Structural & Firesafety Analysis of Gateway Park

A Major Qualifying Project
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I. Abstract

This report focuses on the structural and fire protection analysis of the building located at 68 Prescott Street in Worcester, Massachusetts. This report includes information collected on the renovation of a brownfield site and the masonry building, as part of the Gateway Park Project. Strategies to rehabilitate and reuse the existing masonry exterior were implemented utilizing steel and reinforced concrete structural alternatives with corresponding cost estimates. Additionally, a code analysis was performed. The document concludes with an analysis of the active and passive fire protection systems and a risk assessment of the building.

II. Authorship

Each of the group members contributed equally to the writing and editing of this document, with specific efforts as follows.

1. Introduction – *All Group Members*
2.1 Brownfield Development – *Kaitlin McGillvray*
2.2 Steel Design – *Kathryn Strumolo*
2.3 Reinforced Concrete Design – *Caitlyn Ramig*
2.4 Historic Construction and Masonry – *Kaitlin McGillvray*
2.5 Building Codes and Fire Protection – *Kelly Thompson*
3. Methodology – *All Group Members*
4. Structural Steel Design – *Kathryn Strumolo and Kelly Thompson*
5. Reinforced Concrete Design – *Kaitlin McGillvray and Caitlyn Ramig*
6. Fire Safety Code Analysis –*Kaitlin McGillvray and Caitlyn Ramig*
7. Fire Protection Systems – *Kathryn Strumolo and Kelly Thompson*
8. Risk Assessment – *All Group Members*
9. Conclusions and Recommendations – *All Group Members*

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1. Introduction

The Gateway Park Project, located at 68 Prescott Street in Worcester, MA, is a business venture undertaken by Worcester Polytechnic Institute (WPI) and the Worcester Business Development Corporation (WBDC). The project entails the cleanup of a brownfield site and renovation of an aged masonry building. Other aspects of the project include construction of a new building to serve as laboratory facilities and a parking garage. The development of the 11.5-acre site is the beginning of the master plan for a mixed-use expansion to rejuvenate the 55-acre section of Worcester.

To understand the comprehensive nature of the Gateway Park Project, we conducted supplemental research on the cleanup of this brownfield site. Some important considerations for this project were soil contamination from the past uses of the site and the effect of the contaminants on future uses of the site.

The scope of our work included researching historic masonry construction, performing a structural analysis of the existing masonry building, and designing structural alternatives for the building's interior. We concluded the structural engineering portion of our project with a cost estimate to determine the economics of our suggested alternatives. Then, we proceeded to evaluate the fire protection systems and the building's egress system based on the 2006 International Building Code.

One objective of the Gateway Park renovation project was to structurally update the masonry building for office space and a lecture hall. This presented owners and builders with the challenge of bringing an early 1900's building into compliance with modern code criteria and redesigning the interior spaces. Also, the building plans called for construction of a new brick building, to resemble the masonry building, as well as a connecting building between the aforementioned structures. The demolition of a section of the wall of the old masonry building to provide a connection between buildings posed structural stability design concerns.

The masonry building currently contains the original timber structural framing system. The strength of timber is not as high as other construction materials; therefore, it requires more columns, which places flexibility constraints on the layout of the office space. This report investigates the use of alternative structural systems within the existing masonry shell to provide a more open floor plan. Steel framing systems of varying bay sizes were developed, as well as, various concrete framing systems, including the use of one-way and two way slabs. Considerations, such as depth of construction, were made to evaluate the feasibility of each alternative. We also developed cost estimates to determine which alternatives provided the most beneficial design while maintaining a reasonable cost.

Based on the building standards of life safety, property protection, and mission protection expressed in the 2006 International Building Code, we performed a code analysis of the masonry building. We investigated the building's egress system, and analyzed the active and passive fire protection systems that were installed in the building. Subsequently, we developed methods to perform a risk assessment of the masonry building. After completion of the various structural and fire protection alternatives, cost estimates of the design scenarios were determined.

Realistic constraints addressed in this project include the social implications of revitalizing old industrial districts, health and safety issues, feasibility of structural design alternatives and construction, and the economical constraints that coincide with these issues. The following narrative summarizes the activities performed by our group and illustrates how they meet the realistic constraints embodied in capstone design.

The Master Plan for Gateway Park aims to revitalize the old industrial district in which it is located. The project will result in many important social implications for the City of Worcester and surrounding towns. By rebuilding the site with modern structures and facilities, the project will create new jobs, research opportunities and improved aesthetics in this rundown section of the city.

This project addresses both new and old construction on a site with environmental challenges. The Gateway Park site required a considerable amount of clean up was needed to remove the hazardous wastes and toxins found in the soil. The code requires a specific level of safety to be reach for different occupancies. Additional means to ensure the health and safety of the building's occupants was performing a risk assessment and fire protection analysis. The active and passive fire protection systems were investigated to ensure code compliance.

This project will present structural alternatives for concrete and steel within the existing exterior frame. The alternative designs are intended to provide a more open layout. From a structural standpoint, different alternatives will be developed for the interior design of the building in order to be able to analyze their manufacturability. These alternative designs entail various concrete and steel frames. A cost estimate will be formed to explore the economics of the project and compare the feasibility of each design alternative.

2. Background

Library and field research was conducted in order to understand the concerns of revitalizing Gateway Park. To understand the project's development we completed research on brownfield development, historic masonry construction, and building codes. We also researched structural steel and reinforced concrete construction to obtain a better understanding of the impacts of designing with different materials. We were also able to make two visits to the site as part of our background research and data collection.

2.1 Brownfield Development

The Environmental Protection Agency (EPA) defines brownfields as property, the expansion, redevelopment, or reuse of which may be complicated by the presence or potential presence of a hazardous substance, pollutant, or contaminant.[1] The EPA began its Brownfields Program in 1995, with the purpose of encouraging the clean up and development of brownfield projects. The Brownfields Program is, "designed to empower states, communities, and other stakeholders in economic redevelopment to work together in a timely manner to prevent, assess, safely clean up, and sustainably reuse brownfield[s]…" sites.[2] This is done in hopes that the resulting facilities will increase job opportunities, and utilize the unused area to take developmental pressures off open-areas.

There are many contaminants that may be found in brownfield land. They are grouped into seven categories: halogenated VOCs, nonhalogenated VOCs, halogenated SVOCs, nonhalogenated SVOCs, fuels, metals and metalloids, and explosives.[3]

VOCs are hydrocarbon compounds that evaporate at room temperature, while SVOCs are hydrocarbon compounds that have boiling points greater than 200ºC. The difference between a halogenated and nonhalogenated compound is that halogenated has a halogen (fluorine, chlorine, bromine, or iodine) attached to it. All of the above compounds can be found in areas such as: burn pits, chemical manufacturing plants, disposal areas, electroplating and metal finishing shops, hangers, landfills and more.

[1] United States Environmental Protection Agency. "About Brownfields."
[2] Ibid.
[3] United States Environmental Protection Agency. "Guide to Contaminants and Technologies."

Fuels are chemically created by refining and manufacturing petroleum or natural gas to generate heat or energy. Fuels can contain nonhalogenated VOCs and/or nonhalogenated SVOCs.[4]

Metals, metalloids, and nonmetals are elements that are distinguished by their ionization and bonding properties. Metals are shiny, have a high density, malleable, high melting point, hard, conduct electricity and heat well. On the other hand, metalloids are in the middle of metals and nonmetals and have no unique characteristics. This type of contaminant can be found in artillery and small arms impact areas, battery disposal areas, burn pits, electroplating and metal finishing shops, landfills and more.[5]

Lastly, artificial explosives are manufactured chemical explosives and can typically be found in artillery impact areas, contaminated marines sediments, disposal wells, leach fields, landfills, burial pits, and TNT washout lagoons.[6]

People develop brownfields for several reasons. Location is a large factor in deciding whether to develop land, and brownfields are often in desirable locations within urban areas; however, the economic aspect is equally important to a developer. Cleaning up contaminated land will add a significant amount of cost to the project, so there are often economic incentives to promote development. For example, brownfield land is typically much less to purchase and there are many governmental agencies that will provide grants to projects.

Despite the EPA Brownfields Program, brownfields were not being developed in Massachusetts because of extremely strict codes that resulted from an incident in Woburn in the 1980's where contaminated soil led to many deaths. During this time, the risks of undertaking brownfields projects could not be justified by the benefits. However, in 1998 Massachusetts passed the Brownfield Act, which privatized the process of acquiring and cleaning brownfield land. The job of a License Site Professionals (LSP) was created, in an effort to lessen the burden on the EPA, at the state level, but ultimately at the federal level as well. The LSP's are responsible to go onto a site, evaluate, test and

[4] Ibid.
[5] Ibid.
[6] Ibid.

form an action plan to clean the specific site. Utilizing the private sector enables the developers to clean up the sites and begin redevelopment in a timely fashion.[7]

2.2 Structural Steel Design

For over 150 years, structural steel has been enabling the creation of countless structures. Steel design has been used for various structures such as buildings, bridges, factories and much more. In the case of 68 Prescott Street, the facility provides office space and a lecture hall.

The overall purpose of design is to invent a structure that will satisfy the design requirements. Thus, the structural engineer seeks to design a structural system that will resist and transfer the forces and loads acting on it with adequate safety, and allow for the requirements of stiffness, economy, and harmony.[8] The principal design requirement of any structure is for it to be serviceable and have a functional design. It is common to encounter design challenges due to financial constraints. Not only must structures serve their purpose, but they must also be economical.

