 4.

Table 4: Design Load Comparison Between the *Massachusetts Building Code* and 1965 Worcester Building Code

Load Type	Massachusetts Building Code (psf)	1965 Worcester Building Code (psf)
Live loads	Reading rooms 60 Stack rooms 150 Corridors above first floor 80	Reading rooms 60 Stack Rooms 150
Wind loads	17	15
Snow loads	55	30

3.2: Introduction to the Spatial Layout and Key Features of the New Design

While some modifications were made, the design was developed with the goal of fitting the structure into the existing Gordon Library site location which is on the side of a large hill at the East end of the WPI campus. The main objective of the architectural design was to escape the war-time bunker typology reflected in the existing Gordon library. In order to do this, square footage was sacrificed by carving out a giant 80 foot by 34 foot atrium in the middle of the library. The atrium extends from the ground floor to the roof level and allows light from a skylight at the top of the building to filter through the space. While an atrium is impractical in some ways, in this case it is essential for library users to walk into the space and feel excited and amazed by what they see. Two glass elevators located at opposite corners of the atrium provide service to all four floors of the building. A staircase also drops into the atrium and provides service to the second floor. The staircase combined with the glass elevators give the atrium a very modern feel. Finally, a simple building information model (BIM) of the new design was created using Revit. Renderings of the West, East, and North elevations are shown in Figures 6, 7, and 8.

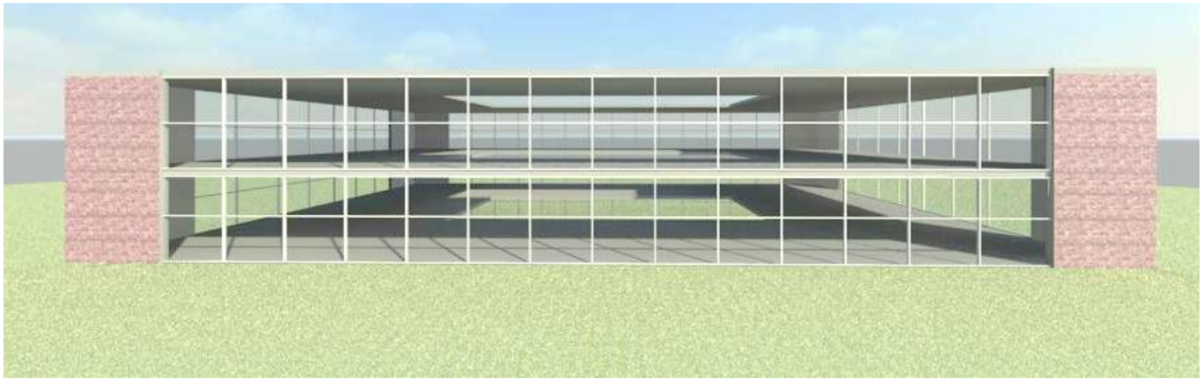

Figure 6: West Elevation Rendering

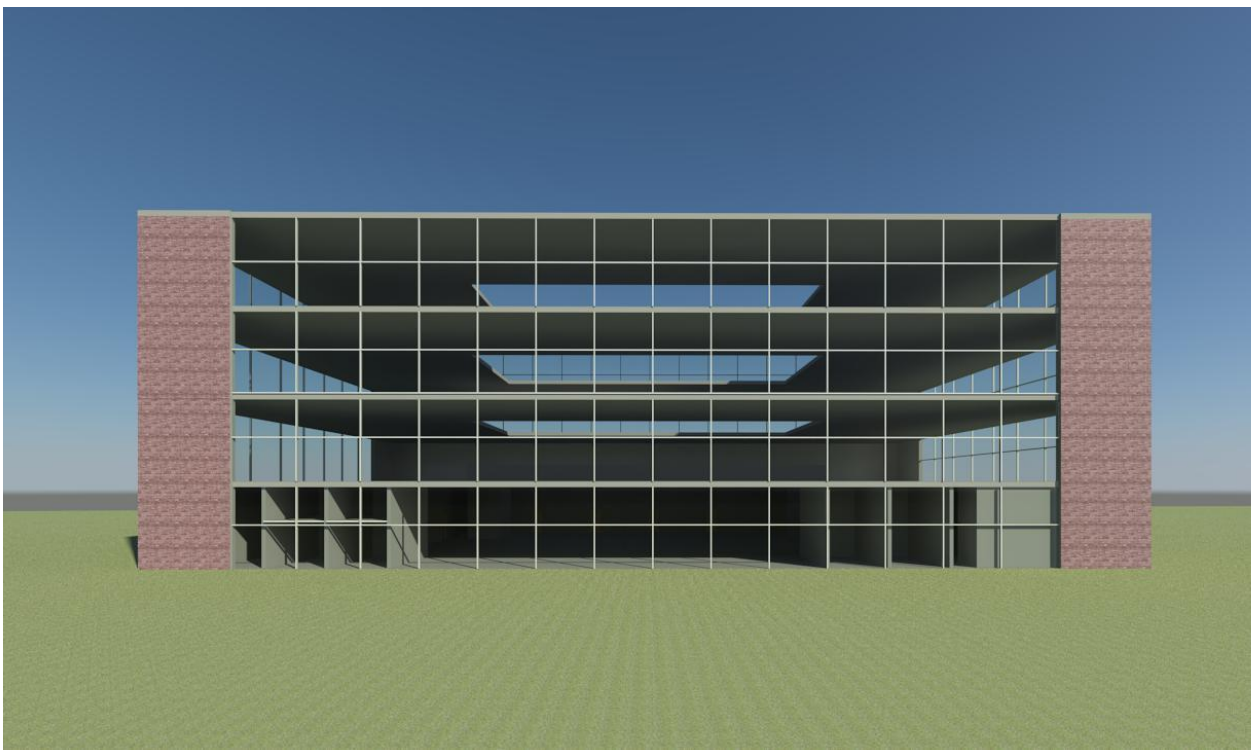

Figure 7: East Elevation Rendering

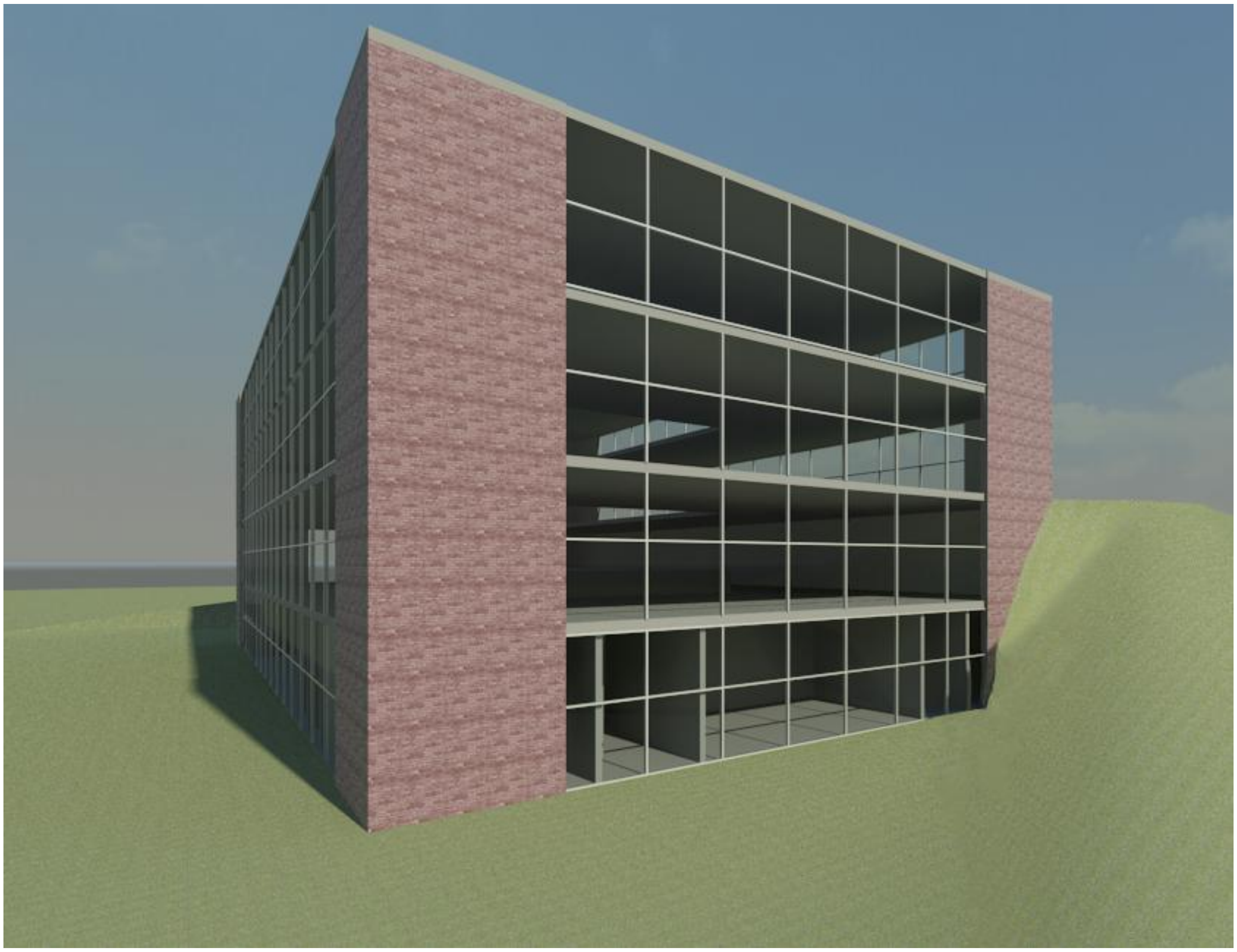

Figure 8: North Elevation Showing Hill

3.2: Column Grid Design

The column grid design was first developed by placing six columns at the perimeter of the atrium in order to provide proper support at the edges of the elevated floor slabs as shown below in Figure 9.

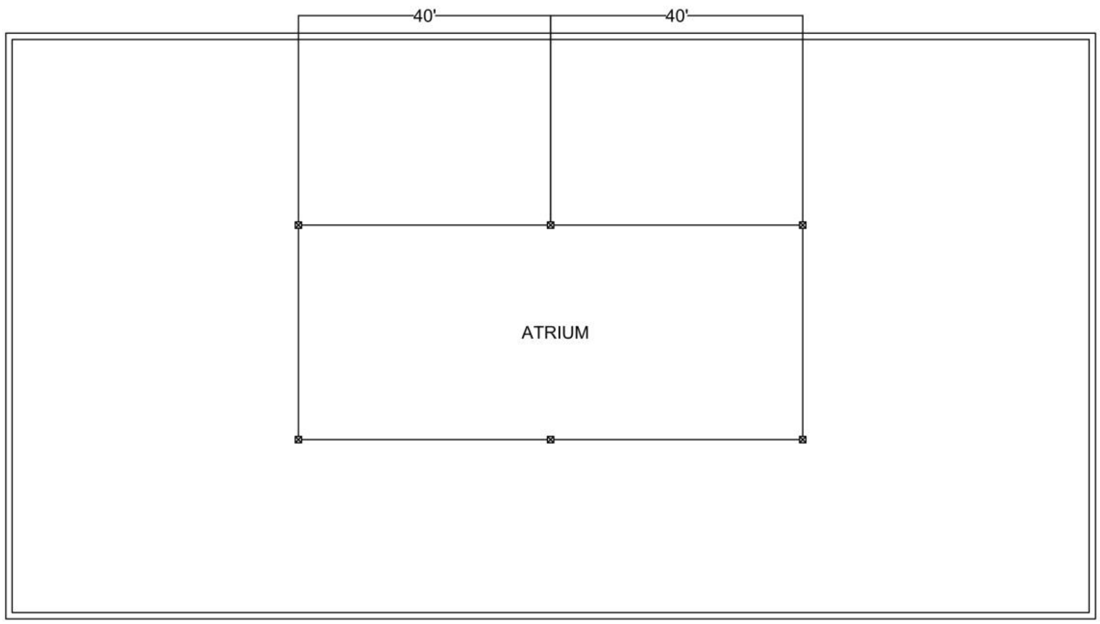

Figure 9: Column Placement – Stage 1

A key component of the column grid design was reducing the number of columns used in the structure in order to create large open floor spaces and contribute to improving constructability. Since column span lengths correlate with member sizes, the spans between columns must be practical in order to minimize the overall cost of the structure. In other words, there is a balance between the number of columns and the spans.

Spans in the range of 20-30 feet were considered in order to maintain the atrium size and fit the new structure into the existing building footprint. The columns were placed according to the grid shown below in Figure 10.

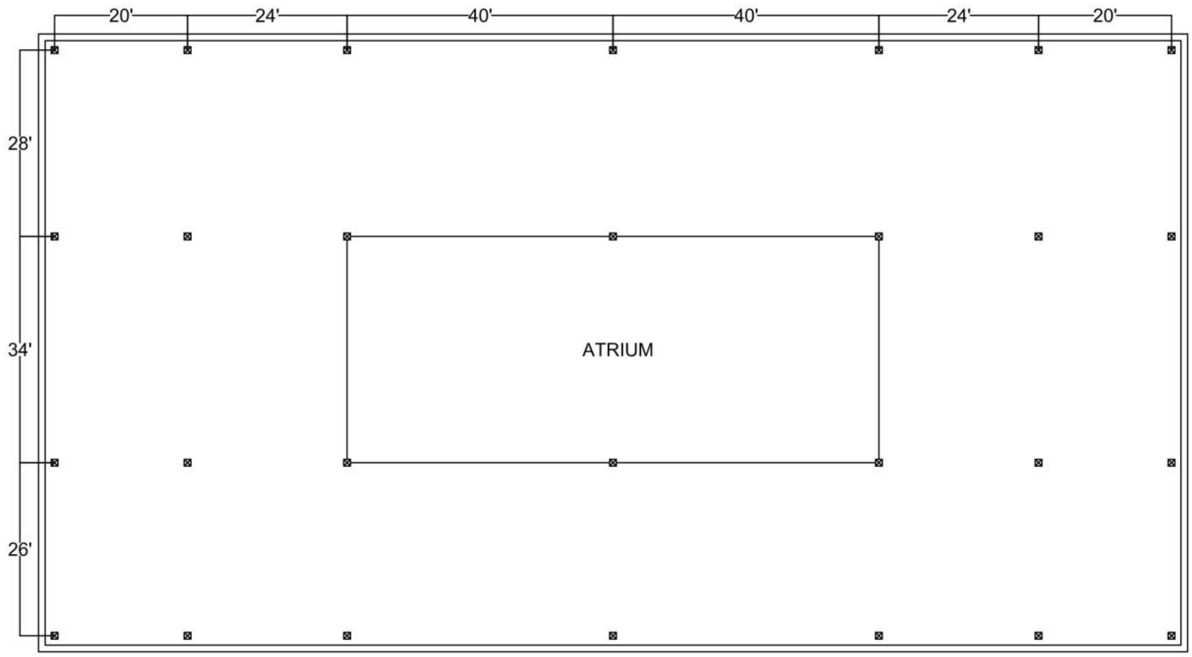

Figure 10: Column Placement – Stage 2

3.3: Interior Floor Plans

The definitions and layout of the interior spaces were determined first by assigning each floor a use-type. The ground floor of the library is a social space; the first floor houses the library's printed materials; the second floor is a group workspace; and the third floor is a quiet study space.

The ground floor layout most notably features six entrances from the Boynton Street Parking Lot, a large art gallery, café, and a computer laboratory as shown below in Figure 11. Entrances from the Boynton Street Parking Lot were primarily created to provide easy access for visitors approaching from the East side of the WPI campus. Currently, these visitors must climb a lengthy staircase that extends from the Boynton Street Parking Lot and traverses the hill that the library is built into.

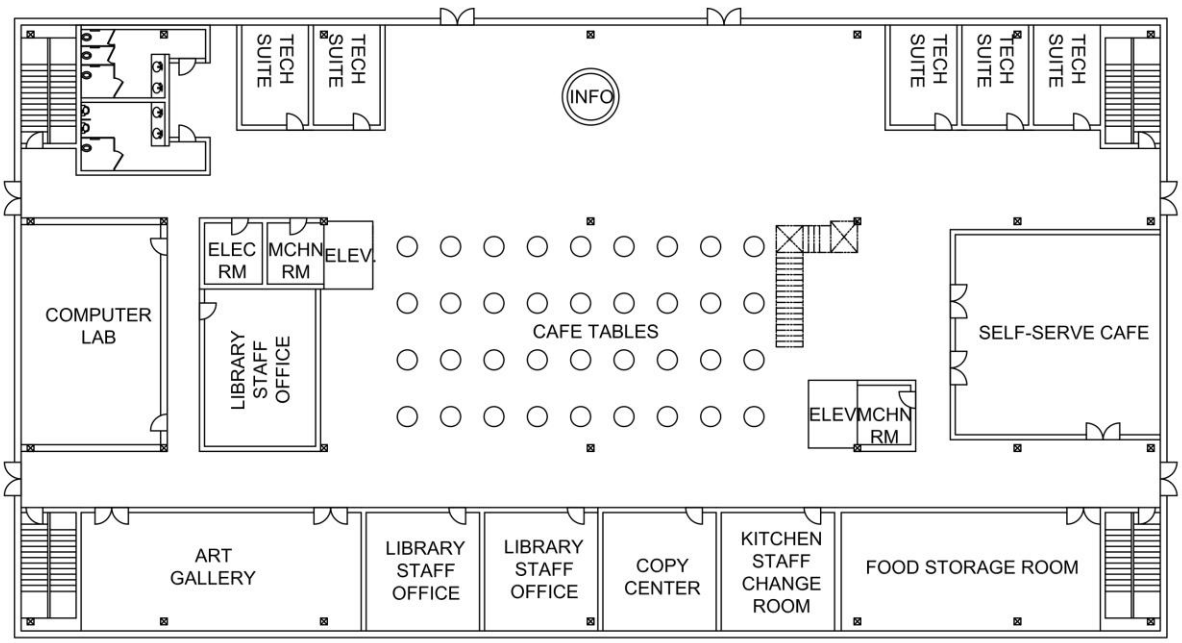

Figure 11: Ground Floor Layout

The self-serve café on the ground floor will offer breakfast, lunch and dinner. Ten percent of the ground floor space or 1,500 square feet was allocated as dining space. A common design rule is to allocate sixty percent of the restaurant space for dining and forty percent for meal preparation [Total Food Service (2013)]. This resulted in a 600 square foot food storage room to serve the ground and third floor cafés. The food storage room simply contains refrigeration and enclosed warming racks. The design intent is for food to be cooked and prepared at the Campus

Center facility so exhaust hoods and ventilation were not designed and incorporated into the library design. A kitchen staff change room was also included in the ground floor plan to enable workers to gather and change before and after their shifts. In addition, the ground floor contains an information desk, a handicap accessible restroom, three library staff offices, a copy center, a computer lab, five tech suites, and an art gallery.

The first floor of the library is the only floor that features books and printed materials. This floor was designed to house book stacks and features two lounge areas around the atrium for students to read books and periodicals as shown below in Figure 12. The book stacks are spaced 36 inches apart which is the design recommendation from the *Whole Building Design Guide* [Whole Building Design Guide (2014)]. A 36-inch spacing also meets the minimum clear width requirement for a single wheel chair in an alcove as prescribed by Section 305.7.1 of the *2010 ADA Standards for Accessible Design* [United States Department of Justice (2010), 109].

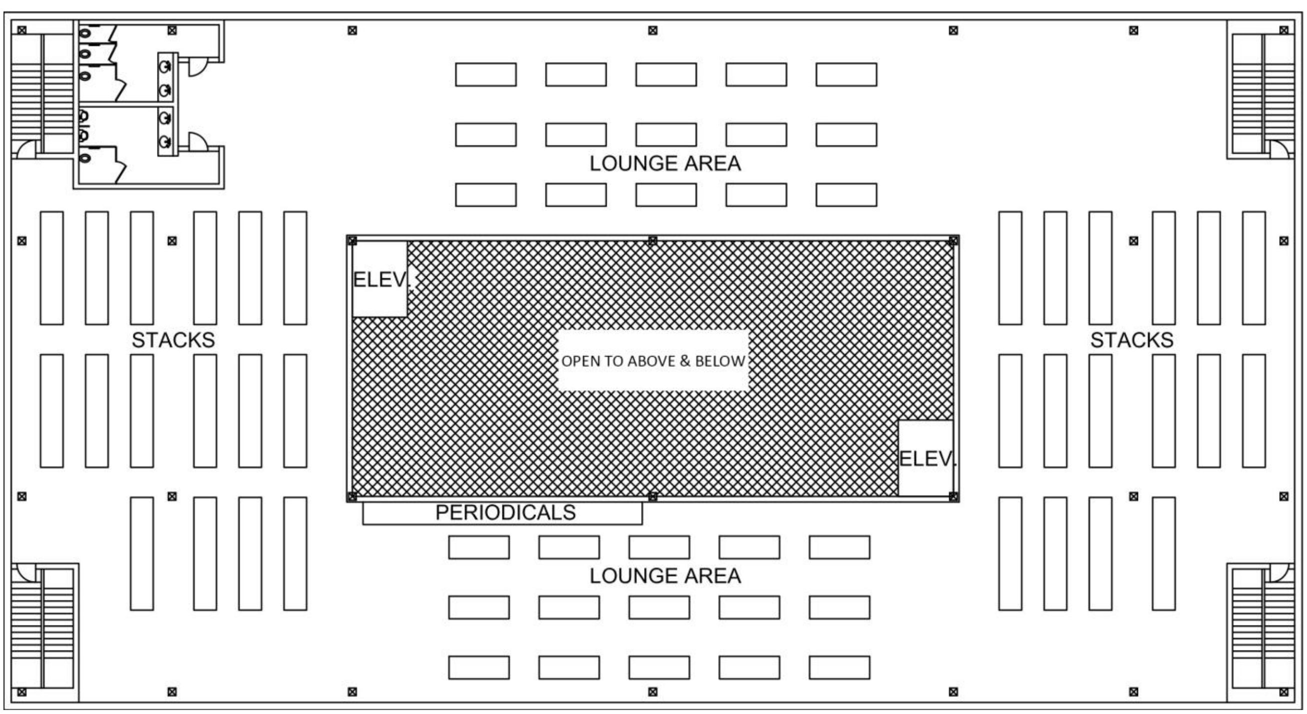

Figure 12: First Floor Interior Floor Plan

The second floor of the library was configured to be a social workspace. The floor plan is shown below in Figure 13. This floor has an entrance that services visitors coming from the main campus level and the West end of the campus. The most notable design feature is a large mixed-use conference room with long tables for students to work collaboratively called the "Living Room" [The American Institute of Architects (2015)]. The Living Room features a floor-to-ceiling curtain wall, which provides a view to the East part of the campus. The second floor also contains a large computer area, eight conference rooms equipped with computers and flat screen TVs called "tech suites," a mini café that serves coffee and pastries, and large open areas for group work and computer access.

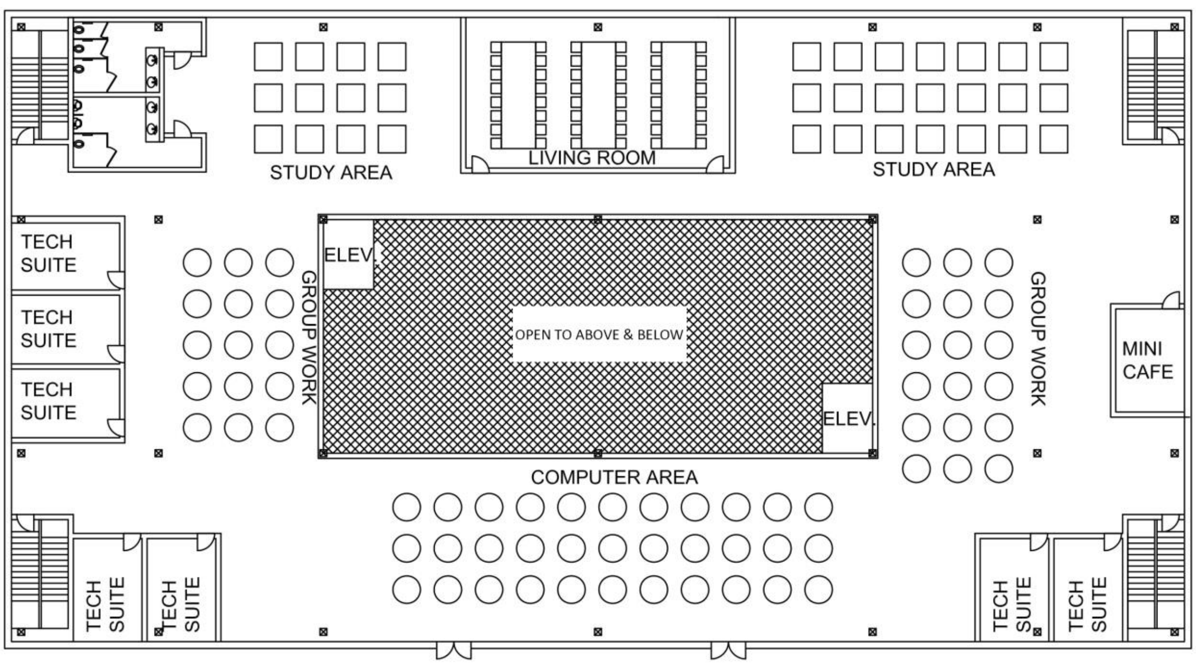

Figure 13: Second Floor Interior Floor Plan

The third floor of the library was designed to be a communal, quiet floor. The layout is shown below in Figure 14. It features seventeen quiet study rooms, seven tech suites, and ample quiet study space with views to the exterior throughout. Reading rooms were sized to be 100 square feet which is adequate space for a desk and two chairs [Fennie, N. (2005)].

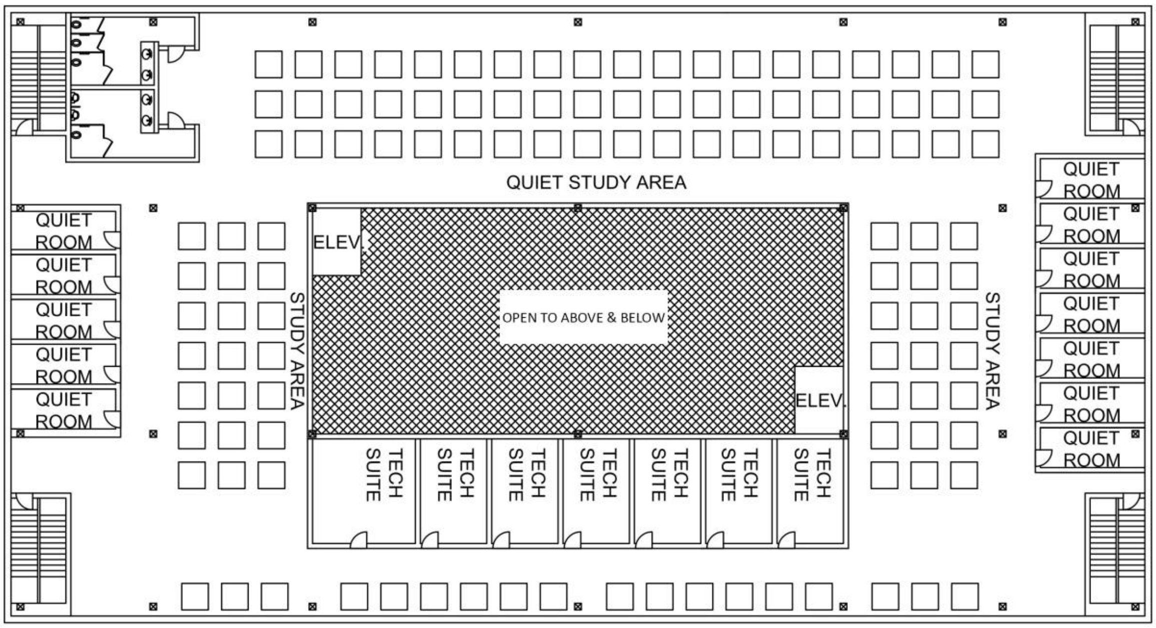

Figure 14: Third Floor Layout

3.4: Design of Common Elements

Tech Suites, accessible restrooms, elevators, and elevator machine rooms are common elements of the design because they occur on all four floors of the library. The handicap accessible restroom was designed using guidelines from *Architectural Graphics Standards* [Ramsey and Sleeper (2007)], a reference used by architects to design buildings and interior spaces. *Architectural Graphics Standards* features illustrations of wheelchairs in various spaces. To help designers comply with the Americans with Disabilities Act standards, it displays the minimum clearances required for a person in a wheelchair to turn a corner and turn around. The *Planning Guide for Accessible Restrooms,* published by Bobrick Washroom Accessories, was also used to size and layout the restrooms. The restroom design is 20 feet by 20 feet and is shown below in Figure 15.

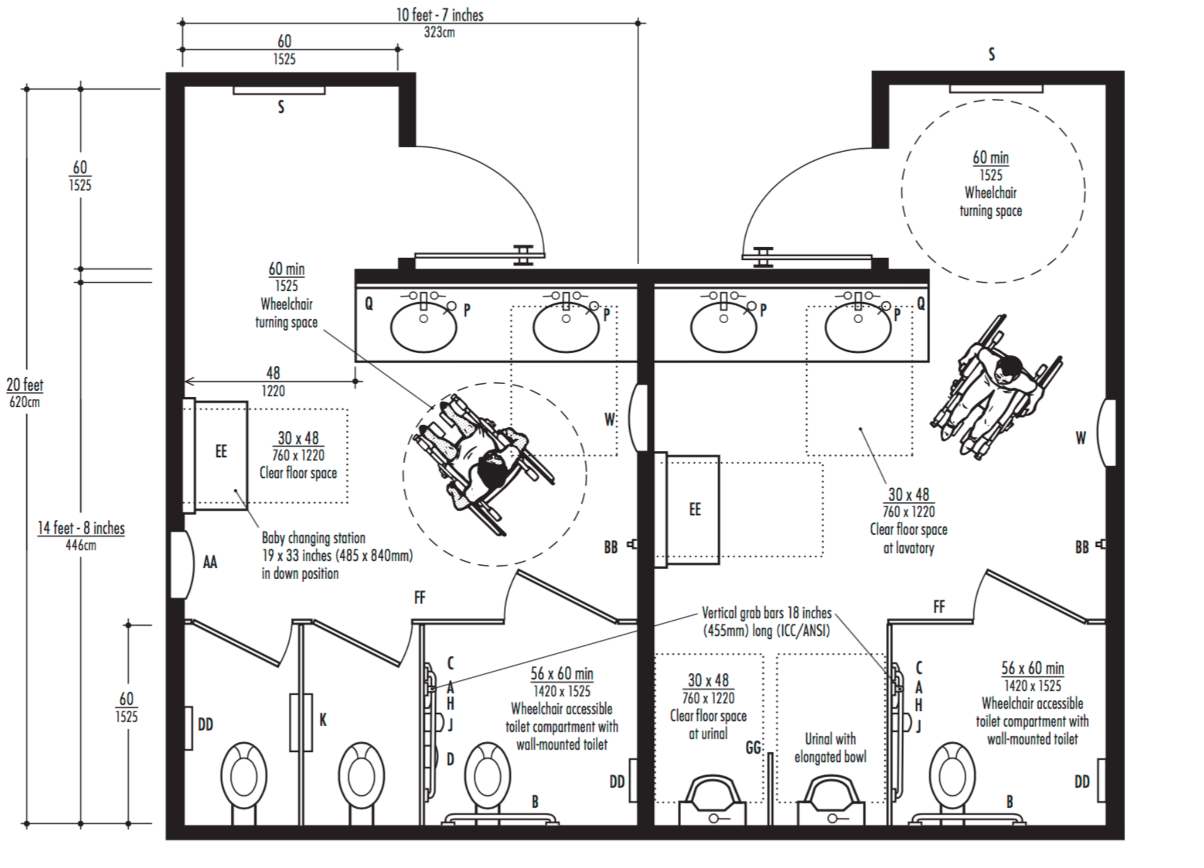

Figure 15: Accessible Restroom Design. Taken from [Bobrick (2012), 12]

Tech Suites were sized to be 10’ x 15’, which according to a space planning guide, is “very prevalent these days and can fit a mid-manager desk and return, two guest chairs and a bookshelf” [Fennie (2005)]. This size will be adequate to fit a wall mounted tv, desk, and chairs for meetings and was chosen for all Tech Suites throughout the building.

Finally, Thyssenkrupp’s *Elevator Planning Guide* (2003) was used to size the two elevator hoistways and machine rooms for the glass elevators located in the atrium.

3.5 Roofing Design

In order to perform a realistic building design, the roofing system was selected using engineering judgement, and the weight of the system was factored into all calculations. Single-ply roofing systems were investigated for use on this project because “compared to bituminous roof membranes, they require less on-site labor, and especially in comparison to built-up roof membranes, they are more elastic and therefore less prone to cracking and tearing as they age” [Allen and Iano (2009), 667]. Investigation into single-ply roofing systems resulted in selecting the EverGuard Extreme TPO roofing system manufactured by GAF. Thermoplastic Polyolefin (TPO) is a single ply-roofing membrane that “offers many of the same benefits as PVC roofing, such as hot-air weldable seams and energy efficiency, but at a lower cost” [Red River Roofing (2014)] A schematic of the roofing system chosen for this project is shown below in Figure 16.

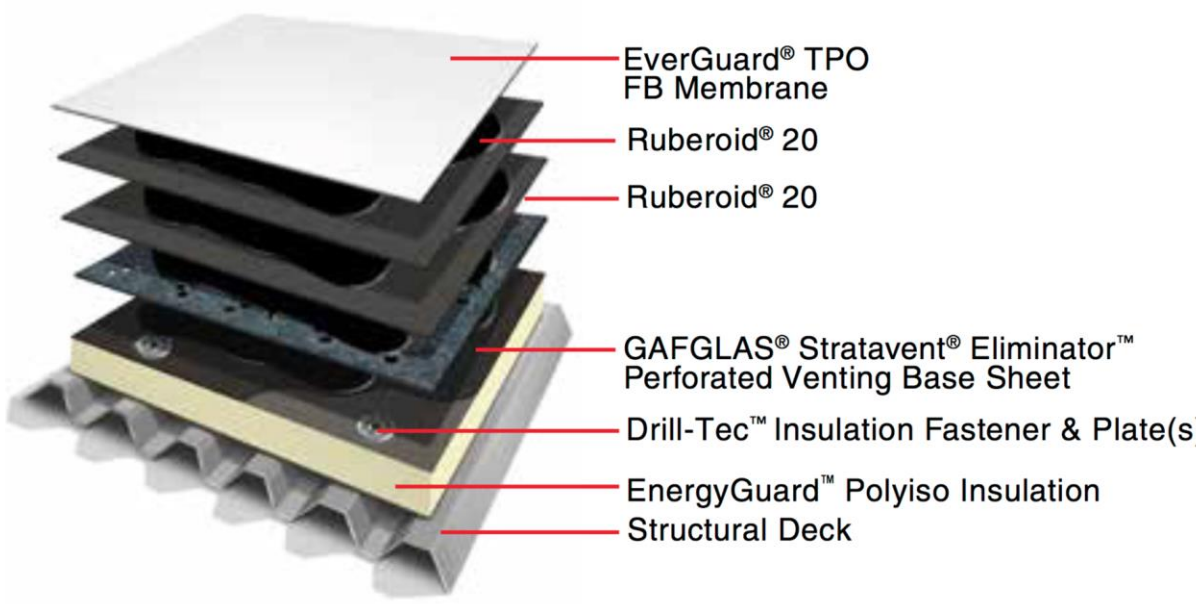

Figure 16: EverGuard Extreme TPO Roofing System. Taken from [GAF (2016)]

3.6: Egress Design

Providing effective means of egress is vital to ensuring the safety of building occupants and is therefore an essential facet of modern building design. Planning for egress early on and in conjunction with space planning mitigates the risk that building designers will have to reconfigure interior layouts to accommodate egress spaces. The first step in determining the required egress means involved classifying the building with respect to its occupancy. Per *780 CMR* Section 303.1, libraries are classified as Group A-3. From the occupancy classification, the length of exit access travel was defined using Table 1006.5 of *780 CMR*. The length of exit access travel for a Group A-3 building with an approved, supervised automatic sprinkler system is 250 feet. The original building design had two stairwells per floor, which provided an exit access travel length of 143 feet. Exit stairway calculations required an additional two stairways to provide sufficient capacity. The set of four stairways decreased the exit access travel length to just 89 feet, which is a 38 percent decrease in travel distance. A map of a typical interior floor plan with exit access travel lengths is shown below in Figure 17.

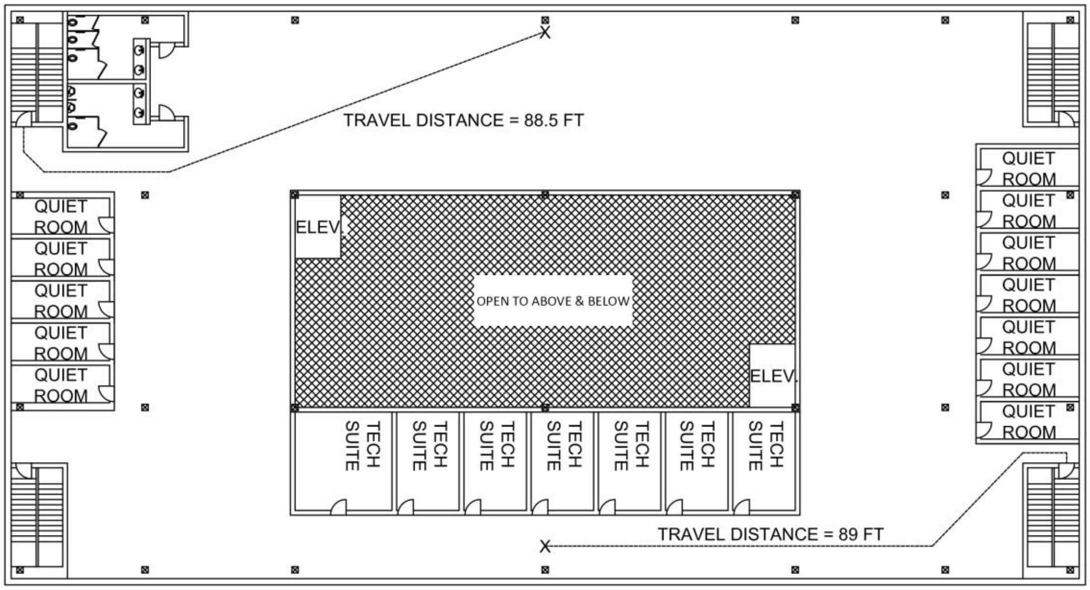

Figure 17: Length of Exit Access Travel Map

Calculating the required number of stairwells and stairway clear distance involved classifying each space according to its occupancy. *NFPA 101* Table 7.3.1.2 provides occupant load factors for the various occupancy types within the library facility. In order to calculate the occupant load, the total square footage per occupancy type was tallied and then divided by the occupant load factors. Results from this process are shown below in Table 5.

Table 5: Occupancy Classification of the Interior Space

Space	Total Area (SQ FT)	Occupant Load Factor	Occupant Load
Kitchen	689	100	7
Business	6930	100	69
Library Stack Areas	3599	100	36
Assembly - less concentrated	37926	15	2528
Industrial (Elevator Rooms)	240	100	2
		Total Occupants	2643
		Occupants Per Floor	661

Section 1007.2 of *MA 780 CMR* stipulates that the minimum stairwell exit size is 48 inches. *NFPA 101* provides a direct method for calculating the minimum required stairwell size to satisfy the occupant load. The equation and relevant factor used to perform this calculation is shown below in Table 6.

Table 6: Minimum Required Stairwell Size Equation

Equation	Stair Factor
$\frac{Occupant\ Load * Stair\ Factor}{Number\ of\ Stairwells}$	0.3

Using the formula shown in Table 6 resulted in a minimum clear distance of 50 inches for 4 stairwells.

Chapter 4: Structural Steel Design

This chapter describes the use of steel as a building material and charts the methods used to design the steel alternative.

4.1: Structural Steel as a Building Material

Structural steel's high strength to weight ratio coupled with its ductility and weldability have afforded it enormous popularity as a building material. One of the main advantages of steel is that, in contrast to load-bearing masonry or cast-in-place concrete, steel is a prefabricated construction material that is manufactured in a factory and assembled on site. This feature greatly enhances construction productivity as steel can be erected rapidly in all seasons.

Despite the benefits of steel, there are several disadvantages to using it as a construction material. First, there is a procurement issue with steel in the sense that only a limited number of steel mills produce the material, and they roll steel shapes according to the projects they schedule. In order to construct a steel building, the shapes needed for the project must fit into a mill's schedule which can cause project scheduling issues and delay the start of construction. This disadvantage can be mitigated if the variation of steel sizes is reduced so that the steel order does not involve a wide schedule for rolling. Other disadvantages of steel construction include its low thermal mass. Thermal mass is a quantity that reflects the ability of a material to absorb and store heat. A major implication of steel's low thermal mass is that "steel conducts heat too rapidly to be in synch with a building's natural heat flows over the day" [Mineral Products Association (2015)]. This results in higher heating costs in the winter months and higher cooling costs in the summer months. In addition, steel members lack inherent fire resistivity and must be protected against structural fires. Spray applied fire resistive materials are often applied to structural steel members but this results in added project cost and time. The implications of steel's low thermal mass and lack of inherent fire resistivity are that they will ultimately increase the cost of the structure.

In any case, steel is still a very competitive material in the building market and was explored as a structural alternative for this project. Design of the steel framework to support an academic library began by utilizing the structural framing plan developed from the interior layout design. The structural steel system included composite beam-and-slab floor systems, W-shape columns, and braced frames composed of W-shape columns and HSS sections.

4.2: Composite Beam-and-Floor Slab Design

Composite floor slab construction was chosen because it is widely considered the highest quality of floor construction and is often specified for steel framed buildings in which serviceability is a primary concern [Liu (2007), 8]. In a composite section, shear studs are welded to steel beams in the field and bond to the concrete slab when it cures. This mechanical bond allows for the transfer of shear force between the concrete slab and steel beams so the two elements act as a single cross section to resist applied loads. Composite action provides two main benefits: improved strength and serviceability. A steel beam joined compositely with concrete can resist 33 to 50 percent more load than its non-composite counterpart [McCormac (2012), 562]. Composite sections are also much stiffer than standard slab construction and enhance serviceability by increasing deflection and vibration resistance.

Composite beam-and-slab floor systems were designed according to the provisions of the *2010 AISC Specification* [American Institute of Steel Construction (2010)] and methods presented in *Structural Steel Design* [McCormac (2012)]. The floor system was designed for un-shored construction in order increase construction productivity. Beams and girders for the roof level were designed independently of those for the library floors due to the differing design loads. A summary of the design loads is presented below in Table 7. In addition, consideration was given to the design load of the skylight, and a preliminary calculation of the skylight load is shown in Appendix B-38.

Table 7: Library Design Loads

Type	Load (PSF)	Reference
Dead Loads		
Concrete (factored 10 % for ponding)	42.9	Vulcraft Steel Roof & Floor Deck
Metal Deck	2.49	Vulcraft Steel Roof & Floor Deck (48)
Acoustical Ceiling Tile	2	McCormac (42)
MEP	5	Engineering Judgement
Skylight	15	Engineering Calculations
Roof MEP + Roofing	3	GAF Commercial Roofing
Live Loads		
Occupancy	150	MA 780 CMR
Wind	17	MA 780 CMR
Snow	55	MA 780 CMR
Seismic	Varies (See Chapter X)	MA 780 CMR
Construction	25	Engineering Judgement
Roof	20	ASCE 7-10

A uniform live load of 150 PSF for library stack rooms was obtained from the *Massachusetts Building Code*. This live load was used throughout the structure in order to provide flexible use of the space and to simplify calculations. According to specification 3.1.2 of *ASCE 7-10*, “In determining dead loads for purposes of design, the actual weights of materials and constructions shall be used provided that in the absence of definite information, values approved by the authority having jurisdiction shall be used.” [ASCE 7-10 (2010), 11] For the purpose of this project, assumed values were used for mechanical, electrical, plumbing (MEP), and ceiling loads. Vulcraft steel decking was chosen for use in the composite floor system, and the dead load of the steel deck and concrete slab was obtained from the manufacturer's catalog [Steel Roof & Floor Deck (2010), 48]. In addition, the weight of the concrete was increased by ten percent to account for the effects of ponding during placement.

Properties of structural steel were based on values given in the *AISC Manual of Steel Construction*. These properties include a modulus of elasticity (E) of 29,000 ksi and a yield strength (Fy) of 50 ksi for W sections rolled from A992 steel. In addition, ¾ inch diameter shear studs were specified throughout the design. The tensile strength Fu = 65 ksi was used for the shear studs, as given by ASTM A108 in Table 2-6 of the *AISC Manual of Steel Construction*. For the concrete slab design, a unit weight of 150 pcf and compressive strength (f'c) of 5,000 psi were defined.

The Vulcraft 1.5VL19 metal decking system was chosen from the Vulcraft catalog [Vulcraft (2008), 48] to serve as the decking for the composite floor construction as opposed to a solid slab. Composite steel decking provides several benefits over solid concrete slabs: they enhance construction productivity, serve as a working platform during the construction process, and provide reinforcement and form for the concrete when construction is finished [ASC Steel Deck (2014)]. A section view of the composite floor slab system is shown in Figure 18.