2.2.1 Development of Structural Steel Design

Although steel was produced as early as 200 B.C. by the Celts, the relevant history starts in the mid-1800's with Sir Henry Bessemer, who developed and patented the first inexpensive industrial process for the mass production of steel. Until Bessemer's Process was introduced, steel was extremely expensive and as a result wrought iron was used during the Industrial Revolution. In Bessemer's process, also referred to as the Basic Oxygen Process, melted iron is poured into a large egg-shaped container called a converter. Blasts of air are pushed through perforations in the bottom of the converter. The resulting metal is a mixture of iron and oxygen along with other elements. Special compounds are then added to remove excess oxygen and restore the correct amount of carbon and other elements for the specific steel application.[9] The Bessemer Process revolutionized the world of construction. Later on in the 1950's the Seimens-Martin

[7] Blais, 2006.
[8] Trahair, N.S. The Behavior and Design of Steel Structures. 39.
[9] Gilbane. Build the Building. 12.

process of basic oxygen steel making was created as an improvement to the Bessemer Process.[10]

Steel design can be an extremely time consuming task depending on the size and complexity of the structure. No matter how complex the project, there are three common resources that structural engineers utilize. The oldest resource and most commonly used is the Allowable Strength Design (ASD), and the most recent resource is the Load and Resistance Factored Design (LRFD). In addition, the American Institute of Steel Construction (AISC) contains a combination of the ASD and LRFD. AISC created a manual with several editions that are frequently being updated. It is also another resource that is widely used by structural engineers to analyze and design steel structures.

2.2.2 Thermal Properties of Steel

The critical temperature for steel is measured to be approximately 540°C. Typically, fires reach this temperature within a few minutes. Structural steel requires external insulation in order to prevent the steel from absorbing enough energy to reach this temperature. First, steel expands, when heated, and once sufficient energy has been absorbed, it softens and loses its structural integrity. This is easily prevented through the use of fireproofing. The use of a bounded fire protection system allows steel structures to have an acceptable fire-resistance rating for building applications. A fire-resistance rating is the duration for which a passive fire protection system can withstand a standard fire resistance test. The addition of fireproofing is necessary to meet the passive fire protection requirements that are mandated through building codes; thus, the cost increases to construct a steel building.[11]

2.2.3 Design of Steel Structures for Fire Safety

It is to be noted that over the past 30 years tremendous progress has been achieved in developing the appropriate design methods for steel structures in fire. Structural behavior in fire is a complex issue and new conclusions are continuously being drawn. Although the standard fire resistance test is a convenient way for ensuring quality control and grading the relative fire performance of different types of structural members,

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[10] "Structural Steel." http://en.wikipedia.org/wiki/Steel
[11] "Structural Steel." http://en.wikipedia.org/wiki/Steel

for a number of reasons, it is not very effective in developing an understanding of realistic structural behavior in a fire.[12]

The objectives of a fire resistant structure are to prevent fire spread from the room of origin to adjacent spaces and to provide a stable structure for safe egress. This forms part of the compartmentation strategy in the fire safety design of the building. The purpose compartmentation is to subdivide a structure into “fire compartments” that may contain single or multiple rooms for the purpose of limiting the spread of fire, smoke and flue gases, in order to enable the three goals of fire protection: life safety, property protection and continuity of operations.[13]

2.3 Reinforced Concrete Design

Both concrete and reinforced concrete are commonly used in construction practices all over the world. From buildings, to dams, bridges, underground structures, water tanks, television towers, and even ships, concrete has proven a satisfactory and economical material. The popular use of the material is credited to the wide availability of reinforcing bars, as well as the elements used to mix concrete. The simplicity of using concrete in construction also contributes to the common practice.

One important characteristic of concrete is its fire resistance. A building’s structure must withstand the properties and effects of a fire for a sufficient amount of time to allow occupants to evacuate and fire personnel to extinguish the fire. A concrete building typically has a 1- to 3-hour fire rating. Other construction materials, such as timber and steel, require fireproofing to achieve this fire rating. In this respect, concrete has an economical advantage as well.

2.3.1 Early Concrete

There have been many documents written about the various buildings of the Roman Empire constructed using concrete as the primary material. However, numerous researchers argue that the first use of a cementitious binding agent, compared to the lime used in ancient mortars, occurred in southern Italy in the second century B.C.[14] A

[12] Wang, Y.C. Steel & Composite Structures: Behavior and Design for Fire Safety. 228.
[13] “Passive Fire Protection.” http://en.wikipedia.org/wiki/Passive_fire_protection.
[14] Shaeffer, R.E. Reinforced Concrete: Preliminary Design For Architects and Builders.

volcanic sand found near Pozzuoli in the bay of Naples, known as pozzuolana was used by the Romans in their cement.[15] This unusual sand reacts chemically with water and lime, solidifying into a rocklike mass. It is known that pozzuolana was used to bind stones together to make concrete in the construction of the Porticus Aemelia, a large warehouse built in 193 B.C.[16]

It is expected that pozzuolans were not used elsewhere due to the lack of availability. As a result, stone and brick masonry remained the common construction materials for most of the world's major buildings for many centuries.

Roman concrete was manually mixed by packing mortar in and around stones of various sizes. This compilation was finished with clay bricks on both sides. According to Shaeffer, the bricks had minimal structural value and were used to facilitate construction and as surface construction.[17] Roman concrete has little resemblance to modern Portland cement concrete. It lacked the plastic characteristics that could flow into a mold or a construction formwork. [18]

Most public buildings in Rome used brick-faced concrete construction for walls and vaults. Built in the second century A.D., the Pantheon was a structural masterpiece of the time.[19] The structure contains many weight-reducing features, such as voids, niches, and small vaulted spaces.[20] The builders of the Pantheon recognized the concept of using heavy aggregates at the ground level and aggregates of decreasing density on each proceeding level in the walls as well as the dome itself. This application reduced the weight to be carried throughout the higher floors. Mainstone states in his text that the Pantheon's clear span of 142 feet created an architectural revolution in terms of the way interior space was perceived.[21]

The rediscovery of concrete occurred in the eighteenth century by the English engineer, John Smeaton while designing the Eddystone Lighthouse off the south coast of England.[22] [23] Smeaton discovered that a mixture of limestone and clay could be used to

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[15] Ibid.
[16] Ibid.
[17] Ibid.
[18] Ibid.
[19] Ibid.
[20] Ibid.
[21] Mainstone, Rowland J. Developments in Structural Form. 116.
[22] MacGregor, James & Wight, James. Reinforced Concrete: Mechanics and Design. Fourth Edition.

make cement that would set under water and would then prove water resistant.[24] [25] Though in turn, Smeaton conservatively used the common approach of mortised stonework on the lighthouse, his discovery sparked others to revive the use of Roman concrete.[26, 27]

2.3.2 Thermal Properties of Concrete

When concrete is exposed to high temperatures, such as those of a fire, it will behave adequately for a substantial amount of time. The surface layers of the concrete expand and eventually cause cracking or spalling off the cooler interior section of the concrete due to the high thermal gradients that occur during a fire.[28] The spalling is irritated if water from a fire hosed is applied too abruptly to cool the surface.

The modulus of elasticity and the strength of concrete decrease when exposed to high temperatures.[29] On the other hand, the coefficient of thermal expansion increases under these conditions. It has been noted that strength reduction and spalling as a result of heat are most common in wet concrete. Thus, fire is most crucial for young concrete.

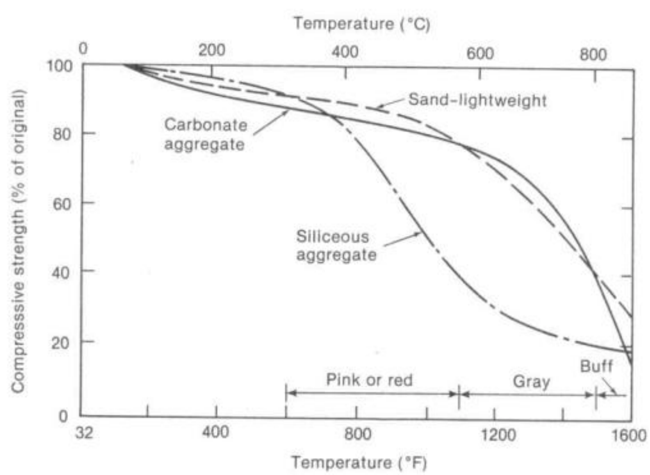

Figure 1: Concrete Strength versus Type of Aggregate

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[23] Cowan, Henry J. Design of Reinforced Concrete Structures.
[24] MacGregor, James & Wight, James. Reinforced Concrete: Mechanics and Design. Fourth Edition.
[25] Cowan, Henry J. Design of Reinforced Concrete Structures.
[26] MacGregor, James & Wight, James. Reinforced Concrete: Mechanics and Design. Fourth Edition.
[27] Cowan, Henry J. Design of Reinforced Concrete Structures.
[28] MacGregor, James & Wight, James. Reinforced Concrete: Mechanics and Design. Fourth Edition.
[29] Ibid.

The compressive strength is not as impacted by temperature as is tensile strength. As shown in Figure 1, the strength reduction varies by type of aggregate.

There are three main types of aggregates: siliceous, lightweight, and carbonate. The quartz in siliceous aggregates, for example, granite, quartzite, schists, and sandstones, will experience a phase change at about 800 to 1000ºF. The lightweight aggregates lose their strength progressively at temperatures exceeding 1200ºF. Concretes composed of carbonate aggregates, such as limestone and dolomite, tend to be rather unaffected by temperature. However, when temperatures reach 1200 to 1300ºF, these aggregates experience a chemical change and quickly lose strength.[30]

Figure 1 also exemplifies the color variance of concretes due to a fire. Such materials as limestone and siliceous aggregates within concrete have a tendency to change color with rising temperatures. This color change is used to indicate the approximate temperature reached by the concrete. Typically, the strength of concrete turned beyond pink is questionable. Concrete that has changed to gray, which is past the pink stage, is commonly badly damaged and should be removed and replaced with a new layer of concrete.

On the contrary, low temperatures have the opposite effect on concrete, increasing the strength of both hardened and moist concrete, given the water does not freeze.[31] Subfreezing temperatures can greatly increase the compressive and tensile strengths as well as the modulus of elasticity of moist concrete. However, dry concrete is not as affected by low temperatures.

The journal article, "Physical Properties of Concrete at Very Low Temperatures" notes that concrete had a strength of 5000psi through compression tests and a strength of 17,000psi at -150ºF. This same concrete was tested oven-dry and at an interior relative humidity of 50 percent, tested a 20 percent increase in compressive strength from the strength at 75ºF. The results of the concrete undergoing the split-cylinder tensile strength showed an increase from 600psi at 75ºF to 1350psi at -75ºF.[32]

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[30] Ibid

[31] From MacGregor: Monfore, G.E. and Lentz, A.E. "Physical Properties of Concrete at Very Low Temperatures." *Journal of the PCA Research and Development Laboratories*, Vol. 4, No. 2, May 1962. 33-39.

[32] Ibid.

2.4 Historic Construction and Masonry

According to the Massachusetts Cultural Resource Information System (MACRIS), the building at 68 Prescott Street was built in 1912; however, an 1876 Sanborn map of the site area places a building at 68 Prescott Street as early as 1870. For the purpose of this project, we will utilize an early 1900's construction time frame.

The Gateway District is a reflection of the importance of Worcester in the industrial north. One of the first uses of 68 Prescott Street was for the making of agricultural machinery but it is historically known as the United States Envelope Company Factory. The U.S. Envelope Company was organized in 1896 as a partnership of ten smaller envelope companies. They occupied the building until 1963, followed by a significant time of vacancy. It was later subdivided for office use.

While we do not know the particular time the facility was constructed, we do know that it is of masonry exterior construction with interior timber framing. Masonry construction consistent to the system currently in place at the Gateway interacts differently than more current masonry buildings. In old frames the weak mortar, in addition to pre-compression stress from the load bearing weight of the wall, resulted in the stresses to be spread throughout the wall rather than concentrated along the diagonal. In modern buildings, exterior walls are only meant for enclosure. The brick is laid in a cement mortar resulting in stiff walls. In current systems, exterior walls are not designed to carry any loads, if a lateral load results in too great of a deflection, the load is forced onto these walls resulting in one large failure in the wall. In old framing systems, instead of one large failure in the wall, the softness in the mortar results in a small scale cracking across the mortar joints along the panel.[33]

2.4.1 Masonry Construction

One of the most common types of construction consists of masonry bearing walls supporting the structural elements that carry the loads of the floors and walls. At the time of construction it was necessary for the masonry walls to be thick enough to carry the loads and resist the lateral forces due to wind loads. The term masonry is defined in

[33] Langenbach, Randolph. "Historic masonry Construction: Bricks, Mortar, and Earthquakes: Historic Preservation vs. Earthquake Safety."

Construction Materials and Processes, third edition, as including "most types of solid or hollow unit building materials which are held together with mortar, for example, brick, stone, hollow clay tile, concrete block, gypsum block and even glass block."[34] Since mortar functions to bond the masonry units together, it is important that mortar have little shrinkage, a resistance to moisture, and strength to resist applied forces.

The standard mortar used during the timeframe was a lime base. Lime mortar was used from ancient times until the late 19th century because it was relatively flexible and would accommodate the movement of buildings due to thermal expansion and contraction. However, lime mortar is now banned from most building codes, as it is weak.[35] Today, Portland cement is used almost exclusively, the exception being if a historic building is being renovated.[36]

2.4.2 Masonry Reconstruction

The masonry building's exterior was deteriorated, and it was necessary to restore the damage during renovations. There are four basic physical causes for deterioration.

- Freeze/thaw cycling
- Wet/dry cycling
- Thermal expansion/contraction
- Salt crystallization

The freeze/thaw cycling is the most common cause of deterioration.[37] The theory is that water freezes in the pores of the masonry unit, narrowing the pores. This leads to the break down of the pores, and in turn causes fracturing of the unit. The wet/dry cycle contains a capillary action, and it can result in a force that exceeds the strength of the unit. The brick absorbs moisture causing the brick to expand, when the brick dries out it shrinks, eventually the brick will fail as a result of this cycle.[38] The thermal expansion/contraction results in the entire structure expanding and contracting with changes in temperature. With inflexible mortar use, the building cannot accommodate this movement. Lastly, when salt crystallizes on the surface of the masonry unit, it is not

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[34] Watson. Construction Materials and Processes. 80.
[35] Ibid. 103.
[36] "Masonry"
[37] Watson. Construction Materials and Processes. 103
[38] "Brick face spalling from sealant"

very harmful; however, if crystallization occurs under the surface of the masonry, it can lead to crumbling and spalling.[39]

There are a number of ways to determine the cause of deterioration of masonry construction. The age of the building is imperative to determining the cause of the failure, due to the difference in construction and mortar as previously discussed. It is also important to consider where in the structure the failure is occurring, if an area of deterioration is near a downspout or windows, the cause of deterioration is due to excessive moisture. However, if the failure is near a masonry opening it could be the result of the loading. The direction (vertical or horizontal), of a crack will also provide insight into the cause of the deterioration. While "a clean crack indicates recent movement; a dirty or previously filled crack may be inactive."[40] In addition, cracks correlating to expansion and contraction may be open one season and closed with the next. Lastly, if there is dusting or flaking of masonry units, there is likely chemical deterioration of the unit.[41]

2.4.3 Timber Construction

Timber has been used as a material as early as the ancient Egyptians in 2500 BC. "In the industrial era of the 19th century timber was used widely for the construction not only of roofs but also furniture, waterwheels, gearwheels, rails of early pit railways, sleepers, signal poles, bobbins and boats."[42] Although timber was used extensively for structural members in the early 1900's for the construction of buildings, it has given way in more recent times to structural steel and reinforced concrete. These construction materials provide more flexibility because of their greater strength and longer spans.

The application of external forces can result in deformation of the timber. The deformation is a result of timber not being truly elastic, it is dependent on time and magnitude of the applied stress and the physical characteristics of the specific wood.[43] In addition, the main causes of deterioration in wood are decay, insects, and fire.

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[39] Ibid. 103.
[40] Old House Web. "General Masonry Inspection- cracking, vowing, spalling, sweeping…and other general issues about masonry structural systems."
[41] Ibid
[42] Dinwoodie. Concrete, Timber, and Metals the nature and behavior of structural materials. 106.
[43] Ibid. 305.

Techniques have been developed to protect wood from the elements of nature, fire, and insects by improving.[44]

2.4.4 Scope of Consigli Construction

The interior framing system within the masonry building was structurally sound at the time of purchase and did not require extensive reconstruction. In the lecture hall located on the first floor, a steel beam was inserted to carry the load in place of columns that were taken out to create better visibility within the room. The wood floors were not level, and in an effort to fix them Consigli poured a concrete slab over the existing floor structure. The interior framing system was kept and no other modifications were made. Each floor was designed by the occupants and partitions were added accordingly.[45]

Consigli Construction also rehabilitated and made modifications to the exterior of the structure. New windows were installed and the brick mortar was replaced, as both were deteriorated with age. Additionally, sections of an exterior wall were removed to create a passageway between the existing structure and the new structure, known as the "the link".[46]

The entire wall was not removed in order to keep the structures separate as they move independent of each other laterally. There is approximately a six-inch to one-foot gap between the two structures. As discussed previously, the historic masonry construction deflects laterally and carries loads differently than current construction.[47]

2.5 Building Codes and Fire Protection

The primary method of regulating building safety is through the implementation of building codes. Building codes establish minimum criteria for safe construction to protect the lives of the public. Some notable organizations and their building codes include the International Code Council (ICC), who publishes the <u>2006 International Building Code</u> (IBC), and the National Fire Protection Association (NFPA), the publisher of the <u>Building Construction and Safety Code: NFPA 5000</u>.

[44] Watson. <u>Construction Materials and Processes</u>. 154.
[45] Johnson, 2006.
[46] Ibid.
[47] Watson. <u>Construction Materials and Processes</u>.

For the purposes of this project, and to create a synergy with our Fire Protection Engineering graduate classes, we will focus on the provisions of the 2006 IBC. The IBC is more widely adopted in the United States than NFPA 5000, and it is expected to form the basis for the next edition of the Massachusetts State Building Code.

The International Code Council, "…was established in 1994 as an umbrella organization consisting of representatives of…" the International Conference of Building Officials (ICBO), Southern Building Code Congress International (SBCCI) and Building Officials and Code Administrators (BOCA).[48] These organizations came together with the ideal of creating a single set of codes to replace the regional codes that had developed. Their success in developing the IBC, as well as many other codes, led to the consolidation of all three groups created an association of more than 50,000 members.[49]

2.5.1 Code Development

The National Board of Fire Underwriters developed the first model building code in the United States in 1905.[50] This 1905 code was an expansion of a code proposed several years earlier for the state of New York.[51] Early codes such as this one were created by the insurance industry to ensure profits by attempting to reduce or prevent fires through the implementation of codes and standards.[52] However, since then building codes have shifted their main goal from protecting insured properties to providing life safety and protecting all properties.