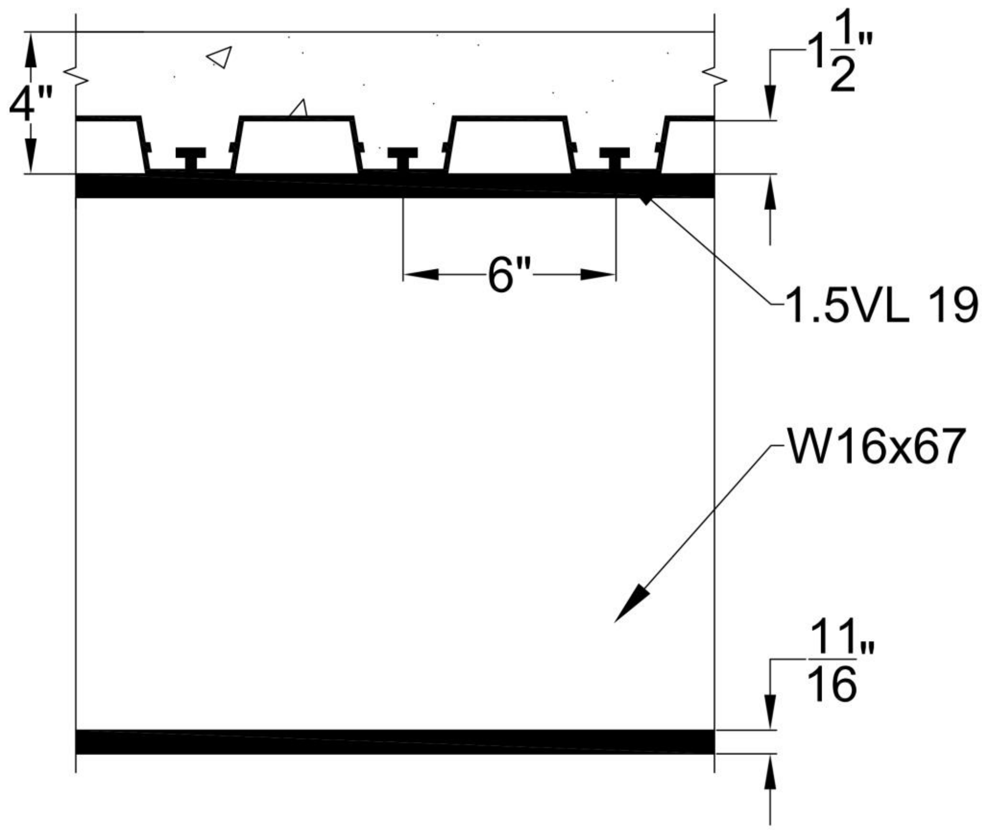

Figure 18: Section View of Typical Composite Floor Slab System

The process for a typical steel beam design is outlined below in bullet form and a more detailed set of calculations is shown in Appendix B-39.

Composite Beam-and-Slab Design Process:

- Determine the bay size and spacing of infill members
- Select metal decking from the Vulcraft catalog that meets the span and live load requirements. Record the slab thickness t_s and weight of the concrete.
- Take the concrete dead load provided in the Vulcraft catalog and increase the load by 10 percent to account for the effects of ponding.
- Sum the total dead and live design loads including concrete, metal deck, MEP, acoustical ceiling, occupancy live, and construction live loads.

- Use the LRFD Load Combination $W_u = 1.2D + 1.6L + 0.5S$ to determine the uniform design load to be resisted by the composite beam [ASCE 7-10 (2010), 7]. S=0 for Ground Floor – Floor 3.
- Determine the design moment to be resisted by the steel beam using the equation: $M_u = \frac{W_u \cdot l^2}{8}$
- Determine the effective width of the concrete slab, b_e, by selecting the smaller of: $2 \cdot \frac{Center\ to\ Center\ Beam\ Span}{8}$

  $2 \cdot \frac{Center\ to\ Center\ Beam\ Spacing}{2}$
- Proceed with the full composite design by assuming the Plastic Neutral Axis is located within the concrete slab. Assume the depth of the compressive stress block a = 2 inches and calculate Y_2, the distance from the centroid of the slab to the top of the steel flange using the equation: $Y_2 = t_s - a/2$
- Use Table 3-19 of the *AISC Manual of Steel Construction* to select a steel shape that provides moment capacity $\phi_b M_n > M_u$
- Verify the depth of the compressive stress block lies within the concrete slab using the equation: $a = \frac{\sum Q_n}{0.85 \cdot b_e \cdot f_c'}$
- Calculate the actual $\phi_b M_n$ by using the value for a calculated above and interpolating with Table 3-19
- Check the beam strength before the concrete hardens by factoring in the beam load, construction live load and treat concrete as a live load.
- Check the beam deflection during construction using service values for the loads:

$\Delta_{cons} = \frac{5 \cdot w_{cons} \cdot l^4}{384 \cdot E \cdot I_x} \leq 1.75", wcons = \ Construction\ live\ load\ +$

$steel\ beam\ and\ steel\ decking\ dead\ load + concrete\ live\ load$

- Check the deflection performance during occupancy using service values for the loads and the lower bound moment of inertia:

  $\Delta_{50\%LL} = \frac{5 \cdot w_{50\%LL} \cdot l^4}{384 \cdot E \cdot I_x} \leq 1"$

  $\Delta_{DL+50\%LL} = \frac{5 \cdot w_{DL+50\%LL} \cdot l^4}{384 \cdot E \cdot I_x} \ \leq L/240$

- Check the in service capacity of the beam and ensure that $\phi_b M_n > M_u$.
- If any of the above checks fail, select a new beam size and repeat the process.
- Calculate the number of shear studs required for the design using the equation:

  $Shear\ studs\ required = \frac{\sum Q_n}{Q_n}$

- Determine the shear stud spacing with the equation: $Shear\ stud\ spacing = \frac{Beam\ Length}{Number\ of\ studs + 1}$

Most of the challenges in selecting steel shapes for the composite floor system were related to deflection requirements for unshored construction. In order to expedite the design process, a Microsoft Excel spreadsheet was created and is shown in Appendix C-01. As a method of improving constructability, repetitive member sizes were specified for similar bays. The only instance where this was not feasible was in members used in the Lateral Force Resisting System (LFRS). After creating and analyzing a Finite Element Model (FEM) of the braced frames, larger steel shapes were required to resist seismic loads. The steel shapes for members in the braced frames were updated accordingly.

Steel Girder design was carried out using an approach similar to the steel beam design. A sample hand calculation is shown in Appendix B-43, and a corresponding Microsoft Excel calculation is provided in Appendix C-04.

The resulting steel framing plans for the roof and level 1-3 are shown below in Figures 19 and 20 respectively.

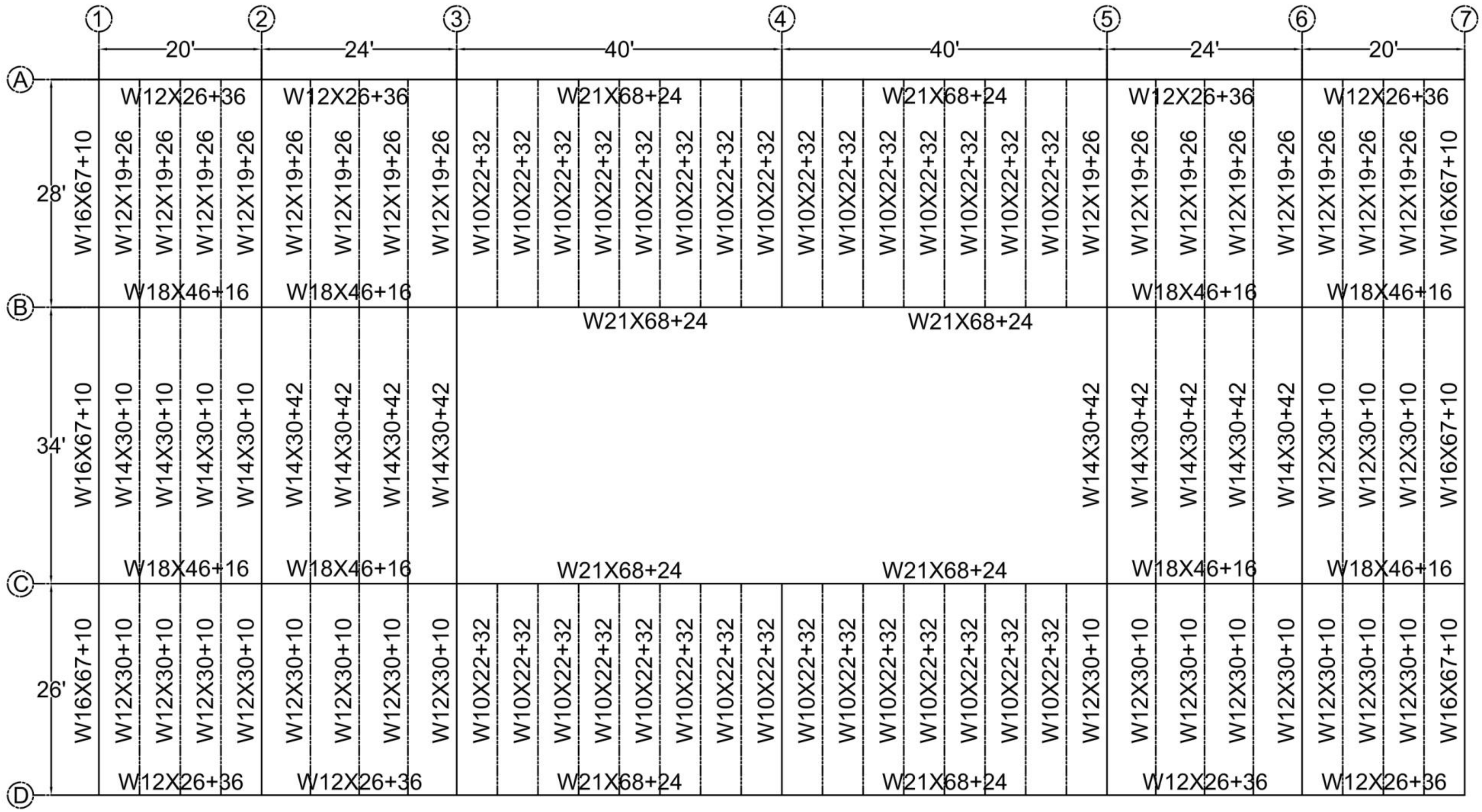

Figure 19: Roof Level Steel Framing Plan

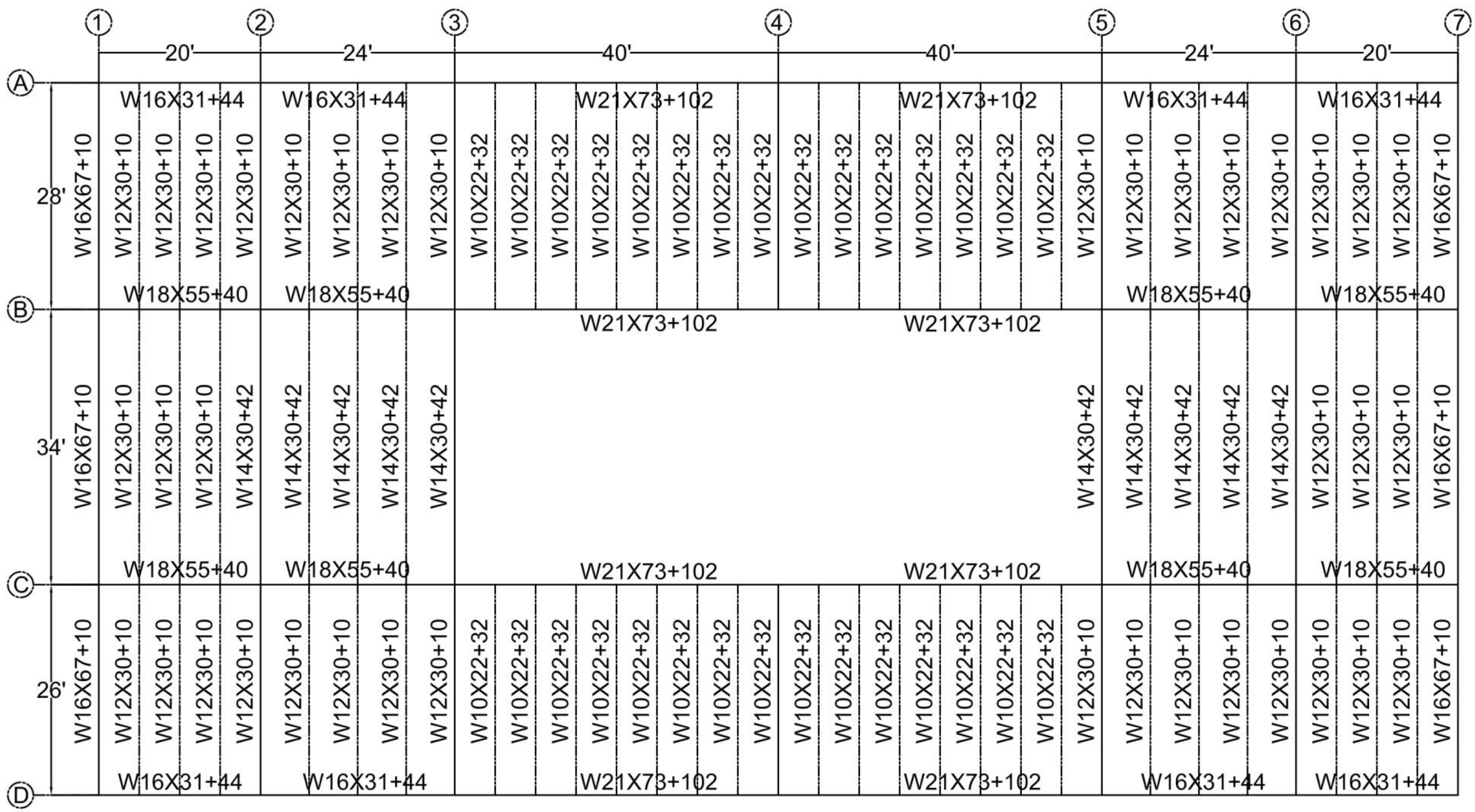

Figure 20: Steel Framing Plan Levels Ground-3

4.3: Steel Column Design

Steel column design was also carried out in accordance with the provisions of the *2010 AISC Specification* and methods provided in *Structural Steel Design* [McCormac (2012)]. Columns with similar tributary areas were designed together as a way to improve constructability. The basic process for column design is outlined below in bullet form. A sample calculation set is also provided in Appendix B-20. In addition, the column schedule which shows all columns and their locations (column marks) is shown below in Figure 21.

Column Design Process:

- Beginning with the top floor, determine the tributary area for each column, and group together columns responsible for similar values of tributary area.
- Determine the design loads on each story level that are associated with each tributary area.
- Investigate the following LRFD Equations
    - $P_u = 1.2D + 1.6L + 0.5S$
    - $P_u = 1.2D + 1.6S + 0.5L$
- Determine the support conditions of the column. Since a braced frame is specified for this project, K=1.0 is an acceptable approach for both the gravity columns and the columns within the frames.
- Use Table 4-1 of the *AISC Manual of Steel Construction* to select a W shape so that $\phi_c P_n > P_u$
- Repeat the process for subsequent floors and account for the additive effect of the column and loads applied to the floors above.

COLUMN SCHEDULE					
FLOOR / COLUMN MARK	1A, 7A, 1B, 7B, 1C, 7C, 1D, 7D	2A, 3A, 5A, 6A, 2D, 3D, 5D, 6D	2B, 3B, 5B, 6B, 2C, 3C, 5C, 6C	4A, 4D	4B, 4C
ROOF					
15'-0" SPLICE	W21X44	W8X35	W8X31	W8X58	W8X35
THIRD 3'-0"					
15'-0" SPLICE	W21X44	W8X35	W8X48	W8X58	W8X58
SECOND 3'-0"					
15'-0" SPLICE	W24X68	W10X54	W10X60	W10X88	W10X68
FIRST 3'-0"					
15'-0" SPLICE	W24X68	W10X54	W10X88	W10X88	W10X88
GROUND 3'-0"					

Figure 21: Steel Column Schedule

4.4: Steel Lateral Force Resisting System Design

After all of the beams, girders, and columns were specified for the structure based on design for gravity loads, the lateral force resisting system was designed using several structural analysis tools. Typical lateral forces considered as part of a structural design include wind and seismic forces.

A seismic and wind force calculator created by Professor Jonathan Ochshorn of Cornell University was used to determine the seismic and wind forces acting on each floor of the structure in accordance with *ASCE 7-10* [Ochshorn (2009)].

In order use the calculator, a number of inputs had to be determined. First, the seismic weight of each floor was determined by summing the total dead load on each floor of the structure. A sample calculation for the seismic weight of each floor is shown in Appendix B-15. The remaining input data for the LFRS is shown below in Table 8. The output from the Seismic and Wind Force Calculator for the North-South braced frame is shown below in Table 9 and also included in Appendix D-01. In addition, a graphical representation of the wind forces acting on each level is provided in Figure 22.

Table 8: LFRS Input Data

Property	Input	Reference
Exposure Class	B	MA 780 CMR
Zone	2	MA 780 CMR
Wind Speed (MPH)	100	MA 780 CMR
S_s	0.24	MA 780 CMR
S_1	0.067	MA 780 CMR
T_L	6	MA 780 CMR
Occupancy Category	3	MA 780 CMR
Site Class	C	MA 780 CMR
Importance Factor	2	MA 780 CMR
K_d	1	MA 780 CMR
K_t	1	MA 780 CMR

Table 9: Seismic and Wind Force Calculator Output

Floor Height Above Grade (ft)	Seismic Weight Per Floor (kips)	Seismic Story Force (kips)	Wind Story Force (kips)	Windward Pressure (psf)	Leeward Pressure (psf)
60	1354.12	64.260	30.953	14.87	-9.29
45	1138.19	39.131	60.403	13.69	-9.29
30	1138.19	24.845	56.980	12.2	-9.29
15	1138.19	11.428	29.179	10.0	-9.29

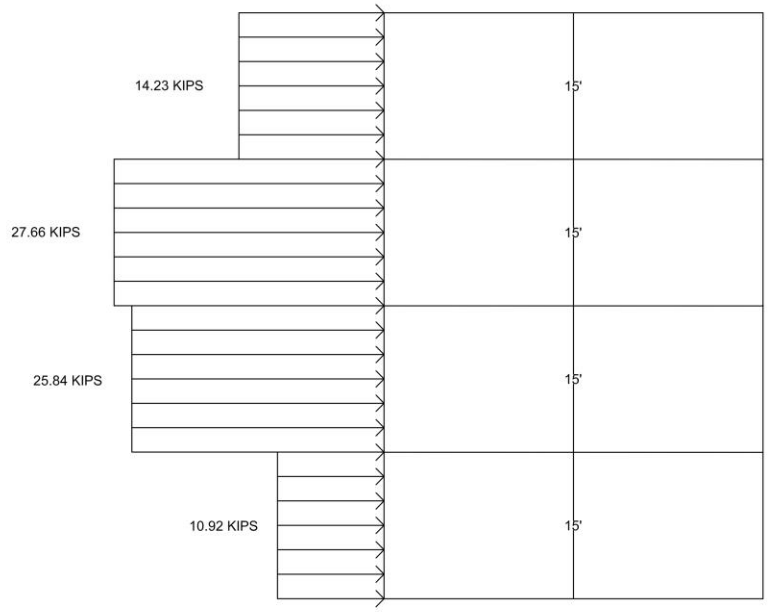

Figure 22: Wind Forces Acting on Each Floor of the Braced Frame

A number of common bracing options were considered for use in this project and are shown in Figure 23 and described in Table 10. Chevron bracing was ultimately selected for its architectural flexibility in terms of where windows and doors can be placed as well as the enhanced ductility it provides.

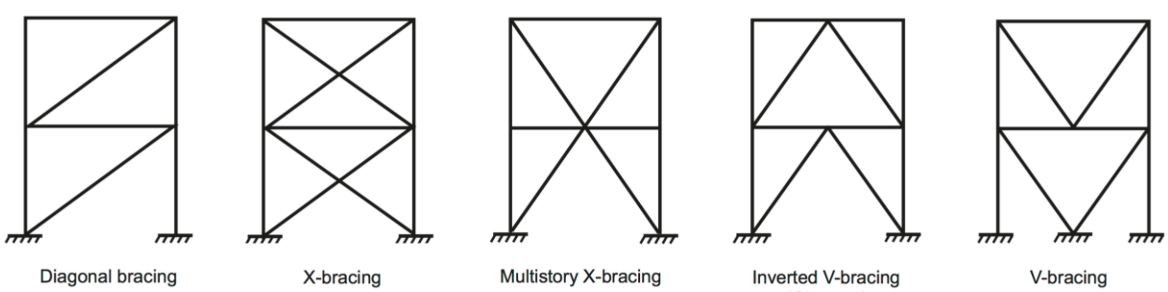

Figure 23: Various Braced Frame Configurations. Taken from: [Hajjar *et.al.* (2013), 5]

Table 10: Common Braced Frame Configurations. Information Sourced from [Hajjar *et.al.* (2013)] and [American Institute of Steel Construction (2014)]

Brace Type	Description
Chevron	• Utilizes intersecting brace connections at beam midspan. • Provides increased architectural flexibility to accommodate windows and doorways.
X-bracing	• Connections located at beam to column joints. • The most common type of bracing. • Commonly used with light bracing on shorter structures. • Effective at transferring story shear to adjacent stories in multistory structures even after fracture and brace buckling.
Eccentric bracing	• Commonly used in seismic regions. • Utilizes intersecting brace connections at beam midspan.
K-bracing	• Utilizes connections at column midspan. • Not permitted in seismic regions.
Knee-bracing	• Remains elastic and stiff during moderate earthquakes.

HSS sections were chosen for the cross braces, and Section 14.2 of the *AISC Seismic Provisions for Structural Steel Buildings* was consulted to determine the minimum size HSS section necessary to provide the required resistance. It should be noted that R=3 was used for the design which greatly simplifies the seismic detailing requirements. Section 14.2 requires "bracing members in K, V, or inverted-V configurations [to] have $KL/r \leq 4\sqrt{E/F_y}$" [AISC (2010), 48]. The lengths of the HSS braces were calculated using the Pythagorean theorem, and the minimum r value required for the HSS section was 3.84 as shown in Appendix B-12. This resulted in the choice of HSS 7x7x1/2 for the braced frames. Column sizes were determined in a slightly different fashion. The column sizes obtained from the gravity system design were input into the braced frame analysis and updated based on the results from the FEM and approximate second order analysis.

An FEM of the braced frame was created using RISA 2D and is shown in Figure 24.

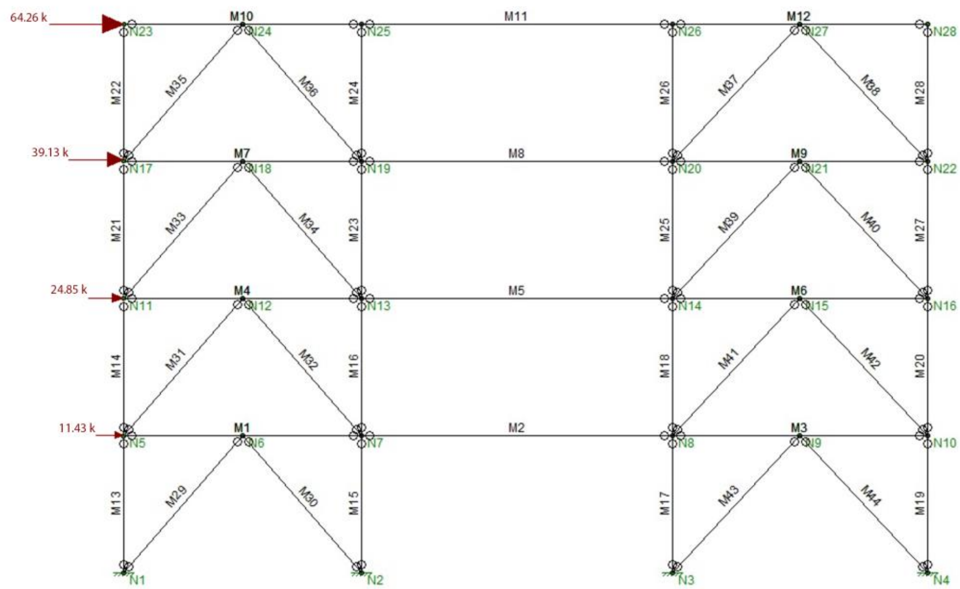

Figure 24: North–South Braced Frame and Seismic Forces at Each Story Level

Since the seismic loading has a far greater impact on the LFRS, wind loading was not considered in the RISA model. The following LRFD load combination equation was investigated: 1.2D + 0.5L +0.2S +1.0E. Two structural analyses were carried out – one analysis for only factored gravity loads (1.2D + 0.5L + 0.2S), and a separate analysis for only the earthquake loads (1.0 E). Results from the RISA model with seismic forces applied to the North-South braced frame including axial force, shear force, and moment diagrams are provided in Figures 25-27.

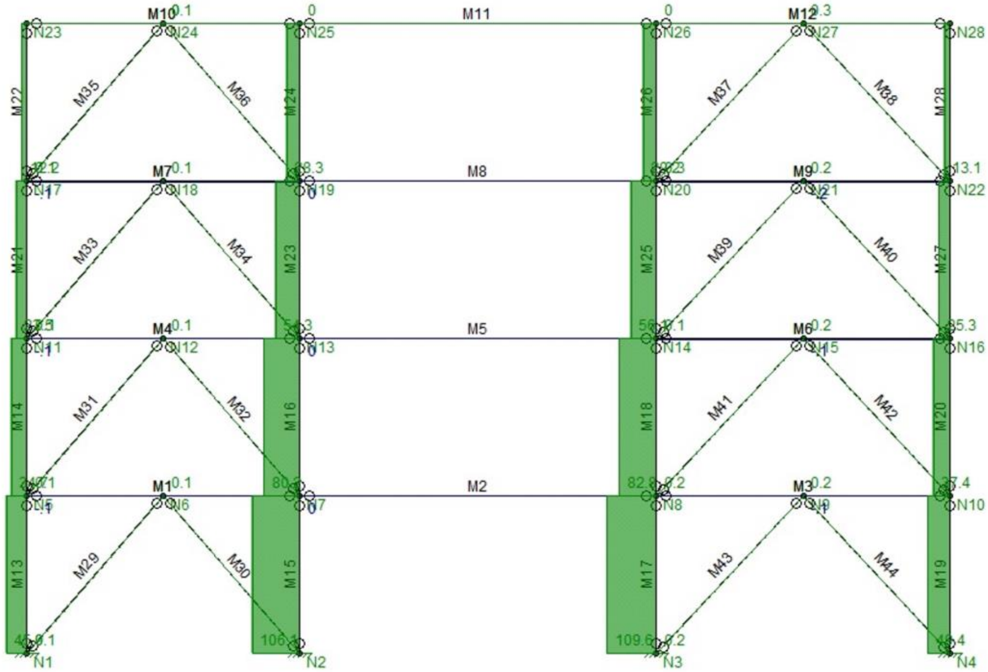

Figure 25: North-South Axial Force Diagram due to Seismic Forces

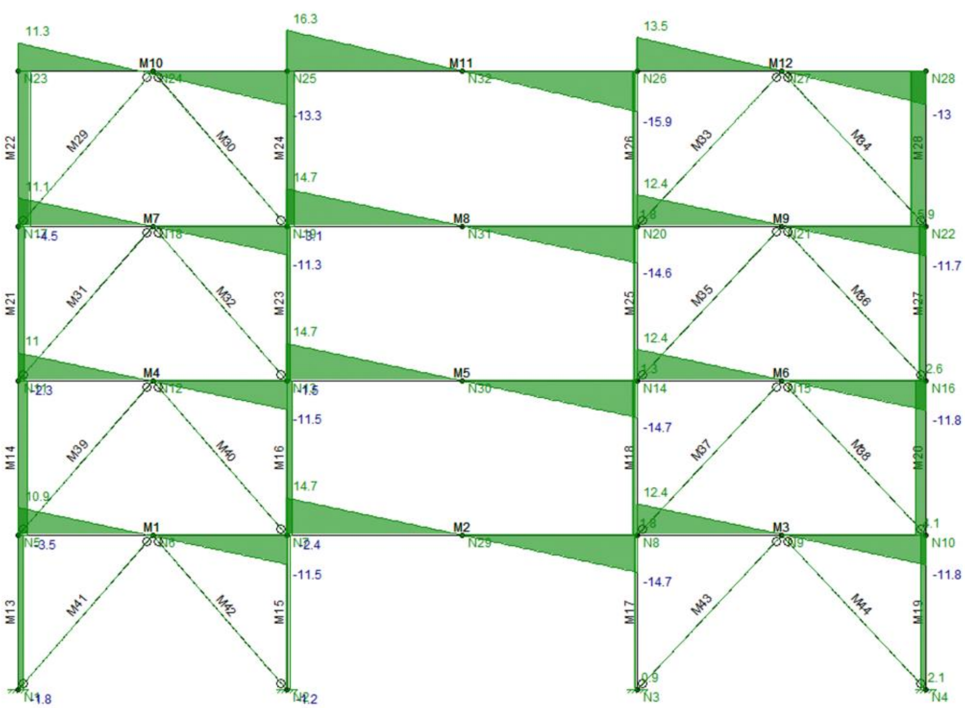

Figure 26: North-South Shear Force Diagram due to Seismic Forces

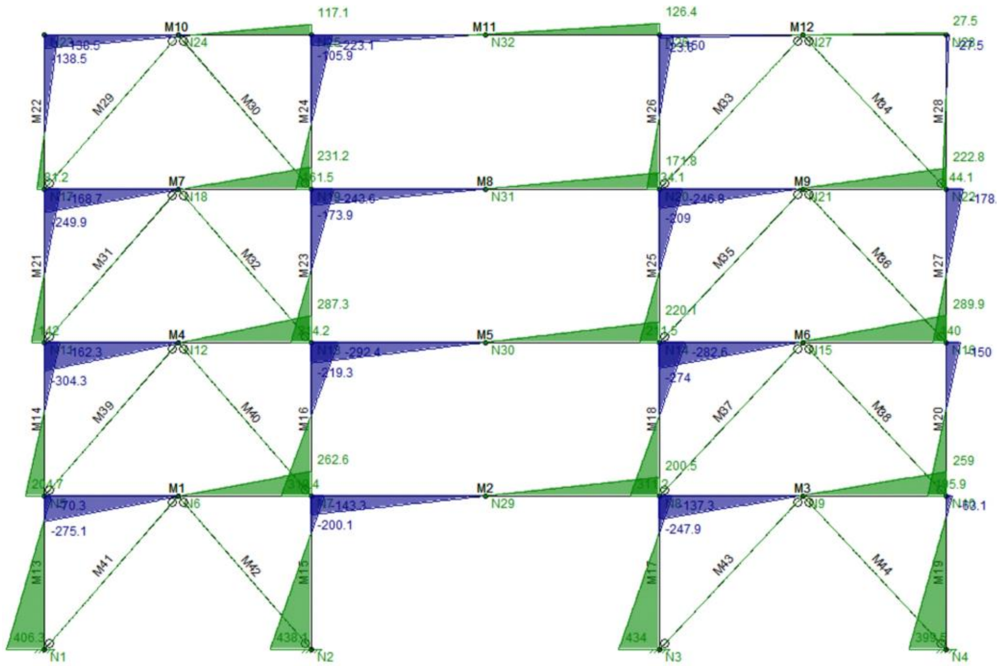

Figure 27: North-South Moment Diagram due to Seismic Forces

The axial force, shear force, and moment diagrams were input to an approximate second-order analysis to check the adequacy of the columns used in the braced frame in accordance with the provisions of Chapters C and H of the *2010 AISC Specification*. The process for this is outlined below in bullet form. Hand calculations for the approximate second-order analysis are shown in Appendix B-01, and a Microsoft Excel spreadsheet used to aid with repetitive calculations is shown in Appendix D-07.

The approximate second-order analysis process following the guidelines of Appendix 8 in the *2010 AISC Specification*:

- Calculate the total elastic critical buckling load for the story using the following equation: $P_{estory} = R_M \cdot \frac{\sum HL}{\Delta H}$ where R_M = 0.85 (conservative) and L=Story height

- Calculate the amplifier B_2 using the following equation: $B_2 = \frac{1}{1-\frac{\propto P_{story}}{P_{estory}}}$ where $\propto = 1\ for\ LRFD$
- Indicate whether the column is in single or reverse curvature
- Calculate C_m using the equation $C_m = 0.6 \pm 0.4\ (M_1/M_2)$ where M_1= smaller factored column end moment due to gravity load (no sway) analysis and M_2= larger factored column end moment due to gravity load (no sway) analysis. Use + for single curvature and – for reverse curvature
- Calculate amplifier B_1 using the equation: $B_1 = \frac{C_m}{1-\frac{\propto P_r}{P_{e1}}} \geq 1$ where $\propto = 1\ for\ LRFD$
- Calculate P_r using the equation Pr = Pnt (for braced frame)
- Calculate M_r using the equation Mr = B1 Mnt + B2 Mlt

The preliminary members for the East-West gravity system were adequate for the LFRS while the approximate second-order analysis indicated that the North-South Members were insufficient for resisting the applied seismic loads. RISA's design module suggested more robust members for the North-South braced frame, and those members were specified in the final design. *AISC* code checks were also performed as an add-on item to verify the RISA calculations, and the updated members passed the code checks.

4.5: Steel Connection Design

Simple column-girder and beam-girder connections were designed in accordance with the *2010 AISC Specification* and methods presented in *Structural Steel Design* [McCormac (2012)]. Design aids provided in Section 9 of the *AISC Manual of Steel Construction* were used to help expedite the simple connection design process. Bolt strength, bolt tearing, angle shear rupture, and angle shear yield limit states were investigated as part of the design process. A sample hand calculation is provided in Appendix B-06, and a Microsoft Excel spreadsheet, shown in Appendix C-07, was created to facilitate repetitive calculations. In addition, a typical column to girder connection is shown below in Figure 28.

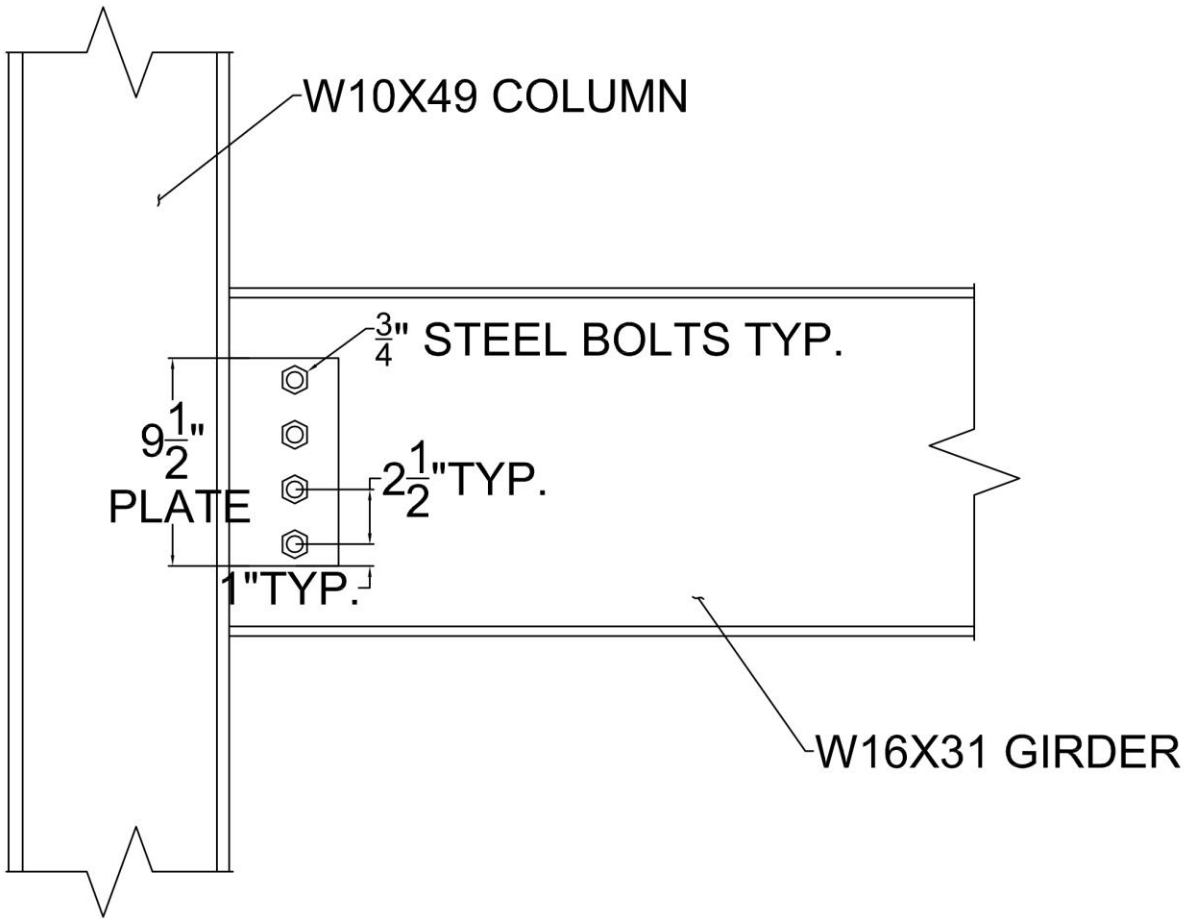

Figure 28: Typical Column to Girder Connection

Chapter 5: Reinforced Concrete Design

This chapter begins with a discussion of reinforced concrete as a building material, and proceeds to describe the salient features of the reinforced concrete design process, and presents the proposed alternative.

5.1: Reinforced Concrete as a Building Material

Reinforced concrete is an alternative structural material considered to support the new library. There are several advantages to using concrete as a structural material. First, the use of local materials in concrete construction saves time since construction can proceed shortly after site excavation for footings. An added benefit of concrete construction is that reinforced concrete does not need to be fireproofed which saves time and money. Concrete also has a very high thermal mass which helps building owners save money on heating and cooling costs.

Disadvantages of concrete construction include the need for formwork, and temperature and weather restrictions on when concrete can be placed.

The column grid for the concrete alternative is shown below in Figure 29 and follows from the architectural layout developed in Chapter 3. Repetitive bay sizes were defined to allow multiple uses of formwork and save construction money and time.

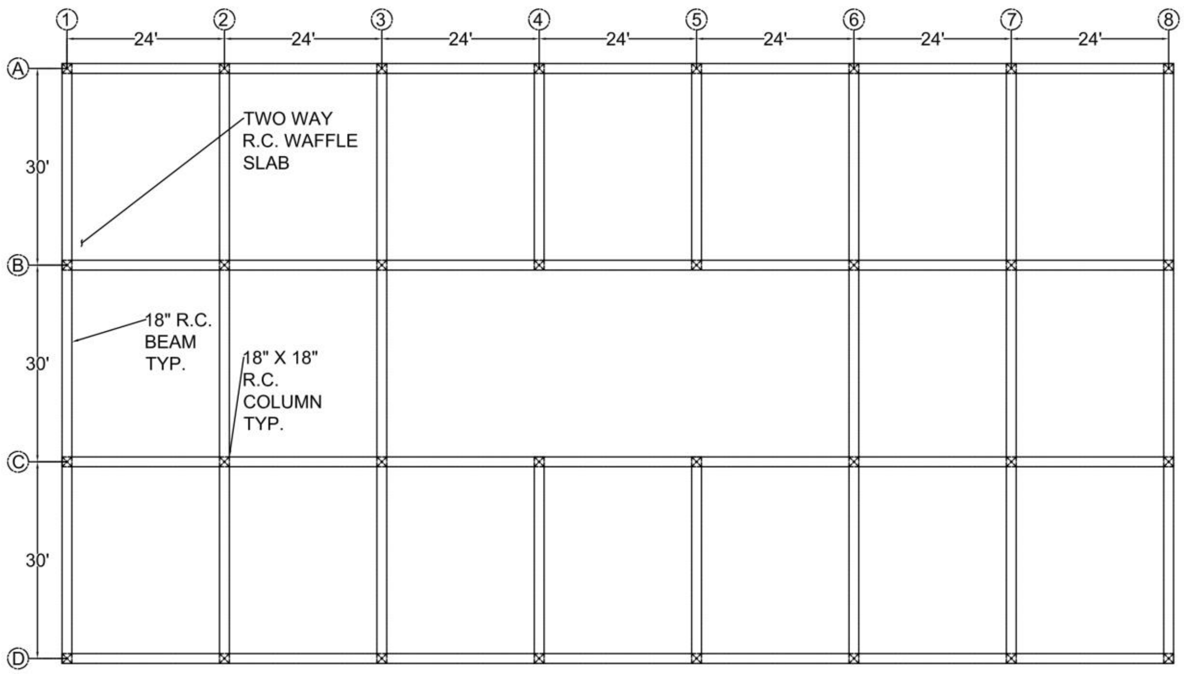

Figure 29: Reinforced Concrete Framing Plan Showing Waffle Slab and Supporting Columns

5.2: Reinforced Concrete Waffle Slab Design

The concrete structure is composed of a two-way waffle slab and reinforced concrete columns. There are numerous types of concrete slabs for varying loading and span lengths. Table 11 provides some of the most common slab types and the typical span lengths for which they are designed. Since the desired bay size for the concrete structure is 30 feet by 24 feet, a two-way waffle slab was considered the most practical design. All concrete members were designed according to the provisions of *ACI 318-11* [American Concrete Institute (2011)] and the methods presented in the textbook *Design of Concrete Structures* [Nilson (2010)].

Table 11: Concrete Slab Types

Slab Type	Typical Span Length (ft)
Flat plate	15-20
Flat slab	13-18
One-way joist	35-50
Two-way joist (waffle slab)	40-50
Two-way slab with beam	20- 30
Banded-beam	35-50

[Portland Cement Association (2005)]

In the preliminary design stage, fire resistance was of key concern in determining the minimum slab thickness. According to Table 601 of *780 CMR*, the floor construction must be designed for a 2-hour fire rating, and from Table 720.1 of *780 CMR*, the minimum design thickness of concrete joists for use in slabs where members are framed into the structure is 11 inches. Furthermore, the minimum concrete insulating material to protect steel reinforcing and tie rods in floor and roof slabs is 1 inch. In addition to the fire resistance requirements, depth of the concrete slab was estimated using the following equations in Chapter 9 of *ACI 318-11* in order to avoid deflection calculations: $L_1/24$ or $L_2/28$ where L_1 is the length of the end bay in inches, and L_2 is the length of a typical interior bay in inches [American Concrete Institute (2011), 127]. Preliminary slab thickness calculations are shown in Appendix B-30. A minimum thickness of 10 inches was used to establish the basis for the geometry of the two-way slab design.

The process for designing the two-way slab is outlined below in bullet form. More detailed hand calculations are presented in Appendix B-52. To help with the iterative design process, a Microsoft Excel spreadsheet was created and is shown in Appendix C-10.

Reinforced Concrete Waffle Slab Design Process:

- Select slab thickness based on span length and fire resistance requirements.
- Select waffle slab dome size.
- Calculate volume displaced by each dome and the total volume of concrete per bay to establish the proper values for dead and live loads.
- Calculate the total moment using the equation $M_u = \frac{W_u \cdot l^2}{8}$. Determine the positive and negative moments by calculating α, the ratio of flexural stiffness of a width of slab bounded laterally by the centerlines of adjacent panels, and refer to an interpolation chart for lateral distribution of slab moments.
- Design ribs for positive and negative bending.
- Check rebar placement and spacing.

30-inch domes were chosen to improve constructability of the system. The reinforcement configuration is shown below in Table 12, and a schematic of the waffle slab is shown in Figure 30.

Table 12: Typical Waffle Slab Reinforcement Configuration

Reinforcement type	Reinforcing Steel
Positive Moment	2#10
Negative Moment	8#11

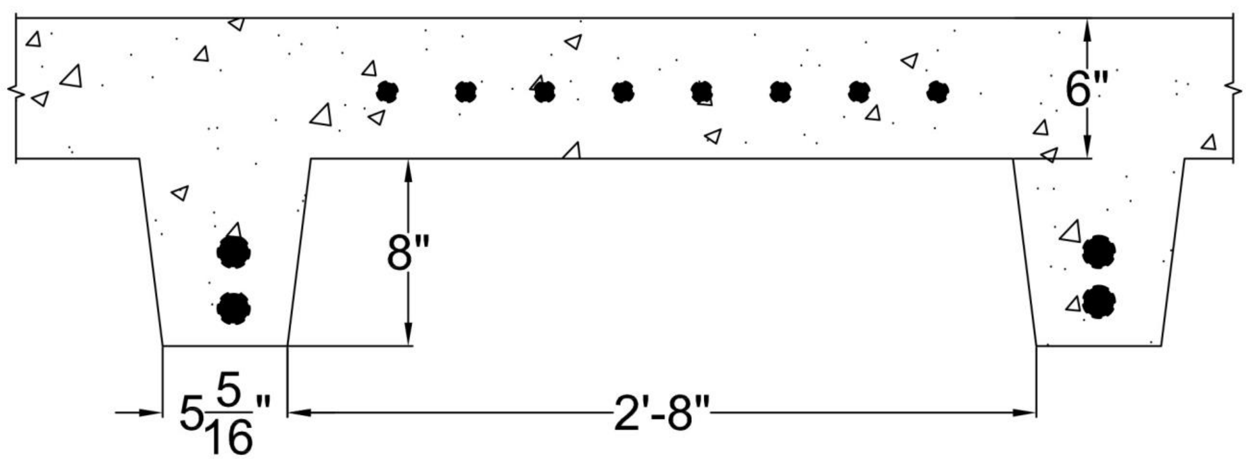

Figure 30: Reinforced Concrete Waffle Slab Schematic

5.3: Reinforced Concrete Column Design

Reinforced concrete column design was carried out in accordance with the provisions of *ACI 318-11* [American Concrete Institute (2011)] and the methods presented in the textbook *Design of Concrete Structures* [Nilson (2010)]. Columns with similar tributary areas were designed together as a way to improve constructability. It should be noted that initial column sizes were established by considering gravity loads only, until a lateral analysis was conducted. A schematic of a typical column cross section and the reinforced concrete column schedule are shown in Figures 31 and 32, respectively. Column marks on re reinforced concrete column schedule refer to the structural plan in Figure 29.