To provide life safety and property protection, codes specify minimum standards for construction quality, which may implement either prescriptive or performance-based design, or elements of both. Prescriptive codes set forth, "…construction requirements according to particular materials and construction methods, rather than to performance criteria."[53] On the other hand, performance-based building codes specify construction standards based on, "…performance criteria rather than to specific building materials, products, or methods of construction." Traditionally, building codes have been based on

[48] Diamantes, David. The Principles of Fire Prevention. 72.
[49] Ibid. 73.
[50] Ibid. 69.
[51] Ibid. 69.
[52] Ibid. 68.
[53] "Explain Prescriptive." http://www.teachmefinance.com/Financial_Terms/prescriptive_code.html

prescriptive design methods because although fire safety system design is very complex, it is still a relatively new and developing field.[54] With underdeveloped technology and a necessity to combat the fire problem, codes were developed based on existing practices in the building industry.[55] However, other engineering fields such as structural design implement performance-based design.

Prescriptive codes such as the IBC have been accepted for more than a century within the fire protection engineering field; however, shifts in the industry are creating a movement toward performance-based code design, such as embodied in NFPA 5000. Since their inception, building codes have expanded to meet the needs of the ever-changing construction environment and new technology. The birth of high-rise building design and underground facilities exemplify some of the challenges posed by the dynamic building design to which today's codes must adapt. These factors, as well as the need for continued safety code development, may be contributing to the movement from prescriptive toward performance-based design.

2.5.2 Code Organization

Building codes create regulations that are specific to the functionality or use of a space. Areas within a building are all given an occupancy classification, by which the jurisdiction's building code may regulate the space's structural elements, fire systems, and other attributes. While some occupancy classifications are typical, such as kitchen or residential spaces, other classification names may vary from code to code. The variation in naming between codes does not imply that there is a corresponding occupancy from one code to the next. Moreover, the parameters by which a space is classified as one occupancy or another also varies between codes. Hence, a space identified as occupancy "X" in one code may or may not have the same classification in another.

With differing occupancy classifications and corresponding regulations, it is expected that more variations will continue to be seen from one code to the next. In terms of the overall organization of codes, NFPA 5000 and the IBC are, perhaps, defined by the distinct framework each code applies. The NFPA 5000 code devotes a chapter to each occupancy classification previously outlined. These chapters list those requirements

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[54] Fitzgerald, Robert. Building Fire Performance Analysis. 2.
[55] Ibid. 2.

specific to the occupancy and then refer the reader to other chapters for broad guidelines on topics such as means of egress. In contrast, the 2006 IBC provides one inclusive chapter for each topic and within that chapter lists exceptions or variations that may occur for a particular occupancy.

Despite some divergence, building codes strive to achieve the same basic health and safety objectives through regulation. Thus, codes generally encompass the same topics and building elements. From height and area limitations to fire protection systems and means of egress, codes establish minimum standards for acceptable construction.

2.5.3 Code Becomes Law

Building codes such as the IBC and NFPA 5000 only serve as model codes. For a building code to be made into law it must be adopted by the authority having jurisdiction (AHJ). Once adopted the AHJ may implement and enforce the building code within the district or zoning area.

Model codes may be adopted as a whole or in part. For example, the Sixth Edition of the Massachusetts Building Code is based on the specifications of the 1993 National Building Code written by the Building Official Code Administrator's (BOCA), another model building code association that merged with others to form the International Code Council.[56] However, the Massachusetts Building Code implements BOCA's 1993 code, while making some significant changes; therefore, this code is unique to the state of Massachusetts.[57]

[56] Galvin, William. "Massachusetts State Building Code: User's Guide to CMR 780, 6th Edition." http://www.mass.gov/Eeops/docs/dps/BuildingCode/780CMRUSG.pdf. 3.
[57] Ibid. 3.

To complete our project objectives and gain a better understanding of what the Gateway Park development entailed we investigated historic construction and masonry. This allowed us insight into some of the issues that might come up in dealing with renovating a 100-year-old masonry building. Also, gaining a better understanding of brownfield development helped us understand the social and economic implications of this project.

These topics, along with our research on structural steel and reinforced concrete design enhanced our methodology and provided us with the proper resources to develop structural alternatives. By understanding the nature of the materials we were using and the design methods associated with them we were able to develop new building frames.

Lastly, building code research developed a basis for all of the project work involving fire protection engineering. Through knowledge of code development and differing code structures we were able to better understand the codes. This understanding was the basis of our investigation of the compliance of the egress safety and fire protection systems of the building. Through completing this background research we were able to focus our report and develop a more succinct methodology.

3. Methodology

Our group performed a structural analysis to get a sense of proportion, sensitivity, and cost for the Gateway Project. To complete the structural analysis we divided our project group into two teams of two people. Each group explored a different construction material, either concrete or steel, to perform a structural analysis and provide several design alternatives. Based on depth of construction and feasibility of the design, both groups then chose the best one or two alternatives from their design to further investigate with cost estimates. The feasibility of each structural system was determined by determining the available vertical space within the constraints of the building.

Additionally, this report addresses several issues within the field of Fire Protection Engineering. A Fire Safety Code Analysis was completed based on the information and measurements collected from site visits. This section addresses issues related to building egress and the different elements that facilitate egress. Also, we investigated the code compliance of the building's Fire Protection Systems, based on the IBC's provisions. Through inspection of the site and by the information provided in the fire protection specifications, we studied the passive and active systems installed in the building. Lastly, we performed a risk assessment to address possible fire scenarios and their implications.

We completed our project with the following objectives:

(1) Understanding historic construction.

(2) Development of a structural analysis and design alternatives with a cost analysis.

(3) Developing a synergy with undergraduate education and Fire Protection Engineering work.

3.1 Gravity Load Design

A load refers to any type of force exerted on an object, which may be in the form of a weight or gravitational force. The most important aspect of a structural engineers' job is to accurately estimate the loading that a structure may endure over its lifetime. In gravity load design, both dead and live loads are investigated. Dead loads are loads of constant magnitude that remain in one position.[58] They consist of the structural frame's own weight and other loads that are permanently attached to the frame. For a steel-framed building, the frame, walls, floors, roof, plumbing, and fixtures are dead loads. Live loads are loads that may change in position and magnitude.[59] They are caused when a structure is occupied, used, and maintained. Publications of methodologies such as the Load and Resistance Factor Design (LRFD) and the American National Standards Institute (ANSI) are necessary to design structures appropriately.

3.1.1 Structural Steel Design

We designed steel structural systems using composite beam and slab design and rolled steel beams. The steel frame design alternatives were designed to withstand loads and forces and act independently from the existing brick exterior of the building. We followed the LRFD method and the provisions of AISC in our design method for the structural steel alternatives.

Our methodology for gravity load design was based on the LRFD provisions of the American Institute of Steel Construction (AISC). This design method is based on the use of *limit states*, or, "…the condition at which a structure or some part of that structure ceases to perform its intended function."[60] This method uses safety factors to increase the scale of the calculated design loads applied to the structure to allow for uncertainties involved in estimating loads.[61] The factored loads are used to calculate the critical or governing load combination that will be used in the design process.[62]

The American Institute of Steel Construction is a non-profit technical institute and trade association established to serve the structural steel design community.[63] AISC has traditionally served the steel construction industry by providing reliable information through publishing technical handbooks.[64] AISC introduced the LRFD method into its handbook for its ability to provide more reliable steel structures under any loading. [65]

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[58] McCormac, Jack. Structural Steel Design. 702.
[59] Ibid. 702.
[60] Ibid. 48.
[61] Ibid. 49.
[62] Ibid. 51.
[63] AISC. "Our Mission: Making Structural Steel the Material of Choice." http://www.aisc.org/Content/NavigationMenu/About_AISC/Mission/Mission.htm
[64] Ibid.
[65] McCormac, Jack. Structural Steel Design. 57.

Also, the language and style used in this method illustrate advances made in structural steel design methods over the years.[66] This method, as opposed to the use of others such as Allowable Stress Design (ASD) may also offer some economic advantage depending on the ratio of the dead and live loads.[67]

The assumptions used to design the steel structural alternatives are as follows:

- Young's Modulus, Modulus of Elasticity (E) = 29,000 ksi
- Minimum Yield Stress (F_y)= 50 ksi
- Tensile Stress (F_u)= 60 ksi
- Concrete Compressive Strength (f'_c) = 3.5 ksi
- Stud diameter of ¾"
- Concrete floor slab thickness = 5"
- Fixed end connections

Loading

The loading conditions for the structure were determined based on the provisions of the IBC. Both the dead and live load conditions were determined for each building level, based on the occupancy of the space.

The primary dead load for our design was the weight of the slab. With unshored construction the beams must support the weight of the wet concrete during construction, as well as construction loads.[68] Thereafter, when the concrete has set and gained strength, all loads carried, "…may be considered to be supported by the composite section."[69] We chose to use a 5-inch slab, which provides a 2-hour fire resistance rating between floors.[70] Weighing approximately 145 lb/ft^3, the slab dead load was determined to be 60.4 pounds per ft^2 (psf).

To determine the total dead load, we assumed the additional following loads:

Mechanical, Electrical and Plumbing (MEP): 5psf

Construction Load: 16psf

Ceiling: 2psf

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[66] Ibid. 57.
[67] Ibid. 57.
[68] Ibid. 518.
[69] Ibid. 518.
[70] MacGregor, James & Wight, James. Reinforced Concrete: Mechanics and Design. Fourth Edition. 436.

Thus, the total unfactored dead load totaled 83.4psf. The live loads for the first through third floors were determined from Table 1607.1 in the IBC, with a 100psf load for the first floor lecture hall with movable seats and a 50psf dead load for the office space on the second and third floors.[71] The live load for the roof was a snow load.[72] The unfactored loads for each floor and the roof are listed in the Table 1.

Table 1: Steel Gravity Design Dead and Live Loads

Floor	Dead Load	Live Load
First	83.4	100
Second	83.4	50
Third	83.4	50
Roof	55	50

We utilized the LRFD method of applying load factors to building loads to account for uncertainty in loading. The load factors applied to either the dead or live load are dependent on the load combinations. Using the load combination equations, sited in the IBC, published by the American Society of Civil Engineers (ASCE) we determined the governing load combination.[73] The two equations most relevant to the design of the gravity systems can be seen below:

$$1.2 * (\text{Dead Load}) + 1.6 * (\text{Live Load})$$

$$1.2 * (\text{Dead Load}) + 1.6 * (\text{Snow Load}) + 0.5 * (\text{Live Load})$$

The governing load combination is equivalent to the equation that produces a critical value. ASCE 7-98 requires that, "…structures and their components are to be designed so their strengths is at least equal to the values obtained with the load combinations."[74] The method of design helps ensure that the calculations and corresponding structural designs are conservative.

[71] 2006 International Building Code. 285.
[72] Ibid. 285.
[73].Ibid. 282.
[74] McCormac, Jack. Structural Steel Design. 50.

From the critical load combinations, we designed the typical bay areas for each floor layout. The three varying bay sizes we chose to investigate were 69' x 27.75', 34.5' x 27.75, and 34.5' x 18.5'. An example layout for the 34.5' x 18.5' bay size is shown in the Figure 2.

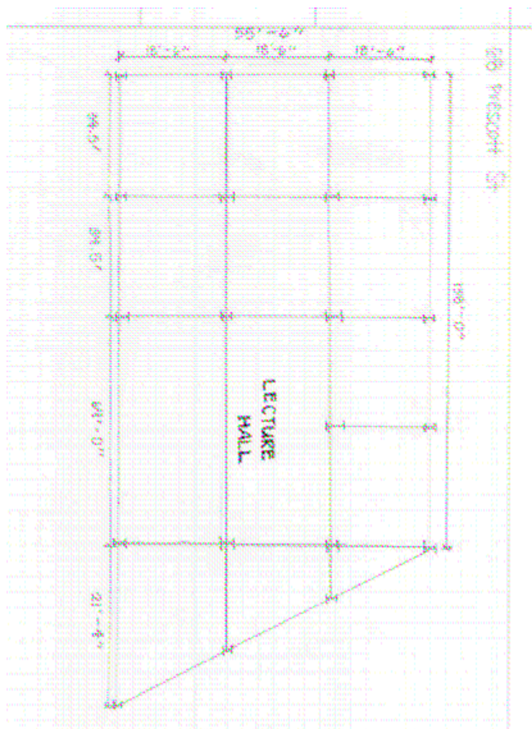

Figure 2: Floor Layout for 34.5' x 18.5' Bay Size

By creating steel frame alternatives based on these floor layouts we explored which option would offer the most flexibility. The flexibility in the gravity system designs was determined by focusing on vertical flexibility allowed by the required depth of the beams and girders.

Beam Design

The beam designs for each layout were prepared with differing spacing based on the number of filler beams for each bay, ranging from 2 to 5 filler beams. The varied spacing of the beams allowed us to explore the effect of tributary width on the load per foot of area (w_u), which is calculated from the governing load equation. Using the w_u value we calculated the governing moment for each beam and the effective flange width. We then proceeded to select W sections with the aid of reference tables from AISC's Steel Design Manual 2005.

To provide connections for each beam we chose to use ¾" diameter studs, which are the typical size stud used for building construction. Based on the area of the stud and the nominal shear strength of the studs, shown in Table 16.1 of McCormac's text, we calculated the number of studs needed for the composite beam and slab system. Acceptable stud spacing was then determined based on the length of the beam and the number of required studs. These spacing were then checked against the minimum and maximum center-to-center spacing permitted by LRFD Specifications.[75]

Girder Design

For each bay size we chose a beam that best fit our criteria for the desired building design. The selected beams were chosen for the vertical flexibility they allowed based on beam dimensions. From this beam weight and corresponding number of filler beams, a girder was designed to complete each bay. The girder design was completed with the same method used for beam design.

To complete the girder design we determined the number of shear studs required per girder, as well as their spacing. From these structural alternatives we chose the best option for the girder and beam system for each bay size based primarily on the depth of construction.

Column design

The column designs for the steel frames were established based on a 2-floor continuation. In other words, the 1st and 2nd floors were designed to have one continuous column run the vertical length of these floors. The same approach was applied to the 3rd and 4th floors. Due to this choice of methodology, the design load for the columns was based on the total load incurred at the lower floor.

Using the column strength equations from the LRFD Specification, we determined the critical or buckling stress for the columns, F_{cr}. Using this value we were able to calculate the required area to support the compressive loading. For each floor we designed corner, exterior and interior columns.

Slant Beam Design (Atypical Area)

The final step in gravity load design for the masonry building was to design the non-uniform or atypical area that was not covered by the typical bay size layout. To

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[75] Ibid. 525.

accomplish this we designed slant beams for the atypical area on the east side of the building. Using the Figure 3 as an example, we can see the atypical or slant beam design area outlined in red.

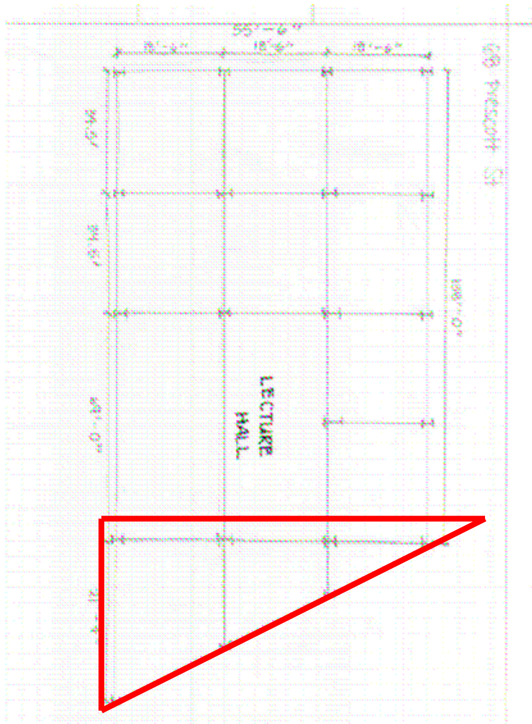

Figure 3: Floor Layout for 34.5' x 18.5' Bay Size

Atypical design was based on triangular loading within the space rather than the distributed loading used in the beam design for the typical areas. However, the subsequent methodology for our slant beam design followed that of beam design for the typical areas, including determining the required number of shear studs and their acceptable spacing.

Some portions of the design work were performed through hand calculation; however, spreadsheets were developed to supplement the manual calculations. Selected examples of the steel design hand calculations can be seen in Appendix F.

Subsequent to the actual design of the steel framing systems, we identified two designs that allowed for the greatest vertical flexibility. This identification was based on the depth of the beams and girders for each alternative that was designed. For the selected designs cost estimates were prepared. The methodology for the cost estimates can be seen in Section 3.3 of the report.

3.1.2 Reinforced Concrete Design

We designed a reinforced concrete frame system using the direct design method, as presented in the American Concrete Institute (ACI) Code Section 318. The direct design method involves visualizing the floor slabs in terms of panels and determining the statical moments within the panels. A set of prescribed coefficients give the negative and positive moments within the panel. However, there are limitations to the direct design method which include the stipulation that there be fairly regular multi-panel slabs. The limitations are given in ACI Section 13.6.1.[76]

In designing the gravity load system for concrete, we conducted design calculations for both one-way and two-way slab systems for the purpose of comparing and analyzing the alternatives both feasibly and economically. The following assumptions were made for the purpose of our calculations:

- F_y = 40 ksi (60 ksi for one way slab design calculations as given in example followed in MacGregor[77])
- f'_c = 3.75 ksi
- F_u = 60 ksi

One-Way Slab Design

Modeling our calculations after examples found throughout the fourth edition of Reinforced Concrete: Mechanics and Design, a textbook by MacGregor and Wight, we began with a one-way slab-and-beam system. For design purposes, a one-way slab is assumed to act as a series of independent, parallel strips with a width of 1-ft.[78] The slab carries the loads to the beams which, in turn, transmits the loads to girders and lastly the columns.