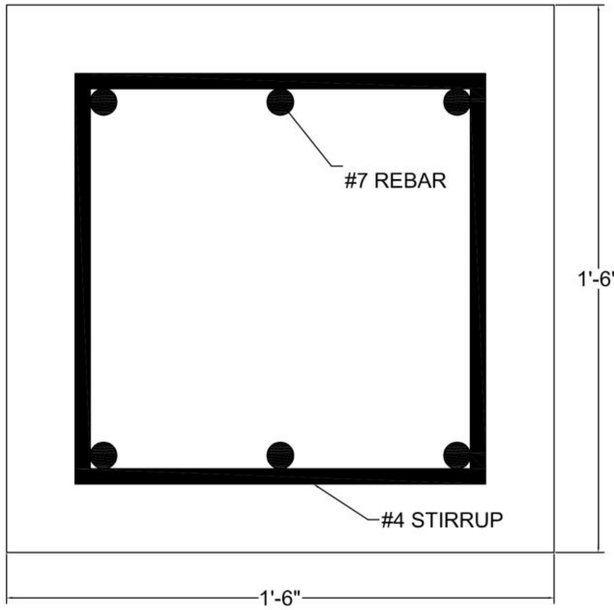

Figure 31: Schematic of Typical Reinforced Concrete Column Section

COLUMN SCHEDULE				
COLUMN MARK / FLOOR / ROOF	3B, 3C, 4B, 4C, 5B, 5C, 6B, 6C 8B	2A, 3A, 4A, 5A, 6A, 7A, 2D, 3D, 4D, 5D, 6D, 7D	2B, 2C, 7B, 7C	1A, 1B, 1C, 1D, 8A, 8B, 8C, 8D
15'-0" SPLICE THIRD 3'-0"	6 #7	8 #6	6 #7	8#6
15'-0" SPLICE SECOND 3'-0"	6 #7	8 #6	6 #7	8 #6
15'-0" SPLICE FIRST 3'-0"	6 #7	8 #6	10 #9	8 #6
15'-0" SPLICE GROUND 3'-0"	6 #7	8 #6	10 #14	6 #10

Figure 32: Reinforced Concrete Column Schedule for Gravity and Seismic Loads

The basic process for column design is outlined below in bullet form.

Column Design Process:

- Beginning with the top floor, determine the tributary area for each column, and group together columns responsible for similar values of tributary area.
- Determine the design loads on each story level that are associated with each tributary area.
- Investigate the following LRFD Equations:
    - $P_u = 1.2D + 1.6L + 0.5S$
    - $P_u = 1.2D + 1.6S + 0.5L$
- Choose steel ties or spirals for reinforcement.
- Determine the gross area of the concrete section based on assumed percentage for As.
- Determined the required reinforcing steel area A_s using the equation: $A_s = \frac{\frac{P_u}{(\alpha \cdot \varphi)} - 0.85 \cdot f_C' \cdot A_g}{F_y}$

  where $\alpha = 0.8 \ and \ \varphi = 0.65$ for ties. Note: *ACI 318-11* requires a minimum steel area of 1% of the gross column area.
- Select the appropriate size and number of steel reinforcing bars to provide the required A_s.
- Calculate the maximum shear (V_{max}).
- Calculate the shear (V_u) at a critical distance of d from the support location.
- Calculate the shear capacity: V_c, φV_c, and $\varphi V_c/2$
- Compare V_u with φ V_c . If $V_u < \varphi V_c$, shear reinforcement is not required.

If shear reinforcement is required, calculate the required size using the following equation: $A_v = 0.75 \cdot \sqrt{f_c'} \cdot \frac{b_w \cdot s}{f_y} \geq 50 \cdot \frac{b_w \cdot s}{f_y}$

Select the shear reinforcement spacing using the equation: $s = \frac{A_v \cdot f_y \cdot d}{\frac{V_{umax}}{\phi} - V_c}$

- Repeat the process for subsequent floors and account for the additive effect of the column loads applied from the floors above.

Ties were chosen to provide reinforcement to the concrete because spiral reinforcement is typically more expensive and specified for locations where seismic activity is of key concern [Weigel (2012), 14]. The cross-sectional dimensions of the columns were originally determined with the desire to create a seamless transition from the columns to the floor slab. The original floor slab was designed as a two way slab with 18 inch beams. This width of 18 inches was kept despite changing the design to a waffle slab in order to use the same column calculations. The minimum column reinforcement is defined by *ACI 318-11* as 1 percent of the gross column area. The use of 6 #7 bars met this requirement.

Most concrete columns required only the minimum reinforcement of 6#7 bars. However, 4 columns required more robust reinforcement due to their location at the atrium slab edge. In addition, a Microsoft Excel spreadsheet was created for the concrete column design and is shown in Appendix C-09.

5.4: Reinforced Concrete Lateral Force Resisting System Design

After the waffle slab and supporting columns were specified for the structure based on design for gravity loads, the lateral force resisting system was designed using several structural analysis tools. Typical lateral forces considered as part of a structural design include wind and seismic forces.

The seismic and wind force calculator created by Professor Jonathan Ochshorn of Cornell University was used to determine the seismic and wind forces acting on each floor of the structure in accordance with *ASCE 7-10* [Ochshorn (2009)].

In order to use the calculator, a number of inputs had to be determined. First, the seismic weight of each floor was determined by summing 20 percent of the roof snow load with the total dead load on each floor of the structure. The remaining input data for the LFRS is the same as the input for the steel design and is shown in Section 4.4, Table 8. The output from the Seismic and Wind Force Calculator for the North-South moment frame is shown below in Table 13.

Table 13: Seismic and Wind Force Calculator Output for the Ordinary Reinforced Concrete Moment Frame

Floor Height Above Grade (ft)	Seismic Weight Per Floor (kips)	Seismic Story Force (kips)	Wind Story Force (kips)	Windward Pressure (psf)	Leeward Pressure (psf)
60	360.0	23.439	14.133	14.87	-6.07
45	360.0	17.235	27.474	13.69	-6.07
30	360.0	11.174	25.671	12.2	-6.07
15	360.0	5.327	23.181	10.0	-6.07

An ordinary reinforced concrete moment frame was selected to provide lateral force resistance for the structural system. A finite element model (FEM) of the moment frame was created using RISA 2D. Since the seismic loading has a far greater impact on the LFRS, wind loading was not considered in the RISA model. The following LRFD load combination equation was investigated: 1.2D + 0.5L +0.2S +1.0E. One structural analyses was carried out for the moment frame.

The column sizes and reinforcing configurations obtained from the gravity system design were input into the moment frame analysis and updated based on the results from the FEM. RISA suggested increasing the amount of reinforcing steel in each column of the moment frame and this change is reflected in the column schedule in Figure 32. Results from the RISA model with seismic forces applied to the North-South moment frame including axial force, shear force, and moment diagrams are provided in Figures 33-36.

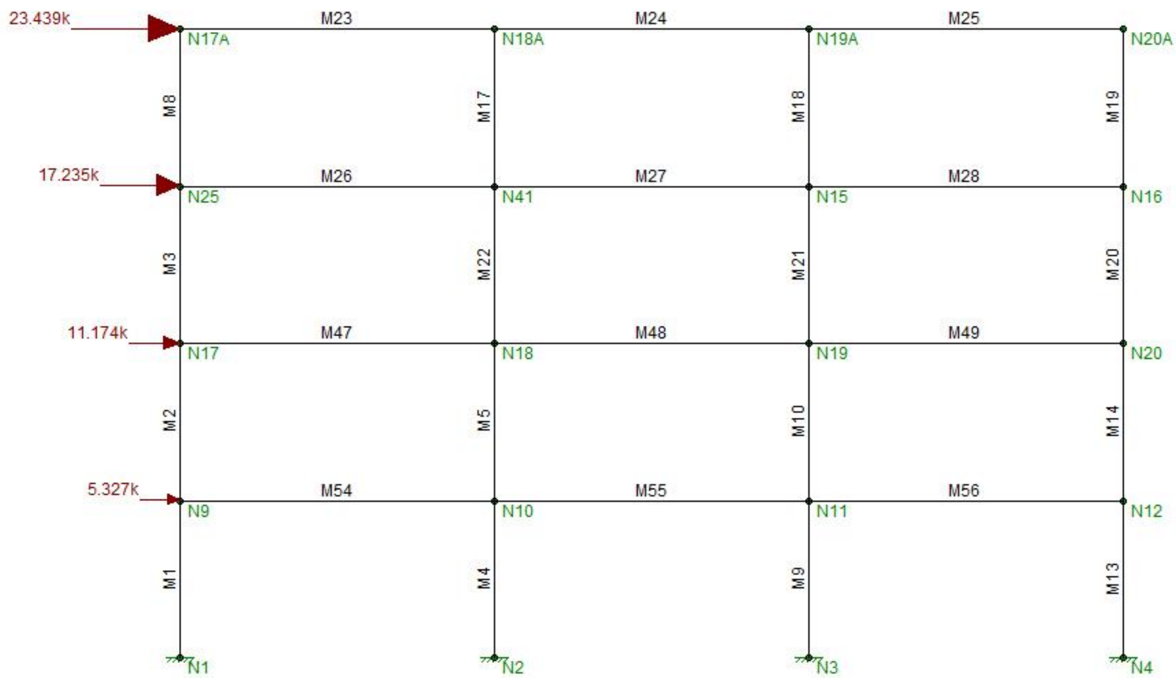

Figure 33: North–South Moment Frame and Seismic Forces at Each Story Level

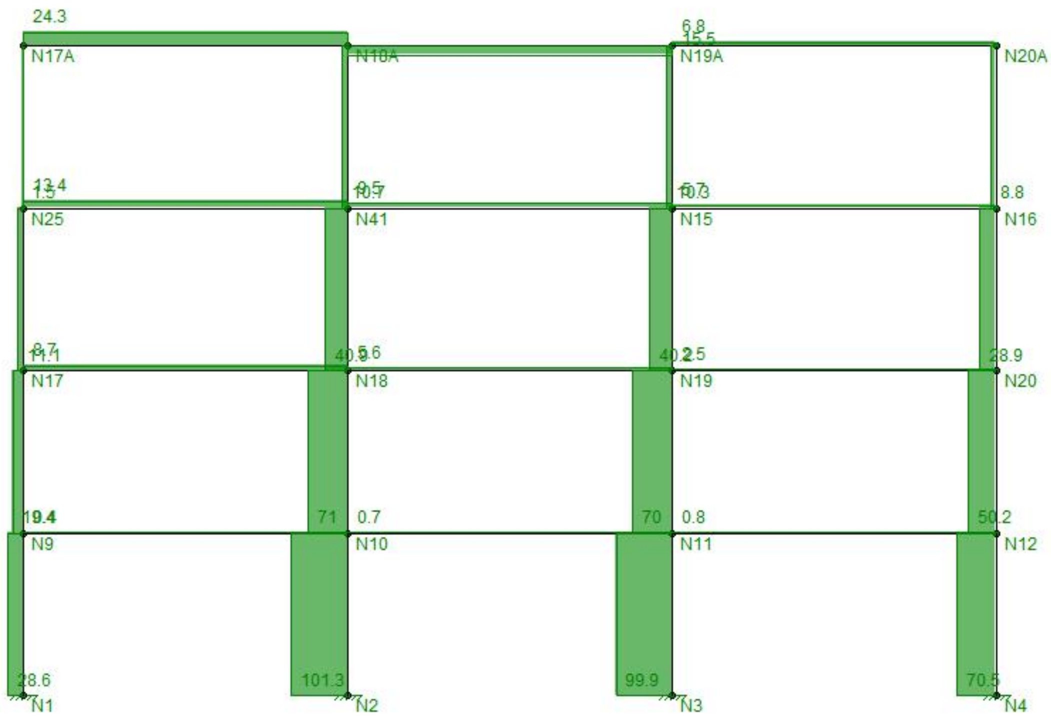

Figure 34: Axial Force Diagram for North-South Moment Frame

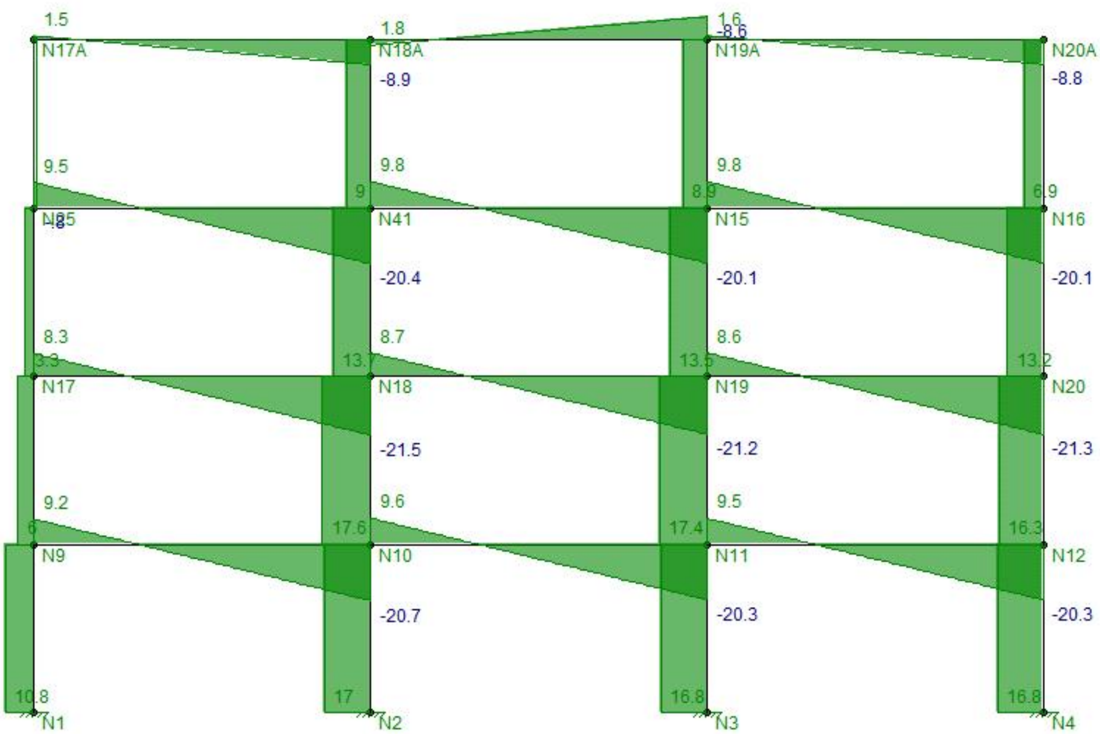

Figure 35: Shear Force Diagram for North–South Moment Frame

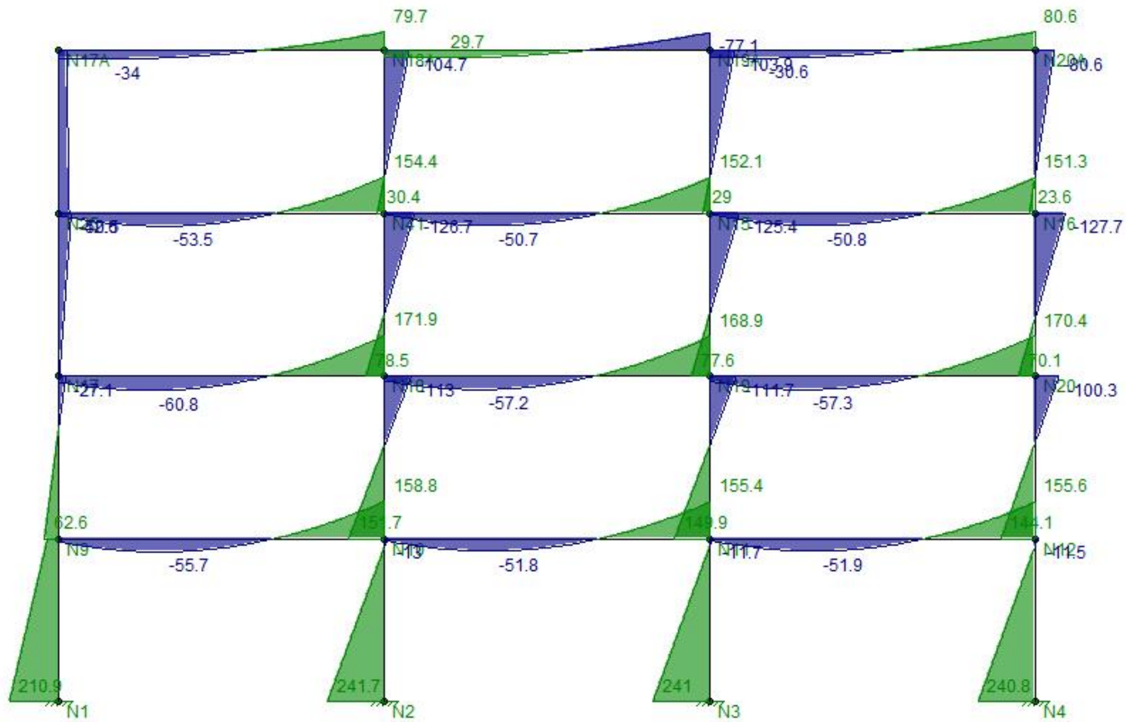

Figure 36: Moment Diagram for North-South Moment Frame

Chapter 6: Foundation Design

Reinforced concrete spread footings were chosen to provide load bearing resistance to the structural systems due to their ease of construction and relatively low cost [Razavi (2016), 145]. In the absence of soil investigation reports, the bearing capacity of the soil was approximated using plans of the existing Gordon Library. The column schedule from the Gordon Library plans tabulated the total column loads at the ground floor level. A Microsoft Excel spreadsheet was developed to allow for input of each column load and footing area from the Gordon Library Plans. The bearing stress of the soil was calculated using the formula: $\sigma_{bearing} = \frac{P}{A_{footing}}$ where P is the column load. Based on the calculations, the maximum bearing capacity of the soil was 8.88 tons/ft^2 which is close to the reported bearing capacity of glacial till soil which is 10 tons/ft^2 [*Massachusetts Building Code* (2010), 92].

Spread footings were designed in accordance with the provisions of *ACI 318-11* [American Concrete Institute (2011)] and methods presented in the textbook *Design of Concrete Structures* [Nilson (2010)]. The process for designing footings is outlined below in bullet form.

Reinforced Concrete Spread Footing Design Process:

- Establish the required footing area using the equation: $A_{required} = \frac{P}{\sigma_{bearing}}$.
- Determine the required footing depth to ensure the footing is below the frost line (typically 4 ft.).
- Determine the pedestal width, c (select a width that can accommodate the column footprint).
- Determine the factored column load P_u for the footing design.

- Calculate the design moment, M_u using the equation $M_u = \frac{P_u \cdot l^2}{2B}$ where B is the width of the footing and l is the distance from the footing edge to the position of the steel reinforcing.
- Calculate the required steel area using the equation $A_s = \frac{f_c' \cdot b}{1.176 \cdot F_y} \cdot d - \sqrt{d^2 - \frac{2.353 M_u}{\varphi \cdot f_c' \cdot b}}$
- Calculate the required bar spacing using the equation: $Spacing = \frac{B}{\#\, of\, rebar - 1}$
- Calculate the development length using the equation: $l_d = (\frac{3}{40} \cdot \frac{F_y}{\lambda \sqrt{f_c'}} \cdot \frac{\psi_e \cdot \psi_t \cdot \psi_s}{\frac{C_b + k_{tr}}{d_b}}) \cdot d_b$

The thickness of the spread footings was established with the goal of resisting shear forces, and the reinforcing steel was designed to resist the anticipated bending forces. The spread footings were designed to be placed four feet below grade which is below the frost line. Four footings were designed to resist varying loads across the building footprint. Table 14 shows the footing designs and the columns those footings support. In addition, Figure 37 shows an example spread footing detail and Figure 38 shows a plan view of the spread footings.

Table 14: Footing Schedule Using Concrete Alternative Loads

Columns	Pu (kips)	The footing size (ft×ft)	Rebar sizing
1A, 8A, 1D, 8D	456.56	5.5×5.5	3 # 6 both directions spaced 33" apart
3B, 3C, 6B, 6C	1062.52	8.5×8.5	5 # 9 both directions spaced 25" apart
2A, 3A, 4A, 5A, 6A, 7A, 2D, 3D, 4D, 5D, 6D, 7D, 1B, 1C, 8B, 8C, 4B, 5B, 4C, 5C	766.56	7×7	5 # 7 both directions spaced 20" apart
2B, 2C, 7B, 7C	1371.32	9.5×9.5	6 # 10 in both directions spaced 22" apart

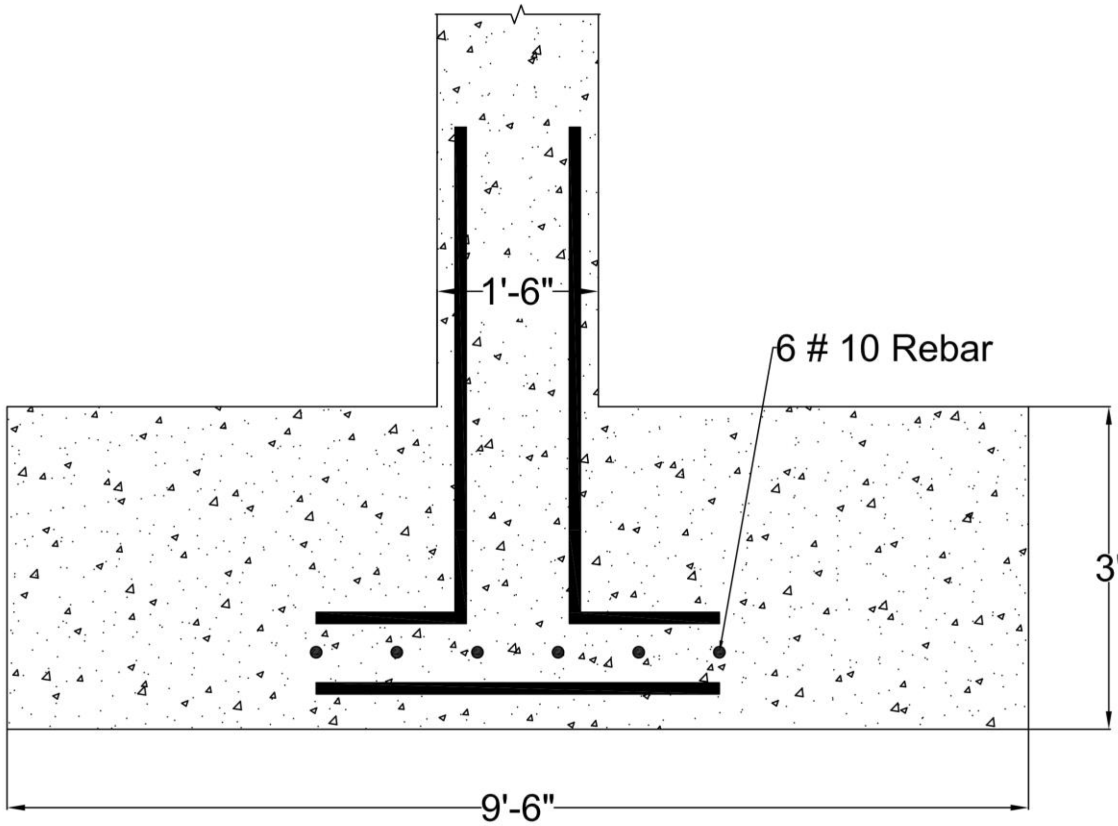

Figure 37: Example Spread Footing Detail

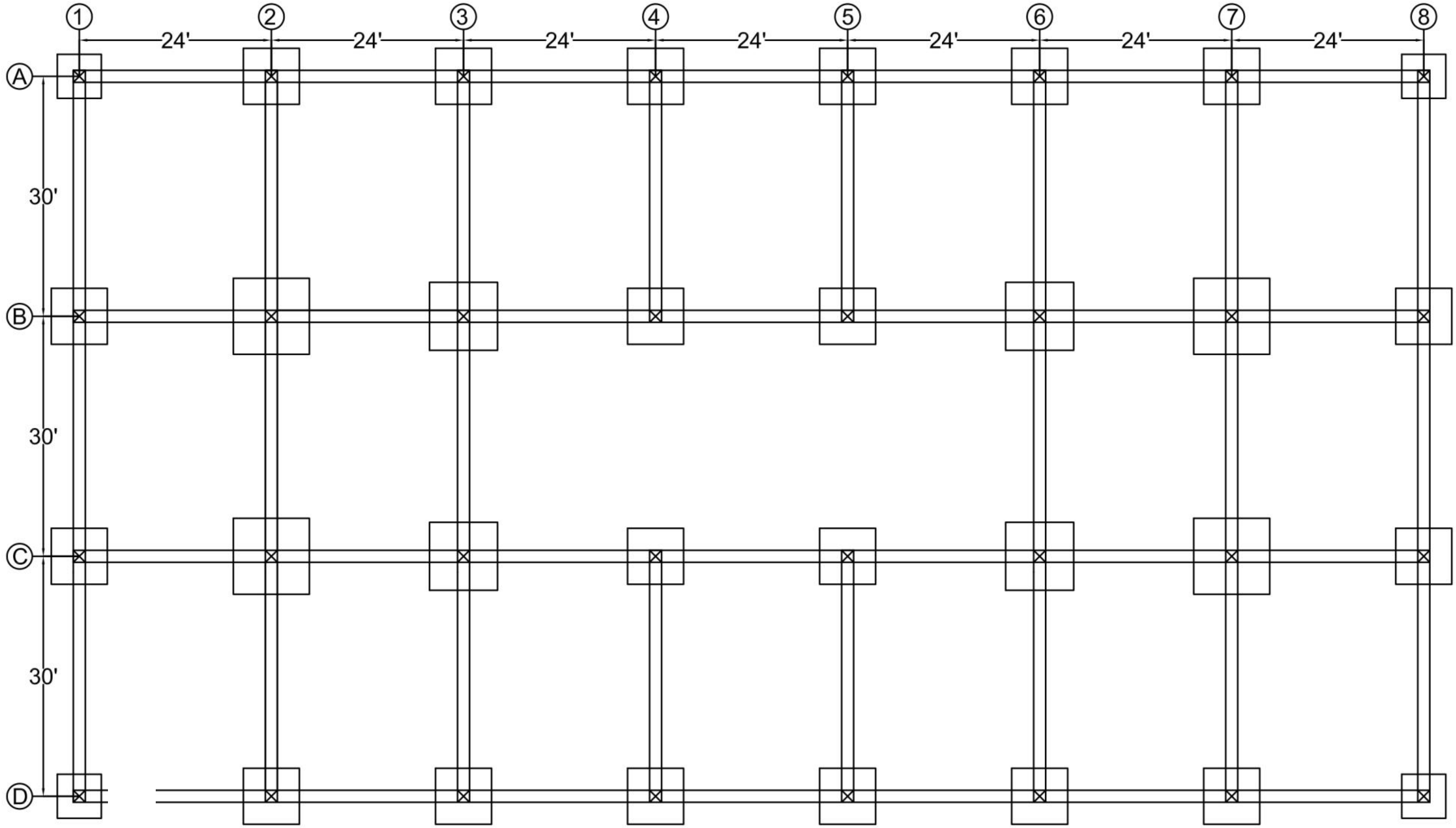

Figure 38: Plan View of Reinforced Concrete Footings

Chapter 7: Cost Estimates and Other Evaluations of the Alternatives

This chapter describes the methods used to determine the costs of the structural alternatives and presents results from the cost estimate.

The cost of the structural alternatives was determined using *RS Means* construction cost data. The *2015 Building Construction Cost Data Book* provides costs for individual building components. This reference was used to calculate the total cost of the structural steel and reinforced concrete for each design alternative. In order to calculate the cost of the structural steel alternative, a Microsoft Excel Spreadsheet was created, and each steel member was entered into the spreadsheet. The cost of each structural member per linear foot was obtained from the *2015 Building Construction Cost Data Book* and multiplied by its total length from the design to establish the cost of the structural frame. Other items required for the steel construction include allowances for the concrete decking and shear studs.

The cost of the reinforced concrete alternative was calculated by multiplying the total volume of all the concrete members by the unit cost of reinforced concrete in dollars per cubic yard. The unit cost for reinforced concrete include an allowance for reinforcing steel and included material, placement, labor, and finishing.

In addition to the structural costs, the completed building will include electrical, mechanical, and a number of other non-structural elements. To price these items, the *2009 Square Foot Costs Book* was used. A building model similar to the proposed library structure was examined, and the cost breakdown for the various systems was used to obtain the corresponding costs for this proposal. The 2009 costs were adjusted for the location of Worcester, Massachusetts and for a 2015 construction start date. Table 15 follows the Uniformat presented in RS Means and shows the breakdown of costs for the structural steel and reinforced concrete alternatives.

Table 15: Construction Costs for the Structural Alternatives

A. Substructure	Cost	Unit	Subtotal	Year Adjustment	Location Adjustment
Standard foundations	4.09	$/S.F.	245,400.00	378,726.19	405,237.03
Slab on grade	1.64	$/S.F.	98,400.00	151,860.87	162,491.13
B. Shell					
Steel Structure	45.88	$/L.F. for each member	5,118,436.75	NA	5,476,727.32
R.C. Structure	113	$/yd^3	4,986,892.94	NA	5,335,975.45
C. Interiors					
Roof covering	2.11	$/S.F.	126,600.00	195,381.97	209,058.71
Doors, Fittings & Partitions	10.15	$/S.F.	609,000.00	939,870.63	1,005,661.57
Stair construction	5.53	$/S.F.	331,800.00	512,067.45	547,912.17
Ceiling, Floor & wall Finishes	15.07	$/S.F.	904,200.00	1,395,453.24	1,493,134.97
D. Services					
Elevators (2)	69,800.00	$/n	139,600.00	215,444.89	230,526.04
Plumbing & Water	3.24	$/S.F.	194,400.00	300,017.82	321,019.06
Rain water drainage	0.51	$/S.F.	30,600.00	47,225.03	50,530.78
Active Fire Protection	19.18	$/S.F.	1,150,800.00	1,776,031.40	1,900,353.59
Electrical & Lighting	15.97	$/S.F.	958,200.00	1,478,791.52	1,582,306.93
Communications	6.42	$/S.F.	385,200.00	594,479.75	636,093.33
E. Equipment & Furnishings	NA	NA			
F. Special Construction	NA	NA			
G. Building Sitework	NA	NA			
Results					
	Reinforced Concrete	**Steel**		Reinforced Concrete Subtotal	13,880,300.75
8% Architect Fee	1,110,424.06	1,121,684.21		Steel Subtotal	14,021,052.63
25% General Contractor Fee	3,470,075.19	3,505,263.16			
Total ($)	18,460,800.00	18,648,000.0 0			
Total ($/SF)	307.68	310.80			

A pie chart showing the cost breakdown for the various components of the steel alternative is provided in Figure 39 as a visual aid for the reader. The shell and building services are by far the most cost intensive components of the project. In addition, a cost breakdown of the shell elements is provided in Tables 16 and 17.

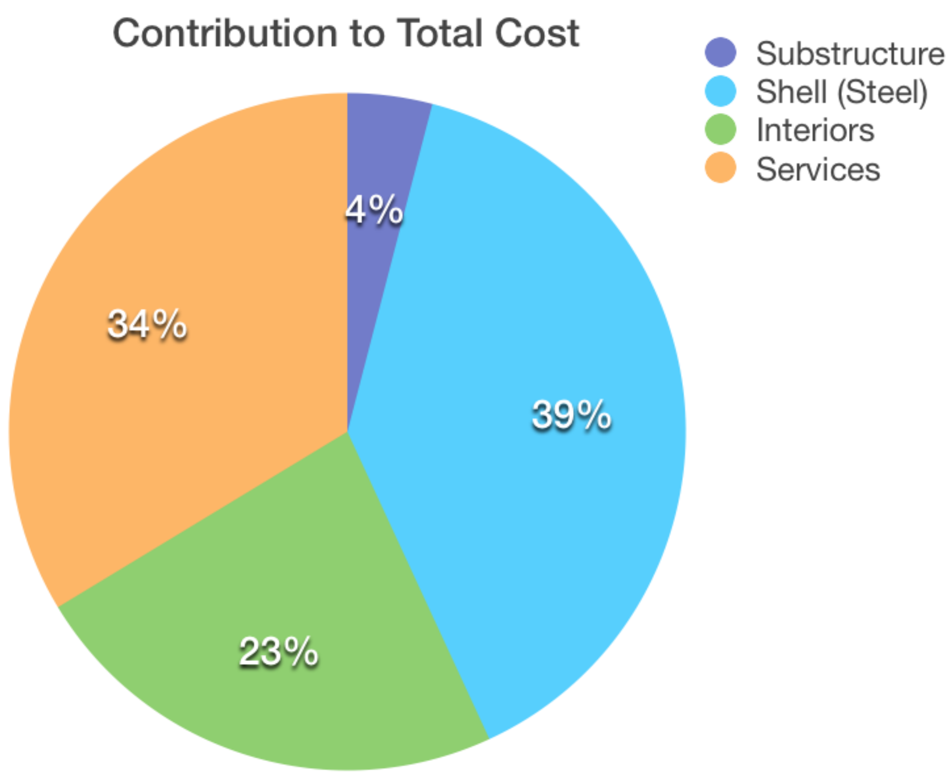

Figure 39: Pie Chart Showing Contribution of Components to the total Cost for the Steel Alternative

Table 16: Shell Cost Breakdown for Steel Alternative Based on a Takeoff

Component	Cost	Percent
Steel Beams & Girders	353,441.44	12.8
Columns	91,834.80	3.3
Studs	22,417.92	0.8
Fireproofing	130,782.49	4.8
Concrete Slab	862,799.41	31.3
Curtain Wall	1,091,813.94	39.7
Brick Masonry Wall	199,710.00	7.3
Cost Per S.F.	45.88	

Table 17: Shell Cost Breakdown for Reinforced Concrete Based on a Takeoff

Component	Cost	Percent
Waffle Slab	710,137.47	26.6
Roof and Grade Slab	391,887.81	14.7
Columns	17,476.44	0.7
Formwork	101,834.28	3.8
Curtain Wall	1,244,754.00	46.7
Brick Masonry Wall	199,710.00	7.5
Cost Per S.F.	44.43	

The cost of the new design was also compared with three library projects currently underway across the United States. This cost comparison is provided below in Table 18.

Table 18: Cost Comparison of Current Library Drawings with New Design

	New Central Library, Austin TX	Metropolitan Library, Columbus OH	City Library, Wichita KS	New Design
Area (SF)	198,000	19,000	95,000	60,000
Cost ($)	90 million	9.8 million	33 million	18.5 million
Cost ($/SF)	454.55	515.79	347.37	307.68
Start Date	Spring 2013	Spring 2013	Spring 2016	Spring 2016
Source	[APLFF (2013)]	[Seman, G. (2016)]	[Ryan, K. (2016)]	Engineering Calculations

Chapter 8: Summary of Findings and Recommendations

This chapter presents a summary of the project work and provides recommendations for implementing and approving upon the design.

8.1: Key Findings

The information age and the prevalence of electronic resources has created a paradigm shift in the way students and educators think about and utilize academic libraries. Some have projected that by 2020, libraries may no longer have circulation desks [Kurt (2012)] while others have put the entire existence of library facilities into question. In order to prevent the end of academic library facilities as we know them, aggressive action must be taken to give library facilities new meaning [Gayton (2008), 60].

This project examined the major ways in which architects and librarians have reshaped the meaning of the academic library as a place where students come to seriously engage academic resources, create new knowledge, and collaborate. The Gordon library at WPI was benchmarked against the criteria developed from the investigation into new academic library design trends, and two structural alternatives in reinforced concrete and structural steel were prepared for this project.

A broad range of innovative architectural features including a skylight, a four-story atrium, and floor-to-ceiling curtain wall were incorporated into the design to maximize daylighting, conserve resources, reduce costs, and improve occupant comfort. The structural alternatives were designed to accommodate the above architectural features, and a cost analysis of the alternatives was performed using *RS Means* construction cost data. The cost analysis includes the cost of the structures, the curtain wall, brick masonry wall, interiors, and building services.

The cost comparison of the new design with three library buildings currently under construction across the United States reveals that the cost estimate for the new design is slightly below average. The average of the square foot costs of the new libraries presented in Table 18 is $439.24 per square foot while the cost for the new design is $307.68 per square foot. This discrepancy could be due to the omission of furniture and electronics costs in the new design cost estimate as well as deviations of actual costs from cost data provided in RS Means.

While the structural steel alternative is certainly a competitive option, the reinforced concrete alternative was chosen for a number of key reasons listed below:

- The reinforced concrete alternative has the lowest cost.
- The reinforced concrete has significant scheduling advantages because steel construction requires significant lead time for procurement.
- The reinforced concrete design is the most constructible alternative due to the repetition of formwork and standard sizes which is highly desirable for the earlier construction start dates it provides.

8.2 Recommendations

The result of this project work is a truly unique space that promotes a productive and comfortable study environment and upholds the relevance of academic libraries. In order for this design to be successfully implemented, a number of challenges will have to be overcome. Raising capital for this project will be a substantial challenge. While residential buildings have a revenue stream associated with room and board charges, other academic buildings must be financed using alternative sources of funding. Another challenge associated with this project is the physical location of the existing building. Since the Gordon Library is built into a large hill on an academic campus, intense construction methods and planning procedures need to be taken in order to minimize disruption of the campus community.

Development of this project could proceed in a number of ways. The architectural layout could be further refined by more accurately approximating the occupant load of the building. This would allow for restrooms and other rooms to be more accurately sized according to the number of users that will occupy these spaces.

Alternative strategies for developing and evaluating the structural alternatives include performing a cost-benefit analysis of each design component in order to create the most cost effective column layout, cladding system, and overall building design.

Investigating the fire safety concerns involved with the four story atrium is also an area of work that could be pursued further. The large open space in the center of the building allows for fire, smoke, heat, and toxic gasses to spread rapidly from floor to floor [Spadafora (2012)]. As a result of this challenge, smoke management and fire suppression systems should be designed to reduce the risk of smoke inhalation and stop the spread of fire throughout the building. A material loss prevention plan should also be developed to protect references in hard copy against losses from fire or other disasters.

The creation of a construction plan that focuses on advancing sustainability, promoting safety, employing the latest construction technologies, ensuring quality, and tightening schedules is also a top priority.

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Gordon Library Redesign Proposal

A Major Qualifying Project proposal to the faculty of Worcester Polytechnic Institute in partial fulfillment of the requirements for the Degree of Bachelor of Science

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Submitted to:

Professor Leonard Albano

October 15th 2015

Abstract

Over the course of this project, we plan to reveal the physical and architectural aspects of academic library design that facilitate a more student-centered pedagogy in order to uphold the relevance of library facilities in the twenty-first century.

The Gordon Library at Worcester Polytechnic Institute will be benchmarked as a case study facility and two structural alternatives will be developed in response to our benchmarking activity. Results of our work will include a finite element analysis of a typical bay in the Gordon Library, framing plans and cost estimates for the alternative designs.

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Chapter 1: Introduction

The purpose of this project is to research aspects of the physical form and architectural quality of library facilities that establish the library as a place for student-centered learning and balance library users' multiplicity of needs.

A list of evaluation criteria was developed as a result of our research and the Gordon Library at Worcester Polytechnic Institute (WPI) was benchmarked against these criteria to reveal the functionality limitations with the existing building. Results from the benchmarking activity helped the project team identify an alternative layout and building design that may be better suited to meet the needs of twenty-first century students.

1.1 Problem Statement

As times change, libraries must adapt to host new types of media and activities necessary to meet the changing size, work habits, and needs of university communities. As such, the level of thought given to library layouts and their compatibility with structural systems should be commensurate with the importance of libraries, or they risk becoming obsolete. Through the course of our research, we have found that the needs and work habits of the WPI community have changed significantly since the Gordon Library was constructed in 1967. These changes are significant enough to explore the use of alternative layouts and structural systems that may be better suited to meet the desires of the current WPI community.

1.2 Scope of Work

Our project team proposes to use the Gordon Library as a case study for evaluating the performance of academic libraries constructed in an era separated from the present not only by time but great advances in building and information technology. To attain this goal, we have established five objectives:

1. Research changing resource types, study habits, desired library services, and amenities.
2. Benchmark the Gordon Library using criteria developed from Objective 1 and identify ways to reduce the demarcation between the interior and exterior environment, improve lighting, group study spaces, and aesthetics.
3. Investigate new layouts and structural configurations in response to our research and benchmarking activity.
4. Develop the structural alternatives by performing engineering calculations to specify the configuration, quantity, and material properties of the structural members that will support the proposed layout.
5. Perform a cost analysis of the structural alternatives in order to perform a comparison between them.

1.3 Report Outline

The following chapters of this report provide background information needed to understand the salient features of our work and sections covering architecture and layout design, structural steel design, reinforced concrete design, and cost analysis. Finally, the report concludes with a summary of results and conclusions.

Chapter 2: Background

In this chapter, a discussion of the background information necessary to understand the underlying historical, social, and technological concepts of our work is presented. In order to demonstrate why a redesign of the library may be appropriate, it is necessary to situate the reader in the era in which the present structure was designed. Such a process will reveal the social and technological conventions that informed the current structure's design. A discussion of emerging technologies and the changing role of the library will follow to demonstrate how a new design can better meet the needs of twenty-first century students.

2.1: The Gordon Library at Worcester Polytechnic Institute

WPI has a long history of growth and has enjoyed a distinctive record of achievement in the sciences and engineering. By 1963, a pivotal year in the university's history, enrollment had reached 1,142 undergraduates, an increase of 44 percent in the last seven years [Worcester Polytechnic Institute (1963), 1]. Meanwhile, the launch of Sputnik in 1957 and the intensification of the Cold War arms race created a significant impetus to improve science and engineering education across the United States.

As one of the premier technical universities on the East Coast, WPI was looking to further increase enrollment and continue to produce engineers of the highest caliber during this period. However, in order to produce a quality engineering curriculum at the graduate and undergraduate level, WPI needed to provide students with access to science and technology information.

At the time, the university lacked a centralized library. A general library located in Boynton Hall contained a wide variety of volumes in literature, economics, history, and art

[Coombs (N.D.), 2]. The remaining academic resources were dispersed amongst the university; each academic department had its own library.

With a desire to expand its collection of books, centralize its resources, provide students with a quiet study environment, and expand into emerging audio-visual and microfilm technologies, the university sought to construct a new library facility.

Constructing a new library was a bold endeavor and required significant capital investment. Fortunately, George C. Gordon, a distinguished alumnus who graduated in 1895, left a bequest of $5,000,000 to the university. [Worcester Polytechnic Institute (1967), 1]. This donation enabled WPI to commission the design and construction of a modern library facility with a capacity for 600 students and 200,000 volumes. The interior design included individual reading tables for concentration, group study rooms, smoking rooms, music rooms, and lounges on each floor. The library cost $2,053,133 [Worcester Polytechnic Institute (1967), 1] and was officially dedicated on October 28, 1967.

Today, the Gordon Library holds over 270,000 volumes of books, more than 4,000 volumes of archival materials and rare books, and provides students access to more than 70,000 electronic journals, books, and databases. The facility has undergone several renovations over the years and now contains computer labs and a library café.

The building is a four-story, reinforced concrete structure with a brick and precast concrete panel facade; a rendering of the architect’s design is shown below in Figure 1. WPI engaged O.E. Nault & Sons of Worcester, Massachusetts as the architect while Harvey and Tracey Consulting Engineers served as the structural engineer of record. The structural system is comprised of two-way waffle slabs on each floor, which transmit gravity loads to concrete columns that vary in size and reinforcement patterns along the building's elevation.

The current interior layout, although modified to accommodate increased technology use and group work, is still influenced significantly by the twentieth century specifications from which the building was tailored. Smoking rooms, music rooms, and the need to store information in the printed medium dominated the building's original design. Aesthetically, the Gordon Library resembles more of a bunker than a library and exudes an unwelcoming and cold feeling as a result.