The ACI Code Section 9.5 gives the minimum thickness of slabs not supporting or attached to partitions or other construction susceptible to large deflection damages.[79] There is no aid for other scenarios. As a result, we chose our slab thickness based on the danger of heat transmission during a fire. The fire rating of a floor is equal to the number of hours of exposure in a standard furnace test needed for the temperature of the

[76] MacGregor, James & Wight, James. Reinforced Concrete: Mechanics and Design. Fourth Edition. 624.
[77] Ibid. 443.
[78] Ibid. 436.
[79] Ibid. 436.

unexposed surface to reach a limiting value. The temperature is a given value, typically 250˚F.[80] To be conservative, we chose a 5-inch slab thickness, which gives a 2-hour fire rating.[81] Consequently, a 5-inch slab thickness yields an exterior bay size of 21'x 18' and an interior bay size of 21'x 19'utilizing to ACI Table 9.5.[82]

Generally, the slab is also supported by steel reinforcement. Concrete cover provides corrosion resistance, fire resistance, and a wearing surface to bond the two materials. Due to the fact that we are dealing with interior building structures, which will not be exposed to weather or in contact with the ground, MacGregor suggests the use of No. 11 bars and smaller, with a ¾-inch clear cover recommended.[83]

In determining the total load, we assumed the following additional dead loads:

Floor cover: 0.5psf

Mechanical equipment: 4psf

Ceiling: 2psf

Thus, with the trial unfactored load, the total dead load equals 69 psf. We have established the live load to be 100 psf, based on assembly occupancy.[84] Subsequently, we applied these loads to the ACI Code load combinations for concrete design from ACI Section 9.2.1 to obtain the governing factored load on a typical section.

A tension-controlled member is defined as one that has an extreme tensile strain greater than or equal to 0.005 at ultimate.[85] Therefore, the slab proves to be tension-controlled and meets the definition of a beam. As a result, we chose the strength-reduction factor $\varnothing$=0.90 for flexure.[86] The next series of calculations was performed to confirm the slab thickness was adequate for the moment and shear.

Additionally, the reinforcement was determined for the slab. The straight-bar arrangement of reinforcement is almost always used in buildings with one-way slabs due to inexpensive costs and ease of construction.[87] The area of reinforcement was computed as A_s/ft of width. Because one-way slabs are designed with 1-ft width, the area of steel

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[80] Ibid. 436.
[81] Ibid, 436.
[82] Ibid. 442.
[83] Ibid, 436.
[84] 2006 International Building Code.
[85] ACI Section 10.3.4
[86] ACI Section 9.3.2.1
[87] MacGregor, James & Wight, James. Reinforced Concrete: Mechanics and Design. Fourth Edition. 437.

required equals the product of the area of a bar times the number of bars per foot. Equation 1 illustrates this relationship:[88]

$$A_s/ft = A_b \left(\frac{12 \text{ in.}}{\text{spacing of bars}} \right)$$

Equation 1: Area of Steel Bars Required

where A_b is the area of one bar. In one-way slabs, the maximum bar spacing is three times the slab thickness or 18 inches, whichever is smaller.[89] However, to maintain crack control, ACI Section 10.6.4 restricts the maximum spacing of flexural reinforcement closest to the tension face of a slab as shown in Equation 2.

$$s = \frac{540}{fs} - 2.5Cc \quad \text{but not greater than} \quad 12\left(\frac{36}{fs}\right) \text{ in.} \qquad \text{(ACI Eqn. 10-5)}$$

Equation 2: Maximum Spacing of Flexural Reinforcement

where fs is the stress in tension steel, and Cc is the clear cover to control the width of cracks on this face of the slab.[90] The calculated s = 12 inches overrides the 18-inch maximum spacing limit previously mentioned.

A spreadsheet was used to repeat the slab design calculations for varying bay sizes. The second scenario includes a 27.5' x 30' bay size; and lastly, the third alternative consists of 20' x 25' exteriors and 15' x 25' interior bays, resulting in a 7.5-inch and 6.43-inch slab thickness, respectively.

Beam Design

Example 4-7 of MacGregor was followed in the design of rectangular beams. This specific example was calculated "when *b* and *h* are unknown."[91] The first step was estimating the dead load of the beam. The weight of a rectangular beam will be

[88] Ibid, Equation 10-5, 437
[89] ACI Section 7.6.5
[90] MacGregor, James & Wight, James. Reinforced Concrete: Mechanics and Design. Fourth Edition. 451.
[91] Ibid. 151.

approximately 10 to 20 percent of the load it must carry.[92] We then computed the factored moment. Next, we had to select a trial steel ratio, ρ, in order to calculate ϕk_n. To select ρ, economic, ductility, and placing considerations were made. We set $\rho = 0.01$ by considering ductility and economy.[93] We then calculated *b* and *d* and revised M_u. Then, it was necessary to calculate the area of reinforcement and select the reinforcing steel corresponding to the minimum area required. The beam design was checked to see if it was tension-controlled and the area of reinforcement was checked for sufficient moment-resistance.

Continuous Girder Design

To design the continuous girders, the fifth edition of Parker and Ambrose's Simplified Design of Reinforced Concrete was used. The first phase of this design was determining the concentrated and distributed loads. The uniformly distributed load includes the weight of the girder as well as the superimposed loads of the floor area tributary to the girder's tributary width.[94] For an approximate design, we considered the total load as a distributed load singly applied and then utilized the moment factors from the ACI Code for positive and negative moments.

In considering flexure, the use of a section with compressive reinforcing at the maximum negative moment of the interior column was considered, allowing reduction in the girder size while enhancing the strength of the girder-column connection.[95] This additional reinforcement also contributes to resisting wind and seismic forces on the building. Parker recommends designing a section with a balancing moment capacity about two-thirds that of the total required resisting moment.[96] The section was checked for adequacy for shear in order to design the stirrup spacing for the maximum shear stress. The role of the stirrups is to act as ties for the compressive reinforcing at the interior columns.

Continuous T-beam Design

The next scenario analyzed was a one-way slab and continuous T-beam. The design of the continuous T-beam was performed for the first scenario discussed in which

[92] Ibid. 151.
[93] Ibid. 151.
[94] Parker, Harry and Ambrose, James. Simplified Design of Reinforced Concrete. Fifth edition. 167-168.
[95] Ibid. 168.
[96] Ibid. 168.

there was a 5-inch slab thickness. The slab of the T-beam acts as the top flange of the beam. The first step in this procedure was to calculate the effective flange width. Per ACI Section 8.10, the width of slab effective as a T-beam flange shall not exceed one-fourth the length of the beam. In addition, the effective overhanging slab width on the sides of the web cannot exceed more than eight times the slab thickness or one-half the distance to the next web, whichever is smallest.

Beam stems were then designed based on the deflection from ACI Table 9-5. Additionally, we calculated the moment capacity. The internal lever arm, jd, was estimated. For singly reinforced beams, the compressive force and the depth of the stress block were calculated based on the tensile force. Then, the compressive force resultant and its line of action were calculated. Lastly, the value obtained for the lever arm was multiplied by either force, compressive or tensile, to compute the moment capacity. It was also necessary to determine the required reinforcing steel for the T-beam.

Two-Way Slab Design

The behavior of a two-way slab in comparison to a one-way slab is that the slab carries load in two directions. This is feasible because the beams are built within the depth of the slab. MacGregor notes that the two-way slab system is an efficient, economical, and commonly used structural system.

There are various forms of two-way slab systems. Flat plates are used for moderately light loads and are often found in apartment buildings. This uniform slab is simply supported by columns. This system is most cost-effective for bay sizes spanning 15 to 20 feet. In the case of larger spans, the thickness required to distribute loads to columns is larger than that required for bending. Therefore, to lighten the slab, save material, and decrease moments, ribs are used to replace the slab located at the mid-spans. This arrangement is known as a waffle slab and is formed with fiberglass or metal forms. This type of slab is typically used for spans from 25 to 40 feet. Futhermore, flat slab systems are used for heavy industrial loads. This system requires thickening the slab near the column with drop panels or by forming a column capital in which the column is flared at the top. This allows the load to transfer to the columns. This scheme is used when loads exceed 100psf and for bays spanning 20 to 30 feet.

As with the continuous T-beam option, we chose to stick with the first scenario bay size of 18"x21" and 19"x21" and a 7.5-inch thick slab thickness was determined to design the two-way slab. It was necessary to determine the moments in a 12-inch wide slab strip for both the East – West direction and North – South direction. As previously mentioned, this is a result of the two-way slab carrying loads in both directions. The next design step was to distribute the moments to the columns and middle strips of the slab. Lastly, we chose our steel reinforcement.

All hand calculations corresponding to the design of the varying reinforced concrete systems can be found in Appendix G.

3.2 Lateral Load Design

The next step of our methodology was to ensure our design is adequate for lateral loadings. Instead of applying the lateral loads to all scenarios, we chose the most desirable of our results. The most desirable scenario was based on feasibility, economics, and vertical flexibility within the space.

Wind Loads

Utilizing the Massachusetts State Building Code, we determined and applied a uniform wind load based on the 90 mile per hour fastest wind speed to the building. This was converted to a point load and applied to each story of the building based on the tributary height and total height. To begin the lateral system design, we implemented the software program RISA-2D Educational Version. This software allows a designer to apply loads to the desired framing system to determine member forces and lateral translations in both the x and y direction. The total deflections of the system were calculated by the software and can be found in Appendix H.

Seismic Loads

The seismic load applied to our building was determined utilizing the Massachusetts State Building Code (MSBC). First, it was necessary to determine the weight of both the steel and concrete framing system: beams, girders, columns and slabs. The weights of the brick exterior, windows, and gypsum partitions were included for the total weight of the building. In addition to the dead load, consideration was made for snow loads where a reduction of 50 percent is permitted. To calculate the seismic base shear we followed the steps outlined in the Massachusetts' State Building Code. First, we found the seismic coefficient with the aid of Consigli Construction's structural cover sheet provided in the drawings. The structural cover sheet classified the building as Seismic Hazard Exposure Group II, indicating seismic performance category C. Additionally, it provided information on the soil profile type which was S_2. Next, the fundamental period of the building was approximated based on height and the moment resisting framing system classification. The seismic base shear was then calculated by multiplying the weight and the seismic coefficient. Next, the vertical distribution factor was determined using a ratio of the weight and height of a single story in relation to the total weight and total height of the building. This factor was then multiplied by the seismic base shear and applied to each story as a point load.