The library has one entrance from campus to the third floor of the building. This entry floor currently features a large open space for computer use and group work along with conference rooms equipped with computers and flat screen TVs called "tech suites" as well as a cafe for students and faculty. Above the main floor is additional flex space for group work, tech suites, a lounge containing newspapers and periodicals, quiet study areas, and book stacks. The second floor of the library is primarily comprised of additional quiet study areas, tech suites, and book stacks. Finally, the ground floor of the library contains a much smaller assortment of compact shelving, group study areas, and the recently renovated university archives and special collections department.

Figure 1: Gordon Library Rendering

2.2: The Future of Libraries

There have been remarkable advances in knowledge sharing and research methods since the 1960s. Today, information is more accessible because of the emergence of the Internet and the prevalence of smartphones and tablet devices. The Internet not only reflects a change in the way researchers access information but also poses a significant challenge to libraries, which must continue to be relevant in an age when information is so readily accessible. Not surprisingly, the proliferation of technology is having tangible effects on university libraries across the country – there has been a sharp decline in the circulation of print sources, a reduction in use of reference services, and falling gate counts [Gayton (2008), 60].

At the same time that advances in technology are threatening the existence of libraries as physical spaces, the traditional notion that libraries are "communal" spaces strictly to support quiet studious activities is also being called into question. One of the driving forces behind this reimagining of the library is a major shift in thinking about learning at the undergraduate level. The classical learning model is one-size fits all. It assumes that students learn best from a teacher and develop and internalize that knowledge independently, in a highly structured environment. We now embrace learning as a highly individualized and complex process that depends on the cognitive abilities and learning styles of each student.

While some students thrive in an environment where information is presented by a professor and studied in a quiet, focused environment, other students enjoy informal learning – they learn from friends, Khan Academy, Youtube videos, and other non-traditional methods. Learning also occurs in different environments – some students learn best in noisy environments like cafes, some learn outside, and others prefer communal environments such as the traditional library [Matthews and Walton (2013), 145].

The type of work students are assigned is also changing. Collaborative group work is playing a much bigger role in undergraduate curricula, particularly in response to the need to develop team players capable of working in a fast-paced, global economy.

In short, there has been a paradigm shift in the way we think about learning, and while the communal model still has a place, learning increasingly "involves a variety of active, problem-solving experiences that engage the learner in the 'social', rather than the 'individual', development of knowledge" [Matthews and Walton (2013), 144].

These changes in thinking about learning and the increased incorporation of group work into undergraduate curricula is leading to the development of library spaces with a wide variety of environments that support the collaboration between students and faculty in their endeavors to learn and to create new knowledge. One of the primary ways designers have supported these new activities is with the addition of creative commons or social spaces such as group study facilities, info commons, cafés, and art galleries [Gayton (2008), 60].

However, at the same time that many academics are excited by the incorporation of social spaces which support collaborative group work and a multiplicity of learning styles, others fear that "the social model undermines something that is highly valued in academic libraries: the communal nature of quiet, serious study. Communal activity in academic libraries is a solitary activity: it is studious, contemplative, and quiet. Social activity is a group activity: it is sometimes studious, not always contemplative, and certainly not quiet" [Gayton (2008), 60]. This view of the social space as a threat to the communal space makes apparent the need to isolate these very different environments.

The library of the future should also be an inviting and friendly space on the bright side of the line between hip and intimidating. Due to the prevalence of electronic resources and remote access, libraries need to remarket themselves as places where students want to study and create new knowledge. One way to accomplish this goal is to design libraries that are aesthetically appealing – libraries should look more like Apple stores and less like bunkers to attract visitors who would otherwise be satisfied accessing the same information from the comfort of their dormitory.

In summary, future libraries need to address the entire range of learning styles and student needs by incorporating both social and communal spaces. Both environments play a role in supporting learning and the development of knowledge but the design of library spaces must take into account the need to keep them separate from one another. Library spaces should also utilize bold, comfortable designs that motivate students to study at the library.

2.3: Structural elements of Library Facilities

Structures are designed to resist vertical and horizontal forces. Vertical forces include dead loads such as the self-weight of a structure and the weight of permanent, non-structural elements like roofing, flooring, and elevators. Live loads from building occupants, furniture, books, and the environment are another class of vertical loads that structural engineers design for. Horizontal forces, on the other hand, include forces from wind and earthquakes. These forces are “put into the special category of lateral live loads due to the severity of their action upon a building and their potential to cause failure” [Peting, D., and Luebkeman, C.H. (1996)]. The structural elements that resist these forces, including slabs, columns, and lateral force resisting structures, will be described in the following sections.

2.3.1: Floor Slabs

Floor slabs are structural elements that resist vertically applied forces and provide occupants with a usable surface to carry out the activities for which a structure was designed to house. Slabs receive and transmit load to other elements in the structural system such as beams, girders, and columns. The simplest type of slab is primarily supported on two opposite sides. In this configuration, the structural action of the slab is one-way. When a load is applied to a one-way slab, a single strip of slab transmits load perpendicularly to the supporting the beams, which in turn, transmit load to columns [MacGregor and Wight (2005), 608]. A slab supported on all four sides is considered to have two-way structural action. In this configuration, one strip of slab transmits load perpendicular to one set of beams, and another strip of slab transmits load perpendicular to another set of beams. Since the slab must transmit load in two directions, it must be reinforced in both directions and is referred to as a two-way slab. It should be noted that a slab supported on all four sides still utilizes one-way structural action if the ratio of length to width of one slab panel is greater than two [Nilson, Darwin, and Dolan (2009), 424].

There are several types of two-way slabs used for different span lengths. For relatively small spans between fifteen and twenty feet, flat plate slabs are used. A flat plate slab is a slab of uniform thickness supported only by columns. For larger spans from twenty five to forty feet, the thickness needed to transmit applied loads to columns exceeds the thickness needed to resist bending moments [MacGregor and Wight (2005), 608]. In such a case, the material of the slab at mid-span is not used efficiently and can be removed to save material and reduce slab moments. This system is referred to as a waffle slab because ribs intersect the areas of removed material creating a waffle-like pattern on the underside of the slab, which is shown below in Figure 2. It should also be noted that the full depth of the slab is maintained in the regions surrounding the columns to allow for load to be transmitted from the slab to the columns.

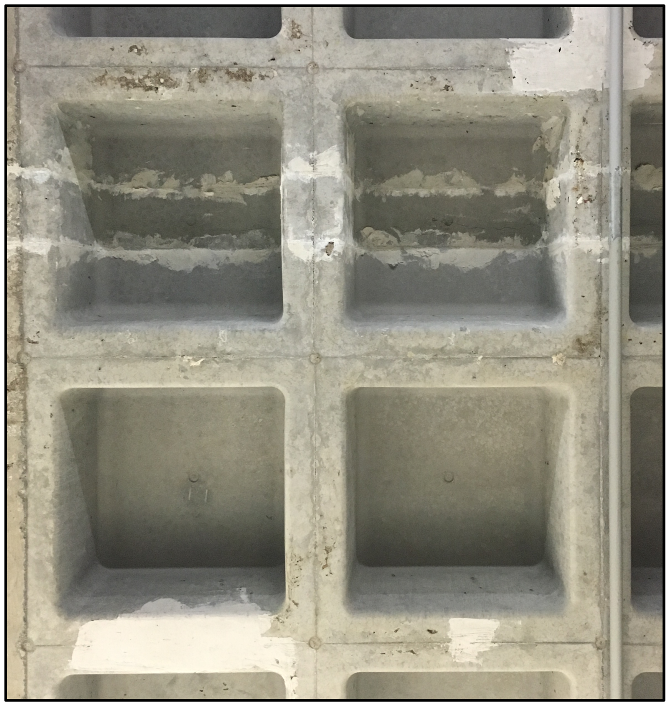

Figure 2: Underside of Waffle Slab on the Ground Floor of the Gordon Library

2.3.2: Columns

Columns are vertical structural members that support axial compressive loads and transmit those loads to a structure's foundation. In a concrete structure, columns are reinforced with longitudinal and transverse reinforcing steel, which vary in configuration depending on the application and loads applied to the column. Longitudinal reinforcing extends from one column into the overlying column where it is lap-spliced with that column's reinforcing. Transverse reinforcing either consists of ties or a spiral. The most common type of column used in non-seismically active regions is the tied column. A tied column consists of longitudinal (vertical) reinforcing bars that are braced with smaller bars along the length of the column. When high strength or high ductility performance is required, the longitudinal reinforcement is arranged in a circle, and a helical or spiral-shaped piece of rebar is wrapped around the longitudinal reinforcing to provide confinement to the concrete as the column attempts to expand laterally

[MacGregor and Wight (2005), 477]. An alternative column type is the composite compression member which is a concrete member reinforced by a structural steel shape, pipe, or tubing. This column type is much less common in modern construction, largely due to increases in the compressive strength of concrete and the development of reinforcing steel with significantly higher yield strength.

2.3.3: Lateral Force Resisting Systems

A lateral force resisting system (LFRS) is a system of horizontal and vertical structural elements that work integrally to resist wind or earthquake loads. Diaphragms make up the horizontal component of the LFRS while shear walls, moment-resisting frames, or a combination of the two can make up the vertical component. A model building that resists lateral loads with diaphragms, moment-resisting frames, and shear walls is shown below in Figure 3.

Diaphragms are the basis for lateral load resisting systems. They most often make up the floors and roof of a building and as such, they are also responsible for resisting gravity loads. Diaphragms are responsible for conjoining the vertical elements of the LFRS and transmit lateral inertial forces to those vertical elements. Diaphragms also provide resistance to out-of-plane forces that develop from wind loads acting on exterior walls and resist thrust from inclined columns [Hooper, *et.al.* (2010), 2]. Diaphragms can transfer lateral forces to interior shear walls, exterior shear walls, or moment-resisting frames [Killian, D.M., and Lee, K.S. (2012), 2] and are required for buildings constructed in Seismic Design Category B, C, D, E, or F. The major components of a diaphragm system include the diaphragm slab, chords, collectors, and connections to the vertical elements of the structure. Diaphragms work integrally with either shear walls or moment-resisting frames to resist lateral forces from wind and earthquakes.

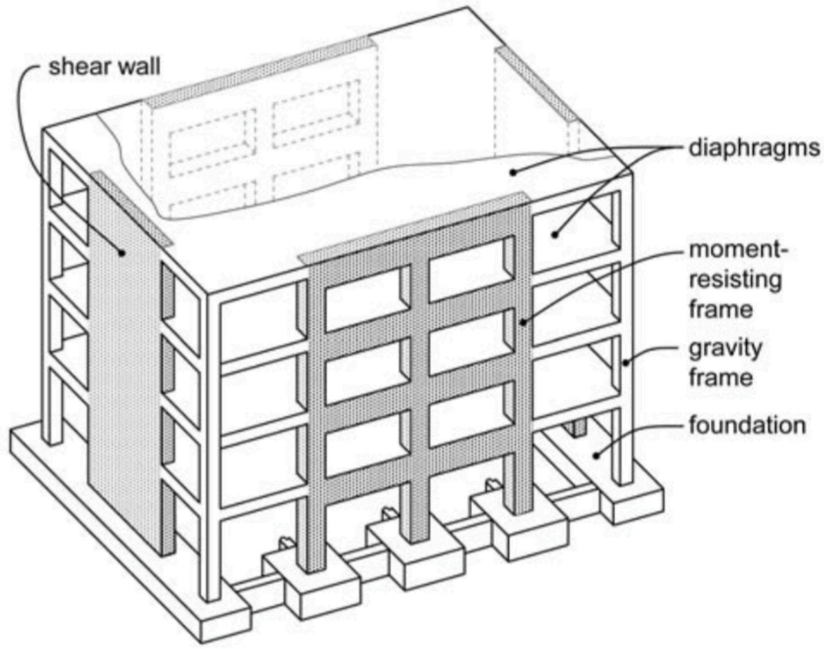

Figure 3: Isometric View of Structural System

[Hooper, *et.al.* (2010), 1]

A moment-resisting frame is composed of interconnected beams and columns that are rigidly connected at their ends to prohibit rotations between the attached members. While the joints of a moment-resisting frame may rotate as a unit, rigid frame members are essentially considered to be continuous through the joints and do not rotate with respect to each other [Schodek (2013), 350]. The advantage to this is that rigid connections restrain columns from freely rotating under laterally applied forces, which could cause a major structural failure.

Shear walls, also known as structural walls, are another example of vertical elements that resist lateral forces applied to a structure. They are primarily responsible for resisting in-plane loads applied along the height of a building. In a reinforced concrete building, shear walls are typically composed of cast-in-place concrete and deformed steel reinforcement [Fields, *et.al.*

(2012), 1], but precast concrete can also be used as a shear wall. There are several types of shear walls: the most basic shear wall is designed to resist combinations of shears, moments, and axial forces while shear walls designed for buildings located in Seismic Design Categories D, E, or F are referred to as special structural walls and must conform to the requirements listed in Chapter 21 of *ACI 318* [Fields, *et.al.* (2012), 2]. The placement of shear walls is also very important. Not only located at the building exterior, shear walls are commonly found on the interior as elevator or stairway cores where they serve a dual purpose of enclosing a space and resisting axial and lateral forces. Shear walls are typically the most cost effective for low to mid-rise buildings where floor-to-floor heights are typically minimized and the added depth required for moment frame members would translate into higher construction costs.

2.3.4: Foundations

Foundations transfer load from the superstructure to the underlying soil or rock. Factors that influence foundation design include the load to be transferred from the building, the behavior of soils under load and their resistance to load, the building code requirements, and the geological conditions of the soil [Das, B. (2011), 1]. There are two main classes of foundations: shallow foundations and deep foundations. Shallow foundations are typically embedded to a depth of three to four times the width of the foundation or less and include spread footings, wall footings, and mat foundations. Drilled shaft and piles make up the second class of foundations and are used in cases where the top layers of the soil have insufficient load bearing capacity.

2.4: Building Codes

A building code is a legal document created to ensure that structures are designed to a standard level of performance, which protects public safety, health, and welfare. Building codes provide minimum strengths of materials, maximum occupancies, and design loads for structures of all kinds.

If a new library were being constructed in Worcester, Massachusetts, in the present day, it would have to comply with the *Eighth Edition, Massachusetts Building Code* (780 CMR). This building code is based off the 2009 *International Building Code* (IBC) produced by the International Code Council (ICC). The *IBC* is a model building code adopted by most localities in the United States and amended through the publication of building codes at the state level. The first edition of the Massachusetts building code was published in 1974. In years prior, the city of Worcester promulgated its own building code, which was used in the design and construction of the Gordon Library.

The current Massachusetts building code, 780 CMR, varies drastically from the 1965 Worcester building code which was used to design the Gordon Library. Significant technical advances in fire protection engineering, earthquake, wind, and snow modeling have changed the way engineers think about designing structures and these changes are reflected in the building code.

To benchmark the performance of the existing building, we plan to perform a comparison between the provisions of 780 CMR and the 1965 Worcester building code, which was obtained from the Worcester Public Library. We will present in tabular form the differences in snow loads, wind loads, and design loads for a library structure.

Another facet of the building code is industry standards. The American Institute of Steel Construction and the American Concrete Institute publish design requirements for steel and concrete structural members, respectively. These requirements are referenced by the *IBC* and must be followed by designers to ensure public safety. Since structural steel shapes produced today vary significantly from those used in the Gordon Library, the *AISC Rehabilitation and Retrofit Guide* was obtained for the benchmarking process.

2.5: Software Tools for Structural Design and Analysis

In order to understand the performance of the existing structure, our project team plans to create a finite element model of the structure. A finite element model is a computer assembly of building elements modeled using their physical and engineering properties and arranged in their desired configurations. Once modeled, loads are applied to the columns, girders, and floors of the model, and the analysis software automatically calculates the resulting stresses and bending moments. We plan to use both *SAP 2000* and *RISA 2D*, which are industry standard finite element analysis programs. Since the two programs make different assumptions and calculate forces and stresses in different ways, we expect to obtain different results, which we will then compare. The primary purpose of using these software tools is to facilitate the process of evaluating the capacity of the existing structure. We also plan to utilize the software to aid in developing the structural alternatives and will utilize the code check features of the software to verify that the structural members satisfy the requirements of the *AISC Steel Construction Manual, 14th Ed.* and *ACI 318-11*.

2.6: Cost Analysis

Once the primary member sizes and structural systems are defined, we are going to perform cost estimates using *RS Means* construction cost data. The cost of the structural alternatives will include material and labor costs for the superstructure and elements of the foundation.

For the reinforced concrete alternative we will determine the total cubic yardage of concrete and the total amount of reinforcing steel required for the superstructure.

In the case of steel we will determine the total weight of steel as well as the amount of decking, and slab material required.

In addition, we plan to analyze the ancillary costs associated with each structure. For example, steel structures require fireproofing material where concrete structures do not. Steel structures also tend to be taller than concrete structures because of the transition between the girders, beams, and structural slabs. This could potentially lead to different curtain wall costs for the alternative superstructures.

Chapter 3: Methodology

This section describes the major objectives of our work and outlines the tasks and primary parties responsible for their completion. As an aid for the reader, we developed a mind map of our methodology and created a series of tables that outline evaluation criteria and specify the team members and resources required to complete project tasks.

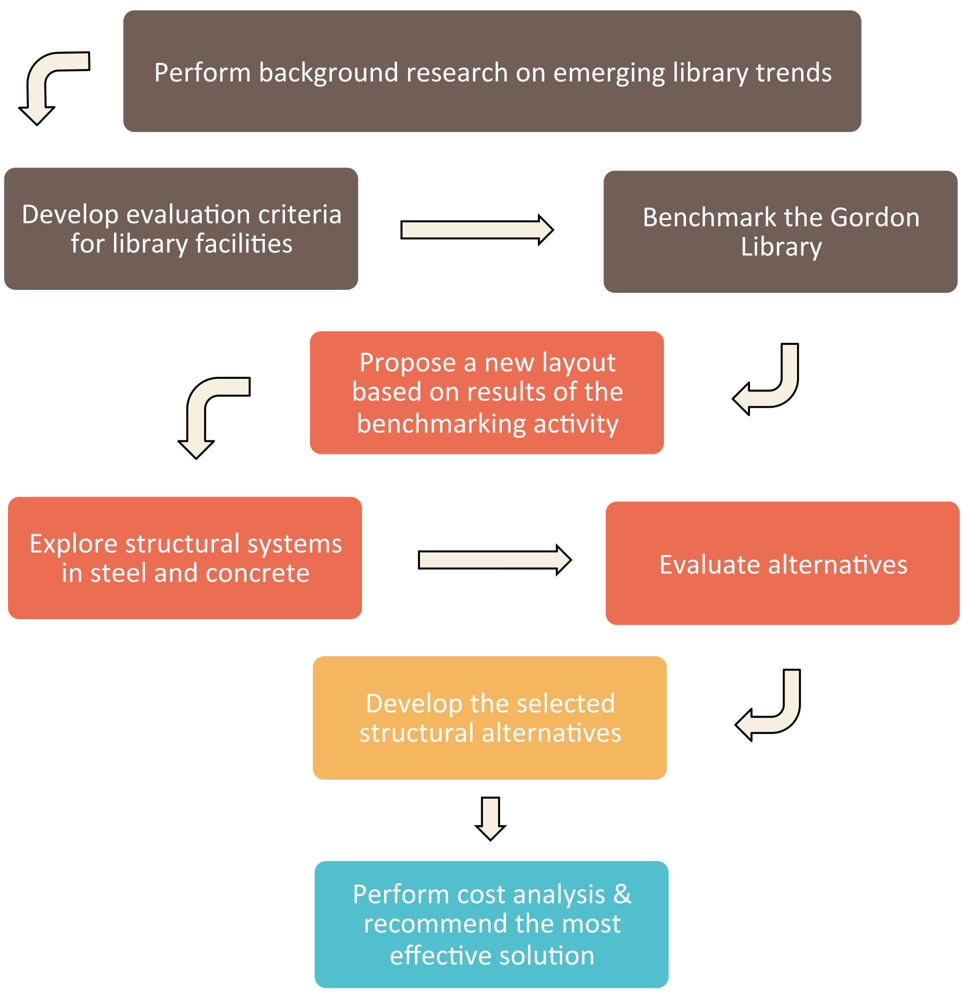

Figure 4: Methodology Mind Map

3.1: Benchmark Existing Building

In order to evaluate the functionality of the Gordon Library, we have developed a list of criteria that modern library facilities should meet, shown below in Table 1. These criteria will be used to benchmark the Gordon Library.

Table 1: Modern Library Layout Evaluation Criteria

Criteria	Key Concepts	Key References
Lighting & Daylighting	Library spaces should be well lit and should make use of daylighting to improve user comfort and productivity	Varrichione and Jarvis (2015) Kilic & Hasirci (2011)
Views to the Exterior	Modern library design focuses on integrating the internal environment with nature and providing adequate views to the exterior is one of the best ways to accomplish this	Kilic & Hasirci (2011) Brand (2006)
Availability of High Quality Communal Spaces	Studious, contemplative, and quiet study spaces remain vitally important to the library experience	Applegate (2009) Gayton (2008) Latimer and Niegard (2008) Lee, Velez, and Yoo-Lee (2013)
Availability of Social Spaces	Library users are increasingly looking to socialize, and work collaboratively. Cafes, art galleries, information commons, and group study spaces are in high demand	Conner (2014) Lee, Velez, and Yoo-Lee (2013) Bryant, Matthews, and Walton (2009)
Accessible	Library facilities should be accessible for those with disabilities and should provide users with multiple access points	Latimer and Niegard (2008) Ramsey and Sleeper (2007)

Aesthetically Pleasing	Library facilities need to catch the attention of passersby and should provide a comfortable and attractive environment for their users	Online research of modern library designs Asher and Duke (2012) Dominiczak (2014)
Balanced Communal and Social Space	There must be an appropriate balance between quiet study areas and social areas. The two settings are distinct and should not interfere with one another	Gayton (2008)

3.1.1: Evaluate Layout

Table 2: Task Breakdown for Evaluating the Existing Layout

Task	Team Member	Tools &Resources
Calculate percentage of windows and number of elevations with views	Rania	Gordon Library plans & tape measure
Evaluate artificial lighting	JP	Gordon Library lighting MQP
Calculate percentage of social space and comment on the quality of the space	JP	Gordon Library plans & tape measure
Calculate percentage of communal space and comment on the quality of the space	Rania	Gordon library site visit
Judge the balance between social and communal spaces	Rania	Gordon Library site visit
Evaluate access (entrances and handicap accessibility)	Rania	Architectural Graphic Standards & Gordon library site visit
Evaluate attractiveness of space	JP	Online research
Evaluate visual impact of columns	JP	Online research

To calculate the total window area, number of elevations with views, and study space area we will make a site visit to the Gordon Library. The plans of the Gordon Library along with a tape measure survey of the building's interior will help us obtain approximate quantities for the above criteria. We also plan to evaluate the current entrance of the Gordon Library and will make suggestions for the new design based on our evaluation. We will judge the attractiveness of the Gordon Library and its potential to draw in users that might not otherwise have a desire to visit the building based on our research into modern library designs. The lighting evaluation will primarily involve reviewing *Lighting Study of the George C. Gordon* Library, a Major Qualifying Project report written by WPI students in 2015. This report will likely inform our design decisions, potentially leading to the inclusion of a skylight and additional windows throughout the building. In addition, a visual review of the Gordon library columns will be performed to assess their impact on the usability of the space.

3.1.2: Evaluate Structure

Table 3: Task Breakdown for Evaluating the Existing Structure

Task	Team Member	Tools & Resources
Determine live loads and gravity loads used to design the existing library	Rania & JP	1965 Worcester Building Code
Verify structural performance of a typical bay	Rania & JP	Gordon Library plans
Create RISA model of existing structure	Rania	Gordon Library plans
Create SAP model of existing structure	JP	Gordon Library plans

In order to get a sense for the structural elements responsible for carrying loads and distributing them to the foundation, a review will be performed using a variety of resources. The first step in evaluating the structure will be a determination of the live loads and gravity loads used to design the Gordon Library. The 1965 Worcester building code will be the primary resource used to complete this task. After we determine the loads used to design the existing structure, we will check the performance of a typical bay using the plans, which give the sizes of the structural members. This performance check will not only indicate that the current structure is safe but will facilitate the process of evaluating the capacity of the existing structure. After this basic performance check, a finite element analysis will be performed using two structural analysis programs: *RISA 2D* and *SAP 2000*. These programs vary in the assumptions and techniques of evaluating structures so a comparison of the results output by these programs will be made. Code checks will also be performed to ensure that the primary members conform to the requirements of the *AISC Steel Construction Manual, 14th Ed.* and *ACI 318-11*.

3.2: Investigate New Designs

The purpose of this step is to determine potential layout and structural features that will provide a space that meets the criteria we have developed.

Table 4: Task Breakdown for Investigating New Designs

Task	Team Member	Tools & Resources
Determine areas of improvement	Rania & JP	Internet research
Propose new layout	Rania & JP	Internet research
Explore options in steel	Rania	Internet research
Explore options in concrete	JP	Internet research

3.2.1: Propose New Layout

A new layout will be proposed based on results from the benchmarking activity and our research into library designs that facilitate a more student-centered pedagogy.

3.2.2: Explore Structural Systems in Steel and Concrete

Options in steel and concrete will be explored to determine the most practical means of supporting the loads to be applied to the new library layout.

3.3 Develop the Selected Structural Alternatives

Each alternative will be evaluated for use on the project, which will require a holistic design process that takes into account the loads to be supported, member sizes, connection types, foundation elements, and the associated costs.

Table 5: Task Breakdown for Developing the Selected Structural Alternatives

Task	Team Member	Tools & Resources
Determine gravity loads & lateral loads	Rania	IBC & 780 CMR Mass Building Code
Develop concrete member sizes	JP	ACI Manual
Develop steel member sizes	Rania	AISC Manual
Design concrete connections	JP	ACI Manual
Design steel connections	Rania	AISC Manual
Design foundation elements	Rania & JP	ACI Manual
Prepare cost analysis	Rania	RS Means Construction Data
Propose high performance concrete mix	JP	Internet research

3.3.1: Design for Gravity and Lateral Loads

The gravity loads and lateral loads to be resisted by the structural alternatives will be determined using the *IBC* and the *Massachusetts Building Code*. A determination of the loads to be supported by the structure is essentially the guiding principle that drives all structural design decisions and is therefore of crucial importance to developing the selected structural alternatives.

3.3.2: Develop Member Sizes

After determining the loads to be supported by the structure, steel members will be designed with the help of the *AISC Steel Manual* and concrete members will be designed using *ACI 318-11*.

3.3.3: Design Connections

Connection designs will be developed for both steel and concrete structures using the *AISC Steel Manual* and *ACI 318-11*.

3.3.4: Design Foundation Elements

In order to design foundations to support the selected structural alternatives, we will use the method of back calculation to establish the bearing capacity of the soil at the Gordon library site. This was performed by dividing the load supported by each column by the footing area as shown in the structural drawings. The structural engineers that designed the library assumed a maximum bearing capacity of 8.88 tons/ft^2 which is reasonable considering that the bearing capacity of Glacial Till, the soil type at the Gordon Library site, is 10 tons/ft^2. This information will allow us to design concrete foundations in accordance with *ACI 318-11.*

3.3.5: Prepare Cost Analysis

Once all members, connections and structural features have been designed, a cost analysis will be performed using RS Means construction cost data. The Cost Analysis will include installed cost along with any ancillary costs such as fireproofing, curtain wall, and window systems.

3.4: Project Schedule

Week	Date	Objectives
		A Term
1	8/31/15 - 9/4/15	Define project scope and objectives
2	9/7/15 - 9/11/15	Continue defining project goals, begin formatting proposal
3	9/14/15 - 9/18/15	Continue formatting proposal, and begin background research
4	9/21/15 - 9/25/15	Perform benchmarking activity inside Gordon Library (take pictures and measurements of the space), and continue background research
5	9/28/15 - 10/2/15	Perform preliminary calculations to benchmark the structure, and continue background research
6	10/5/15 - 10/9/15	Work on proposal
7	10/12/15 - 10/15/15	Work on and submit proposal
		B Term
8	10/27/15 - 10/30/15	Begin steel design (roof framing plan)
9	11/2/15 - 11/6/15	Finish roof framing plan, begin level 1-4 framing plan, begin column design
10	11/9/15 - 11/13/15	Finish level 1-4 framing plan, finish column design, begin steel LFRS design
11	11/16/15 - 11/20/15	Complete column design, Begin two-way slab design
12	11/30/15 - 12/4/15	Finish two-way slab design, complete steel LFRS design, begin concrete beam design
13	12/7/15 - 12/11/15	Complete concrete beam design, begin concrete column design, begin concrete shear wall design
14	12/14/15 - 12/17/15	Finish concrete column and shear wall design, revise proposal, and submit B Term Deliverable
		C Term
15	1/14/16 - 1/22/16	Begin cost analysis and format final report
16	1/15/16 - 1/29/16	Complete cost analysis and work on final report
17	2/1/16 - 2/5/16	Work on final report
18	2/8/16 - 2/12/16	Work on final report
19	2/15/16 - 2/19/16	Submit draft of final report
20	2/22/16 - 2/26/16	Make revisions to final report
21	2/29/16 - 3/4/16	Continue revising final report and submit final report

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MEMBER: M1

BEAM: W16X31 $\Rightarrow I_G = 375 \text{ in}^4$

COLUMN: W10X49 $\Rightarrow I_C = 272 \text{ in}^4$

COLUMN

$r_x = 4.35$, $r_y = 2.54$

$A_g = 14.4 \text{ in}^2$

$$G = \frac{\Sigma(I_c/L_c)}{\Sigma(I_g/L_g)} = \frac{2 \cdot 272/15'}{375/15} = 1.45 = G_B$$

$$\text{CHECK } \frac{K_y \cdot L}{r_y} = \frac{1 \cdot 15' \cdot 12''/ft}{2.54} = \underline{\underline{70.87}}$$

BIGGER VALUE GOVERNS

NOMOGRAPH $\Rightarrow K_x = 1.38$ $\quad \dfrac{1.38 \cdot 12''/FT \cdot 15 \text{ FT}}{4.35} = 57.10$

$$F_e = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2 \cdot 29000}{(70.87)^2} = 56.99$$

$$F_{cr} = (0.658^{50/56.99}) \cdot 50 = 34.63$$

$$\phi P_n = 0.9 \cdot F_{cr} \cdot A_g = 0.9 \cdot 34.63 \cdot 14.4 \text{ in}^2 = 448.81 \text{ KIPS} = P_c$$

TABLE 3-2

$\left.\begin{array}{l} L_p = 8.97 \\ L_r = 31.6 \\ L_b = 15' \end{array}\right\} L_b > L_p$ CONSIDER LATERAL TORSIONAL BUCKLING

$$M_p = Z_x \cdot F_y = 60.4 \cdot 50 \frac{K}{in^2} = 3020 \text{ K} \cdot \text{in} = 251.67 \text{ K} \cdot \text{Ft}$$

$$0.7 \cdot F_x \cdot S_x = 0.7 \cdot 50 \frac{\text{Kip}}{in^2} \cdot 54.6 = 1911 \text{ K} \cdot \text{in} = 159.25 \text{ K} \cdot \text{Ft}$$

$$\text{INTERPOLATE } M_n = 251.67 - (251.67 - 159.25) \cdot \left(\frac{15 - 8.97}{31.6 - 8.97}\right)$$

$$M_n = 227.04 \Rightarrow \phi M_n = 204.34 \text{ Ft} \cdot \text{k} = M_{cx}$$

$$\phi M_{rx} = 143 \text{ kip-ft}$$

$$\frac{P_r}{P_c} = \frac{49.10}{448.81} = 0.109 < 0.2 \Rightarrow \text{H1-1b}$$

$$\text{H1-1b: } \frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0 ; \quad \frac{49.10}{2 \cdot 448.81} + \left(\frac{18.39}{204.34} + 0\right) = 0.154 < 1$$

$\therefore$ OK

$$G = \frac{\Sigma(I_c/L_c)}{\Sigma(I_g/L_g)} = \frac{2 \cdot 272/15'}{238/26'} = 3.96$$

2 COLUMNS

$I_c = 272\ in^4$
$I_g = 238\ in^4$
$G_A = 1$

NONOGRAPH $K_x = 1.62$

Check $\frac{K_y \cdot L}{r_y} = \frac{1 \cdot 15' \cdot 12''/ft}{2.54} = \underline{\underline{70.87}}$

$\frac{K_x \cdot L}{r_x} = \frac{1.62 \cdot 15' \cdot 12''/ft}{4.35} = 67.03$

Bigger value governs

$\Downarrow$

$$F_e = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2 E}{(70.87)^2} = \frac{\pi^2 \cdot 29000}{70.87^2} = 56.99$$

$$F_{cr} = \left(0.658^{50/56.99}\right) \cdot 50 = 34.63\ ksi$$

$\phi P_n = 0.9 \cdot F_{cr} \cdot A_g$

$0.9 \cdot 34.63 \cdot 14.4\ in^2 = 448.80\ KIPS = P_c$

$WLB = \frac{h}{t_w} \leq 3.76\sqrt{\frac{E}{F_y}}$

$23.1 \leq 90.5$ ∴ WEB IS COMPACT

$FLB = \frac{b_f}{2t_f} = 8.93 \leq 0.38\sqrt{\frac{E}{F_y}} = 9.2$ ∴ FLANGE COMPACT

TABLE 3-2 DATA

$L_p = 8.97'$
$L_r = 31.6'$
$L_b = 15'$

$L_b > L_p$ so consider lateral torsional buckling

$M_p = Z_x \cdot F_y = 60.4\ in^3 \cdot 50\frac{K}{in^2} = 3020\ K \cdot in = 251.67\ K \cdot ft$

$0.7 \cdot F_y \cdot S_x = 0.7 \cdot 50 \frac{Kip}{in^2} \cdot 54.6 = 1911\ K \cdot in = 159.25\ K \cdot ft$

INTERPOLATE $M_n = 251.67 - (251.67 - 159.25)\left(\frac{15 - 8.97}{31.6 - 8.97}\right)$

$M_n = 227.04 \Rightarrow \phi M_n = 204.34\ ft \cdot k = M_{cx}$

$M_{rx} = 95.4$

BEAM W16X67 $\Rightarrow I_G = 954 \text{ in}^4$

COLUMN W24X68 $\Rightarrow I_C = 1830 \text{ in}^4$

$$G = \frac{\Sigma (I_c / L_c)}{\Sigma (I_g / L_g)} = \frac{2 \cdot 1830 / 15'}{954 / 26'} = 6.65$$

$$\text{Check } \frac{K_y \cdot L}{r_y} = \frac{1 \cdot 15' \cdot 12''/ft}{1.87} = \underline{\underline{96.26}}$$ BIGGER VALUE GOVERNS

NOMOGRAPH : $K_x = 1.0$ $\quad \frac{K_x \cdot L}{r_x} = \frac{1 \cdot 15' \cdot 12''/ft}{9.55} = 18.85$

$G_A = 1$, $G_B = 6.65$

$\Downarrow$

$$F_e = \frac{\pi^2 E}{(kL/r)^2} = \frac{\pi^2 \cdot 29000}{(96.26)^2} = 30.89$$

$$F_{cr} = \left(0.658^{50/30.89}\right) \cdot 50 = 25.39$$

$\phi P_n = 0.9 \cdot F_{cr} \cdot A_g = 0.9 \cdot 25.39 \cdot 20.1 \text{ in}^2 = 459.31 \text{ Kips} = P_c$

TABLE 3-2

$\left.\begin{array}{l} L_p = 6.61 \\ L_r = 18.9 \end{array}\right\}$ $L_b > L_p \Rightarrow$ CONSIDER LATERAL TORSIONAL BUCKLING
$L_b = 15'$

$M_p = Z_x \cdot F_y = 170 \cdot 50 \frac{k}{in^2} = 8500 \text{ k}\cdot\text{in} = 708.33 \text{ k}\cdot\text{ft}$

$0.7 \cdot F_y \cdot S_x = 0.7 \cdot 50 \frac{\text{Kip}}{in^2} \cdot 154 = 5390 \text{ k}\cdot\text{in} = 449.17 \text{ k}\cdot\text{ft}$

INTERPOLATE $M_n = 708.33 - (708.33 - 449.17) \cdot \left(\frac{15 - 6.61}{18.9 - 6.61}\right)$

$M_n = 531.41 \Rightarrow \phi M_n = 478.27 \text{ ft}\cdot\text{k} = M_{cx}$

$M_{rx} = 269 \text{ kip}\cdot\text{ft}$

$\frac{P_r}{P_c} = \frac{138.99}{459.31} = 0.302 \Rightarrow$ H1-1a

EQUATION H1-1a $\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}}\right)$

$$= 0.302 + \frac{8}{9}\left(\frac{269}{478.27}\right) = 0.80195 < 1.0 \therefore OK$$

W24X68 IS ADEQUATE

① CHECK SHEAR CAPACITY OF BEAM W12X30

$$\phi V_n = \phi \cdot 0.6 \cdot F_y \cdot A_w \cdot C_v$$

CHECK $\frac{h}{t_w} \leq 2.24 \sqrt{\frac{E}{F_y}}$ $\quad F_y = 50$

$41.8 \leq 53.95$ ✓

$\phi = 1, \; C_v = 1$

$\phi V_n = 1 \cdot 0.6 \cdot 50\text{ksi} \cdot A_w = d \cdot t_w = 12.3 \cdot 0.260 = 95.94 \text{ Kips}$

$V = \frac{WL}{2} \qquad \frac{1.85 \text{ Kip/Ft} \times 26\text{ft}}{2} = 24.05 \text{ kips} < \phi V_n$

$A_b \Rightarrow \frac{5}{8}''$ ø BOLTS $\Rightarrow \frac{\pi}{4}\left(\frac{5}{8}\right)^2 = 0.3068 \text{ in}^2$

BOLT STRENGTH : $\phi R_n = 2 \cdot \phi \cdot F_v \cdot A_b$

$\phi R_n = 2 \cdot 0.75 \cdot 54 \cdot 0.3068 = 24.85$

$\# \text{ of bolts} = \frac{V_u}{\phi R_n} = \frac{24.05}{24.85} = 0.967 = 1 \text{ bolt}$

$\Rightarrow 2$ bolts

BOLT TEARING : $L_c = 1 - \frac{1}{2}\left(d_b + \frac{1}{8}\right) = 0.625''$

TOTAL CAPACITY $\quad \phi \cdot 1.2 \cdot L_c \cdot t \cdot F_u + 2 \cdot \phi \cdot 2.4 \cdot d_b \cdot t \cdot F_u \geq \frac{1}{2} \cdot 24.05$

$166.52 \geq 12.03$

$t \geq 0.07$

ANGLE SHEAR RUPTURE

$\phi R_n = \phi \cdot 0.6 \cdot F_u \cdot \left(L - n\left(d_b + \frac{1}{8}\right)\right) t$

TRY $L = 6$
$n = 2$

$0.75 \cdot 0.6 \cdot 58 \cdot \left(6 - 2\left(\frac{5}{8} + \frac{1}{8}\right)\right) t \geq \frac{1}{2} \cdot 24.05$

$117.45 t \geq 12.025$

$t \geq 0.102$

ANGLE SHEAR YIELD

$\phi R_n = \phi \cdot 0.6 \cdot F_y \cdot L \cdot t \geq 12.025$

$1 \cdot 0.6 \cdot 50 \cdot 6 \cdot t \geq 12.025$

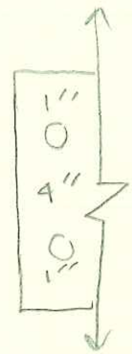
$t \geq 0.067''$

ANGLE SHEAR RUPTURE GOVERNS

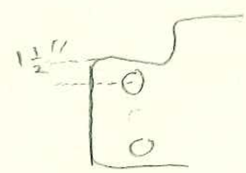
$t \geq 0.102 \Rightarrow \frac{1}{4}'' = 0.25''$

CHECK $L \geq \frac{T}{2}$ FOR STABILITY

$\frac{T}{2} = 10\frac{1}{8} / 2 = 5.0625 < 6''$



USE $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{4}$



NET HEIGHT : $1\frac{1}{2} + 4'' = 5.5''$

A_{nv} = Net shear area = $\left[5.5 - 0.5\left(\frac{5}{8} + \frac{1}{8}\right)\right] \cdot t_w = 0.260''$

$A_{nv} = 1.33 \text{ in}^2$

SHEAR RUPTURE

$0.6 \cdot F_u \cdot A_{nv} = 0.6 \cdot 58\text{ksi} \cdot 1.33 \text{ in}^2 = 46.371$

$A_{nt} = \left[1\frac{1}{2}'' - \frac{1}{2}\left(\frac{5}{8} + \frac{1}{8}\right)\right] \cdot t_w = 0.26 = 0.293 \text{ in}^2$

TENSION RUPTURE $U_{bs} \cdot F_u \cdot A_{nt}$

$= 1 \cdot 58 \cdot 0.293 = 16.97^K$

$A_{gv} = 5.5 \times \frac{1}{4} = 1.375 \text{ in}^2$

SHEAR YIELD = $0.6 \cdot F_y \cdot A_{gv} = 0.6 \cdot 36\text{ksi} \cdot 1.375 = 29.7^K$

R_n = Shear rupture + tension rupture $\leq$ Shear yield + tension rupture

$63.341 \leq 46.67$

$\therefore R_n = 46.67 \Rightarrow \phi R_n = 35.00^K > 24.05 \Rightarrow$ OK

TRY L = 5"

ANGLE SHEAR RUPTURE

$\phi R_n = 0.75 \cdot 0.6 \cdot 58 \cdot (5'' - 3(5/8 + 1/8)) \cdot t \geq 12.025$

$71.78\,t \geq 12.025$

$t \geq 0.167''$

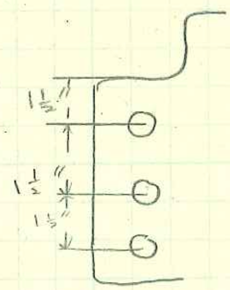
ANGLE SHEAR YIELD

$\phi R_n = \phi \cdot 0.6 \cdot F_y \cdot L \cdot t \geq 12.025$

$1 \cdot 0.6 \cdot 50 \cdot 5 \cdot t \geq 12.025$

$150\,t \geq 12.025$

$t \geq 0.08$



ANGLE SHEAR RUPTURE GOVERNS

$t \geq 0.167 \Rightarrow \frac{1}{4}'' = 0.25''$

$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{4}$ ANGLE

NET HEIGHT $= 1\frac{1}{2} + 1\frac{1}{2} + 1\frac{1}{2} = 4.5''$

A_{nv} = Net Shear area =

$4.5 - 0.5\left(\frac{5}{8} + \frac{1}{8}\right) \cdot t_w = 0.260'' = 1.073\ in^2$

$0.6 \cdot F_u \cdot A_{nv} = 0.6 \cdot 58ksi \cdot 1.073\ in^2 = 37.34$ = SHEAR RUPTURE

$A_{nt} = \left[1\frac{1}{2}'' - \frac{1}{2}\left(\frac{5}{8} + \frac{1}{8}\right)\right] \cdot t_w = 0.260'' = 0.293\ in^2$

TENSION RUPTURE : $U_{bs} \cdot F_u \cdot A_{nt} = 1 \cdot 58 \cdot 0.293 = 16.97^k$

$A_{gv} = 4.5'' \times \frac{1}{4} = 1.125\ in^2$

SHEAR YIELD = $0.6\,F_y \cdot A_{gv} = 0.6 \cdot 36ksi \cdot 1.125 = 24.3^k$

R_n = SHEAR RUPTURE + TENSION RUPTURE ≤ SHEAR YIELD + TENSION RUPTURE

$54.31 \leq 41.27$

$\therefore R_n = 41.27 \Rightarrow \phi R_n = 0.75 \cdot 41.27 = 30.95 > 24.05 \Rightarrow OK$

① CHECK SHEAR CAPACITY OF GIRDER : W16X31

$$\phi V_n = \phi \cdot 0.6 \cdot F_y \cdot A_w \cdot C_v$$

$$\text{CHECK } \frac{h}{t_w} \leq 2.24\sqrt{\frac{E}{F_y}}$$

$$51.6 < 53.95 \checkmark$$

$\phi = 1 \quad Cv = 1$

$\phi V_n = 1 \cdot 0.6 \cdot 50ksi \cdot A_w = d \cdot t_w = 15.9 \cdot 0.275 \cdot 1$

$$\phi V_n = 131.175 \text{ KIPS}$$

$V = \frac{WL}{2}$ ON GIRDER $\frac{4.59 \frac{KIP}{ft} \cdot 20ft}{2} = $ Vu 45.9 KIPS $< \phi V_n$ OK

$A_b \Rightarrow \frac{5}{8}$" Ø BOLTS $\Rightarrow \frac{\pi}{4}(5/8)^2 = 0.3068 in^2$

BOLT STRENGTH : $\phi R_n = 2 \cdot \phi \cdot F_v \cdot A_b$

$\phi R_n = 2 \cdot 0.75 \cdot 54ksi \cdot 0.3068in^2 = 24.85$ / bolt

# of bolts $= \frac{V_u}{\phi R_n} = \frac{45.9}{24.85} = 1.8 \Rightarrow$ 4 bolts to keep 1" edge distance

Evaluate three limit states

① BOLT TEARING : $L_c = 1 - \frac{1}{2}(d_b + \frac{1}{8}) = 1 - \frac{1}{2}(\frac{5}{8} + \frac{1}{8}) = 0.625"$