3.3 Cost Estimating

In an effort to compare the designs obtained from the two different construction materials, we calculated cost estimates for the interior structural systems. This allowed the project group to consider both flexibility and economic aspects for determining the most desirable layout of the building. The 65th Annual Edition (2007) of the R.S. Means: Building Construction Cost Data was utilized to prepare the cost estimates.

For the cost estimate of the space, it was important to consider the costs of demolition and gutting the building. Prior to its renovation, the interior of the building boasted a very open floor plan, containing only a few columns for support. According to the Project Manager for Consigli, Steve Johnson, the highest cost involved in the process was the asbestos abatement. The total cost for gutting the interior, including abatement, totaled $300,000. However, this cost does not include the allowance for demolition of the interior columns and the floor system, as they are still in place today. This cost was integrated into our cost estimate to calculate the cost per square foot of renovating the space.

To determine the cost for the structural steel alternatives the weight of the beams, girders and columns were calculated for each layout. The aggregate steel cost was calculated using a cost of $3500 per ton. The 5-inch floor slabs of ready-mix concrete with strength of 3500 psi, were valued at $116 per cubic yard.[97] Finally, the costs of

[97] RS Means Square Foot Costs: 26th Annual Edition. 174.

shear studs for the composite system were added at a cost of $3 each. Combined with the aforementioned cost of gutting the building, we were able to determine the cost per square foot of installing our steel frame designs.

To estimate the costs of the reinforced concrete system, we followed the prices that apply to structural concrete, which include the price of labor, materials and equipment. Ready-mix concrete with a strength of 4000 psi costs $119 per cubic yard. For the placement of concrete per use of a pump, the following prices were applied: $28 per yd^3 for slabs less than 6-inch; $24.50 per yd^3 for a slab thickness between 6 and 10 inches; and $43 per yd^3 for large beam and girders. The columns were excluded from the cost estimates because we chose to design columns only for our most desirable layout.

For details of the cost estimate results for both of the construction materials please refer to Chapters 5 and 6.

3.4 Fire Safety Code Analysis

This section was completed utilizing the IBC and following the outline of a report, completed by R.W. Sullivan, a fire protection engineering firm in Boston, Massachusetts. Prior to a January 2007 site visit, a list was developed of measurements that were referenced in IBC Chapter 10-Means of Egress, which needed to be gathered.

List of Measurements:

- Stairwell width/height
- Stair tread/height/depth
- Handrail height
- Handrail diameter
- Handrail distance from wall
- Landing width/height
- Exit travel distance
- Number of exits
- Dead end corridor
- Corridor width
- Doorway width/height
- Doors opening into stairwells

- Direction of door swing
- Proper Illumination and exit signs
- Lecture Hall, aisle width/length

The building height and area limitations were determined based on the plans provided to the group by Consigli Construction. We made a comparison between the requirements of the IBC and the installed features within the facility to determine the building's compliance with the current building code provisions.

3.5 Fire Protection Systems

This section of the report addresses both the active and passive fire protection systems installed in the masonry building. We were able to complete this section by reviewing project plans and fire protection specifications and performing two walkthroughs of the masonry building. The first walkthrough was in September 2006 at the beginning of the interior renovation. Our second walk through was in January 2007, during the later stages of renovation. In addition to our visual reviews, information was gathered from the tours with the Project Manager for Consigli, Steve Johnson. The fire protection specifications can be seen in Appendix I.

During our site visits we completed a visual investigation of the building's active fire protection system. The main features that were addressed were the automatic sprinkler system and standpipes. However, other features of the active fire protection systems such as portable fire extinguishers, manual pull stations, and audible alarms were also inspected. As well as visual inspection, we also collected information verbally from the project manager. We focused on information regarding different components of the active fire protection system, such as the sprinklers and fire control panel. All of the information gathered is listed on the data collection sheet in Appendix J.

Additionally, we explored the passive fire protection features of the building. The passive fire protection of a building is dependent on the building's layout as well as its materials and methods of construction. The main components we looked for in the passive fire protection were rate of fire growth, compartmentation, and emergency egress. These three categories best summarize the strength or weaknesses of a building's passive fire protection systems. Information on the construction materials used and the building's layout and egress routes was gathered from the building plans provided by Consigli.

3.6 Risk Assessment

To complete our risk assessment we identified scenarios that could occur in the masonry building and what their implications. To do this we investigated existing threats and the events that may occur leading to full room involvement (FRI). The sequence of necessary steps would include presence of an ignition source, no automatic suppression, no manual suppression, and sufficient fuel and ventilation for the fire to grow.

It is important to note that in completing a risk assessment there may be several different objectives to accomplish. These objectives are dependent on the different points of view regarding the space under consideration. However, the main objectives in any fire scenario are life safety, mission protection, and property protection.

This assessment provides awareness of the different factors that may come under consideration in a risk assessment, and does not include probabilities or statistics. To identify some of the key risks for consideration we studied the Concepts Trees provided in the NFPA's Fire Protection Handbook. These diagrams provide relationships of fire prevention and damage control strategies.[99] "The fire safety concepts tree provides an overall structure with which to analyze the potential impact of fire safety strategies." [100] The principal branches of the Fire Safety Concepts Tree can be seen in the Figure 4.

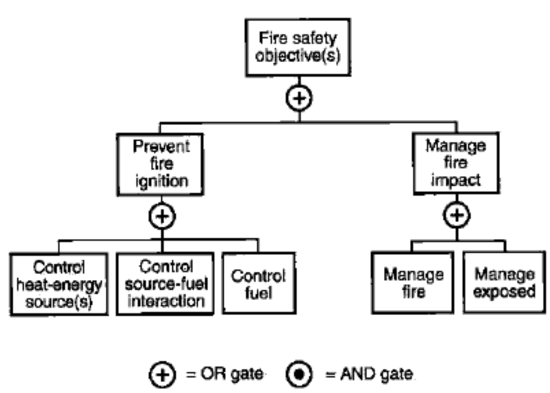

Figure 4: Principal Branches of the Fire Safety Concepts Tree[98]

[98] NFPA. Fire Protection Handbook: Nineteenth Edition. 2-38.
[99] Ibid. 2-38.
[100] Ibid. 2-38.

For our risk scenarios, we examined pieces of the NFPA's decision trees and looked at negative outcomes. Branches of the diagram suggest preventing fire ignition and controlling fuel; thus, we chose a fire scenario to study the available fuels in the masonry building. Also mentioned is managing the fire by means of suppression. To assess the risk of the masonry building, we identified scenarios in which manual and/or automatic suppression would or would not occur. Lastly, we explored the hazards implicit in not renovating or remodeling a stairwell that does conform to building codes. Occupants would likely still use this stairwell as an egress point; however, the building owner would be liable for legal damages including the possible injury and death of building occupants. Thus, this scenario identified risk associated with life safety.

We have provided a narrative to raise awareness of the factors under consideration in risk assessment. Statistical data would be needed to complete a risk analysis and assessment of any scenarios. Such information may be found from an assortment of resources; however, it is important to use data that is both appropriate and from a reputable source.

4. Structural Steel Design

The following section presents the framing plans for the bay sizes considered in the design process. The tables that summarize the results of the steel structural alternative design follow each layout. From the design alternatives, we chose the best two based primarily on the depth of construction.

4.1 Gravity Load Design Summary Tables

The summary tables provided herein were developed based solely on the gravity loads for the building. The term gravity loads includes the dead loads associated with the use of the space, as well as allowances for the weight of the steel frame, MEP, ceiling assembly, and exterior walls. Gravity live loading also includes the basic office live load attributed to the masonry building and snow loads for the roof of the building. For a complete listing of the loads attributed to each floor please refer to section 3.1.1 of this report.

This section addresses the design of a typical bay and the atypical area, including the lecture hall located on the first level of the building. Framing plans for the typical floor areas and the slant beam design are provided.

4.1.1 Bay Size: 34.5' x 27.75'

The subsequent section summarizes the layout and design for the 34.5' x 27.75' bay size. For this scenario, we investigated the use of three to five filler beams running perpendicular to the girders in each bay. The layout is shown in Figure 5.