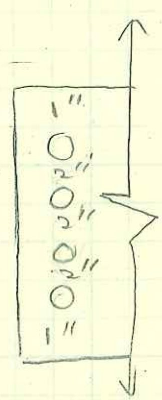
TOTAL CAPACITY : $\phi \cdot 1.2 \cdot L_c \cdot t \cdot F_u + 2 \cdot \phi \cdot 2.4 \cdot d_b \cdot t \cdot F_u \geq \frac{1}{2} \cdot 45.9$

$= 0.75 \cdot 1.2 \cdot 0.69t \cdot 58 + 2 \cdot 0.75 \cdot 2.4 \cdot 5/8 \cdot t \cdot 58$

$36.018t + 130.5t \geq 22.95$

$t \geq 0.138$

L=8"
n=4



② ANGLE SHEAR RUPTURE

$\phi R_n = \phi \cdot 0.6 \cdot F_u \cdot (L - n(d_b + 1/8")) t$

$0.75 \cdot 0.6 \cdot 58 \cdot (8 - 4(0.5/8 + 0.125)) \cdot t \geq 22.95$

$130.5t \geq 22.95$

$t \geq 0.176$

ANGLE SHEAR YIELD

$$\phi R_n = \phi \cdot 0.6 \cdot F_y \cdot L \cdot t \geq 22.95$$

$$1 \cdot 0.6 \cdot 50 \cdot 8\,t \geq 22.95$$

$$240\,t \geq 22.95$$

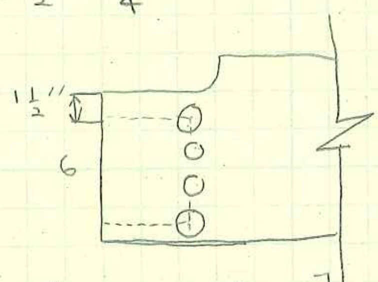
$$t \geq 0.0956$$

BOLT TEARING GOVERNS $\Rightarrow t \geq 0.176$ $\Rightarrow \frac{1}{4} = 0.25''$

CHECK if $L \geq \frac{T}{2}$ FOR STABILITY $\Rightarrow T = 13.625$

$$\frac{T}{2} = 6.8125'' < 8'' \therefore \text{OK}$$

USE $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{4}$



NET HEIGHT

$$1\tfrac{1}{2} + 2 + 2 + 2 = 7.5''$$

$$A_{nv} = \text{Net shear area} = \left[7.5'' - 0.5\left(\frac{5}{8} + \frac{1}{8}\right)\right] \cdot t_w = 0.275''$$

$$A_{nv} = 1.96\,in^2$$

SHEAR RUPTURE

$$0.6 \cdot F_u \cdot A_{nv} = 0.6 \cdot 58ksi \cdot 1.96\,in^2 = 68.21$$

$$A_{nt} = \left[1\tfrac{1}{2}'' - \tfrac{1}{2}\left(\frac{5}{8} + \frac{1}{8}\right)\right] \cdot t_w = 0.275 = 0.31\ in^2$$

TENSION RUPTURE $U_{bs} \cdot F_u \cdot A_{nt}$

$$= 1 \cdot 58ksi \cdot 0.31\,in^2 = 17.98^k$$

$$A_{gv} = 7.5'' \times \frac{1}{4}'' = 1.875\,in^2$$

$$\text{SHEAR YIELD} = 0.6 \cdot F_y \cdot A_{gv} = 0.6 \cdot 36ksi \cdot 1.875\,in^2 = 40.5^k$$

R_n = Shear rupture + tension rupture $\leq$ Shearyield + tension rupture

$$68.21 + 17.98^k \leq 40.5^k + 17.98$$

$$86.19 \nless 58.48$$

$$\therefore R_n = 58.48k \Rightarrow \phi R_n = 0.75 \cdot 58.48 = 43.86 < 45.9k$$

$\Rightarrow$ USE SPREADSHEET $\therefore$ FAIL

Interaction equations

$$G = \frac{\Sigma(I_c/L_c)}{\Sigma(I_g/L_g)} = \frac{\overset{2\ \text{COLUMNS}}{2} \cdot 272/15'}{375/20'} = 1.93$$

$I_c = 272\ in^4$

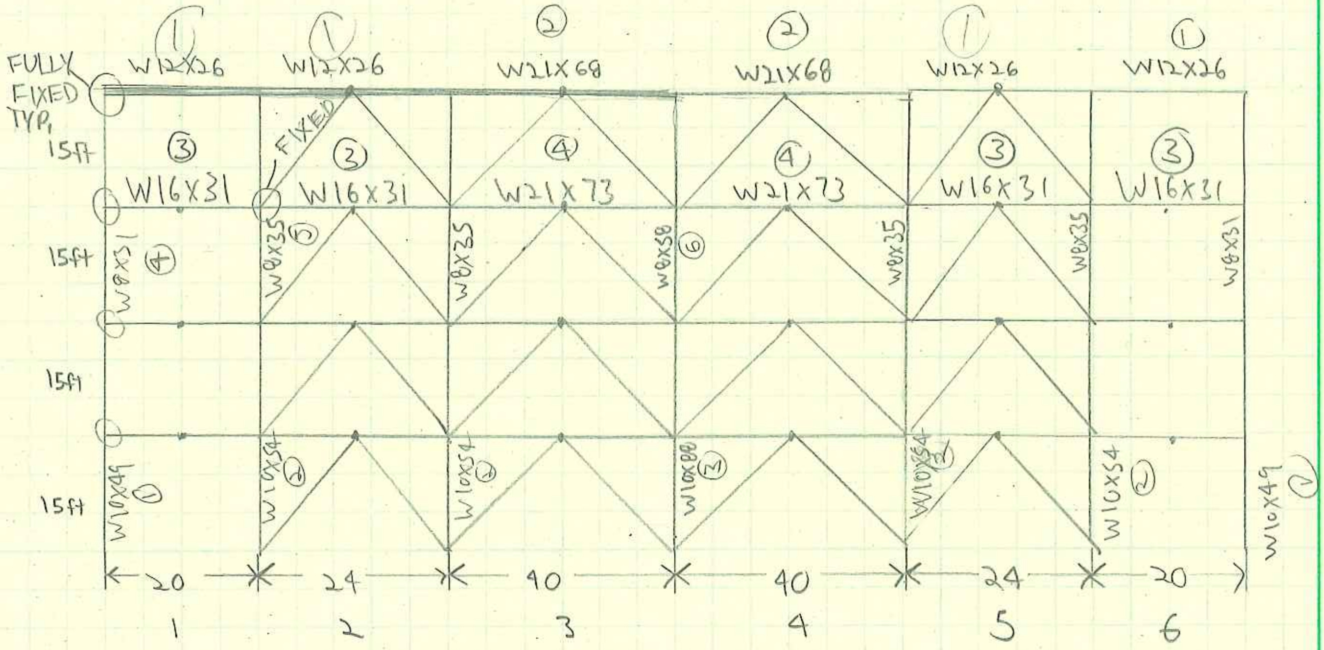
$I_g = 375\ in^4$

$G_A = 1.0$

NOMOGRAPH $K_x = 1.42$

CHECK $\frac{\overset{=1}{K_y} \cdot L}{r_y} = \frac{1 \cdot 15' \cdot 12''/ft}{2.54}$

$$\frac{K_x \cdot L}{r_x} = \frac{1.42 \cdot 15' \cdot 12''/ft}{4.35}$$



r BAY 2&5 $\geq (1 \cdot 19\text{ft} \cdot 12''/\text{ft}) \div 100 = 2.28$

r BAY 3&4 $\geq (1 \cdot 25\text{ ft} \cdot 12''/\text{ft}) \div 100 = 3.0$

BRACING FOR BAY 2&5

	WT.	b/t	r	thickness
HSS 6X6X5/16	23.34 #/Ft	17.6	2.31	0.291

BRACING FOR BAY 3&4

	WT.	b/t	r	thickness
HSS 8X8X1/2	48.85 #/Ft	14.2	3.04	0.465

LOAD CASES:
- 1.2D + 0.5L + 0.2S *
- 1.0E *

} Second order analysis

- 1.2D + 0.5L + 0.5S
- 1.6W

} Second order analysis

SEISMIC
64.26 →
39.131 →
24.845 →
11.428 →

WIND
14.228 →
27.656 →
25.836 →
13.136 →

ROOF DEAD : 61.49 X 14ft = 0.8609 KIP/FT
ROOF SNOW : 55 X 14ft = 0.77 KIP/FT
ROOF LIVE : 20 X 14ft = 0.28 KIP/FT
FLOOR 1-4 DEAD : 9.49 x 14 = 0.1329 KIP/FT
FLOOR 1-4 LIVE : 150 x 14 = 2.1 KIP/FT

HALFWAY INTO FRAME
: 28/2 = 14

AISC 14.2 BRACING MEMBERS

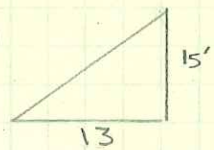
- MEET PROVISIONS OF 8.2b "SEISMICALLY COMPACT"

RECTANGULAR HSS $\frac{b}{A} \leq 16.1 = 0.64\sqrt{\frac{29000\,ksi}{46\,ksi}}$

$$\frac{KL}{r} \leq 100 = 4\sqrt{\frac{29000\,ksi}{46\,ksi}}$$

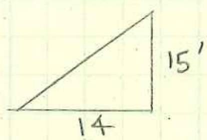
$k = 1.0$

BAY 1 L =



$\sqrt{15^2+13^2} = 19.85\ ft$

BAY 3 L

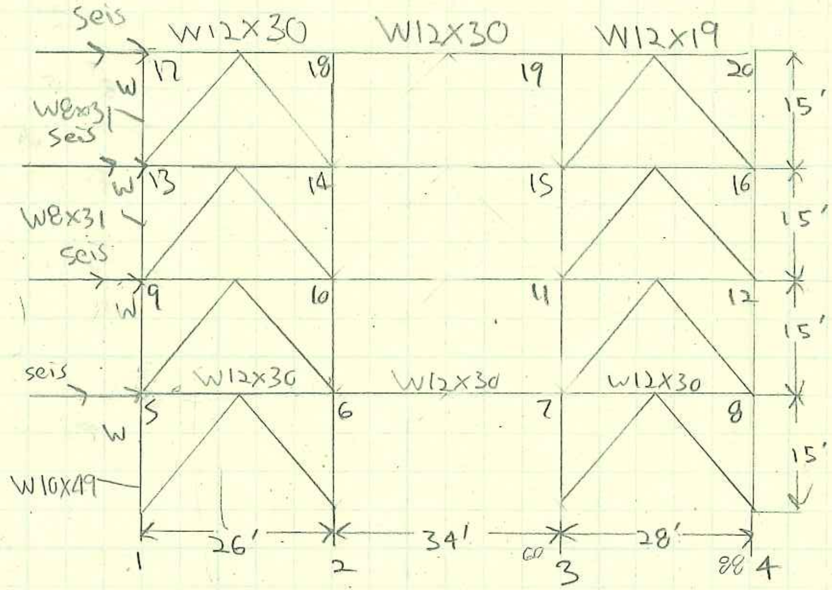


$\sqrt{15^2+14^2} = 20.52 \approx 20\,ft$

$r_{BAY3} \geq (1.0 \cdot 20\,ft \cdot 12''/ft) \div 100 = 2.4''$

$\Rightarrow$ Select HSS w/ r > 3.84

	Wt	b/t	r
HSS 7 X 7 X $\frac{1}{2}$	42.05 #/ft	12.1	2.63



BASE SHEAR SEISMIC : 19,031 lb

WIND : 20,238 lb

ROOF DEAD 20 + 39 + 2.49

DEAD ↓ CONCRETE ↓ METAL DECK ↓

CROSS BRACE : HSS $7 \times 7 \times \frac{1}{2}$

WALL THICKNESS : 0.465"

ROOF DEAD

$61.49 \frac{lb}{ft^2} \times 10 ft = 0.6149$ KIP/FT

ROOF SNOW

$55 \frac{lb}{ft^2} \times 10 ft = 0.55$ KIP/FT

ROOF LIVE $= 20 \frac{lb}{ft^2} \times 10 ft = 0.2$ KIP/FT

FLOOR 1-4 DEAD $= 9.49 \frac{lb}{ft^2} \times 10 = 0.0949$ KIP/FT

FLOOR 1-4 LIVE $= 150 \frac{lb}{ft^2} \times 10 = 1.5$ KIP/FT

NODES :

X	Y
0	0
26	0
60	0
88	0
0	15
13	15
26	15
60	15
74	15
88	15
⋮	

Seismic : $64.26 / 2 = 32.13$

$39.131 / 2 = 19.57$

$24.845 / 2 = 12.42$

$11.428 / 2 = 5.714$

LOAD CASES : • $1.2D + 0.5L + 0.2S$

• $1.0E$

ROOF

BEAMS : 5 X 19 X 28 X 2
5 X 19 X 28 X 2
9 X 22 X 28 X 2
5 X 30 X 34 X 2
5 X 30 X 34 X 2
5 X 19 X 26 X 2
5 X 19 X 26 X 2
9 X 22 X 26 X 2
= 2152 Lb = 2.152 KIP

GIRDERS 26 X 20 X 4
46 X 20 X 4
26 X 24 X 4
46 X 24 X 4
68 X 40 X 4
68 X 40 X 4
= 34432 lb = 34.43 KIP

COLUMNS
31 X 26 X 15) ÷ 2 = 6570 lb = 6.57 KIP
35 X 2 X 15)

METAL DECK : 2.49 $\frac{lb}{ft^2}$ X 90.83 X 170.83 = 38.64 KIP

CONCRETE : 39 $\frac{lb}{ft^2}$ X 90.83 X 170.83 = 605.14 KIP

MEP : 7 $\frac{lb}{ft^2}$ X 90.83 X 170.83 = 108.6 KIP

ROOF DEAD = 20 $\frac{lb}{ft^2}$ X 90.83 X 170.83 = 310.33 KIP

TOTAL DEAD = 1105.86 KIP

LIVE LOAD = 20 $\frac{lb}{ft^2}$ X 90.83 X 170.83 = 310.32 KIP

SNOW LOAD = 55 $\frac{lb}{ft^2}$ X 90.83 X 170.83 = 853.41 KIP

W = DEAD LOAD + 20% SNOW LOAD + 25% LIVE LOAD

1105.86 + 170.68 + 77.58 ⇒ $\boxed{W_{x\,ROOF} = 1354.12 \text{ KIP}}$

2ND FLOOR

BEAMS

30 X 28 X 5 X 4
22 X 28 X 9 X 2
30 X 34 X 5 X 4
30 X 26 X 5 X 4
22 X 26 X 9 X 2
= 74.18 KIP

GIRDERS

31 X 20 X 4
31 X 24 X 4
73 X 40 X 8
55 X 20 X 4
55 X 24 X 4
= 38.496 KIP

COLUMNS

$$\left.\begin{array}{l} 40 \times 8 \times 15 \\ 45 \times 8 \times 15 \\ 60 \times 8 \times 15 \\ 68 \times 4 \times 15 \end{array}\right) \div 2 = 10.74 \text{ KIP}$$

FLOOR AREA = $(90.83 \times 170.83) - (76 \times 30) = 13236.49\ FT^2$

METAL DECK = $2.49 \frac{lb}{Ft^2} \times 13236.49\ Ft^2 = 32.95$ KIP

MEP + CEILING = $7 \frac{lb}{Ft^2} \times 13236.49\ Ft^2 = 92.66$ KIP

CONCRETE : $39 \frac{lb}{Ft^2} \times 13236.49\ Ft^2 = 516.22$ KIP

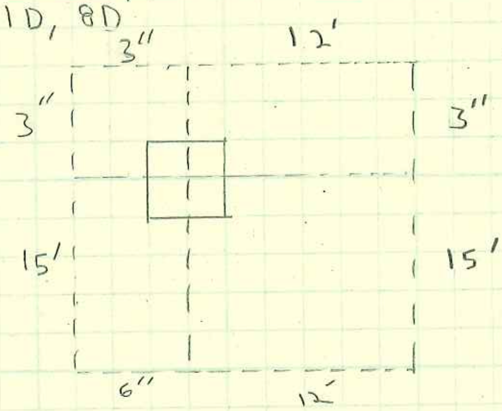
TOTAL DEAD LOAD = 641.83 KIP

LIVE = $150 \frac{lb}{Ft^2} \times 13236.49\ Ft^2 = 1985.47$ KIP

W_x = DEAD LOAD + 25% LIVELOAD = 1138.19 KIP

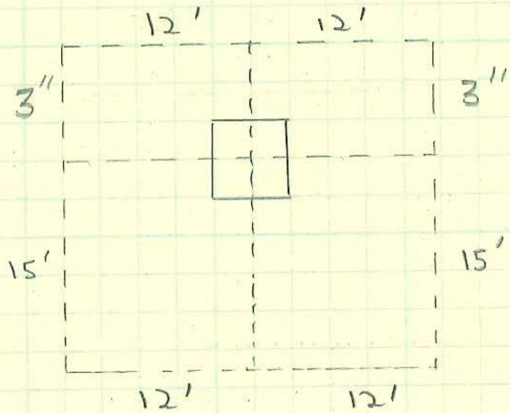
USE FOR FLOOR 1-3

1A, 8A, 1D, 8D



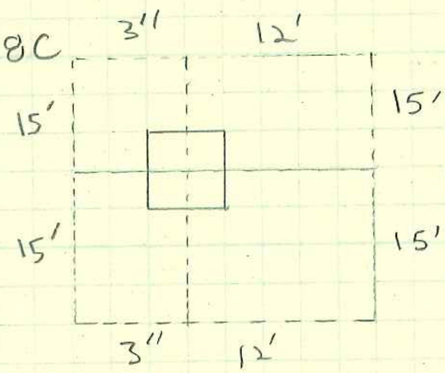
$$AREA = \frac{3''}{12} \times \frac{3''}{12} + \frac{3''}{12} \times 12' + 15' \times 12' + 15' \times \frac{3''}{12} = \underline{186.81\ ft^2}$$

2A, 3A, 4A, 5A, 6A, 7A,
2D, 3D, 4D, 5D, 6D, 7D



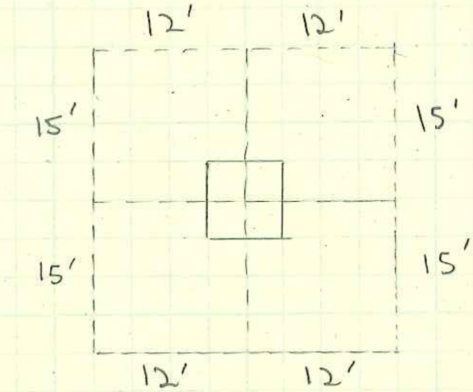
$$AREA = \frac{3''}{12} \times 12' + 12' \times \frac{3''}{12} + 15' \times 12' + 15' \times 12' = 366\ ft^2$$

1B, 1C, 8B, 8C



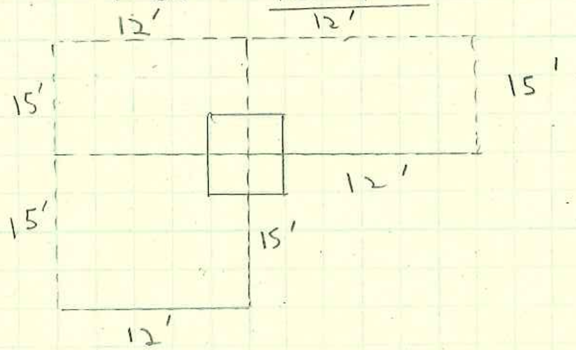
$$AREA = \left(\frac{3''}{12} + 12'\right) \times 30' = 367.5\ ft^2$$

2B, 2C, 7B, 7C



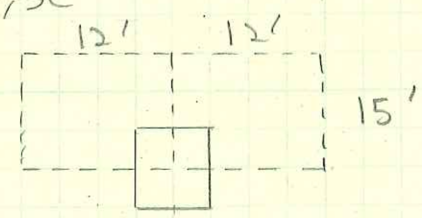
AREA = 24' x 30' = 720 ft²

3B, 3C, 6B, 6C



AREA = 12' x 15' + 12' x 15' + 12' x 15' = 540 ft²

4B, 5B, 4C, 5C



AREA = 24' x 15' = 360 ft²

*DESIGN 2 2A, 3A, 4A, 5A, 6A, 7A, 2D, 3D, 4D, 5D, 6D, 7D, 1B, 1C, 8B, 8C, 4B, 5B, 4C, 5C

for AREA = 367.5 ft²

EACH COLUMN SUPPORTS:

1 30FT BEAM, 1 24FT BEAM, CONCRETE SLAB TRIBUTARY AREA, MEP, LIVE LOAD, CEILING LOAD, ROOF DEAD, ROOF LIVE

* PERIMETER COLUMNS ONLY SUPPORT $\frac{1}{2}$ 30FT BEAM BUT FOR CONSTRUCTABILITY ALL COLUMNS WILL BE DESIGNED THE SAME

30 FT. BEAM LOAD $= \frac{28}{12} \times \frac{18}{12} \times 30\text{ FT} \times 150\frac{\text{lb}}{\text{FT}^3} = 15.75\text{ KIPS}$

24 FT. BEAM LOAD $= \frac{28}{12} \times \frac{18}{12} \times 24\text{ FT} \times 150\frac{\text{lb}}{\text{FT}^3} = 12.6\text{ KIPS}$

SLAB THICKNESS = 10

$\Rightarrow \frac{10}{12} \times 150\frac{\text{lb}}{\text{FT}^3} = 0.125\text{ KIPS/FT}^2$

MEP & CEILING = 0.007 KIP/FT^2

ROOF DEAD = 0.02 KIP/FT^2

$(0.007 + 0.02 + 0.125)$

TOTAL DEAD LOAD = $0.152\text{ KIPS/FT}^2 + 28.35\text{ KIPS}$

ROOF LIVE LOAD = 0.02 KIPS/FT^2

SNOW LOAD = 0.055 KIPS/FT^2

LOAD COMBO: $1.2D + 1.6S + 0.5L$

$P_u = 1.2(0.152 \times 186.81 + 28.35) + 1.6(0.055 \times 186.81) + 0.5(0.02 \times 186.81)$

$\boxed{P_u = 86.4\text{ KIPS}}$

1ST FLOOR COLUMNS

2B, 6B, 2C, 6C, 3B, 3C, 5B, 5C

$P_u = 1.2D + 1.6L = 206.62$ KIP

P_u ABOVE = 545.48 KIP

WT. COLUMN ABOVE = $\left(15\text{ FT} \times 60 \frac{\text{Lb}}{\text{FT}}\right) \div 1000 = 0.9$ KIP

TOTAL $P_u = 753$ KIP

USE W10X88 PROVIDES 831 KIP

4B, 4C, 4A, 4D

$P_u = 1.2D + 1.6L = 181.81$ KIP

P_u ABOVE = 600.9 KIP

WT. COLUMN ABOVE $\left(68 \frac{\text{lb}}{\text{ft}} \times 15\text{ ft}\right) \div 1000 = 1.02$ KIP

TOTAL $P_u = 783.73$ KIP

USE W10X88 ⇒ PROVIDES 831 KIP

3A, 3D, 5A, 5D, 2A, 6A, 2D, 6D

$P_u = 1.2D + 1.6L = 142.64$ KIP

P_u ABOVE = 308.0 KIP

WT. COLUMN ABOVE = $\left(15\text{ FT} \times 45 \frac{\text{Lb}}{\text{ft}}\right) \div 1000 = 0.675$ KIP

⇒ UPDATE TO 54 ⇒ = 0.81 KIP ⇒ WILL NOT MAKE A DIFFERENCE

TOTAL $P_u = 451.32$ KIP

USE W10X54 : PROVIDES $\phi_c P_n = 495$ KIP

1B, 1C, 7B, 7C, 1A, 7A, 1D, 7D

$P_u = 100.58$ KIP

P_u ABOVE = 265.90 KIP

WT. COLUMN ABOVE = $\left(40 \frac{\text{lb}}{\text{ft}} \times 15\text{ ft}\right) \div 1000 = 0.6$ KIP

TOTAL $P_u = 367.1$ KIP

USE W10X49 : PROVIDES 449 KIP

2ND FLOOR COLUMNS

2B, 6B, 2C, 6C, 3B, 3C, 5B, 5C

$P_U = 1.2D + 1.6L = 206.62$ KIP } SAME B/C SAME LOADS & AREA
P_U ABOVE = 338.14 KIP
WT. COLUMN ABOVE = $\left(15 \text{ FT} \times 48 \frac{\text{lb}}{\text{Ft}}\right) \div 1000 = 0.72$ KIP

TOTAL $P_U = 545.48$ KIP

USE W10 X 60 : PROVIDES $\phi_c P_n = 556$ KIP > $P_U = 545.48$ KIP

4B, 4C, 4A, 4D

$P_U = 1.2D + 1.6L = 181.81$ KIP

P_U ABOVE = 418.22 KIP
WT. COLUMN ABOVE = $\left(58 \frac{\text{lb}}{\text{ft}} \times 15 \text{ Ft}\right) \div 1000 = 0.87$ KIP

TOTAL $P_U = 600.9$ KIP
USE W10 X 68 : PROVIDES $\phi_c P_n = 629$ KIP

3A, 3D, 5A, 5D, 2A, 6A, 2D, 6D

$P_U = 1.2D + 1.6L = 142.64$ KIP
P_U ABOVE = 164.85 KIP
WT. COLUMN ABOVE = $\left(15 \text{ FT} \times 35 \frac{\text{Lb}}{\text{Ft}}\right) \div 1000 = 0.525$ KIP

TOTAL $P_U = 308.0$ KIP

USE W10 X 45 : PROVIDES $\phi_c P_n = 333$ KIP

1B, 1C, 7B, 7C, 1A, 7A, 1D, 7D
$P_U = 1.2D + 1.6L = 100.58$ KIP
P_U ABOVE = 164.85 KIP
WT. COLUMN ABOVE = $(31 \times 15 \text{ FT}) \div 1000 = 0.465$
TOTAL $P_U = 265.90$ KIP
USE W8 X 40 ⇒ PROVIDES 298 KIP

COLUMN 2B

P_u COLUMN ABOVE: 131.05 Kip
WT. COLUMN ABOVE: 15 FT X 31 LB/FT = 0.465 KIP

BEAM WT = 30 LB/FT

CONC. AREA = 682 FT^2

BEAM SPACING = 5 FT ∴ CONSERVATIVE

BEAM LOAD = 6 LB/FT^2

GIRDER LOAD = 55 LB/FT

LOADS:

MEP & CEILING: 7 $\frac{LB}{FT^2}$

METAL DECK: 2.49 LB/FT^2

CONCRETE: 42.9 LB/FT^2

OCCUPANCY LIVE = 150 LB/FT^2

$P_u = 1.2D + 1.6L$

$1.2(52.39 \times 682 FT^2 + 55') + 1.6(150 \times 682 FT^2) = 206.62$ KIP

TOTAL P_u = 0.465 KIP + 131.05 KIP + 206.62 KIP = 338.14 KIP

KL = 15 & PINNED @ BOTH ENDS SO K = 1

AISC TABLE 4-1 W8X48 PROVIDES 367 KIP > P_u = 338.14 KIP

USE W8X48 FOR 2B & 6B

USE FOR 3B, 3C, 5B, 5C B/C TRIB AREA = 669 FT^2

USE FOR 2B, 6B, 2C, 6C B/C TRIB AREA = 682 FT^2

3RD FLOOR COLUMNS

4B, 4C, 4A & 4D   AREA = 600 FT^2

OVERDESIGN: 4A & 4D HAVE SMALLER AREA & LIGHTER COLUMN ABOVE

BEAM LOAD = 22 Lb/FT / 5 FT SPACING = 4.4 Lb/Ft^2

GIRDER LOAD = 73 Lb/Ft

LOADS: MEP & CEILING: 7 $\frac{lb}{FT^2}$

METAL DECK: 2.49 $\frac{lb}{Ft^2}$

CONCRETE: 42.9 $\frac{Lb}{Ft^2}$

Occupancy LIVE: 150 $\frac{lb}{FT^2}$

$P_u = 1.2 D + 1.6 L$

$= 1.2(52.39 \times 600 FT^2 + 73) + 1.6(150 \times 600 FT^2) = 181.81$ KIP

P_u COLUMN ABOVE: 235.88 KIP

WT. COLUMN ABOVE = 35 $\frac{Lb}{FT}$ X 15 FT = 0.525 KIP

TOTAL P_u = 418.22 KIP

W8 X 58 PROVIDES $\phi_c P_n = 450$ KIP > P_u = 418.22 KIP

3A, 3D, 5A, 5D, 2A, 6A, 2D, 6D

AREA = 470.67 FT^2

CONSERVATIVE BEAM LOAD = 30 Lb/FT / 5 Ft = 6 Lb/Ft^2
GIRDER LOAD = 73 Lb / FT

$P_U = 1.2((52.39 \times 470.67) + 73) + 1.6(150 \times 470.67) = 142.64$ KIP

P_U ABOVE : $1.2(78.4 \cdot 470.67 + 68) + 1.6(55 \cdot 470.67) + 0.5(20 \cdot 470.67)$
$= 90.48$ KIP

COLUMN ABOVE WT $= \left(15\ FT \times 31 \frac{Lb}{FT}\right) / 1000 = 0.465$ KIP

TOTAL $P_U = 233.59$ KIP
USE W8X35 : PROVIDES $\phi_c P_n = 261$ KIP

1B, 1C, 7B, 7C

AREA = 332 FT^2

P_U ABOVE $= 1.2(78.4 \cdot 332 + 26) + 1.6(55 \cdot 332) + 0.5(20 \cdot 332)$
$= 63.80$ KIP

$P_U = 1.2(52.39 \times 332 + 31) + 1.6(150 \times 332) = 100.58$ KIP

WT. COLUMN ABOVE : $(31\ lb/FT \times 15\ Ft) / 1000 = 0.465$ KIP

TOTAL $P_U = 164.85$ KIP $\Rightarrow$ W8X31 PROVIDES 230 KIP

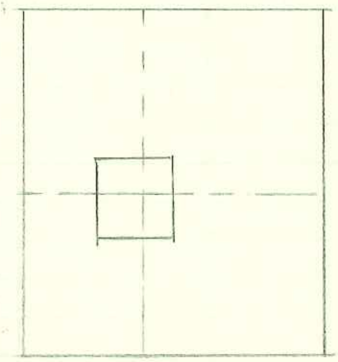
$\Rightarrow$ USE FOR 1A, 7A, 1D, 7D AS WELL
ASSUME THEY HAVE SAME TOTAL P_U = 164.85 KIP

EACH COLUMN SUPPORTS 1 GIRDER, BEAM LOAD, CONCRETE TRIBUTARY AREA, MEP, LIVE LOAD, CEILING, METAL DECK, ROOF WT, ROOF LIVE, ROOF DEAD

BEAM SPACING = 5FT * CONSERVATIVE

BEAM WT. = 30 $\frac{lb}{ft}$ *

DESIGN FOR LARGE SPAN: 28'



BEAM LOAD = 30 $\frac{lb}{FT}$ / 5FT = 6 LB/FT² *

GIRDER LOAD = 46 lb *

CONC. TRIB AREA = 682 FT²

CONCRETE LOAD = 42.9 $\frac{lb}{FT^2}$

METAL DECK LOAD = 2.49 $\frac{Lb}{FT^2}$

SNOW: 55 $\frac{LB}{FT^2}$

MEP LOAD & CEILING = 7 $\frac{LB}{FT^2}$

ROOFING: 2.6 $\frac{LB}{FT^2}$ + M&E ROOF = ROOF DEAD = 20 PSF

TOTAL DEAD LOAD = 77.4 $\frac{lb}{ft^2}$ + 46 $\frac{lb}{ft}$

ROOF LIVE: 20 $\frac{LB}{FT^2}$

$P_u = 1.2D + 1.6L + 0.5S$

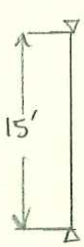
$1.2(78.4 \times 682\,FT^2 + 46) + 1.6\left(20\frac{lb}{FT^2} \times 682\,FT^2\right) + 0.5\left(55\frac{lb}{FT^2} \times 682\,FT^2\right)$

HIGHER LOAD FACTOR

$P_u = 1.2D + 1.6S + 0.5L$

$1.2(78.4 \times 682\,FT^2 + 46) + 1.6\left(55\frac{lb}{FT^2} \times 682\,FT^2\right) + 0.5\left(20\frac{lb}{FT^2} \times 682\,FT^2\right)$

$P_u = 131.05$ KIPS



COLUMN PINNED AT BOTH ENDS => K = 1

KL = 15

USE AISC TABLE 4-1

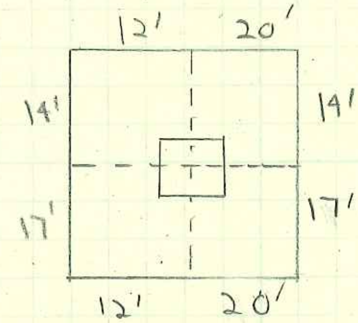
W8 X 31 PROVIDES 230 KIP $\phi_c P_n$ > P_u = 131.05 KIP

USE W10X31 FOR 2B, 3B, 5B, 6B, 2C, 3C, 5C, 6C, 1A, 1B, 1C

COLUMN 3B

CONC. TRIB AREA: 992 FT^2

GO WITH HIGHEST BEAM WT, SMALLEST SPACING



BEAM WT: 30 $\frac{lb}{FT}$

BEAM LOAD: $30 \frac{lb}{FT} / 5 FT = 6 LB/FT^2$

GIRDER LOAD: 68 $\frac{LB}{FT}$

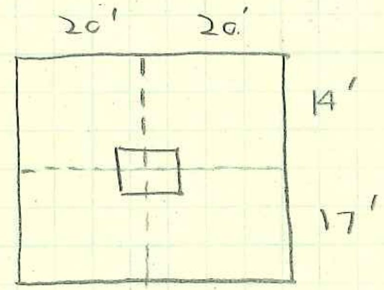
SAME LOADS EXCEPT BEAM & GIRDER

$P_U = 1.2(78.4 \times 992 FT^2 + 68) + 1.6(55 \frac{lb}{FT^2} \times 992 FT^2) + 0.5(20 \frac{lb}{FT^2} \times 992 FT^2)$

$P_U = 190.63$ KIP

SAME SUPPORT CONDITIONS

USE W8X31



COLUMN 4B

CONC. TRIB. AREA: 1240 FT^2

BEAM WT.: 22 $\frac{lb}{FT}$

BEAM SPACING: 5 FT

BEAM LOAD = 4.4 LB/FT^2

SAME LOADS EXCEPT BEAM & GIRDER

$P_U = 1.2(76.8 \frac{lb}{FT^2} \times 1240 FT^2 + 68) + 1.6(55 \frac{lb}{FT^2} \times 1240 FT^2) + 0.5(20 \times 1240)$

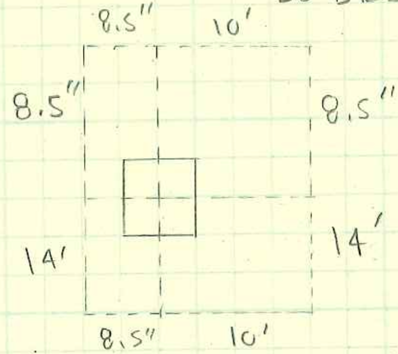
$P_U = 235.88$ KIP

SAME SUPPORT CONDITIONS

USE W8X35 : PROVIDES $\phi_c P_n = 261 > 235.88 = P_U$

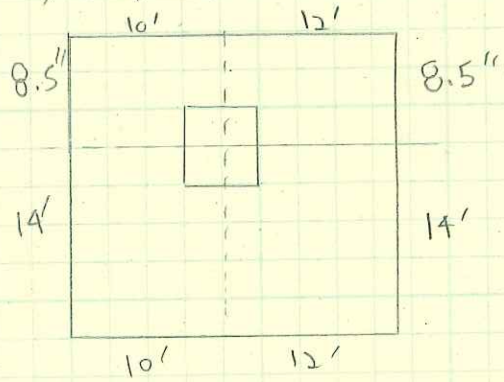
USE FOR 4B & 4C

1A, 7A, 1D, 7D * 1D & 7D HAVE SLIGHTLY SMALLER AREA SO DESIGN FOR 1A & 7A



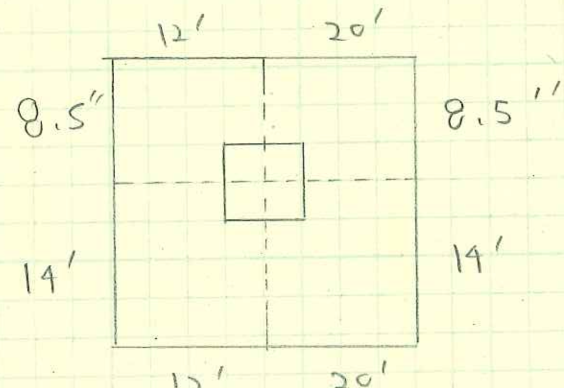
$$AREA = 14 \times \frac{8.5}{12} + \frac{8.5}{12} \times \frac{8.5}{12} + 10 \times \frac{8.5}{12} + 14 \times 10 = 157.5\ ft^2$$

2A, 6A, 2D, 6D



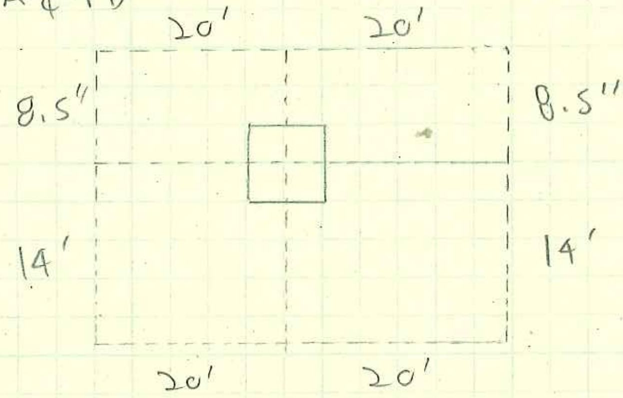
$$10 \times \frac{8.5}{12} + 12 \times \frac{8.5}{12} + 14 \times 10 + 14 \times 12 = 323.58\ ft^2$$

3A, 3D, 5A, 5D



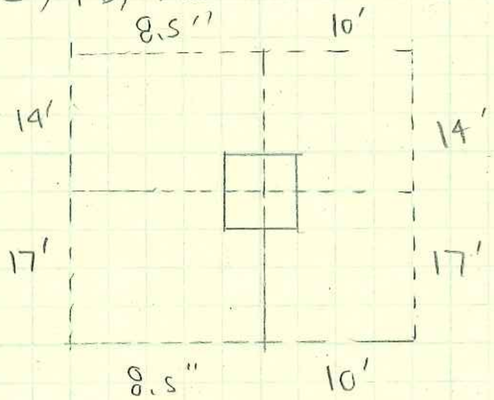
$$AREA: \frac{8.5''}{12} \times 12' + 20' \times \frac{8.5''}{12} + 14' \times 20' + 14' \times 12' = 470.67\ ft^2$$

4A & 4D



$AREA = 20 \times \frac{8.5}{12} + 20 \times \frac{8.5}{12} + 14 \times 20 + 14 \times 20 = 588.33\ FT^2$

1B, 1C, 7B, 7C

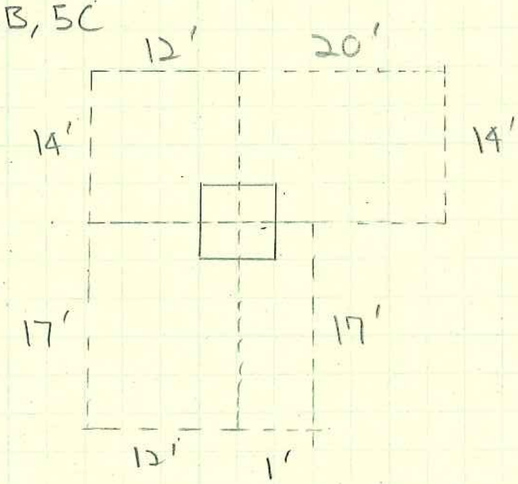


$AREA = (10 \times 14) + (10 \times 17) + \left(\frac{8.5}{12} \times 14\right) + \left(17 \times \frac{8.5}{12}\right) = 332\ FT^2$

2B, 6B, 2C, 6C

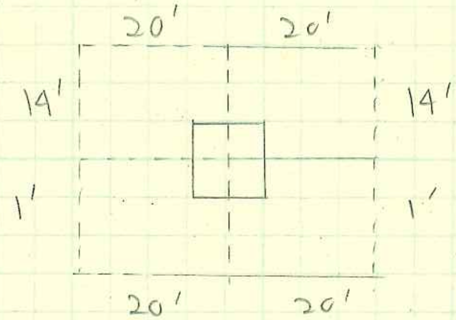
ALREADY CALCULATED FOR 4TH FLOOR COLUMNS : $682\ FT^2$

3B, 3C, 5B, 5C



$AREA = 14 \times 12 + 20 \times 14 + 17 \times 12 + 17 \times 1 = 669\ FT^2$

4B & 4C



AREA = (20 × 14) + (20 × 14) + (1 × 20) + (1 × 20) = 600 FT^2

1) CHOOSE LAYOUT & TYPE OF SLAB : TWO WAY SLAB WITH BEAMS

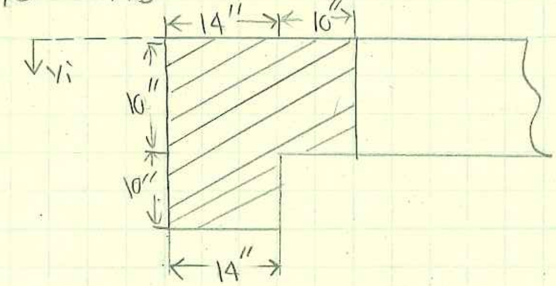
DMD ADEQUACY ? ✓

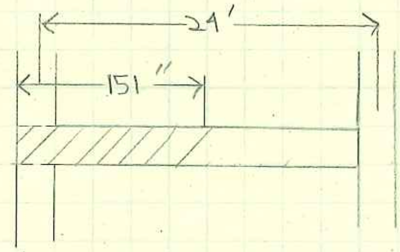
2) CHOOSE SLAB THICKNESS : $\frac{24 \cdot 12}{24} = 12''$

$\frac{24 \cdot 12}{28} = 10.28$

EDGE BEAM TYP.

10" SLAB





$C_g = \Sigma y_i \cdot A_i / A_g$

$$\frac{10 \cdot (14 \times 20) + 5 \cdot (10 \times 10)}{(14 \times 20) + (10 \times 10)} = 8.68'' \text{ FROM TOP OF SLAB}$$

$$I_g = \sum_{i=1}^{n} I_i + A_i d_i^2 \quad d_i \to (C_g - y_i)$$

$$\frac{1}{12} \cdot 14 \cdot 20^3 + (14 \times 20) \cdot (8.68 - 10)^2 + \frac{1}{12} \cdot 10 \cdot 10^3 + (10 \times 10) \cdot (8.68 - 5)^2$$

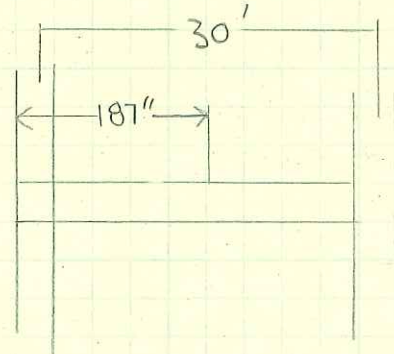
$I_b = 12008.77 \text{ in}^4$

$I_s = 151'' \times \frac{10^3}{12} = 12583.33 \text{ in}^4$

BEAM TO STIFFNESS RATIO

$\alpha = \frac{I_b}{I_s} = 0.95$

CHECK :



SAME EDGE BEAM

$I_s = 187 \times \frac{10^3}{12} = 15583.33$

$\alpha = \frac{I_b}{I_s} = 0.77$

AVG. $\alpha = \alpha_m = \frac{0.77 + 0.95}{2} = 0.86$

RATIO OF LONG TO SHORT CLEAR SPANS $= \frac{28.83}{22.83} = 1.26 = \beta$

$(30 \cdot 12 - 14)/12 = 28.83$

$(24 \cdot 12 - 14)/12 = 22.83$

l_n = LONGER CLEAR SPAN OF SLAB PANEL

$= ((30\text{ FT} \times 12'') - 14'') = 346''$

FOR SLABS WITH BEAMS BETWEEN SUPPORTS, ACI 9.5.3.3 GIVES MINIMUM THICKNESS

FOR $0.2 < \alpha_m < 2.0$: $h = \dfrac{l_n\left[0.8 + (f_y / 200{,}000)\right]}{36 + 5\beta(\alpha_m - 0.2)}$

BUT NOT LESS THAN 5"

$$\frac{346\left[0.8 + (60{,}000/200{,}000)\right]}{36 + 5 \cdot 1.26\,(1.25 - 0.2)} = 8.93''$$

PROCEDE WITH 10" DESIGN

$\frac{10''}{12} \times 150 \frac{lb}{ft^3} = 125 \frac{lb}{ft^2}$ SLAB DEAD LOAD

TOTAL DEAD LOAD = 132 PSF

$q = 1.2D + 1.6L = 1.2(132\text{ PSF}) + 1.6(150\text{ PSF}) = 398.4\text{ PSF}$

FOR THE LONG SPAN DIRECTION, FOR THE SLAB STRIP CENTERED ON THE INTERIOR COLUMN LINE, THE TOTAL STATIC DESIGN MOMENT IS:

$M_o = \frac{1}{8} \cdot 0.3984 \times 24 \times 28.83^2 = 993.41$ ft-KIPS

DISTRIBUTED : 993.41×0.65 = Neg. DESIGN MOMENT = 645.72 ft-KIPS

993.41×0.35 = POS. DESIGN MOMENT = 347.69 ft-KIPS

COLUMN STRIP HAS A WIDTH OF $2 \times 30/4 = 15$ ft

$\frac{l_2}{l_1} = \frac{24}{30} = 0.8$

$\alpha_{f1} \frac{l_2}{l_1} =$ 30 FT LONG BEAM

INTERPOLATION CHART FOR LATERAL DISTRIBUTION OF SLAB MOMENTS

$0.77 \times 0.8 = 0.616$

GRAPH A.4 GIVES 82 PERCENT OF THE NEGATIVE EXTERIOR MOMENT OR 529.49 ft-KIPS IS TAKEN BY THE COLUMN STRIP, OF WHICH 85% OR 450.06 ft-KIPS, IS TAKEN BY THE BEAM AND 79.42 ft-KIPS BY THE SLAB. THE REMAINING 116.23 KIPS IS TAKEN BY THE SLAB MIDDLE STRIP.

82%. INT. NEG MOMENT 450.00 BEAM

POS: 75% 260.77 : COLUMN STRIP 221.65 BEAM

FOR THE SHORT DIRECTION

$M_0 = \frac{1}{8} \cdot 0.3984 \times 30 \times 22.83^2 = 778.69$ ft-KIPS

EXTERIOR NEGATIVE MOMENT:

$0.16 \cdot 778.69$ ft-KIPS $= 124.6$ Ft-KIPS

POSITIVE MOMENT:

$0.57 \cdot 778.69$ ft-KIPS $= 443.85$ Ft-KIPS

INTERIOR NEGATIVE MOMENT:

$0.7 \cdot 778.69$ Ft-KIPS $= 545.08$ Ft-KIPS

EDGE BEAM TORSIONAL CONSTANT 14 x 20 in. RECTANGULAR BEAM WITH A 10" x 10" PROJECTING FLANGE

EQ 13.6

$$C = \left(1 - 0.63 \times \frac{14}{20}\right) \cdot \frac{14^3 \times 20}{3} + \left(1 - 0.63 \times \frac{10}{10}\right) \cdot \frac{10^3 \times 10}{3}$$

$= 11459.3$

$\frac{l_2}{l_1} = \frac{30}{24} = 1.25 \qquad \alpha_{f1} \frac{l_2}{l_1} = 0.95 \cdot 1.25 = 1.19$

$$B_t = \frac{11459.3}{2 \times I_s = 12583.33 \text{ in}^4} = 0.455$$ Eq. 13.5

GRAPH A.4

69% POSITIVE MOMENT = 306.26 FT-KIPS } COLUMN STRIP
69% EXTERIOR NEGATIVE MOMENT = 85.97 FT-KIPS } COLUMN STRIP
69% INTERIOR NEGATIVE MOMENT = 376.11 Ft-KIPS } COLUMN STRIP

COLUMN LINE BEAM WILL ACCOUNT FOR 85% OF THE COLUMN STRIP MOMENT

	BEAM MOMENT	SLAB COLUMN STRIP	SLAB MIDDLE STRIP
EXTERIOR NEGATIVE - 24 FT SPAN	73.07	12.90	38.63
POSITIVE - 24 FT SPAN	260.32	45.94	137.59
INTERIOR NEGATIVE - 24 FT SPAN	319.69	56.42	168.98

No. 4 BAR DIAMETER = 0.5"

SHORT DIRECTION POSITIVE STEEL PLACED FIRST, FOLLOWED BY LONG DIRECTION POSITIVE STEEL

#4 BAR $d = h - \left(\text{cover} + \frac{\phi}{2}\right) = 7 - \left(0.75 + \frac{0.5}{2}\right) = 6''$ & 5.5

DESIGN OF SLAB REINFORCEMENT

(1)	(2) LOCATION	(3) M_u ft-KIPS	(4) b, in	(5) d, in	(6) $M_u \times 12/b$ ft-KIPS/ft	(7) ρ	(8) A_s, in^2	(9) No. of #4 bars
24 ft SPAN TWO HALF-COLUMN STRIPS	Ex. Neg.	12.9	166	6	0.93	0.0021	2.09	11 → 12 3 rows of 4
	POS.	45.94	166	6	3.32	0.0021	2.09	11 → 12 3 rows of 4
	INT. NEG	56.42	166	6	4.08	0.0021	2.09	11 → 12 3 rows of 4
MIDDLE STRIP	EX. NEG	38.63	180	6	2.58	0.0021	2.27	12 (3 rows of 4)
	POS.	137.59	180	6	9.17	0.0049	5.29	27 (3 rows of 9)
	INT. NEG	168.98	180	6	11.27	0.006	6.48	33 (3 Rows of 11)
30 FT SPAN EXTERIOR HALF-COLUMN STRIP	NEG.	79.42	58	5.5	16.43	0.011	3.51	18 (3 rows of 6)
	POS.	39.12	58	5.5	8.09	0.005	1.60	8 → 9 (3 rows of 3)
MIDDLE STRIP	NEG.	116.2	130	5.5	10.73	0.007	5.01	25 (2 rows of 9, 1 row of 7)
	POS.	86.92	130	5.5	8.02	0.0050	3.58	18 (3 rows of 6)
INTERIOR HALF COLUMN STRIP	NEG.	79.42	58	5.5	16.43	0.0109	3.48	18 (3 rows of 6)
	POS.	42.77	58	5.5	8.85	0.0056	1.79	9 (3 rows of 3)

#4 NOT PRACTICAL

MIN. STEEL AREA EQUAL TO 0.0018 X GROSS CONCRETE AREA TO CONTROL TEMP & SHRINKAGE CRACKING

12" SLAB STRIP: $0.0018 \times 7 \times 12 = 0.151 \, in^2$

30 Ft DIRECTION $\rho_{min} = \dfrac{0.151}{5.5 \times 12} = 0.0023$

24 Ft DIRECTION $\rho_{min} = \dfrac{0.151}{6 \times 12} = 0.0021$

ρ FROM TABLE A.9

$A_s = \rho \cdot b \cdot d$

TABLE A.12 ⇒ # OF BARS

SHEAR CAPACITY OF SLAB

$$V_u = 0.3984\left(15 - \frac{14}{2 \times 12} - \frac{6}{12}\right) = 5.54 \text{ KIPS}$$

SLAB DESIGN SHEAR STRENGTH $= 0.75 \times 2\sqrt{5000} \times 12 \times \frac{6}{1000}$

$= 7.64$ KIPS ; WELL ABOVE SHEAR APPLIED AT FACTORED LOADS

EACH BEAM MUST BE DESIGNED FOR ITS SHARE OF THE TOTAL STATIC MOMENT

BEAM MOMENTS
30 FT SPAN

EXT. NEGATIVE MOMENT : 450 FT - KIPS
INT. NEGATIVE MOMENT : 450 FT - KIPS
POS. MOMENT : 221.65 FT - KIPS

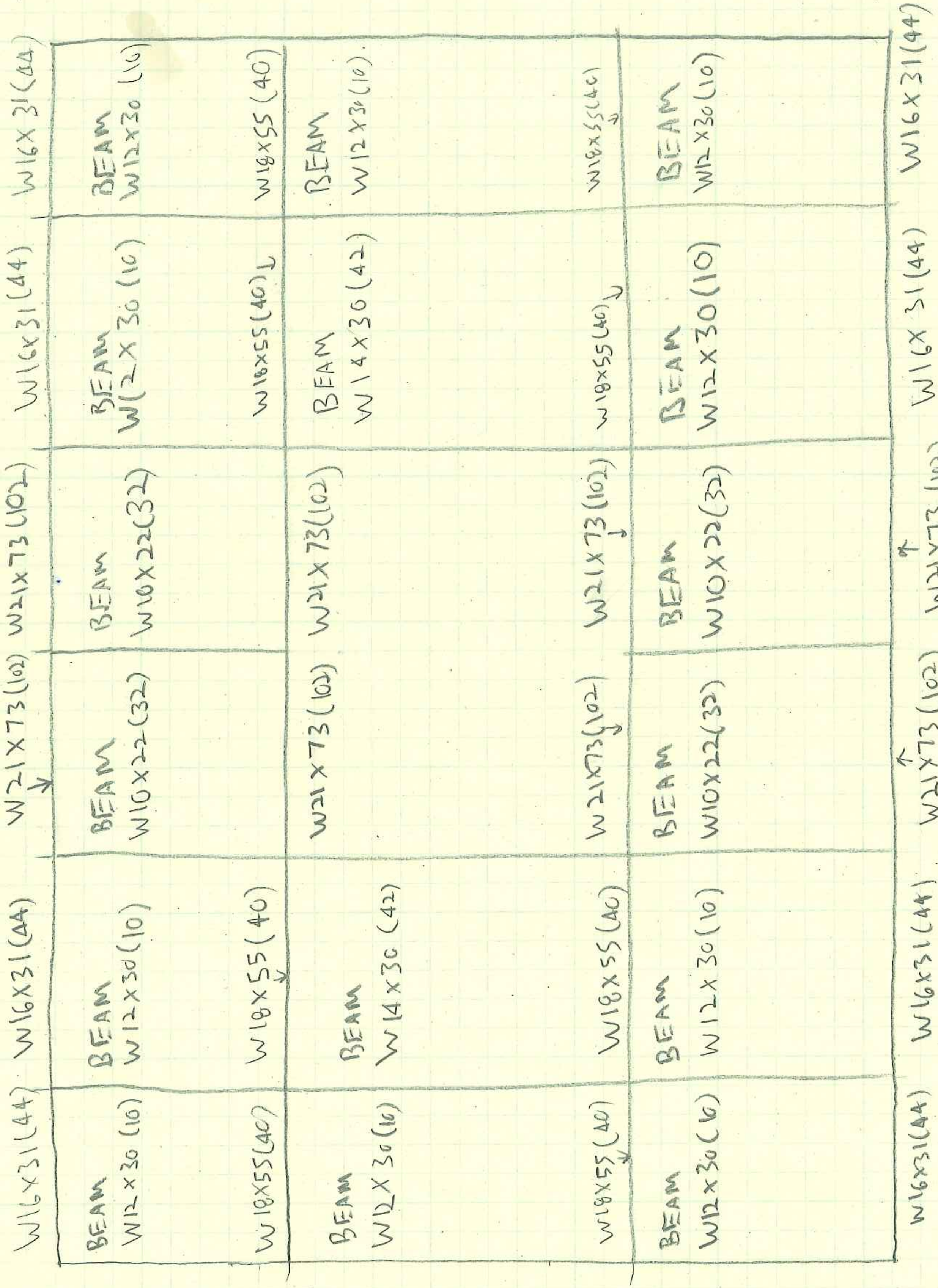
24 FT SPAN

EXT. NEGATIVE MOMENT : 73.07 FT - KIPS
INT. NEGATIVE MOMENT : 319.69 FT - KIPS
POS. MOMENT 260.32 FT - KIPS

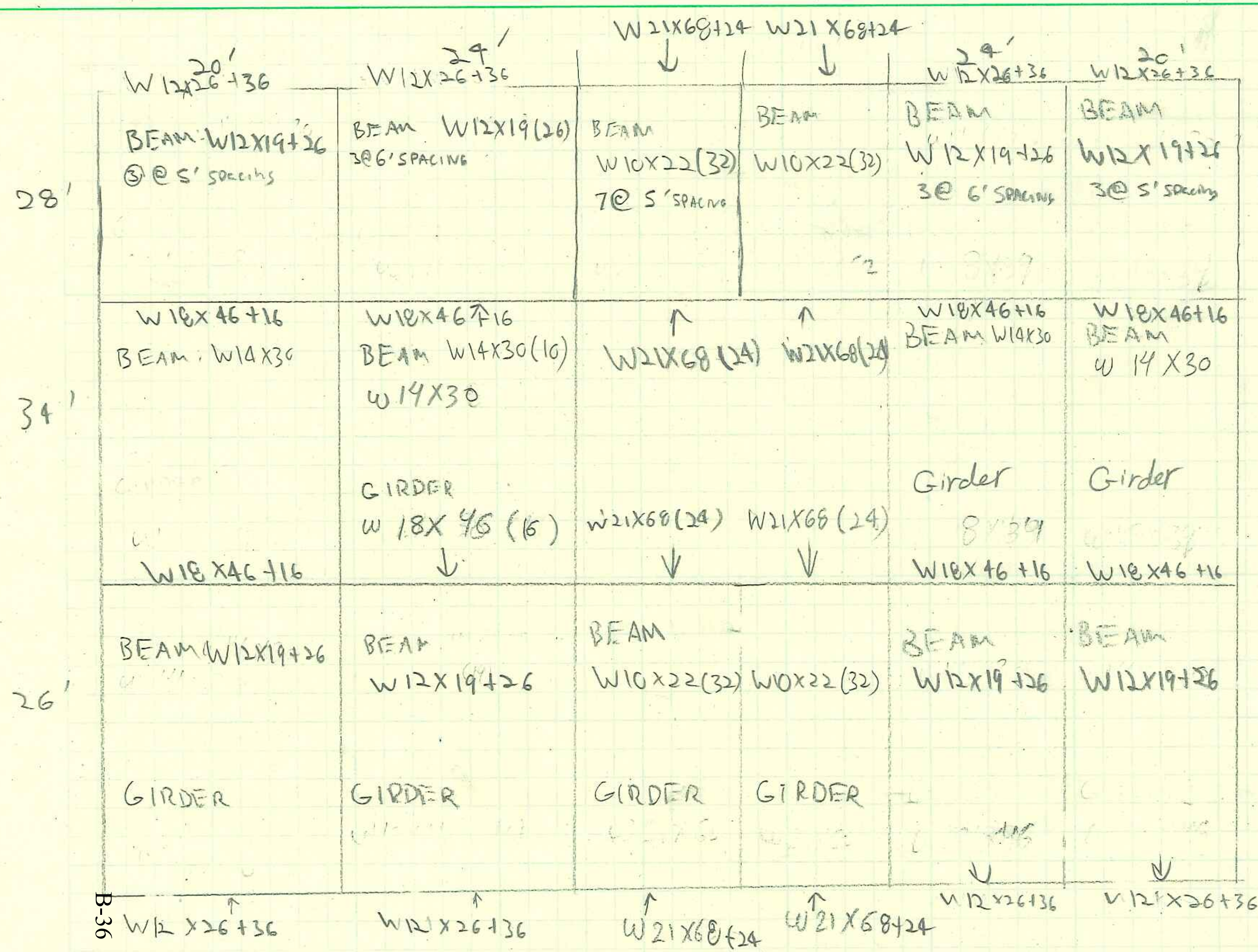
PRELIMINARY STEEL FRAMING PLAN

LEVEL 1-4 FRAMING PLAN

I



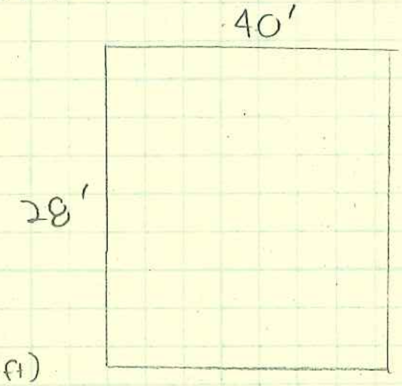
ROOF PLAN


40'
40'
W21X68+24
W21X68+24
20'
24'
24'
20'
W12X26+36
W12X26+36
W12X26+36
W12X26+36
28'
34'
26'
BEAM W12X19+26
③ @ 5' spacings
BEAM W12X19(26)
3@6' SPACING
BEAM
W10X22(32)
7@ 5' SPACING
BEAM
W10X22(32)
BEAM
W12X19+26
3@ 6' SPACING
BEAM
W12X19+26
3@ 5' SPACING
W18X46+16
BEAM: W14X30
W18X46+16
BEAM W14X30(16)
W14X30
W21X68(24)
W21X68(24)
W18X46+16
BEAM W14X30
W18X46+16
BEAM
W 14X30
GIRDER
W 18X 46 (16)
W21X68(24)
W21X68(24)
Girder
Girder
W18X46+16
W18X46+16
W18X46+16
BEAM W12X19+26
BEAM
W12X19+26
BEAM
W10X22(32)
W10X22(32)
BEAM
W12X19+26
BEAM
W12X19+26
GIRDER
GIRDER
GIRDER
GIRDER
W12X26+36
W12X26+36
W21X68+24
W21X68+24
W12X26+36
W12X26+36

LOADS
- CONCRETE : 42.9 PSF
- METAL DECK : 2.49 PSF
- ROOF DEAD : 20 PSF
- ROOF LIVE : 20 PSF
- CONSTRUCTION LIVE : 25 PSF
- INTERIOR 4TH FLOOR MEP & CEILING : 7 PSF

TOTAL DEAD = 72.4 PSF

SNOW LOAD : 55 PSF



LOAD COMBINATIONS : 1.2D + 1.6L + 0.5S

$1.2\left(72.4 \frac{lb}{ft^2} \times 5ft\right) + 1.6\left(20 \frac{LB}{FT^2} \times 5ft\right) + 0.5\left(55 \frac{lb}{ft^2} \times 5ft\right)$

7 INFILL BEAMS => 5' SPACING

= 0.732 KIP/FT

* FOR FUTURE CALCULATIONS THIS WILL GOVERN

OR 1.2D + 1.6S + 0.5L

$1.2\left(72.4 \frac{lb}{ft^2} \times 5ft\right) + 1.6\left(55 \frac{lb}{ft^2} \times 5ft\right) + 0.5\left(20 \frac{lb}{ft^2} \times 5ft\right)$

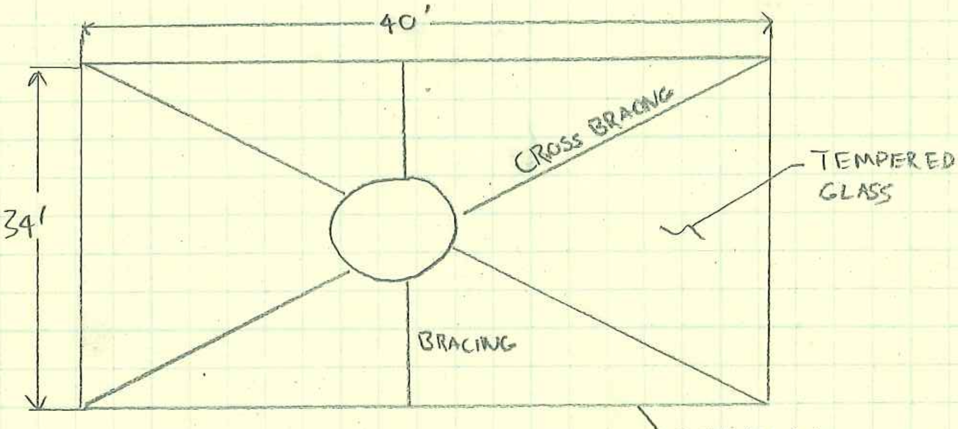
= 0.924 KIP/FT * GOVERNS

M_u = 90.59 KIP·FT => W10X12 EXCEEDS COMPACT LIMIT FOR FLEXURE => TRY W10X22

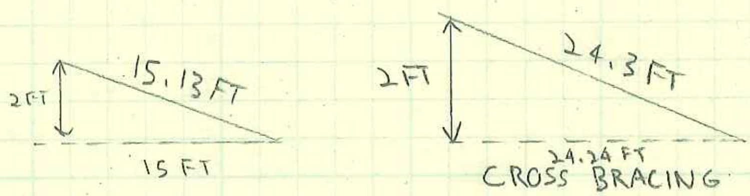
=> CONTINUED WITH EXCEL

APPROX SKYLIGHT LOAD

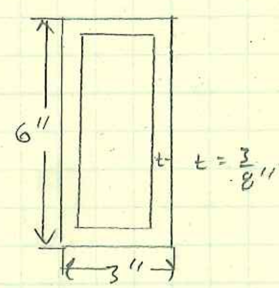
PLAN VIEW



ELEVATION



SUPPORTS ARE STRUCTURAL STEEL ⇒ APPROXIMATE CROSS SECTION CONSERVATIVELY



CIRCLE LENGTH =
$2\pi R$
$= 4 \cdot \pi = 12.56$ FT

STEEL DENSITY : $506 \frac{lb}{FT^3}$

GLASS DENSITY : $156 \frac{lb}{FT^3}$

$6" - 2 \cdot \frac{3}{8}" \times 3" - 2 \cdot \frac{3}{8}"$

$= 5.25" \times 2.25" = 11.81 \text{ in}^2$

$6" \times 3" = 18"$

NET STEEL AREA = 6.19 in^2

WT. CROSS BRACING :

$24.3 \text{FT} \times 4 \text{ PIECES} \times \frac{6.19 \text{ in}^2}{144 \text{ in}^2} \times 506 \frac{lb}{FT^3} = 2114.19 \text{ lb}$

WT. BRACING

$15.13 \text{FT} \times 2 \text{ PIECES} \times \frac{6.19 \text{ in}^2}{144 \text{ in}^2} \times 506 \frac{lb}{FT^3} = 658.18 \text{ lb}$

WT. CIRCLE

$12.56 \text{ FT} \times 1 \text{ PIECE} \times \frac{6.19 \text{ in}^2}{144 \text{ in}^2} \times 506 \frac{lb}{Ft^3} = 273.19 \text{ lb}$

WT. GLASS

$40' \times 34' \times \frac{1"}{12"/FT} \times 156 \frac{lb}{Ft^3} = 17680 \text{ lb}$
CONSERVATIVE *

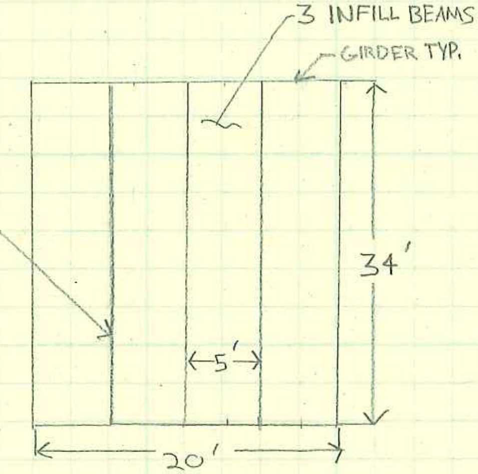
TOTAL WT. = 20726 lb

UNIFORM LOAD SKYLIGHT = $15.24 \frac{lb}{ft^2}$

* SINCE THIS WILL TAKE PLACE OF ROOF DEAD LOAD WHERE THERE IS NO ROOF, CURRENT DESIGN IS ADEQUATE

COMPOSITE BEAM DESIGN

MA BUILDING CODE CLASSIFICATION : TYPE I-B
∴ FLOOR CONSTRUCTION : 2 HOUR FIRE RATING
TABLE 720.1 OF 780 CMR GIVES MIN $t_s = 2''$

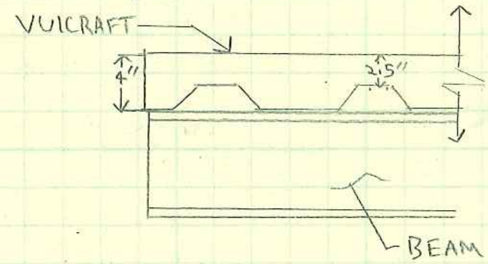


SPACING = 5' O.C.

USE VULCRAFT METAL DECKING
FOR 5' SPAN LENGTH USE 1.5VL19
⇒ LIVE LOAD CAPACITY OF 400 PSF
- UNSHORED CONSTRUCTION

VULCRAFT CONC. WT. = 39 PSF
INCREASE 10% FOR PONDING = 42.9 PSF

$f'_c = 5\text{ KSI}$



TOTAL SLAB DEPTH = 4"

LOADS

- CONCRETE : 42.9 PSF
- METAL DECK : 2.49 PSF
- MEP : 5.0 PSF
- CEILING : 2.0 PSF
- OCCUPANCY LIVE : 150 PSF
- CONSTRUCTION LIVE : 25 PSF

$W_u = 1.2D + 1.6L$

$W_u = 1.2(52.4 \frac{lb}{ft^2} \times 5ft) + 1.6(150 \frac{lb}{ft^2} \times 5ft)$

$W_u = 1514.4 \frac{lb}{ft} \Rightarrow M_u = \frac{W_u \cdot l^2}{8} = \frac{\frac{1514.4 \frac{lb}{ft}}{1000 \frac{lb}{kip}} \cdot (34ft)^2}{8}$

$M_u = 218.83\ K \cdot ft$

$b_e \leq 2 \cdot \frac{\text{LENGTH}}{8} = 2 \cdot \frac{34 \times 12}{8} = 102''$

$b_e \leq 2 \cdot \frac{\text{SPACING}}{2} = 2 \cdot \frac{5 \times 12}{2} = 60''$

LESSER VALUE GOVERNS ⇒ $b_e = 60''$

FULL COMPOSITE DESIGN

ASSUME $a = 2''$

W12x26 $I_x = 204\ in^4$

$Y_2 = t_s - \frac{a}{2} = 4 - \frac{2}{2} = 3''$

FROM TABLE 3-19, $\phi M_n = 261$ KIP·FT ⇒ W12X26

VERIFY THAT THE LOCATION OF THE COMPRESSIVE STRENGTH BLOCK IS WITHIN THE CONCRETE SLAB

$$a = \frac{\Sigma Q_n}{0.85 \cdot f'_c \cdot b_e} = \frac{383}{0.85 \cdot 5ksi \cdot 60''} = 1.5''$$

COMPUTE ACTUAL $\phi_b M_n$: $Y_2 = 4 - \frac{1.5}{2} = 3.25''$

LINEAR INTERPOLATION USING TABLE 3-19

$X_1 : 3.0 \quad Y_1 : 261$
$X_2 : 3.25 \quad Y_2 : \phi_b M_n$
$X_3 : 3.5 \quad Y_3 : 275$

$$\phi_b M_n = Y_1 + \frac{(X_2 - X_1)(Y_3 - Y_1)}{(X_3 - X_1)} = 267.97 \text{ ft·KIP}$$

$> M_U = 218.83$ ft·kip

∴ OK

CHECK BEAM STRENGTH BEFORE CONCRETE HARDENS

- CONCRETE TREATED AS LIVE LOAD

$$W_U = 1.2(\underbrace{0.026 \tfrac{KIP}{Ft}}_{\text{BEAM WT.}} + \underbrace{0.00249 \tfrac{KIP}{FT^2}}_{\text{DECK WT.}} \cdot 5ft) + 1.6(\underbrace{0.0429 \tfrac{KIP}{FT^2}}_{\text{CONC. WT.}} \times 5FT + \underbrace{0.025 \tfrac{KIP}{FT^2}}_{\text{CONS LIVE LOAD}} \times 5FT)$$

$$W_{U\,cons} = 0.589 \frac{KIP}{FT} \Rightarrow M_U = \frac{0.589 \frac{KIP}{FT} \cdot (34FT)^2}{8} = 85.11 \text{ FT-KIP}$$

< 140 KIP·ft OK

$\phi_b M_p = 140$ KIP·FT } TABLE 3-19

DEFLECTION CHECKS : UNFACTORED LOADS

$$\Delta_{CONSTRUCTION} = \frac{5 W_{cons} \ell^4}{384 \cdot E \cdot I} = \frac{\frac{5 \cdot 0.377 \frac{KIP}{FT}}{12} \times \left(34' \times \frac{12''}{FT}\right)^4}{384 \cdot 29000 \cdot \underbrace{204\ in^4}_{I_x}}$$

$$W_{cons} = \underbrace{0.026}_{\text{BEAM}} + 0.025 \frac{KIP}{FT^2} \times 5FT + 0.0429 \frac{KIP}{FT^2} \times 5FT + 0.00249 \frac{KIP}{FT^2} \times 5FT$$

$W_{cons} = 0.377 \frac{KIP}{FT}$ $\quad \Delta_{CONS} = 1.92'' > MAX = 1.75''$

∴ W12 X 26 FAILS

PICK NEW SECTION

TRY W12X30

$Y_2 = 3 \Rightarrow$ PROVIDES 302 FT·KIP

$$a = \frac{\Sigma Q_n}{0.85 \cdot F'c \cdot be} = \frac{440\ KIP}{0.85 \cdot 5KSI \cdot 60''} = 1.73''$$

$$Y_2 = 4 - \frac{1.73}{2} = 3.14''$$

LINEAR INTERPOLATION USING TABLE 3-19

$X_1 : 3.0$ $Y_1 : 285$
$X_2 : 3.14$ $Y_2 : \phi M_n$
$X_3 : 3.5$ $Y_3 : 302$

$\phi M_n = 306.39 > M_u = 218.82$ FT·KIP ∴ OK

CHECK BEAM STRENGTH BEFORE CONCRETE HARDENS
○CONCRETE TREATED AS LIVE LOAD

$$W_u = 1.2\left(0.030\frac{KIP}{FT} + 0.00249\frac{KIP}{FT^2} \cdot 5FT\right) + 1.6\left(0.0429\frac{KIP}{FT^2} \cdot 5FT + 0.025\frac{KIP}{FT^2} \cdot 5FT\right)$$

$$W_{u_{CONS}} = 0.594\frac{KIP}{FT} \Rightarrow M_u = \frac{0.594\frac{KIP}{FT} \cdot (34FT)^2}{8} = 85.85\ FT \cdot KIP$$

$\phi_b M_p = 162$ FT·KIP $> M_u = 85.85$ FT·KIP ∴ OK

DEFLECTION CHECKS : UNFACTORED LOADS

$$\Delta_{CONS} = \frac{5 W_{cons} l^4}{384 EI} = \frac{5 \cdot \frac{0.3819\frac{KIP}{FT}}{12} \times \left(34' \times \frac{12''}{FT}\right)^4}{384 \cdot 29000 \cdot 238\ in^4} = 1.66'' < 1.75''$$
∴ OK

$$W_{CONS} = 0.030 + 0.025\frac{KIP}{FT^2} \times 5FT + 0.0429\frac{KIP}{FT^2} \times 5FT + 0.0249\frac{KIP}{FT^2} \times 5FT$$

$$W_{CONS} = 0.3819\frac{KIP}{FT}$$

$$\Delta_{LL50\%} = \frac{5 \cdot \frac{0.375\frac{KIP}{FT}}{12} \cdot \left(34' \times \frac{12''}{FT}\right)^4}{384 \cdot 29000 \cdot 617.53} = 0.630'' < 1.00''\ MAX$$
∴ OK

$I_{LOWER BOUND}$ - INTERPOLATION = 617.53 in^4

$$\Delta_{DL+50\%LL} = \frac{5 \cdot \frac{0.6369\frac{KIP}{FT}}{12} \cdot \left(34' \times \frac{12''}{FT}\right)^4}{384 \cdot 29000 \cdot 617.53} = 1.119'' < \frac{L}{240} = 1.7$$
∴ OK

$$W_{DL+50\%LL} = \underset{\text{CONC WT.}}{0.0429\frac{KIP}{FT^2} \times 5FT} + \underset{\text{MEP+CEILING+METAL DECK}}{0.00949\frac{KIP}{FT^2} \times 5} + \underset{\text{50\% LL}}{0.375\frac{KIP}{FT}} + \underset{\text{BEAM WT}}{0.03\frac{KIP}{FT}}$$

CHECK IN SERVICE CAPACITY :

$$W_u = 1.2\left(0.0429\frac{KIP}{FT^2} \times 5FT + 0.00949\frac{KIP}{FT^2} \times 5FT + 0.03\frac{KIP}{FT}\right) + 1.6\left(0.15\frac{KIP}{FT^2} \times 5FT\right)$$

$$W_u = 1.55\frac{KIP}{FT} \Rightarrow M_u = \frac{1.55\frac{KIP}{FT} \cdot 34'^2}{8} = 224\ KIP \cdot FT$$

$$\phi_b M_n = 306.39 > M_u = 224\ KIP \cdot FT \therefore ok$$

FULLCOMPOSITE SHEAR STUD DESIGN

$$\frac{\Sigma Q_n}{Q_n} = \frac{440}{21.5} = 20.47 \times 2 \Rightarrow 41\ STUDS$$

$Q_n = 21.5$

$1\frac{3}{4}$" STRONG STUD PER RIB

$$MIN\ SPACING = 6D = 6 \cdot \frac{3}{4}'' = 4.5''$$

$$MAXSPACING = 8 \cdot t_s \text{ or } 36'' = 8 \cdot 4'' = 32''$$

$$S = \frac{L}{\#STUDS + 1} = \frac{34 \cdot 12}{48 + 1} = 8.33''$$

CHECK PARTIAL COMPOSITE

$$\frac{\Sigma Q_n}{Q_n} = \frac{110}{21.5} = 5.11 \Rightarrow 10\ STUDS$$

$$PNA\ 7 \Rightarrow \Sigma Q_n = 110$$

$$a = \frac{\Sigma Q_n}{0.85 \cdot f'c \cdot be} = \frac{110\ KIP}{0.85 \cdot 5KSI \cdot 60''} = 0.431''$$

$$Y_2 = 4 - \frac{0.431}{2} = 3.784''$$

$$SPACING = \frac{34 \cdot 12}{10 + 1} = 36.4''\ SPACING \Rightarrow 36''\ SPACING$$

NEW I_{LB} INTERPOLATION = 839.78 in^4

$$\Delta_{LL\ 50\%} = \frac{5wl^4}{384EI} = 0.463'' < 1''\ MAX$$

$$\Delta_{DL+50\%LL} = \frac{5wl^4}{384EI} = 0.823'' < \frac{L}{240} = 1.7'' \therefore ok$$

NEW ϕM_N INTERPOLATION = 350.84 > M_u = 218.82 FT·KIP

$$\boxed{BEAM\ DESIGN\ BAY\ B = W12X30\ (10)\ \tfrac{3}{4}''\ SPACED\ 36''\ APART}$$

* AMENDED TO W14 X 30 TO REDUCE VARIABILITY

TRIBUTARY WIDTH : 31 FT } GIRDER THAT SPANS COLUMNS B1 & B2 + B6 & B7
GIRDER THAT SPANS C1 & C2 HAS TRIBUTARY WIDTH OF 30 FT ∴ OVER DESIGN

BEAM WT. $30 \frac{lb}{ft}$

BEAM SPACING : 5 FT

GIRDER LENGTH = 20 FT

BEAM LOAD = $\frac{30 \frac{lb}{ft}}{5 ft} = 6 \frac{lb}{ft^2}$

DEAD LOAD, BEAM LOAD

$$W_u = 1.2\left(52.4 \frac{lb}{ft^2} \times 31 FT + 6 \frac{lb}{ft^2} \times 31 ft\right) + 1.6\left(150 \frac{lb}{ft^2} \times 31 FT\right)$$

$$W_u = 9.612 \frac{Kip}{FT}$$

$$M_u = \frac{w_u l^2}{8} = \frac{9.612 \ Kip/ft * (20 \ ft)^2}{8} = 480.61 \ Kip.ft$$

$$be = \frac{2 * length}{8} = \frac{2 * 20'}{8} = 5 \ ft = 60 \ in \Leftarrow \text{governs}$$

$$be = 2 * \frac{spacing}{2} = \frac{2 * 31 * 12}{2} = 372 \ in$$

assume $a = 2''$

$$Y_2 = t_s - \frac{a}{2} = 4 - \frac{2}{2} = 3''$$

Table 3-19 W 18 x 40 Provides $\phi M_n = 529$ Kip.ft

$$\Sigma Q_n = 590$$

$\phi M_n > M_u$ ∴ OK

VERIFY THAT THE LOCATION OF THE LOCATION OF THE COMPRESSIVE STRENGTH BLOCK IS WITHIN THE SLAB

$$a = \frac{\Sigma Q_n}{0.85 \cdot f'_c \cdot be} = \frac{590}{0.85 \cdot 5ksi \cdot 60''} = 2.31''$$

ACTUAL $Y_2 = 4 - \frac{2.31}{2} = 2.84''$

$\phi_b M_n$ Interpolation = 522.10 KIPFT > MU = 482.45 Kip.ft ∴ OK

2.5 507
2.84 $\phi_b M_n$
3.0 529

CHECK STRENGTH BEFORE CONCRETE HARDENS

$$W_U = 1.2\left(0.040\,\frac{KIP}{FT} + 0.00249\,\frac{KIP}{FT^2}\cdot 31\,ft + 0.066\,\frac{KIP}{FT^2}\cdot 31\,FT\right) + 1.6\left(0.0429\,\frac{KIP}{FT^2}\cdot 31\,FT + 0.025\,\frac{KIP}{FT^2}\cdot 31\,FT\right)$$

BEAMS (0.066) — METAL DECK (0.00249)

$$W_{U_{cons}} = 3.73\,\frac{KIP}{FT} \Rightarrow M_{U_{cons}} = \frac{3.73\,\frac{KIP}{FT}\cdot(20\,FT)^2}{8} = 186.58\,KIP.FT$$

$< \phi_b M_p = 294$
TABLE 3-19

DEFLECTION CHECKS - UNFACTORED LOADS

$$\Delta_{CONS} = \frac{5\,W_{cons}\,l^4}{384\,EI} = \frac{5\cdot\frac{2.41\frac{KIP}{FT}}{12}\cdot\left(20'\times\frac{12''}{FT}\right)^4}{384\cdot 29000\cdot 612\,in^4} = 0.488'' < 1.75''\ MAX$$

($612\,in^4 \leftarrow I_x$)

$$W_{cons} = 0.006\,\frac{KIP}{FT^2}\cdot 31\,FT + 0.00299\,\frac{KIP}{FT^2}\cdot 31\,FT + 0.0429\,\frac{KIP}{FT^2}\cdot 31\,FT + 0.025\,\frac{KIP}{FT^2}\cdot 31\,FT + 0.04\,\frac{KIP}{FT} = 2.41\,\frac{KIP}{FT}$$

$$\Delta_{LL50\%} = \frac{5\cdot\frac{2.325}{12}\cdot\left(20'\times\frac{12''}{FT}\right)^4}{384\cdot 29000\cdot 1424.9\,in^4} = 0.203'' < 1''\ MAX$$

($1424.9\,in^4 \leftarrow I_x$ Lower bound)

$$\frac{150\,\frac{lb}{FT^2}}{2} = 0.075\,\frac{KIP}{FT^2}\times 31\,FT = 2.325\,\frac{KIP}{FT}$$

$$\Delta_{LL50\%+DL} = \frac{5\cdot\frac{4.175}{12}\cdot\left(20'\times\frac{12''}{FT}\right)^4}{384\cdot 29000\cdot 1424.9\,in^4} = 0.364'' < \frac{L}{240} = 1''$$

$\therefore$ OK

($1424.9\,in^4 \leftarrow I_x$ Lowerband)

$$W_{DL+50\%LL} = 4.175\,\frac{KIP}{FT}$$

$$W_{DL} = 52.4\,\frac{lb}{FT^2}\times 31\,FT + 6\,\frac{lb}{FT^2}\times 31\,FT + 40\,\frac{lb}{FT} = 1.8504\,\frac{KIP}{FT}$$

$$\frac{L}{240} = \frac{20'\times 12''}{240} = 1''$$

CHECK IN SERVICE CAPACITY

$$W_U = 1.2\left(52.39\,\frac{lb}{FT^2}\times 31\,FT + 6\,\frac{lb}{FT^2}\times 31\,FT + \frac{40\,lb}{FT}\right) + 1.6\left(150\,\frac{lb}{FT^2}\times 31\,FT\right) = 9.66\,\frac{KIP}{FT}$$

$$M_U = \frac{9.66\,\frac{KIP}{FT}\cdot 20\,FT^2}{8} = 483.01\,KIP.FT < \phi_b M_N = 522.10\,KIP.FT$$

$\therefore$ OK

FULL COMPOSITE SHEAR STUD DESIGN

$$\frac{\Sigma Q_n}{Q_n} = \frac{590\ K}{17.2} = 34.3 \implies 68 \text{ studs}$$

MIN SPACING $= 6D = 6 \cdot \frac{3}{4}'' = 4.5''$

MAX SPACING $= 8 \cdot t_s = 32''$ or $36''$

use 6" spacing

Partial composite..

$\phi M_n = 487.96$ FT-KIPS

$\Sigma Q_n = 511$

$\# \text{ of studs} = \frac{\Sigma Q_n}{17.2} = 29.7$

use 60 studs

GIRDER DESIGN = W 18x40 (60)

- USE WAUSAU 3" FACE SUPERWALL

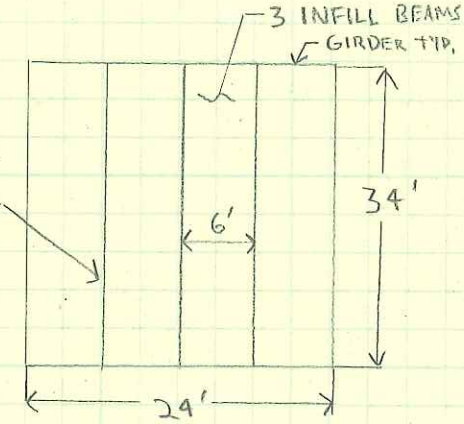
- MULLION HEIGHT : 180"

- MULLION WIDTH : 54"

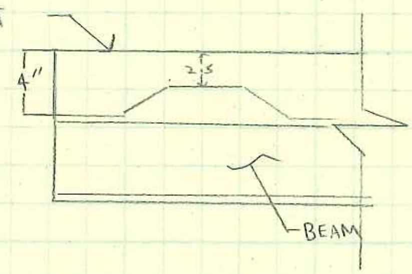
- DESIGN FOR L/240 + 1/4" DEFLECTION

- GRAVITY LOAD OF CURTAIN WALL SUPPORTED AT LOWEST LEVEL WITH CONNECTIONS AT ALL OF THE UPPER FRAMED LEVELS PROVIDING LATERAL SUPPORT ONLY.

USE VULCRAFT METAL DECKING
6' SPAN LENGTH ⇒ USE 1.5 VL19
⇒ LIVE LOAD CAPACITY OF 344 PSF
° UNSHORED CONSTRUCTION
VULCRAFT CONCRETE WT. = 39 PSF
INCREASE 10% FOR PONDING = 42.9 PSF



SPACING = 6' O.C.



<u>LOADS</u>

- CONCRETE : 42.9 PSF
- METAL DECK : 2.49 PSF
- MEP : 5.0 PSF
- CEILING : 2.0 PSF
- OCCUPANCY LIVE : 150.0 PSF
- CONSTRUCTION LIVE : 25.0 PSF

$$W_U = 1.2D + 1.6L \Rightarrow 1.2\left(52.4\,\frac{lb}{ft^2} \times 6FT\right) + 1.6\left(150\,\frac{lb}{FT^2} \times 6FT\right)$$

$$W_U = 1.817\,\frac{KIP}{FT} \Rightarrow M_U = \frac{W_U \cdot l^2}{8} = \frac{1.817\,\frac{KIP}{FT} \cdot (34FT)^2}{8}$$

$$M_U = 262.59\ KIP \cdot FT$$

$$b_e \le 2 \cdot \frac{LENGTH}{8} = 2 \cdot \frac{34 \times 12}{8} = 102''$$

$$b_e \le 2 \cdot \frac{SPACING}{2} = 2 \cdot \frac{6 \times 12}{2} = 72''$$

LESSER VALUE GOVERNS ⇒ $b_e = 72$

FULL COMPOSITE DESIGN
ASSUME $a = 2''$

$$Y_2 = t_s - \frac{a}{2} = 4 - \frac{2}{2} = 3''$$

FROM TABLE 3-19 $\phi M_n = 302$ KIP·FT ⇒ W12 X 30

VERIFY LOCATION OF COMPRESSIVE STRENGTH BLOCK

$$a = \frac{\Sigma Q_n}{0.85 \cdot f'_c \cdot b_e} = \frac{383}{0.85 \cdot 5KSI \cdot 72''} = 1.44''$$

COMPUTE ACTUAL $\phi_b M_n$: $Y_2 = 4 - \frac{1.44}{2} = 3.28''$

LINEAR INTERPOLATION USING TABLE 3-19

$X_1 : 3$ $\quad Y_1 : 261$
$X_2 : 3.28$ $\quad Y_2 : \phi_b M_n$ $\Rightarrow \phi_b M_n = 310.99$ KIP.FT $> M_U = 262.59$ KIP.FT
$X_3 : 3.5$ $\quad Y_3 : 275$

$\therefore$ OK

CHECK BEAM STRENGTH BEFORE CONCRETE HARDENS

$$W_U = 1.2\left(0.030\,\frac{KIP}{FT} + 0.00249\,\frac{KIP}{FT^2} \times 6FT\right) + 1.6\left(0.0429\,\frac{KIP}{FT^2} \times 6FT + 0.025\,\frac{KIP}{FT^2} \times 6FT\right)$$

BEAM — METAL DECK — CONCRETE — CONS LIVE LOAD

$$W_{U\,CONS} = 0.706\,\frac{KIP}{FT} \Rightarrow M_U = \frac{0.706\,\frac{KIP}{FT} \cdot (34\,FT)^2}{8} = 101.98 \text{ KIP.FT}$$

$< \phi_b M_p = 162$ KIP.FT
$\therefore$ OK

DEFLECTION CHECKS : UNFACTORED LOADS

$$\Delta_{CONS} = \frac{5\,W_{cons}\,l^4}{384EI} = \frac{5 \cdot \frac{0.452\,\frac{KIP}{FT}}{12} \cdot \left(34' \cdot \frac{12''}{FT}\right)^4}{384 \cdot 29000 \cdot \underbrace{204\,in^4}_{I_x}}$$

$$W_{CONS} = 0.030 + 0.025\,\frac{KIP}{FT^2} \cdot 6FT + 0.0429\,\frac{KIP}{FT^2} \cdot 6FT + 0.00249\,\frac{KIP}{FT^2} \cdot 6FT$$

$$= 0.452\,\frac{KIP}{FT}$$

$\Delta_{CONS} = 1.97'' > 1.75''$ MAX $\therefore$ FAIL

TRY BIGGER SECTION

TRY W14X30
$Y_2 = 3$ PROVIDES 329 FT·KIP

$$a = \frac{\Sigma Q_n}{0.85 \cdot f'_c \cdot b_e} = \frac{443}{0.85 \cdot 5KSI \cdot 72''} = 1.45'' \qquad Y_2 = 4 - \frac{1.45}{2} = 3.275''$$

LINEAR INTERPOLATION USING 3-19

3	325
3.28	?
3.5	370

$\Rightarrow \phi M_n = 337.84$ KIP·FT $> M_U = 262.59$ KIP·FT $\therefore$ OK

SAME BEAM WEIGHT AS PREVIOUS TRIAL $\therefore M_U = 101.98$ KIP·FT < 177 KIP.FT
$\therefore$ OK

DEFLECTION CHECKS : UNFACTORED LOADS

$$\Delta_{CONS} = \frac{5 \cdot \frac{0.452}{12} \cdot \left(34' \cdot 12''\right)}{384 \cdot 29000 \cdot \underbrace{291\,in^4}_{I_x}} = 1.61'' < 1.75'' \text{ MAX} \therefore \text{OK}$$

$$\Delta_{LL\,50\%} = \frac{5 \cdot \frac{0.45\,\frac{KIP}{FT}}{12} \cdot \left(34' \cdot 12\frac{''}{FT}\right)^4}{384 \cdot 29000 \cdot 749.85\,in^4} = 0.62'' < 1''\,MAX$$

$\therefore$ OK

$\uparrow I_{LB}$

$$\Delta_{DL+50\%LL} = \frac{5 \cdot \frac{0.794\,\frac{KIP}{FT}}{12} \cdot \left(34' \cdot 12\frac{''}{FT}\right)^4}{384 \cdot 29000 \cdot 749.85\,in^4} = 1.10'' < \frac{L}{240} = 1.7''$$

$\therefore$ OK

CHECK IN SERVICE CAPACITY

$$W_U = 1.2\left(0.0429\,\frac{KIP}{FT^2} \cdot 6FT + 0.00949\,\frac{KIP}{FT^2} \cdot 6FT + 0.03\,\frac{KIP}{FT}\right) + 1.6\left(0.15\,\frac{KIP}{FT^2} \cdot 6FT\right)$$

$$W_U = 1.85\,\frac{KIP}{FT} \Rightarrow M_U = \frac{1.85\,\frac{KIP}{FT} \cdot (34')^2}{8} = 267.79\,KIP \cdot FT$$

$< \phi_b M_n = 337.84\,KIP \cdot FT$

$\therefore$ OK

FULL COMPOSITE SHEAR STUD DESIGN

$$\frac{\Sigma Q_n}{Q_n} = \frac{443\,KIP}{17.2} = 25.76 \Rightarrow 50\text{ STUDS}$$

MIN SPACING = $6D = 6 \cdot \frac{3}{4}'' = 4.5''$

MAX SPACING = $8 \cdot t_s$ or $36'' = 8'' \cdot 4'' = 32''$

use 6" Spacing

CHECK PARTIAL COMPOSITE

$$\frac{\Sigma Q_n}{Q_n} = \frac{248}{17.2} = 14.4 \Rightarrow 29\text{ studs}$$

$PNA\,4 \Rightarrow \Sigma Q_n = 248$

$$a = \frac{\Sigma Q_n}{0.85 \cdot f'_c \cdot b_e} = \frac{248}{0.85 \cdot 5\,ksi \cdot 72''} = 0.81''$$

$$Y_2 = 4 - \frac{0.81}{2} = 3.59''$$

use 6" Spacing

NEW I_{LB} INTERPOLATION = $495.11\,in^4$

$\Delta_{LL\,50\%+DL} = 1.663'' < 1.7''\,MAX \therefore OK$

$\Delta_{LL\,50\%} = 0.942'' < 1''\,MAX$

$\phi_b M_n$ INTERPOLATION = 248.55 FT·KIP < M_u = 267.79 FT·KIP

For Partial composite:

$\phi_b M_n = 286.6$ Kip·ft

$\Sigma Q_n = 248$ KIP   Table 3-19

$\# \text{ of studs} = \dfrac{\Sigma Q_n}{Q_n} = \dfrac{248}{17.2} = 14.42$

use 29 studs

at spacing 6"

Beam Design: W 14X30 (29) 3/4" studs

BAY E & L | GIRDER DESIGN | 1/1

TRIBUTARY WIDTH: 31 FT } GIRDER THAT SPANS COLUMNS B2 & B3 + B5 & B6

BEAM WT.: $30 \frac{LB}{FT}$

BEAM SPACING: 6 FT   GIRDER LENGTH = 24 FT

$$\text{BEAM LOAD} = \frac{30 \frac{LB}{FT}}{6\ FT} = 5 \frac{LB}{FT^2}$$

$$W_u = 1.2\left(52.4 \frac{LB}{FT^2} \times 31FT + 5\frac{LB}{FT} \times 31FT\right) + 1.6\left(150 \frac{LB}{FT^2} \times 31FT\right)$$

$$W_u = 9.575 \frac{KIP}{FT} \Rightarrow M_u = \frac{W_u l^2}{8} = \frac{9.575 \frac{KIP}{FT} \cdot (24FT)^2}{8} = 689.39\ KIP\cdot FT$$

$$b_e = \frac{2 \cdot LENGTH}{8} = 2 \cdot \frac{24' \times \frac{12''}{FT}}{8} = 72''$$

$$b_e = \frac{2 \cdot SPACING}{2} = 2 \cdot \frac{31' \times \frac{12''}{FT}}{2} = 372''$$

LESSER VALUE GOVERNS $b_e = 72''$

ASSUME $a = 2''$

$$Y_2 = t_s - \frac{a}{2} \Rightarrow 4 - \frac{2}{2} = 3''$$

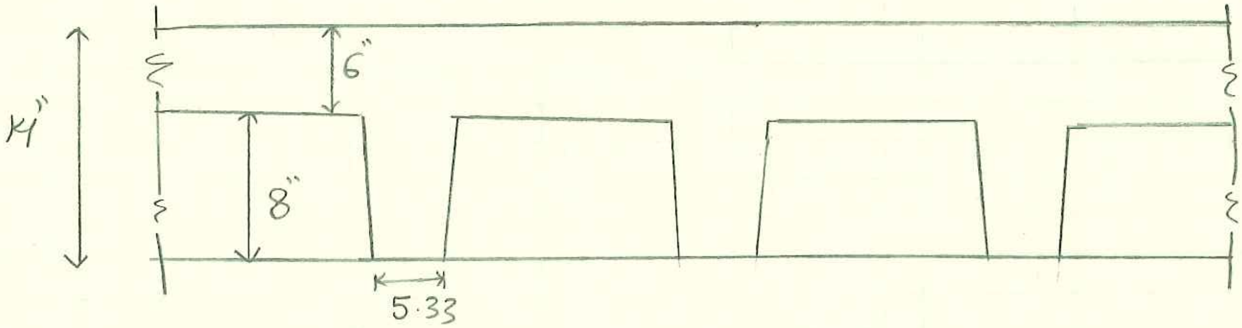
TABLE 3-19 ⇒ W18X55 PROVIDES 732 FT·KIP

⇒ USED EXCEL CALCULATIONS FOR THE REMAINING CALCULATIONS

GIRDER DESIGN: W18X55 (40) $\frac{3}{4}$" STUDS SPACED EVENLY

Concrete Slab Design

Waffle slab



From Table 6-1 [Concrete starcture book]
at $F_y = 60000\,psi$

$$h \geq \frac{l_n}{33} = \frac{30' * 12 - 18''}{33} = 10.4''$$

→ Select 6 in slab Thickness with 8" in deep Pans with Total depth 14 in.

each 30 in * 30 in * 10 in dome displaces 4.92 ft^3 of Concrete

A Typical bay Contains 80 domes

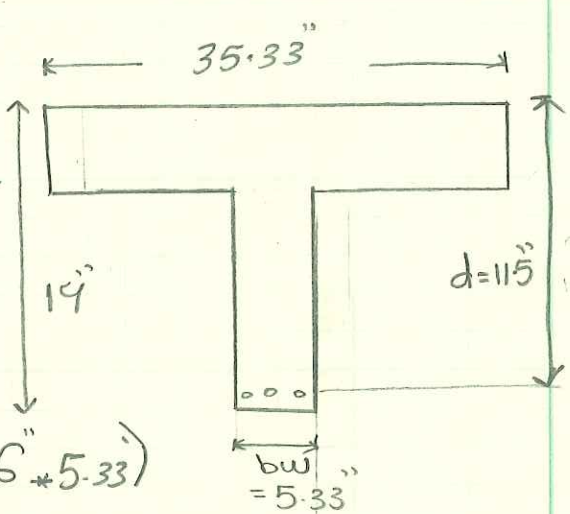
Total Volume of Concrete in the bay is:-

$$Volume = 30' * 24' * \frac{14}{12} - 80 * 4.92\,ft^3 = 446.4\,ft^3$$

The average Concrete Thickness is:-

$$t_{avg} = \frac{446.4\,ft^3}{30 * 24} * 12 = 7.44\,in$$

For Positive moment..



$M_u = 224.58$ K-ft

assume $a = h_f$

$T = C$

$A_s F_y = 0.85 f'_c a b$

$A_s * (60000 Psi) = (0.85 * 5000 Psi * 6" * 5.33")$

$A_s = 2.27 \ in^2$

$M_u = \phi M_n = \phi A_s F_y (d - a/2)$

$$A_s = \frac{M_u}{\phi F_y (d - a/2)} = \frac{(224.58 * 12) \ K \cdot in}{0.9 * 60 \ ksi * (11.5 - 3)}$$

$A_s = 5.87 \ in^2 > 2.27$ o.k

$$a = \frac{A_s F_y}{0.85 f'_c b} = \frac{5.87 in^2 * 60 ksi}{0.85 * 5 ksi * 35.33 in} = 2.35 < 6" = h_f$$

so ⇒ in The Flang

$M_n = TZ = A_s f_y (d - a/2)$

$$A_s = \frac{M_u}{\phi * F_y (d - a/2)} = \frac{(224.58 * 12) \ kip.in}{0.9 * 60 \ ksi * (11.5 - 2.35/2)}$$

$\boxed{A_s = 4.839 \ in^2}$ ⇒ use 5 #9 - bar

$A_s = 1 in^2$

$$\rho = \frac{A_s}{bd} = \frac{4.839 \ in^2}{5.33 in * 11.5 in} = 0.079$$

$$c = a/B_1 = \frac{2.35}{0.8} = 2.94$$

check for ductility

$$\varepsilon = \varepsilon_u \left(\frac{d-c}{c}\right) = 0.003 * \left(\frac{11.5 - 2.94}{2.94}\right) = 0.0087 > 0.005$$

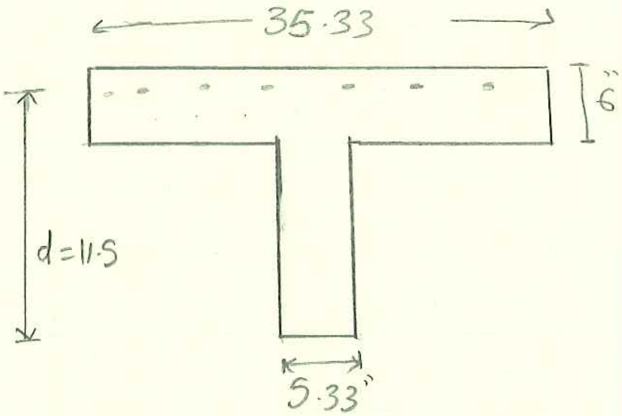
ductile

o.K

For negative moment

$M_u = 417.58$ Kips.ft

assume $a = h_f = 6''$



$$A_s = \frac{0.85 f'_c a b}{F_y}$$

$$A_s = \frac{0.85 * 5^{ksi} * 6'' * 5.33}{60 \text{ ksi}}$$

$A_s = 2.27 \text{ in}^2$

$M_u = \Phi M_n = \Phi A_s F_y (d - a/2)$

$$A_s = \frac{M_u}{\Phi F_y (d - a/2)} = \frac{417.58 * 12 \text{ Kips.in}}{0.9 * 60 * (11.5 - 3)} = 10.92 \text{ in}^2 > A_s$$

$$a = \frac{A_s F_y}{0.85 f'_c b} = \frac{10.92 \text{ in}^2 * 60 \text{ ksi}}{0.85 * 5 \text{ ksi} * 35.33 \text{ in}} = 4.36 < h_f = 6''$$

so ⇒ in The Flang

$M_n = A_s F_y (d - a/2)$

$$\frac{M_u}{\Phi} = A_s * F_y * (d - a/2)$$

$$\frac{417.58 * 12}{0.9} = A_s * 60 \text{ ksi} * \left(11.5 - \frac{4.36}{2}\right)$$

$A_s = 9.96 \text{ in}^2$

check For ductility

$$B_1 = 0.85 - 0.05 \frac{f'_c - 4000}{1000} = 0.85 - 0.05 \frac{5000 - 4000}{1000}$$

$B_1 = 0.8$

$$c = \frac{a}{B_1} = \frac{4.36}{0.8} = 5.45 \text{ in}$$

4/6

$$\varepsilon_t = \varepsilon_u \left(\frac{d-c}{c}\right) = 0.003 * \left(\frac{11.5 - 5.45}{5.45}\right)$$

$\varepsilon_t = 0.0033 < 0.005$

not ductile

$\phi = 0.483 + 83.3\,\varepsilon_t = 0.483 + 83.3 * 0.0033$

$\phi = 0.76$

$M_n = A_s F_y (d - a/2)$

$$A_s = \frac{M_u}{\phi F_y (d - a/2)} = \frac{417.58 * 12 \text{ Kip.in}}{0.76 * 60 \text{ Ksi} * (11.5 - \frac{4.36}{2})}$$

$A_s = 11.79 \text{ in}^2$

use 8 #11 bars

$A = 1.56 \text{ in}^2$

Waffle Slab For corridoL

*For POSiTiVe momenT

$M_u = 150.43 \ K.ft$

$T = C$

$A_s F_y = 0.85 f'_c a b$

$A_s * (60000 \ Psi) = 0.85 * 5000 \ Psi * 6'' * 5.33''$

$A_s = 2.27 \ in^2$

$M_u = \phi M_n = \phi A_s F_y (d - a/2)$

$A_s = \dfrac{M_u}{\phi F_y (d - a/2)} = \dfrac{150.43 * 12 \ Kip.in}{0.9 * 60 \ ksi * (11.5'' - 3'')} = 3.9 \ in^2 > 2.27 \ in^2$

$a = \dfrac{A_s F_y}{0.85 f'_c b} = \dfrac{3.9 \ in^2 * 60 \ ksi}{0.85 * 5 \ ksi * 35.33} = 1.57 < h_f = 6''$

Check For ductiliTy :-

$C = \dfrac{a}{B_1} = \dfrac{1.57}{0.8} = 1.95 \ in$

$\varepsilon_t = \varepsilon_u \left(\dfrac{d - c}{c}\right) = 0.003 * \left(\dfrac{11.5 - 1.95}{1.95}\right) = 0.015 > 0.005$ ✓

ductile → use $\phi = 0.9$

$M_n = A_s F_y (d - a/2)$

$\dfrac{M_u}{\phi} = A_s F_y (d - a/2)$

$A_s = \dfrac{(150.43 * 12) \ K.in}{0.9 * 60K * (11.5 - \frac{1.57}{2})} = \underline{\underline{3.12 \ in^2}}$

use 4 #8 bar

$A_s = 0.79$

* For negative moment:-

$M_u = 279.37$ Kip. Ft

$A_s F_y = 0.85 f'_c a b$

$A_s * 60\ ksi = 0.85 * 5 * 6'' * 5.33 \Rightarrow A_s = 2.27\ in^2$

$M_u = \phi M_n = \phi A_s F_y (d - a/2)$

$$A_s = \frac{M_u}{\phi F_y (d - a/2)} = \frac{(279.37 * 12)\ kip.in}{0.9 * 60 * (11.5 - 3)}$$

$A_s = 7.3\ in^2 > 2.27$ o.k

$$a = \frac{A_s F_y}{0.85 f'_c b} = \frac{7.3\ in^2 * 60\ ksi}{0.85 * 5ksi * 35.33\ in} = 2.92\ in < 6''$$

check for ductility:-

$$c = \frac{a}{\beta_1} = \frac{2.92}{0.8} = 3.65\ in$$

$$\varepsilon_t = \varepsilon_u \left(\frac{d-c}{c}\right) = 0.003 * \left(\frac{11.5 - 3.65}{3.65}\right) =$$

$= 0.0065 > 0.005$ ductile, $\phi = 0.9$

$$M_n = \frac{M_u}{\phi} = A_s F_y (d - a/2)$$

$$A_s = \frac{M_u}{\phi F_y (d - a/2)} = \frac{(279.37 * 12)\ kip.in}{0.9 * 60\ kip * (11.5 - \frac{2.92}{2})}$$

$A_s = 6.18\ in^2$

5 # 10 bar

$A_s = 1.27\ in^2$

For steel

Foundation design:- Soil Capacity $= 8$ tons/ft^2

$= 16$ Kips/ft^2

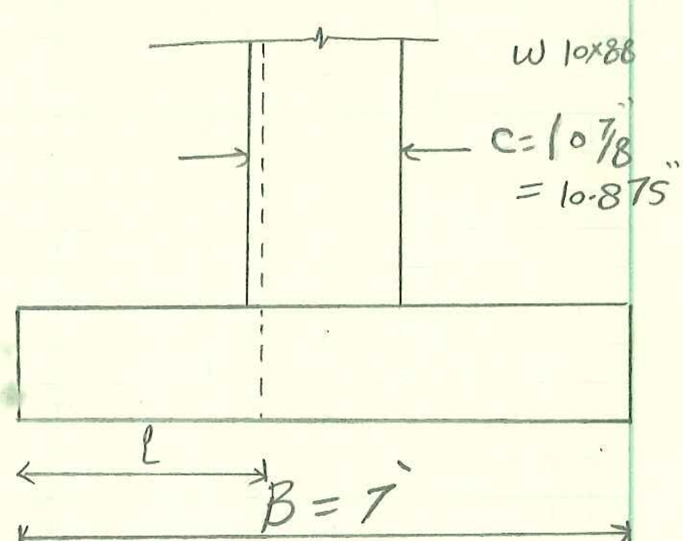
For Column The largest P_u:-

$P_u = 783.73$ Kips

$$P = \frac{F}{A} \Rightarrow A = \frac{F}{P} = \frac{783.73 \text{ Kips}}{16 \frac{\text{Kips}}{ft^2}} = 48.98 \, ft^2$$

$A = 48.98 \, ft^2$

use $7' \times 7'$ Footing



Reinforcing bars

Table 8.1
CoChuto ; Don AID P.

Thickness $= 36''$

$$l = \frac{B - C}{2} = \frac{7' - 0.9063'}{2}$$

$l = 3.05' = 36.6''$

$$M_u = \frac{P_u l^2}{2B} = \frac{783.73 * 1000 * (36.6'')^2}{2 * 7 * 12} = 6249127.136 \text{ in.lb}$$

$$A_s = \left(\frac{F'_c b}{1.176 F_y}\right)\left(d - \sqrt{d^2 - \frac{2.353 M_u}{\phi F'_c b}}\right)$$

$$= \left(\frac{3000 * 84}{1.176 * 60000}\right)\left(36 - \sqrt{(36)^2 - \frac{2.353 * 6249127.136}{0.9 * 3000 * 84}}\right)$$

$= 3.57 * 0.912 = 3.26 \, in^2$

use 6 #6 (both direction)

$A_s = 0.6\ in^2$

$A_g = 0.6 * 6 = 3.6\ in^2$

$$\text{spacing} = \frac{7*12}{6-1} = \frac{84''}{5} = 16.8''$$

use 16" spacing

-Development length

$ld_{supplied} = 36.6'' - 3'' = 33.6''$

$$\frac{c_b + K_{tr}}{d_b} = \frac{3+0}{6/8} = 4$$

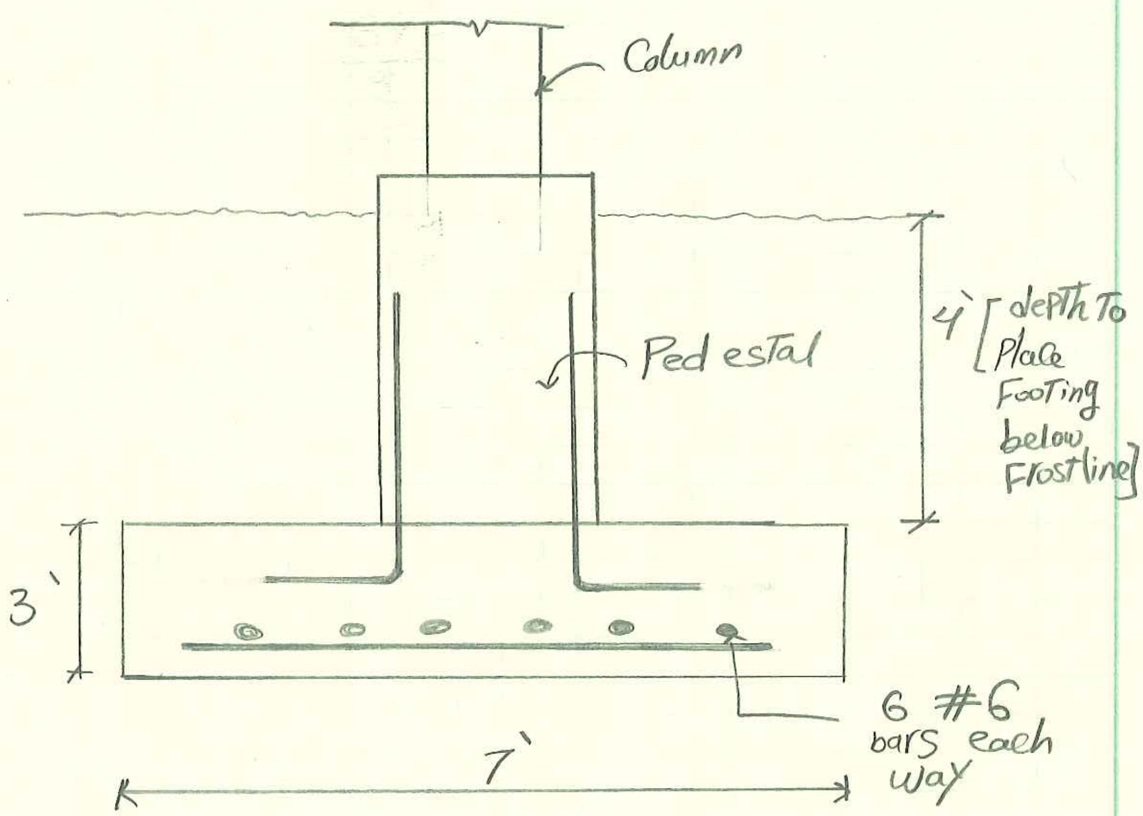
$$\frac{l_d}{d_b} = \left(\frac{3}{40} * \frac{F_y}{\lambda\sqrt{f'_c}} * \frac{\psi_e\ \psi_t\ \psi_s}{\frac{c_b + K_{tr}}{d_b}}\right)$$

$$\frac{l_d}{d_b} = \left(\frac{3}{40} * \frac{60000}{\sqrt{3000}} * \frac{1*1*1}{4}\right) = 20.54''$$

$l_d = 20.54 * 6/8 = 15.4'' < 36.6''$ o.K

use 6#6 in both direction

spacing 16"


Column
Pedestal
4' [depth To Place Footing below Frostline]
3'
6 #6 bars each way
7'

For Concrete

Foundation design:- Soil Capacity $= 8 \text{ tons}/ft^2$
$= 16 \dfrac{\text{Kips}}{ft^2}$

$$P = \frac{F}{A} \Rightarrow A = \frac{F}{P}$$

Column:- 2B, 2C, 7B, 7C

$P_u = 1371$ Kips

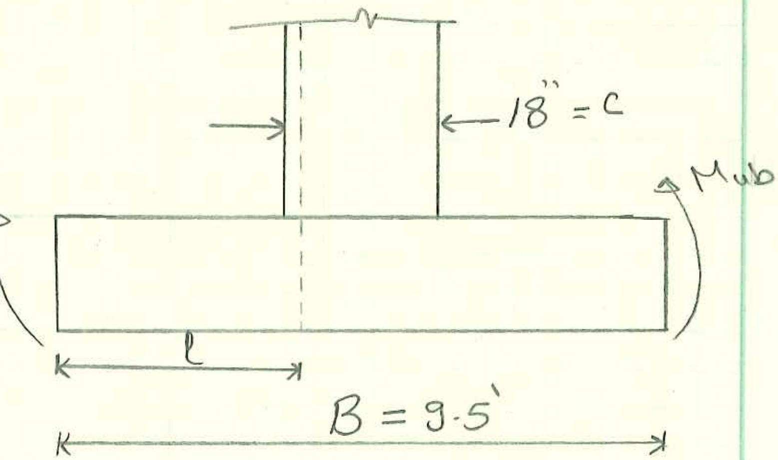
$$A = \frac{1371 \text{ Kips}}{16 \frac{\text{Kips}}{ft^2}} = 85.68 \; ft^2 \Rightarrow 86 \; ft^2$$

Use 9.5 x 9.5 Footing

Reinforcing bars

Table 8.1

Cocluto, DonALD P.



Thickness = 36"

$$l = \frac{B - c}{2}$$

$$= \frac{9.5' - 1.5'}{2} = 4' = 48''$$

$$M_u = \frac{P_u l^2}{2B} = \frac{1371 * 1000 * (48'')^2}{2 * 9.5 * 12} = 13854315.8 \text{ in.lb}$$

$$A_s = \left(\frac{F_c' b}{1.176 F_y}\right)\left(d - \sqrt{d^2 - \frac{2.353 M_u}{\phi F_c' b}}\right)$$

$$A_s = \left(\frac{3000 * 114}{1.176 * 60000}\right)\left(36 - \sqrt{36^2 - \frac{2.353 * 1385431 5.8 \text{ lb.in}}{0.9 * 3000 * 114}}\right)$$

$= 4.85 * 1.5 = 7.29 \text{ in}^2$

use 6 #10 bars (both direction)

$A_s = 1.27 \text{ in}^2$

$A_g = 1.27 * 6 = 7.62 \text{ in}^2$

$$\text{Spacing} = \frac{9.5 * 12}{6-1} = \frac{114}{5} = 22.8''$$

use Spacing 22''

Development length

l_d supplied $= 48'' - 3'' = 45''$

$$\frac{C_b + K_{tr}}{d_b} = \frac{3+0}{10/8} = 2.4$$

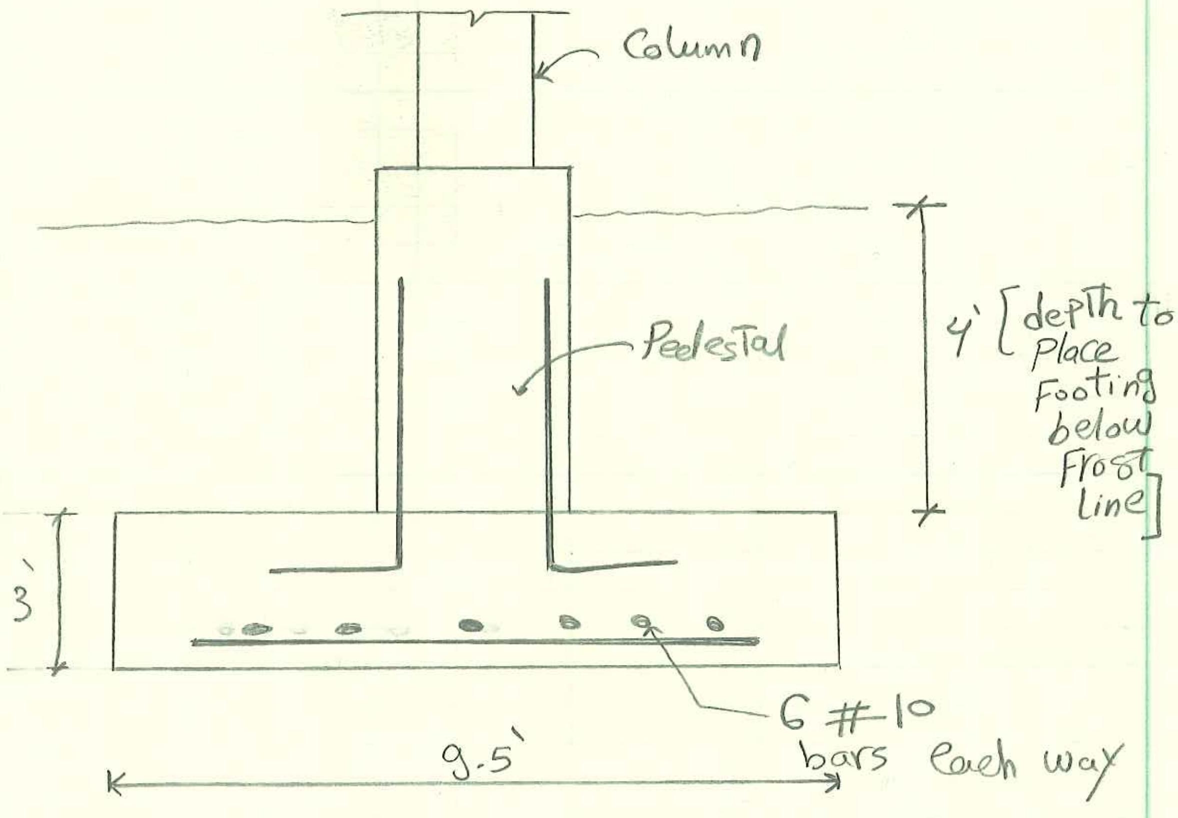
$$\frac{l_d}{d_b} = \left(\frac{3}{40} * \frac{F_y}{\lambda\sqrt{f_c'}} * \frac{\psi_e \psi_t \psi_s}{\frac{C_b + K_{tr}}{d_b}}\right)$$

$$\frac{l_d}{d_b} = \left(\frac{3}{40} * \frac{60000}{1 * \sqrt{3000}} * \frac{1*1*1}{2.4}\right) = 34.23''$$

$$l_d = 34.23'' * \frac{10''}{8} = 42.8'' < 45'' \quad \underline{\text{O.K}}$$

use 6 #10 in both direction

Spacing 22''


Column
Pedestal
4' [depth to Place Footing below Frost line]
3'
6 #10 bars each way
9.5'

Foundation design

* For Columns 1A, 8A, 1D, 8D

$P_u = 456.56$ Kips

soil Capacity = 8 tons / ft^2 = 16 Kips/ft^2

$$A = \frac{F}{P} = \frac{456.56 \text{ Kips}}{16 \text{ Kips/ft}^2} = 28.54 \text{ ft}^2$$

use 5.5 X 5.5 Footing

Reinforcing bars Table 8.1 Cocluto, Don AlD P.

Thickness 36"

$$l = \frac{B - c}{2}$$

$$= \frac{5.5' - 1.5'}{2} = 2' = 24"$$

$$M_u = \frac{P_u l^2}{2B} = \frac{456.56 \text{ Kips} * (24")^2 * 1000}{2 * 5.5" * 12}$$

$= 1,992,261.82$ in. lb.

$$A_s = \left(\frac{f'_c \, b}{1.176 F_y}\right)\left(d - \sqrt{d^2 - \frac{2.353 M_u}{\phi f'_c \, b}}\right)$$

$$A_s = \left(\frac{3000 * 66}{1.176 * 60000}\right)\left(36 - \sqrt{36^2 - \frac{2.353 * 1992261.82 \text{ lb.in}}{0.9 * 3000 * 66}}\right)$$

$A_s = 2.81 * 0.37 = 1.04 \text{ in}^2$

use 3 # 6 bars (both direction)

$A_s = 0.44 \text{ in}^2$

$A_g = 3 * 0.44 = 1.32 \text{ in}^2$

$$\text{Spacing} = \frac{5.5 * 12}{2} = 33"$$

Developement length

$l_{d\,supplied} = 24'' - 3'' = 21''$

$$\frac{C_b + K_{tr}}{d_b} = \frac{3 + 0}{0.750} = 4$$

$$\frac{l_d}{d_b} = \left(\frac{3}{40} * \frac{F_y}{\lambda\sqrt{f'_c}} * \frac{\psi_e \psi_t \psi_s}{\frac{C_b + K_{tr}}{d_b}}\right)$$

$$\frac{l_d}{d_b} = \left(\frac{3}{40} * \frac{60000}{1 * \sqrt{3000}} * \frac{1}{4}\right) = 20.54$$

$l_d = 20.54 * 0.750 = 15.4 < 21''$ o.K

use 3 # 6 both direction

For Columns 3B, 3C, 6B, 6C

$P_u = 1062.52\,Kips$

$$A = \frac{1062.52\ Kips}{16\ Kips/ft^2} = 66.41\ ft^2$$

use 8.5 * 8.5 Footing

Reinforcing bars :-

Thickness = 36"

$$l = \frac{B-C}{2} = \frac{8.5' - 1.5'}{2} = 3.5' = 42"$$

$$M_u = \frac{P_u l^2}{2B} = \frac{1062.52\ Kips * (42)^2 * 1000}{2 * 8.5 * 12} = 918767.941\ lb.in$$

$$A_s = \left(\frac{F'_c b}{1.176 F_y}\right)\left(d - \sqrt{d^2 - \frac{2.353 M_u}{\Phi F'_c b}}\right)$$

$$= \left(\frac{3000 * 102}{1.176 * 60000}\right)\left(36 - \sqrt{36^2 - \frac{2.353 * 918767.941}{0.9 * 3000 * 102}}\right)$$

$= 4.34 * 1.107 = \underline{\underline{4.8\ in^2}}$

use 5 #9

$A_s = 1\ in^2 \Longrightarrow A_g = 1 * 5 = 5\ in^2$

$$\text{spacing} = \frac{8.5 * 12}{5-1} = 25.5"$$

use spacing 25"

Development length :-

l_d supplied = 42 - 3 = 39"

$$\frac{C_b + K_{tr}}{d_b} = \frac{3+0}{1.128} = 2.7$$

$$l_d = \left(\frac{3}{40} * \frac{F_y}{\lambda \sqrt{f'_c}} * \frac{\psi_e \psi_t \psi_s}{\frac{c_b + K_{tr}}{d_b}} \right) * d_b$$

$$= \left(\frac{3}{40} * \frac{60000}{1 * \sqrt{3000}} * \frac{1*1*1}{2.7} \right) d_b = 30.43 * 1.128$$

$$= 34.33 < 39 \quad \text{o.k}$$

use 5 #9 both direction

* For Columns :

2A, 3A, 4A, 5A, 6A, 7A, 2D, 3D, 4D, 5D, 6D, 7D,
1B, 1C, 8B, 8C, 4B, 5B, 4C, 5C

$P_u = 766.56$ KiPs

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

$$A = \frac{766.56 \text{ KiPS}}{16 \text{ KiPs/ FT}^2} = 47.91 \text{ FT}^2$$

use 7 * 7 Footing

Reinforeing bars :

Thickness 36"

$$l = \frac{B - C}{2} = \frac{7 - 1.5'}{2} = 2.75' = 33''$$

$$M_u = \frac{P_u l^2}{2B} = \frac{766.56 \text{ KiPs} * 1000 * (33)^2}{2 * 7 * 12}$$

$$= 4968951.43 \text{ lb.in}$$

$$A_s = \left(\frac{f'_c b}{1.176 F_y} \right) \left(d - \sqrt{\frac{2.353 M_u}{\phi f'_c b}} \right)$$

$$A_s = \left(\frac{3000 * 84}{1.176 * 60000}\right)\left(36 - \sqrt{36^2 - \frac{2.353 * 4968951.43 \text{ lb.in}}{0.9 * 3000 * 84}}\right)$$

$$= 3.57 * 0.723 = 2.58$$

use 5 #7

$A_s = 0.6 \text{ in}^2$

$A_g = 0.6 * 5 = 3 \text{ in}^2$

$$\text{spacing} = \frac{7 * 12}{5 - 1} = 21''$$

use 20" spacing

- Development length

$l_{d\,supplied} = 33 - 3 = 30''$

$$\frac{c_b + K_{tr}}{d_b} = \frac{3 + 0}{0.875} = 3.4$$

$$\frac{l_d}{d_b} = \left(\frac{3}{40} * \frac{F_y}{\lambda\sqrt{F'_c}} * \frac{\psi_e \psi_t \psi_s}{\frac{c_b + K_{tr}}{d_b}}\right)$$

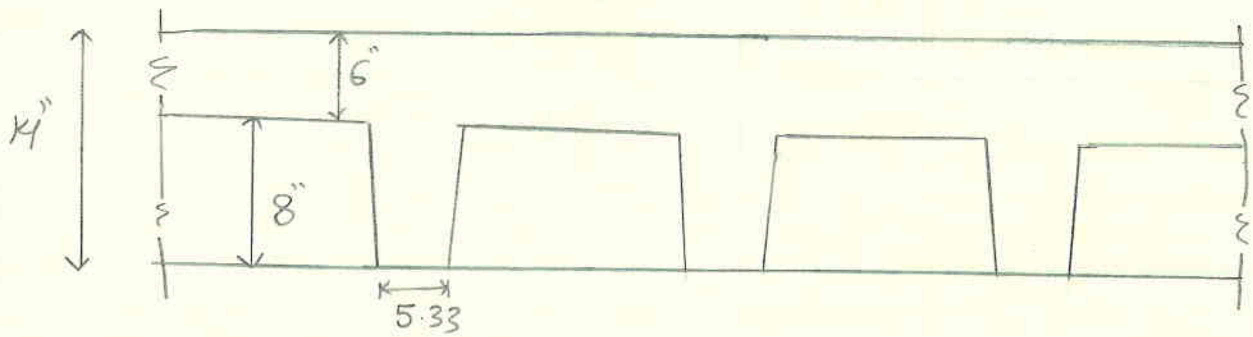
$$= \left(\frac{3}{40} * \frac{60000}{1 * \sqrt{3000}} * \frac{1 * 1 * 1}{3.4}\right) = 24.16$$

$l_d = 24.16 * 0.875 = 21.144'' < 30''$ o.k

use 5 # 7 in both direction

Concrete Slab Design

Waffle slab



From Table 6-1 [Concrete Starcture book]
at $F_y = 60000\,psi$

$$h \geq \frac{l_n}{33} = \frac{30 * 12 - 18''}{33} = 10.4''$$

→ Select 6in slab Thickness with 8" in deep Pans with Total depth 14 in.

each 30 in * 30 in * 10 in dome displaces 4.92 ft^3 of Concrete

A Typical bay Contains 80 domes

Total volume of Concrete in The bay is:-

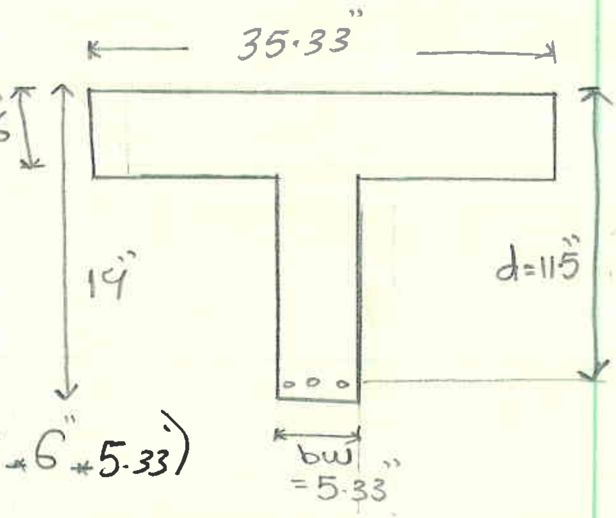
$$Volume = 30' * 24' * \frac{14}{12} - 80 * 4.92\ ft^3 = 446.4\ ft^3$$

The average Concrete Thickness is:-

$$t_{avg} = \frac{446.4\ ft^3}{30 * 24} * 12 = 7.44\ in$$

For Positive moment ..

$M_u = 224.58$ K-ft



assume $a = h_f$

$T = C$

$A_s F_y = 0.85 f'_c a b$

$A_s * (60000 Psi) = (0.85 * 5000 Psi * 6" * 5.33")$

$A_s = 2.27 \ in^2$

$M_u = \phi M_n = \phi A_s F_y (d - a/2)$

$$A_s = \frac{M_u}{\phi F_y (d - a/2)} = \frac{(224.58 * 12) \ K.in}{0.9 * 60 \ ksi * (11.5 - 3)}$$

$A_s = 5.87 \ in^2 > 2.27$ o.K

$$a = \frac{A_s F_y}{0.85 \ f'_c b} = \frac{5.87 in^2 * 60 \ ksi}{0.85 * 5 \ ksi * 35.33 \ in} = 2.35 < 6" = h_f$$

so ⇒ in The Flang.

$M_n = TZ = A_s f_y (d - a/2)$

$$A_s = \frac{M_u}{\phi * F_y (d - a/2)} = \frac{(224.58 * 12) \ kip.in}{0.9 * 60 \ ksi * (11.5 - 2.35/2)}$$

$\boxed{A_s = 4.839 \ in^2}$ ⇒ use 5 #9 - bar
$A_s = 1 in^2$

$$\rho = \frac{A_s}{bd} = \frac{4.839 \ in^2}{5.33 in * 11.5 in} = 0.079$$

$$c = a/B_1 = \frac{2.35}{0.8} = 2.94$$

check For ductility

$$\varepsilon = \varepsilon_u \left(\frac{d-c}{c}\right) = 0.003 * \left(\frac{11.5 - 2.94}{2.94}\right) = 0.0087 > 0.005$$

ductile
o.K

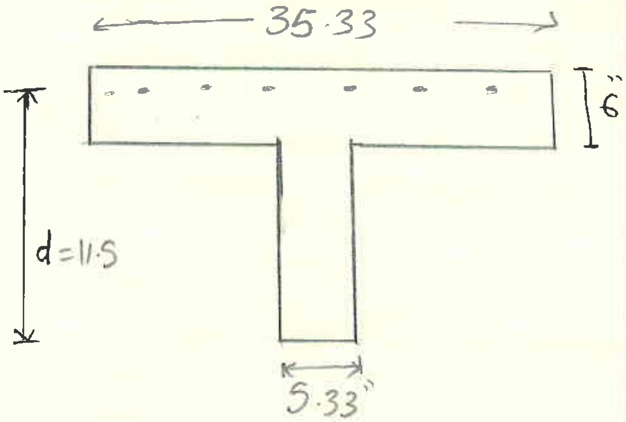
For negative moment

$M_u = 417.58$ Kips.ft

assume $a = h_f = 6''$

$$A_s = \frac{0.85 f'_c a b}{F_y}$$

$$A_s = \frac{0.85 * 5^{ksi} * 6'' * 5.33}{60 \text{ ksi}}$$



$A_s = 2.27 \text{ in}^2$

$$M_u = \Phi M_n = \Phi A_s F_y (d - a/2)$$

$$A_s = \frac{M_u}{\Phi F_y (d - a/2)} = \frac{417.58 * 12 \text{ kips.in}}{0.9 * 60 * (11.5 - 3)} = 10.92 \text{ in}^2 > A_s$$

$$a = \frac{A_s F_y}{0.85 f'_c b} = \frac{10.92 \text{ in}^2 * 60 \text{ ksi}}{0.85 * 5 \text{ ksi} * 35.33 \text{ in}} = 4.36 < h_f = 6''$$

so ⇒ in The Flang

$M_n = A_s F_y (d - a/2)$

$$\frac{M_u}{\Phi} = A_s * F_y * (d - a/2)$$

$$\frac{417.58 * 12}{0.9} = A_s * 60 \text{ ksi} * \left(11.5 - \frac{4.36}{2}\right)$$

$A_s = 9.96 \text{ in}^2$

check For ductility

$$B_1 = 0.85 - 0.05 \frac{f'_c - 4000}{1000} = 0.85 - 0.05 \frac{5000 - 4000}{1000}$$

$B_1 = 0.8$

$$c = \frac{a}{B_1} = \frac{4.36}{0.8} = 5.45 \text{ in}$$

$$\varepsilon_t = \varepsilon_u \left(\frac{d-c}{c}\right) = 0.003 * \left(\frac{11.5-5.45}{5.45}\right)$$

$\varepsilon_t = 0.0033 < 0.005$

not ductile

$\Phi = 0.483 + 83.3\,\varepsilon_t = 0.483 + 83.3 * 0.0033$

$\Phi = 0.76$

$M_n = A_s F_y (d - a/2)$

$$A_s = \frac{M_u}{\Phi F_y (d - a/2)} = \frac{417.58 * 12 \text{ Kip.in}}{0.76 * 60 \text{ Ksi} * \left(11.5 - \frac{4.36}{2}\right)}$$

$A_s = 11.79 \text{ in}^2$

use 8 #11 bars

$A = 1.56 \text{ in}^2$

Waffle Slab For corridoL

*For positive momenT

$M_u = 150.43 \quad K.ft$

$T = C$

$A_s F_y = 0.85 f'_c a b$

$A_s * (60000\ Psi) = 0.85 * 5000 Psi * 6'' * 5.33''$

$A_s = 2.27\ in^2$

$M_u = \phi M_n = \phi A_s F_y (d - a/2)$

$$A_s = \frac{M_u}{\phi F_y (d - a/2)} = \frac{150.43 * 12\ Kip.in}{0.9 * 60\ ksi * (11.5 - \frac{3}{2})} = 3.9\ in^2 > 2.27\ in^2$$

$$a = \frac{A_s F_y}{0.85 f'_c b} = \frac{3.9\ in^2 * 60\ ksi}{0.85 * 5\ ksi * 35.33} = 1.57 < h_f = 6''$$

Check for ductiLiTy :-

$$C = \frac{a}{B_1} = \frac{1.57}{0.8} = 1.95\ in$$

$$\varepsilon_t = \varepsilon_u \left(\frac{d - c}{c}\right) = 0.003 * \left(\frac{11.5 - 1.95}{1.95}\right) = 0.015 > 0.005 \checkmark$$

ductile $\rightarrow$ use $\phi = 0.9$

$M_n = A_s F_y (d - a/2)$

$$\frac{M_u}{\phi} = A_s F_y (d - a/2)$$

$$A_s = \frac{(150.43 * 12)\ K.in}{0.9 * 60K * (11.5 - \frac{1.57}{2})} = \underline{\underline{3.12\ in^2}}$$

use 4 #8 bar

$A_s = 0.79$

* For negative moment:-

$M_u = 279.37$ Kip.Ft

$A_s F_y = 0.85 f'_c a b$

$A_s * 60\,Ksi = 0.85 * 5 * 6'' * 5.33 \Rightarrow A_s = 2.27\ in^2$

$M_u = \Phi M_n = \Phi A_s F_y (d - a/2)$

$$A_s = \frac{M_u}{\Phi F_y (d - a/2)} = \frac{(279.37 * 12)\ kip.in}{0.9 * 60 * (11.5 - 3)}$$

$A_s = 7.3\ in^2 > 2.27$ o.k

$$a = \frac{A_s F_y}{0.85 f'_c b} = \frac{7.3\ in^2 * 60\ ksi}{0.85 * 5\,ksi * 35.33\,in} = 2.92\,in < 6''$$

check for ductility:-

$$c = \frac{a}{\beta_1} = \frac{2.92}{0.8} = 3.65\ in$$

$$\varepsilon_t = \varepsilon_u \left(\frac{d-c}{c}\right) = 0.003 * \left(\frac{11.5 - 3.65}{3.65}\right) =$$

$= 0.0065 > 0.005$ ductile, $\Phi = 0.9$

$$M_n = \frac{M_u}{\Phi} = A_s F_y (d - a/2)$$

$$A_s = \frac{M_u}{\Phi F_y (d - a/2)} = \frac{(279.37 * 12)\ kip.in}{0.9 * 60\,kip * (11.5 - \frac{2.92}{2})}$$

$A_s = 6.18\ in^2$

5 # 10 bar

$A_s = 1.27\ in^2$

Foundation design

* For Columns 1A, 8A, 1D, 8D

$P_u = 456.56$ Kips

soil Capacity = 8 tons / ft^2 = 16 Kips/ft^2

$$A = \frac{F}{P} = \frac{456.56 \text{ Kips}}{16 \text{ Kips}/ft^2} = 28.54 \; ft^2$$

use 5.5 X 5.5 Footing

Reinforcing bars Table 8.1 Cocluto ; Don AlD P.

Thickness 36"

$$l = \frac{B - c}{2}$$

$$= \frac{5.5' - 1.5'}{2} = 2' = 24"$$

$$M_u = \frac{P_u l^2}{2B} = \frac{456.56 \text{ Kips} * (24")^2 * 1000}{2 * 5.5" * 12}$$

$= 1,992,261.82$ in. lb.

$$A_s = \left(\frac{F_c' b}{1.176 F_y}\right)\left(d - \sqrt{d^2 - \frac{2.353 M_u}{\phi F_c' b}}\right)$$

$$A_s = \left(\frac{3000 * 66}{1.176 * 60000}\right)\left(36 - \sqrt{36^2 - \frac{2.353 * 1992261.82 \text{ lb.in}}{0.9 * 3000 * 66}}\right)$$

$A_s = 2.81 * 0.37 = 1.04 \, in^2$

use 3 # 6 bars (both direction)

$A_s = 0.44 \, in^2$

$A_g = 3 * 0.44 = 1.32 \, in^2$

$$\text{Spacing} = \frac{5.5 * 12}{2} = 33"$$

Develofement length

$l_{d\,supplied} = 24'' - 3'' = 21''$

$$\frac{C_b + K_{tr}}{d_b} = \frac{3 + 0}{0.750} = 4$$

$$\frac{l_d}{d_b} = \left(\frac{3}{40} * \frac{F_y}{\lambda \sqrt{f_c'}} * \frac{\psi_e \, \psi_t \, \psi_s}{\frac{C_b + K_{tr}}{d_b}}\right)$$

$$\frac{l_d}{d_b} = \left(\frac{3}{40} * \frac{60000}{1 * \sqrt{3000}} * \frac{1}{4}\right) = 20.54$$

$l_d = 20.54 * 0.750 = 15.4 < 21''$ o.K

use 3 # 6 both direction

For Columns 3B, 3C, 6B, 6C

$P_u = 1062.52 \text{ Kips}$

$$A = \frac{1062.52 \text{ Kips}}{16 \text{ Kips/ft}^2} = 66.41 \text{ ft}^2$$

Use 8.5 * 8.5 Footing

Reinforcing bars :-

Thickness = 36"

$$l = \frac{B - c}{2} = \frac{8.5' - 1.5'}{2} = 3.5' = 42"$$

$$M_u = \frac{P_u l^2}{2B} = \frac{1062.52 \text{ Kips} * (42)^2 * 1000}{2 * 8.5 * 12} = 918767 2.941 \text{ lb.in}$$

$$A_s = \left(\frac{F'_c b}{1.176 F_y}\right)\left(d - \sqrt{d^2 - \frac{2.353 M_u}{\Phi F'_c b}}\right)$$

$$= \left(\frac{3000 * 102}{1.176 * 60000}\right)\left(36 - \sqrt{36^2 - \frac{2.353 * 9187672.941}{0.9 * 3000 * 102}}\right)$$

$= 4.34 * 1.107 = \underline{\underline{4.8 \text{ in}^2}}$

use 5 #9

$A_s = 1 \text{ in}^2 \Longrightarrow A_g = 1 * 5 = 5 \text{ in}^2$

$$\text{spacing} = \frac{8.5 * 12}{5 - 1} = 25.5"$$

use spacing 25"

Development length :-

l_d supplied = 42 - 3 = 39"

$$\frac{C_b + K_{tr}}{d_b} = \frac{3 + 0}{1.128} = 2.7$$

$$l_d = \left(\frac{3}{40} * \frac{F_y}{\lambda \sqrt{f'_c}} * \frac{\psi_e \psi_t \psi_s}{\frac{c_b + K_{tr}}{d_b}} \right) * d_b$$

$$= \left(\frac{3}{40} * \frac{60000}{1 * \sqrt{3000}} * \frac{1*1*1}{2.7} \right) d_b = 30.43 * 1.128$$

$$= 34.33 < 39 \quad o.k$$

use 5 #9 both direction

* For Columns:

2A, 3A, 4A, 5A, 6A, 7A, 2D, 3D, 4D, 5D, 6D, 7D,
1B, 1C, 8B, 8C, 4B, 5B, 4C, 5C

$P_u = 766.56$ Kips

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

$$A = \frac{766.56 \text{ Kips}}{16 \text{ Kips/ft}^2} = 47.91 \text{ ft}^2$$

use 7 * 7 Footing

Reinforeing bars:

Thickness 36"

$$l = \frac{B - C}{2} = \frac{7' - 1.5'}{2} = 2.75' = 33''$$

$$M_u = \frac{P_u l^2}{2B} = \frac{766.56 \text{ Kips} * 1000 * (33)^2}{2 * 7 * 12}$$

$$= 4968951.43 \text{ lb.in}$$

$$A_s = \left(\frac{f'_c b}{1.176 F_y} \right) \left(d - \sqrt{\frac{2.353 M_u}{\phi f'_c b}} \right)$$

$$A_s = \left(\frac{3000 * 84}{1.176 * 60000}\right)\left(36 - \sqrt{36^2 - \frac{2.353 * 4968951.43 \text{ lb.in}}{0.9 * 3000 * 84}}\right)$$

$$= 3.57 * 0.723 = 2.58$$

use $5\#7$

$A_s = 0.6 \text{ in}^2$

$A_g = 0.6 * 5 = 3 \text{ in}^2$

$$\text{spacing} = \frac{7 * 12}{5-1} = 21''$$

use $20''$ spacing

* Development length

$l_{d\,supplied} = 33 - 3 = 30''$

$$\frac{c_b + K_{tr}}{d_b} = \frac{3+0}{0.875} = 3.4$$

$$\frac{l_d}{d_b} = \left(\frac{3}{40} * \frac{F_y}{\lambda\sqrt{F_c'}} * \frac{\psi_e \psi_t \psi_s}{\frac{c_b + K_{tr}}{d_b}}\right)$$

$$= \left(\frac{3}{40} * \frac{60000}{1 * \sqrt{3000}} * \frac{1*1*1}{3.4}\right) = 24.16$$

$l_d = 24.16 * 0.875 = 21.144'' < 30''$ o.k

use $5\#7$ in both direction

Foundation design:- Soil Capacity $= 8\ tons/ft^2$
$= 16\ \frac{Kips}{ft^2}$

$$P = \frac{F}{A} \Rightarrow A = \frac{F}{P}$$

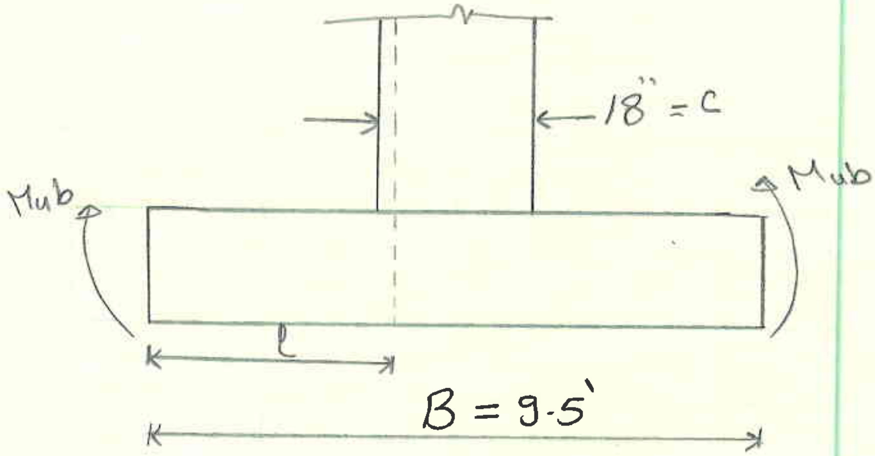
Column:. 2B, 2C, 7B, 7C

$P_u = 1371$ Kips

$$A = \frac{1371\ Kips}{16\ \frac{Kips}{ft^2}} = 85.68\ ft^2 \Rightarrow 86\ ft^2$$

use 9.5 x 9.5 Footing

Reinforcing bars
Table 8.1
Coduto, Don ALD P.



Thickness = 36"

$$l = \frac{B - c}{2}$$

$$= \frac{9.5' - 1.5'}{2} = 4' = 48"$$

$$M_u = \frac{P_u l^2}{2B} = \frac{1371 * 1000 * (48")^2}{2 * 9.5 * 12} = 13854315.8\ in.lb$$

$$A_s = \left(\frac{F_c' b}{1.176 F_y}\right)\left(d - \sqrt{d^2 - \frac{2.353 M_u}{\phi F_c' b}}\right.$$

$$A_s = \left(\frac{3000 * 114}{1.176 * 60000}\right)\left(36 - \sqrt{36^2 - \frac{2.353 * 13854315.8 \text{ lb.in}}{0.9 * 3000 * 114}}\right)$$

$= 4.85 * 1.5 = 7.29 \text{ in}^2$

use 6 #10 bars (both direction)

$A_s = 1.27$

$A_g = 1.27 * 10 = 12.7 \text{ in}^2$

$$\text{Spacing} = \frac{9.5 * 12}{6-1} = \frac{114}{5} = 22.8''$$

use spacing 22"

Development length

l_d supplied $= 48'' - 3 = 45''$

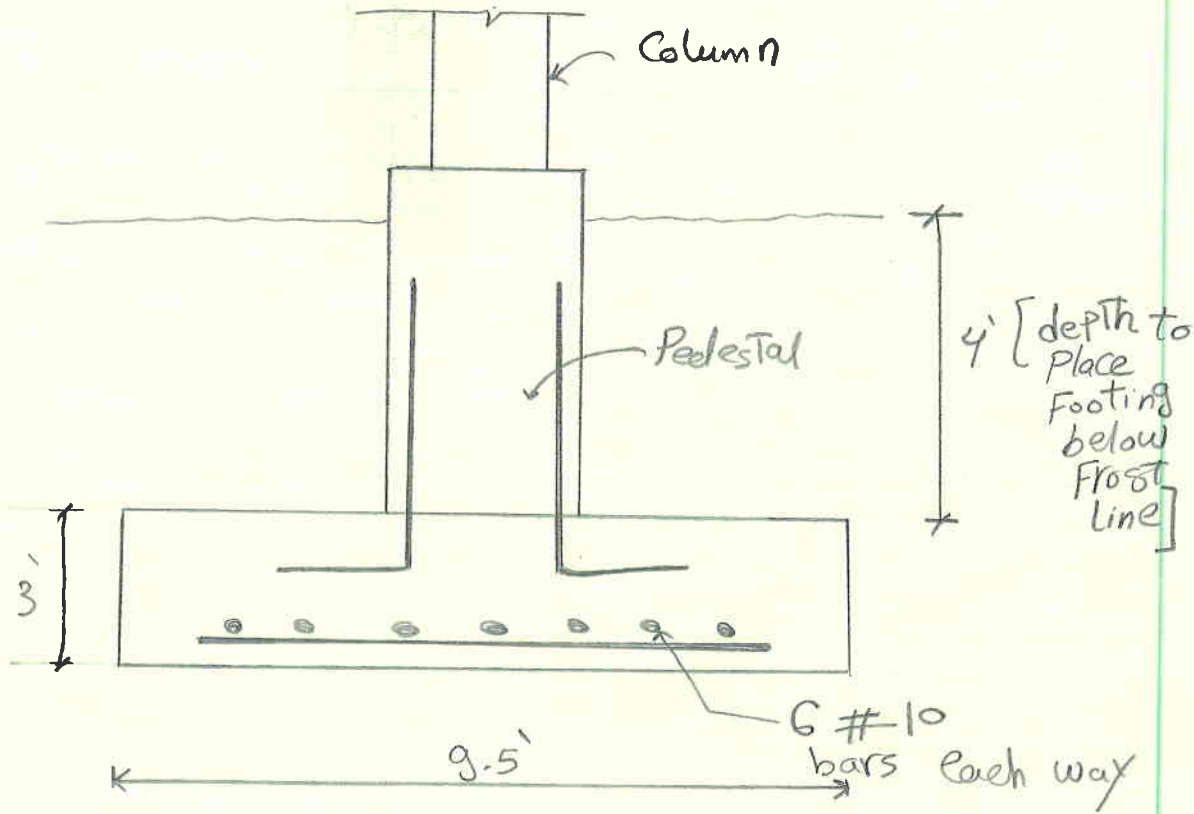
$$\frac{C_b + K_{tr}}{d_b} = \frac{3+0}{10/8} = 2.4$$

$$\frac{l_d}{d_b} = \left(\frac{3}{40} * \frac{F_y}{\lambda\sqrt{f'_c}} * \frac{\psi_e \psi_t \psi_s}{\frac{C_b + K_{tr}}{d_b}}\right)$$

$$\frac{l_d}{d_b} = \left(\frac{3}{40} * \frac{60000}{1 * \sqrt{3000}} * \frac{1*1*1}{2.4}\right) = 34.23''$$

$$l_d = 34.23'' * \frac{10''}{8} = 42.8'' < 45'' \qquad \underline{\text{O.K}}$$

use 6 #10 in both direction
spacing 22"



Column
Pedestal
4' [depth to Place Footing below Frost Line]
3'
6 #10 bars each way
9.5'

Gordon Library Redesign
JP Connors & Rania Attala

Bay:		B	
Tributary Width		31	ft
Beam Spacing		5	ft
Beam Weight		30	lb/ft
Beam load		6	lb/ft^2
Girder Length		20	ft
Vulcraft metal deck choice		1.5VL19	
Total Slab Thickness		4	inches
Concrete f'c		5	ksi
Loads			
MEP		5	psf
Ceiling		2	psf
Metal Deck		2.49	psf
Concrete (Vulcraft)		39	psf
Occupancy Live		150	psf
Concrete wt. Adjusted for Ponding		42.9	psf
Construction Live Load		25	psf
W_u=1.2D +1.6L		9.612	kip/ft
$M_u=(W_u*L^2)/8$		480.61	kip*ft
a assumption		2	inches
Assumed Y2		3	inches
Capacity given Y2 Value (Table 3-19)		529	kip*ft
Check Capacity>Mu?		YES	
Beam Selection		W18x40	
Weight Girder		40	lb/ft
Ix		612	in^4
Ix lower bound		1424.90	in^4
Interpolation			
	3	1450	
	2.84		
	3.5	1530	
b_e		60	inches
b_e		372	inches
Governing b_e		60	inches
∑Qn (Table 3-19)		590	
a		2.31	inches
Actual Y2		2.84	inches
φbMn Interpolation		522.10	kip*ft

3	529	
2.84	φbMn	
3.5	551	
Check Capacity>Mu?	YES	
φbMp (Table 3-19)	294	ft*kip
Capacity Before Concrete Hardens		
Concrete as live load and construction Live load		
wucons=1.2D+1.6L	3.732	kip/ft
Mucons	186.58	kip*ft
Check?	YES	
Δ_{Cons} =($5wl^4$)/384EI	0.48845639	inches
Max	1.75	inches
Check?	YES	
$\Delta_{LL50\%}$ =($5wl^4$)/384EIlowerbound	0.203	inches
Max	1.00	inches
Check?	YES	
$\Delta_{LL50\%+DL}$ =($5wl^4$)/384EIowerbound	0.36373522	
L/240	1	
Check?	YES	
In Service Capacity		
Wu	9.660108	kip/ft
Mu=(Wu*L2)/8	483.01	kip*ft
Check?	YES	
Full Composite Shear Stud Design		
Qn (1 3/4" strong stud per rib)	21.5	
∑Qn/Qn	27.44	
Total Studs	54.88	
Min Spacing	4.5	inches
Max Spacing	32	inches
Spacing	6.66	inches
Partial Composite Shear Stud Design		
Y1 for PNA (7)	4.27	
∑Qn for PNA 7	148	
a	0.580	
Y2	3.710	
∑Qn/Qn	7	
Number of Studs	14	
Spacing	16.2	inches
Ilb interpolation		
3.5	979	
3.710	New ILB	

	4	1010	
New I Lb		992.01	
$\Delta_{LL50\%}$ =($5wl^4$)/384EI		0.291	
Max		1	inch
Check?		YES	
$\Delta_{LL50\%+DL}$ =($5wl^4$)/384EI		0.522	
Max		1	
Check?		YES	
φbMn Interpolation			
	3.5	407	
	3.710	φbMn	
	4	412	
φbMn		409.10	
Check?		FAIL	
		USE FULL COMPOSITE	

Gordon Library Redesign
JP Connors & Rania Attala

Bay:	E&L	
Length	34	ft
Width	24	ft
# Infill Beams	3	
Spacing	6	ft
Vulcraft metal deck choice	1.5VL19	
Total Slab Thickness	4	inches
Concrete f'c	5	ksi
Loads		
MEP	5	psf
Ceiling	2	psf
Metal Deck	2.49	psf
Concrete (Vulcraft)	39	psf
Occupancy Live	150	psf
Concrete wt. Adjusted for Ponding	42.9	psf
Construction Live Load	25	psf
W_u=1.2D +1.6L	1.817208	kip/ft
$M_u=(W_u*L^2)/8$	262.59	kip*ft
a assumption	2	inches
Assumed Y2	3	inches
Capacity given Y2 Value (Table 3-19)	329	kip*ft
Check Capacity>Mu?	YES	
Beam Selection	W14X30	
Weight Beam	30	lb/ft
Ix	291	in^4
Ix lower bound	749.85	in^4
Interpolation		
3	725	
3.28		
3.5	770	
b_e	102	inches
b_e	72	inches
Governing b_e	72	inches
∑Qn (Table 3-19)	443	
a	1.45	inches
Actual Y2	3.28	inches
φbMn Interpolation	337.84	kip*ft
3	329	

3.28	φbMn	
3.5	345	
Check Capacity>Mu?	YES	
φbMp (Table 3-19)	177	ft*kip
Capacity Before Concrete Hardens		
Concrete as live load and construction Live load		
wucons=1.2D+1.6L	0.706	kip/ft
Mucons	101.98	kip*ft
Check?	YES	
Δ_{Cons} =($5wl^4$)/384EI	1.61165542	inches
Max	1.75	inches
Check?	YES	
$\Delta_{LL50\%}$ =($5wl^4$)/384EIlowerbound	0.622	inches
Max	1.00	inches
Check?	YES	
$\Delta_{LL50\%+DL}$ =($5wl^4$)/384EIowerbound	1.10	
L/240	1.7	
Check?	YES	
In Service Capacity		
Wu	1.853208	kip/ft
Mu=(Wu*L2)/8	267.79	kip*ft
Check?	YES	
Full Composite Shear Stud Design		
Qn (1 3/4" strong stud per rib)	21.5	
∑Qn/Qn	20.60	
Total Studs	42.00	
Min Spacing	4.5	inches
Max Spacing	32	inches
Spacing	9.49	inches
Partial Composite Shear Stud Design		
Y1 for PNA (7)	2.8	inches
∑Qn for PNA 7	111	
a	0.363	inches
Y2	3.819	inches
∑Qn/Qn	5	
Number of Studs	10	
Spacing	35.8	inches
Ilb interpolation		
3.5	483	
3.819	New ILB	
4	502	

New I Lb		495.11	
$\Delta_{LL50\%}$ =($5wl^4$)/384EI		0.942	
Max		1	inch
Check?		YES	
$\Delta_{LL50\%+DL}$ =($5wl^4$)/384EI		1.663	
Max		1.7	
Check?		YES	
φbMn Interpolation			
	3.5	246	
	3.819	φbMn	
	4	250	
φbMn		248.55	
Check?		FAIL	
		NEED FULL COMPOSITE	

Connection Calculations
Check shear capacity of beam

Beam	W12X30	
Phi Vn	95.94	
T	10.13	
d	12.30	
tw	0.26	
Fy	50.00	
h/tw	41.80	
2.24 * SQRT (E/Fy)	53.95	
Check?	OK	
Wu	1.85	kips/ft
L	26.00	ft
V	24.05	Kips
Phi Vn Check	YES	
Bolt Diameter	0.63	
Bolt Strength	24.85	
Number of bolts	0.97	
Number of bolts	3.00	
Stability check	5.06	
Lc	0.63	
Total Capacity	163.13	
t	0.07	
Minimum L	5.06	
L	6.00	
Angle Shear Rupture	0.12	
Angle Shear Yield t	0.07	
Use	0.25	
Bolt spacing	2.00	
Net Height	5.50	
Net Shear Area	0.94	
Shear rupture	32.80	
Ant	0.29	
Tension rupture	16.97	
Agv	1.38	
Shear Yield	29.70	
Rn1	49.76	
Rn2	46.67	
Rn	46.67	
Phi RN	35.00	
Check	YES	

JP Connors & Rania Attala
Soil Bearing Capacity Calculation

Load (Kips)	Footing Type	Footing Area (in^2)	Bearing Capacity ($Tons/ft^2$)
615	4	81	3.80
642	5	90.25	3.56
615	4	81	3.80
370	2	49	3.78
370	2	49	3.78
870	2	49	8.88
740	6	100	3.70
740	6	100	3.70
615	4	81	3.80
740	6	100	3.70
740	6	100	3.70
740	6	100	3.70
747	6	100	3.74
720	6	100	3.60
720	6	100	3.60
720	6	100	3.60
642	5	90.25	3.56
505	7	66	3.83
505	3	72.25	3.49
496	7	66	3.76
496	7	66	3.76
362	4	81	2.23
285	2	49	2.91
80	1	12.25	3.27

Type	Area (ft^2)
1	12.25
2	49
3	72.25
4	81
5	90.25
6	100
7	66
8	20.25
9	16.875

Bearing
Capacity of
Glacial Till: 10 $tons/ft^2$

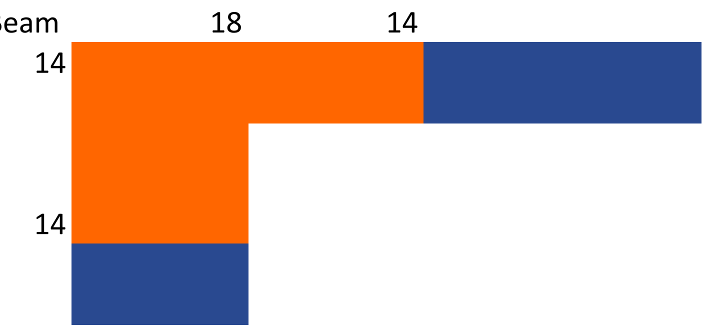
Concrete Column Design

Floor	3	
Columns	3B, 3C, 6B, 6C	
Area	540	Ft^2
Beam 1 Area	504	in^2
Beam 2 Area	504	in^2
Beam 1 Length	24	Ft
Beam 2 Length	30	Ft
Tot. Beam/Girder Load	28.35	KIPS
Slab Thickness	14	inches
Unit Wt. Conc.	150	PCF
Slab Load	0.175	Kips/Ft^2
MEP & Ceiling	0.007	Kips/Ft^2
Live Load	0.15	Kips/Ft^2
Total Dead Load	126.63	Kips
Load Combo	1.2D+1.6L	
Pu Column Above	780.96	Kips
Pu	1062.52	Kips
Alpha(ties)	0.8	
Phi (ties)	0.65	
Ag	324	
f'C	5	
Fy	60	
As	11.105	
Use	5#14	

Gordon Library Redesign
Rania Attala & JP Connors

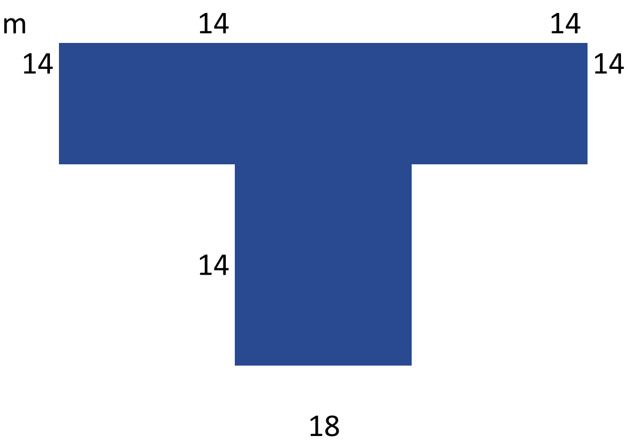
Slab Thickness	14	inches
Unit Weight Concrete	150	PCF
F'c	5000	psi
Fy	60000	psi
Slab Dead Load	175.0	lb/ft^2
MEP Load	5	PSF
Ceiling Load	2	PSF
Total Dead Load	182	
Live Load	150	PSF
Slab Load Combination	1.2D+1.6L	
Wu Slab Load	458.4	
Edge width 1	15.8	Ft.
Edge width 2	24	Ft.
Edge width 3	30	Ft.
Is Edge width 1	43218	in^4
Is Edge width 2	65856	in^4
Is Edge width 3	82320	in^4
alpha 15.8 ft width	1.00	
alpha 20 ft width	0.76	
alpha 25 ft width	0.61	
Average Alpha	0.79	
Ratio long to short clear span (Beta)	1.27	

Edge Beam



4Hf	56	
Hw	14	
Effective Flange Projection		14
Cg	12.04	
Ib	43044.2133	

Int. Beam



Cg	10.9375	in. from the top
Ig	50135.17	in^4

**alpha>0.2<2, minimum h=	9.47	
Height Check	OK	
SHORT SPAN		
Short Span, slab-beam strip on int. column line Mo	870.24	
Half strip width	7.88	
Full strip width	15.00	
Negative design moment	565.66	
Positive design moment	304.59	
L2/L1	1.25	
Alpha L2/L1	0.95	
% Neg. Moment by column Strip {Graph A.4}	68	%
Neg. Moment by column strip	384.65	
85% Beam:	326.95	ft-kips
15% Slab	57.70	ft-kips
Slab middle strip	181.01	ft-kips
% Pos. Moment by column Strip {Graph A.4}	68	%
Pos.Moment by column strip	207.12	
85% Beam:	176.05	
15% Slab	31.07	
Slab middle strip	97.47	
Short Span, slab-beam strip at edge Mo	456.88	
Negative design moment	296.97	
Positive design moment	159.91	
L2/L1	1.25	
Alpha L2/L1	1.24	

% Neg. Moment by column Strip {Graph A.4}	68	%
Neg. Moment by column strip	201.94	
85% Beam:	171.65	
15% Slab	30.29	
Slab middle strip	95.0306175	
% Pos. Moment by column Strip {Graph A.4}	68	%
Pos.Moment by column strip	108.74	
85% Beam:	92.43	
15% Slab	16.31	
Slab middle strip	51.17	
LONG SPAN Mo	1117.01	
Strip Width	12.00	ft
Exterior Negative Design Moment	178.72	
Positive Design Moment	636.69	
Edge Beam Torsional Constant	37125.0	
L2/L1	0.8	
Alpha L2/L1	0.60902778	
Beta t	0.28186508	
% Neg. Moment by column Strip {Graph A.4}	79	
Neg. Moment by column strip	141.19	
85% Beam:	120.01	
15% Slab	21.18	

Interior Slab - beam strip - 24 ft span	Beam Moment	Column Strip Moment	Middle Strip Moment
Negative	326.95	57.70	181.01
Positive	176.05	31.07	97.47

Exterior Slab- beam strip - 24 ft span			
Negative	171.65	30.29	95.03
Positive	92.43	16.31	51.17

Slab middle strip	37.53
% Pos. Moment by column Strip {Graph A.4}	73
Pos.Moment by column strip	464.79
85% Beam:	395.07
15% Slab	69.72
Slab middle strip	171.91
Interior Negative Design Moment	781.90434
% Neg. Moment by column Strip {Graph A.4}	79
Neg. Moment by column strip	617.70
85% Beam:	525.05
15% Slab	92.66
Slab middle strip	164.20
Slab Reinforcement Design:	
Slab cover	2.5
Slab rebar diameter (#7)	0.875
d in 30 ft direction	10.1875
d in 24 ft direction	11.0625
Minimum steel area	0.3024

	Beam Moment	Column-Strip Slab Moment	Middle-Strip Slab Moment
Exterior Negative - 30 ft Span	120.01	21.18	37.53
Positive - 30 ft span	395.07	69.72	171.91
Interior negative - 30 ft span	525.05	92.66	164.20

30 ft pmin	0.00247362
24 ft pmin	0.00227797
Area #7 bars	0.6 in^2

	Location	Mu (ft-k	b (in)	d (in)	Mu x 12/b (ft-kips/ft)	p	As (in^2)	# of No. 7 bars
30 ft span (2 half column strips)	Ext. Neg.	21.18	126	10.1875	2.02	0.00247362	3.1752	6
	Positive	69.72	126	10.1875	6.64	0.00247362	3.1752	6
	Int. Neg.	92.66	126	10.1875	8.82	0.00247362	3.1752	6
Middle Strip	Ext. Neg.	37.53	144	10.1875	3.13	0.00247362	3.6288	7
	Positive	171.91	144	10.1875	14.33	0.00270377	3.96643059	7
	Int. Neg.	164.20	144	10.1875	13.68	0.00258	3.78486	7
24 ft span Ext. half-column strip	Negative	30.29	76.5	11.0625	4.75	0.00227797	1.9278	4
	Positive	16.31	76.5	11.0625	2.56	0.00227797	1.9278	4
Middle Strip	Negative	181.01	180	11.0625	12.07	0.00227736	4.5347931	8
	Positive	97.47	180	11.0625	6.50	0.00227797	4.536	8
Interior half-column strip	Negative	57.70	76.5	11.0625	9.05	0.00227797	1.9278	4
	Positive	31.07	76.5	11.0625	4.87	0.00227797	1.9278	4

	phi Mn value	P values
Linear interpolation		
	10.6	0.002
	12.07	0.0023
	15.9	0.003

Vu	4.7344125
Phi vc	14.0802638

Space	Total Area (SQ FT)	Occupant Load Factor	Occupants Allowed
Kitchen	689	100	7
Business	6930	100	69
Library Stack Areas	3599	100	36
Assembly - less concentrated	37926	15	2528
Industrial	240	100	2
		Total Occupants	2643
		Occupants per 2 floors	1321

Third Floor Occupant Load	661
Minimum Clear Width	50

Clear width (in.)	100
Stair factor	0.3
Number of stairwells	4
Capacity per stairwell	333
Total capacity	1333

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Seismic and Wind Force Calculator

Jonathan Ochshorn

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Directions: Enter general data (city, importance factor), seismic data (site class, seismic force resisting system), and wind data (exposure category, plan and parapet dimensions, and coefficients for directionality and topography). Then, enter values for story heights above grade and seismic weight (approximately equal to the dead load) for each story. Start at the highest floor (i.e., the roof), and work down to the lowest above-grade floor level. ***Press "update" button***.

Story forces for wind and seismic loading will be displayed to the right of the values entered for seismic weight. In this way, the magnitude of wind and seismic forces may be compared for a given building on a given site. Note that there are some limitations for the use of this calculator: the building is assumed to be rectangular, and is limited to 20 stories (for buildings with more stories, an approximate calculation can be obtained by combining the seismic weight of two adjacent stories and entering the average height above grade). Calculations are based on analytic procedures for rigid buildings, neglecting internal pressures (wind), and equivalent lateral force procedures (seismic) as described in ASCE/SEI 7-05, *Minimum Design Loads for Buildings and Other Structures*. Plan dimensions for wind loading calculations are shown in Fig. 1. To obtain wind story forces from calculated wind pressures, windward and leeward pressures are combined into a single set of forces, as shown in Fig. 2. Account is taken of higher wind pressure on parapets. Story forces for seismic loading are shown in Fig. 3.

More detailed explanations and examples can be found in my text.

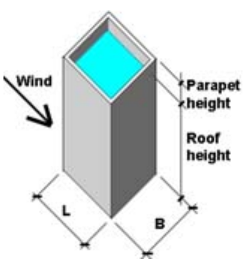

Fig. 1. *Wind direction and plan dimensions.*

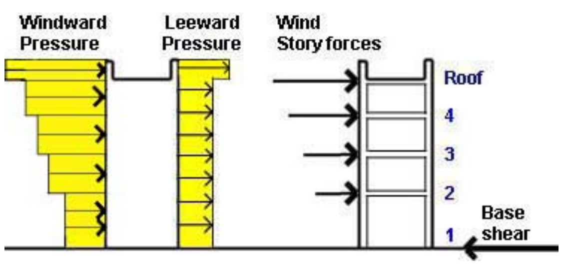

Fig. 2. *Building sections comparing windward and leeward pressures with wind story forces and base shear.*

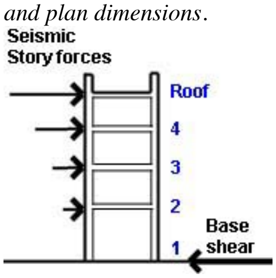

Fig. 3. *Building section with seismic story forces and base shear.*

floor/roof height above grade (ft)	seismic weight per floor (lb)	seismic story force (lb)	wind story force (lb)	windward pressure (psf)	leeward pressure (psf
60	1,354,120	**64260**	**14228**	14.87	-6.02
45	1,138,190	**39131**	**27656**	13.69	-6.02
30	1,138,190	**24845**	**25836**	12.2	-6.02
15	1,138,190	**11428**	**13136**	10	-6.02
14	0		**10915**	10	-6.02
	0				

D-1

0	

base shear = 139664 lb 91771 lb

general data

since city = 'Other,' enter these values

city	wind speed (mph)	seismic Ss	seismic S1	seismic TL
Other	100	0.24	0.067	6

importance factor	plan dimensions (ft) L	B
II - normal	170.83	90.83

wind data

exposure	Kd	Kt	parapet height above roof (ft)
B	1	1	0

seismic data

lat-force-resist system: main category

moment-resisting frame systems

sub-category

01. Special reinforced concrete shear walls

01. Steel ecc-br frames, mom-res, conn at cols away from lir

04. Ordinary steel moment frames ***<- this applies***

01. Steel eccentrically braced frames (no limits)

01. Special steel concentrically braced frames

01. Special steel moment frames

Steel systems not specifically detailed for seismic resistance

no limits

(your seismic design category is B)

site class (soil)

C = dense soil or soft rock

paramenters for calculation of period	C_T	x	T (sec)
Use default values shown:	0.028	0.8	0.741

Checks:

errors: **0**

sub not permitted 0

D-2

exceeds height	0
floor heights are not in descending order	0
wind speed must be > 0	0
seismic Ss out of range	0
seismic S1 out of range	0
seismic TL out of range	0
plan dimension L must be > 0	0
plan dimension B must be > 0	0
heights must be > 0	0
weights must be > 0	0
Kt out of range (should be between 1 and 3)	0

First posted Aug. 3, 2009 | Last updated Aug. 3, 2009

D-3

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Seismic and Wind Force Calculator

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Directions: Enter general data (city, importance factor), seismic data (site class, seismic force resisting system), and wind data (exposure category, plan and parapet dimensions, and coefficients for directionality and topography). Then, enter values for story heights above grade and seismic weight (approximately equal to the dead load) for each story. Start at the highest floor (i.e., the roof), and work down to the lowest above-grade floor level. ***Press "update" button***.

Story forces for wind and seismic loading will be displayed to the right of the values entered for seismic weight. In this way, the magnitude of wind and seismic forces may be compared for a given building on a given site. Note that there are some limitations for the use of this calculator: the building is assumed to be rectangular, and is limited to 20 stories (for buildings with more stories, an approximate calculation can be obtained by combining the seismic weight of two adjacent stories and entering the average height above grade). Calculations are based on analytic procedures for rigid buildings, neglecting internal pressures (wind), and equivalent lateral force procedures (seismic) as described in ASCE/SEI 7-05, *Minimum Design Loads for Buildings and Other Structures*. Plan dimensions for wind loading calculations are shown in Fig. 1. To obtain wind story forces from calculated wind pressures, windward and leeward pressures are combined into a single set of forces, as shown in Fig. 2. Account is taken of higher wind pressure on parapets. Story forces for seismic loading are shown in Fig. 3.

More detailed explanations and examples can be found in my text.

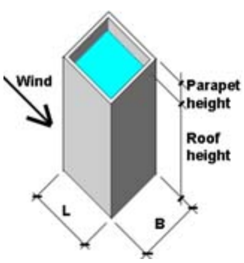

Fig. 1. *Wind direction and plan dimensions.*

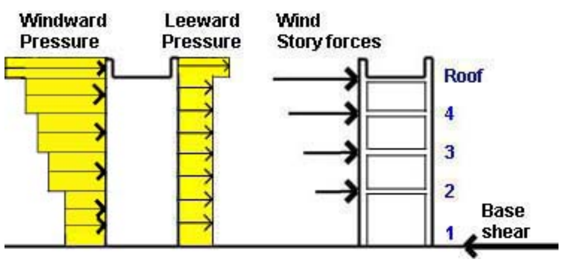

Fig. 2. *Building sections comparing windward and leeward pressures with wind story forces and base shear.*

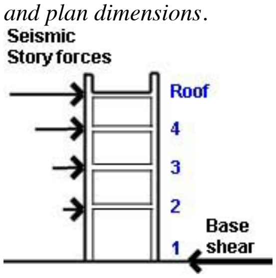

Fig. 3. *Building section with seismic story forces and base shear.*

floor/roof height above grade (ft)	seismic weight per floor (lb)	seismic story force (lb)	wind story force (lb)	windward pressure (psf)	leeward pressure (psf
60	1,354,120	**64260**	**30953**	14.87	-9.29
45	1,138,190	**39131**	**60403**	13.69	-9.29
30	1,138,190	**24845**	**56980**	12.2	-9.29
15	1,138,190	**11428**	**29179**	10	-9.29
14	0		**24723**	10	-9.29
	0				

D-4

0	

base shear = 139664 lb 202238 lb

general data

since city = 'Other,' enter these values

city	wind speed (mph)	seismic Ss	seismic S1	seismic TL
Other	100	0.24	0.067	6

importance factor	plan dimensions (ft) L	B
II - normal	90.83	170.83

wind data

exposure	Kd	Kt	parapet height above roof (ft)
B	1	1	0

seismic data

lat-force-resist system: main category

moment-resisting frame systems

sub-category

01. Special reinforced concrete shear walls

01. Steel ecc-br frames, mom-res, conn at cols away from lir

04. Ordinary steel moment frames *<- this applies*

01. Steel eccentrically braced frames (no limits)

01. Special steel concentrically braced frames

01. Special steel moment frames

Steel systems not specifically detailed for seismic resistance

no limits

(your seismic design category is B)

site class (soil)

C = dense soil or soft rock

paramenters for calculation of period	C_T	x	T (sec)
Use default values shown:	0.028	0.8	0.741

Checks:

errors: **0**

sub not permitted 0

D-5

exceeds height	0
floor heights are not in descending order	0
wind speed must be > 0	0
seismic Ss out of range	0
seismic S1 out of range	0
seismic TL out of range	0
plan dimension L must be > 0	0
plan dimension B must be > 0	0
heights must be > 0	0
weights must be > 0	0
Kt out of range (should be between 1 and 3)	0

Disclaimer: This calculator is not intended to be used for the design of actual structures, but only for schematic (preliminary) understanding of structural design principles. For the design of an actual structure, a competent professional should be consulted.

First posted Aug. 3, 2009 | Last updated Aug. 3, 2009

JP Connors & Rania Attalla

Approximate Second-Order Elastic Analysis Column M 15

Column load effects from analysis

Factored axial force Pnt from no-sway analysis (gravity loads)	108.09
Factored axial force Plt from sway analysis (lateral loads)	30.36
Factored moment Mnt from no-sway analysis (gravity loads)	12.21
Factored moment Mlt from sway analysis (lateral loads)	438.12

Lateral deflection (story drift) from analysis

Total story shear ΣH (lateral loads input to deflection analysis for the story)	139.66
Lateral deflection (drift) for story ΔH (obtained from deflection analysis and loading ΣH)	1.19

Amplifier B2

Total elastic critical buckling load for the story

$$P_{e\ story} = R_M \frac{\sum HL}{\Delta H}$$

18032.57

where Rm =0.85 (conservative) and L = story height (same units as ΔH)

P_{story} = total vertical load supported by the story using appropriate load combination equations 311.45

$$B_2 = \frac{1}{1 - \frac{\alpha P_{story}}{P_{estory}}}$$

1.02

where α = 1.0 for LRFD

Amplifier B_1

M_1= smaller factored column end moment due to gravity load (no sway) analysis	5.93
M_2= larger factored column end moment due to gravity load (no sway) analysis	7.67
Indicate: single or reverse curvature	Reverse

Cm = 0.6 ± 0.4 (M1/M2)	0.29
Use + for single curvature (*hurt*)	
Use - for reverse curvature (*help*)	
Required second-order axial strength $P_r = P_{nt} + B_2 P_{lt}$	138.99
Elastic critical buckling load for column $P_{el} = \pi^2 EI/(K_1L)^2$ where K_1 = 1.0 Note: This load capacity refers to the no-sway case (gravity loading)	16166.05
$$B_1 = \frac{C_m}{1 - \frac{\alpha P_r}{P_{e1}}} \geq 1$$	0.29
where α = 1.0 for LRFD	1.00
Required second-order strength values	
Pr = Pnt	108.09
Mr = B1 Mnt + B2 Mlt	458.03
where Mnt, Mlt, B1, and B2 are defined above	