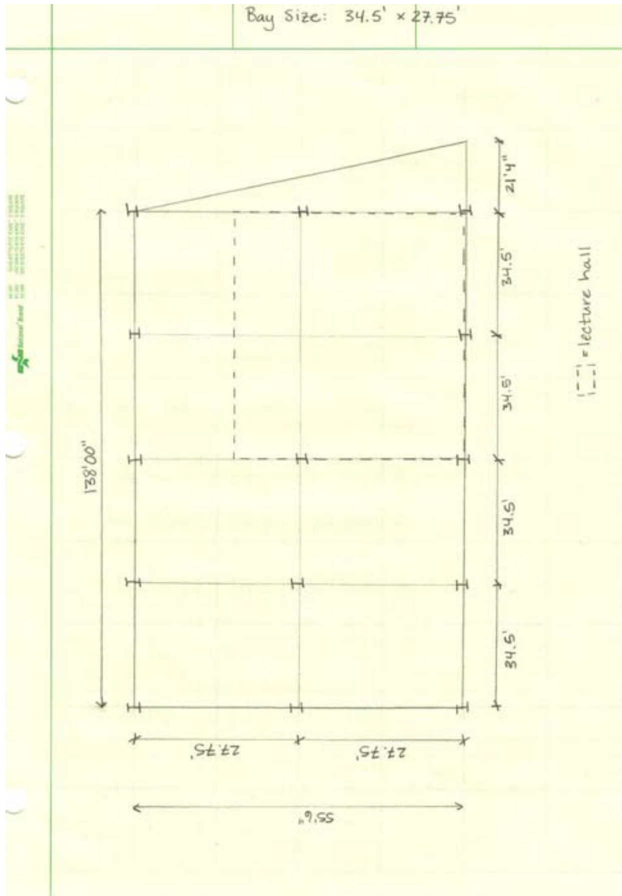

Figure 5: Layout for 34.5' x 27.75' Bay Size

The design of our steel structural alternatives began with designing the typical building areas. Thus, each floor was divided into bays of equal size for design purposes. The atypical areas were designed subsequent to the typical areas.

For the this floor layout special considerations were made for the lecture hall on the first floor. The lecture hall on the first floor boasts a larger bay size to eliminate the column that would be present if the 34.5' x 27.75' bay size was used throughout. Thus, the floor layout for the lecture hall is more practical for its use, as well as aesthetically pleasing. The beam design summary tables can be seen in Table 2.

Table 2: Beam Design for 34.5' x 27.75' Bay Size

Floor	Bay Size	Beam	Number of Filler Beams	Critical Design Moment (ft-kips)	Depth of Construction	# of Studs For Floor
First	34.5' x 27.75'	w14 x 26	3	147	14"	0
		w10 x 26	4	112	10"	0
		w10 x 22	5	90	10"	0
Second & Third	34.5' x 27.75'	w10 x 26	3	103	10"	0
		w10 x 19	4	77	10"	0
		w10 x 17	5	63	10"	0
Fourth	34.5' x 27.75'	w10 x 22	3	84	10"	0
		w10 x 19	4	64	10"	0
		w8 x 18	5	51	8"	0

Following our methodology, from our beam design we were able to design girders that would withstand the building's loading as well as the additional weight of the beams. The girder designs correspond with the beam designs for the varying number of filler beams used. A complete summary of the girder designs for the 34.5' x 27.75' bay size is shown in Table 3.

Table 3: Girder Design for 34.5' x 27.75' Bay Size

Floor	Bay Size	Girder	# of Filler Beams	Critical Design Moment (ft-kips)	Depth of Construction	# of Studs For Floor
First	34.5' x 27.75'	w21 x 44	3	86	21"	0
		w18 x 35	4	64	18"	0
		w18 x 35	5	51	18"	0
First Floor Lecture Hall	34.5' x 27.75'	w40 x 167	3	342	40"	0
		w36 x 135	4	257	36"	0
		w33 x 130	5	205	33"	0
Second, Third, & Fourth	34.5' x 27.75'	w16 x 31	3	43	16"	0
		w14 x 26	4	32	14"	0
		w12 x 26	5	26	12"	0

By using the girder and beam layout shown in the Figure 6, we were able to best utilize the vertical space in the masonry building.

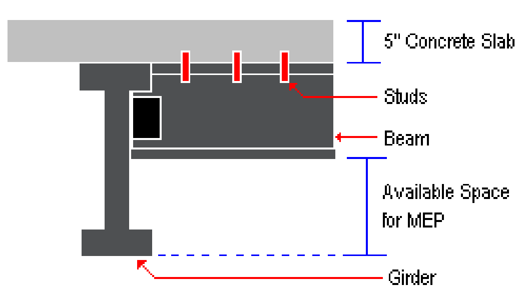

Figure 6: Beam & Girder Layout

The total depth for each beam and girder configuration was determined by adding the girder depth, slab thickness and any additional vertical allowances made for MEP. Additional MEP allowances were determined based on the slab and beams being joined together compositely. Thus, we determined the available space by subtracting the beam depth from the girder depth. For our design, we required a 6-inch MEP space below the beams. If the available space we determined was less than 6-inch than the ceiling would be suspended lower to make up the difference. This adjustment to ceiling height was added to the total depth of construction. The Table 4 summarizes the total depth of construction for each beam and girder design.

Table 4: Depth of Construction for 34.5' x 27.75' Bay Size

Floor	Bay Size	Number of Filler Beams	Beam	Girder	Beam & Girder Depth	Slab, MEP, & Suspended Ceiling Depth	Total Depth of Construction
First	34.5' x 27.75'	3	w14 x 26	w21 x 44	21"	5" slab, no additional MEP	26"
		4	w10 x 26	w18 x 35	18"	5" slab, no additional MEP	23"
		5	w10 x 22	w18 x 35	18"	5" slab, no additional MEP	23"
		Lecture Hall Design	--	w40 x 211	40"	5" slab, no additional MEP	45"
Second & Third	34.5' x 27.75'	3	w10 x 26	w16 x 31	16"	5" slab, no additional MEP	21"
		4	w10 x 19	w14 x 26	14"	5" slab, 2" MEP & ceiling allowance	21"
		5	w10 x 17	w12 x 26	12"	5" slab, 4" MEP & ceiling allowance	21"
Fourth	34.5' x 27.75'	3	w10 x 22	w16 x 31	16"	5" slab, no additional MEP	21"
		4	w10 x 19	w14 x 26	14"	5" slab, 2" MEP & ceiling allowance	21"
		5	w8 x 18	w12 x 26	12"	5" slab, 2" MEP & ceiling allowance	19"

Subsequently, we performed column design for each floor layout. The columns were designed for a two-floor continuation. Thus, the design for the first and second floors was identical. A summary of column design can be seen in Table 5.

Table 5: Column Design for 34.5' x 27.75' Bay Size

Floor	Bay Size	Column	Size	Pu (kips)	Area Req'd (in^2)
First & Second	34.5' x 27.75'	Interior	w14 x 82	421	19.07
		Exterior	w12 x 35	211	9.54
		Corner	w10 x 30	105	4.77
Third & Fourth	34.5' x 27.75'	Interior	w12 x 50	312	14.13
		Exterior	w10 x 30	156	7.07
		Corner	w10 x 15	78	3.53

The structural alternatives design ended with the design of the atypical area on the east side of the building. For this section of the building we employed slant beam design for each floor. This design is summarized in Table 6.

Table 6: Atypical Area (Slant Beam) Design

Floor	Slant Beam	Interior Beams	Length (ft)	# of Studs
First	w24 x 62		59.5	
		w 8 x 10	21.3	
		w 8 x 10	10.7	106
Second & Third	w18 x 55		59.5	
		w 8 x 10	21.3	
		w 8 x 10	10.7	196
Fourth	w18 x 40		59.5	
		w 8 x 10	21.3	
		w 8 x 10	10.7	78

The steel structural alternative that offers the best balance between depth of construction and total weight of steel is the use of 4 filler beams in the 34.5' by 27.75' bay size. For this alternative the filler beams were W10 x 26 for the first floor and W 10 x 19 for floors two and three and the roof. The girders chosen for the first floor were W18 x 35, and W14 x 26 for the second floor through roof level.

The framing plans for this bay size can be seen in the Figure 7 and Figure 8. Both the typical and atypical, or slant beam, designs are shown.

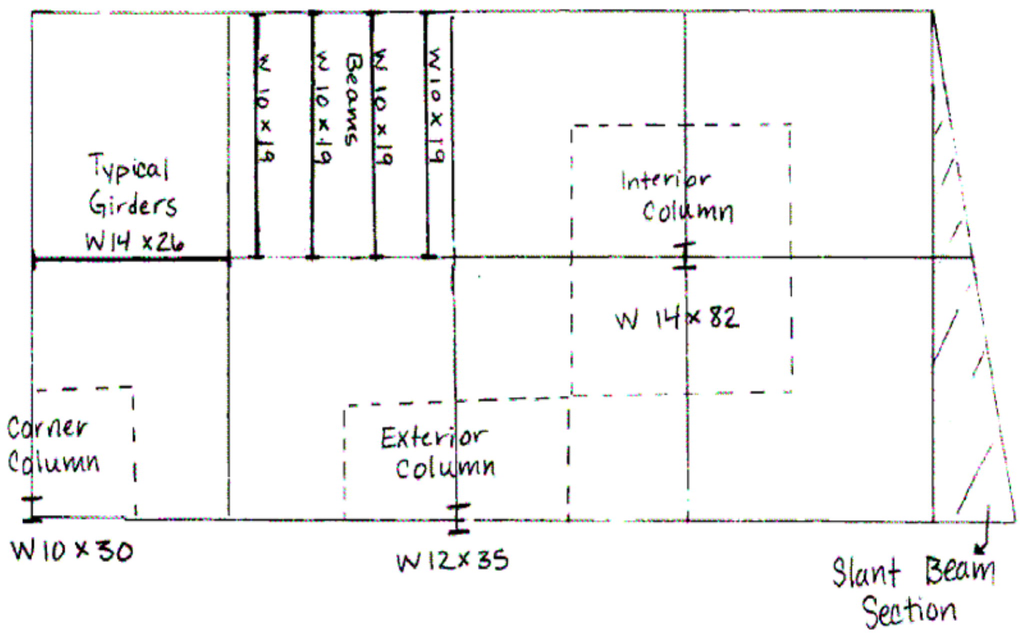

Figure 7: Framing Plan for Typical Areas in 34.5' x 27.75' Bay Size

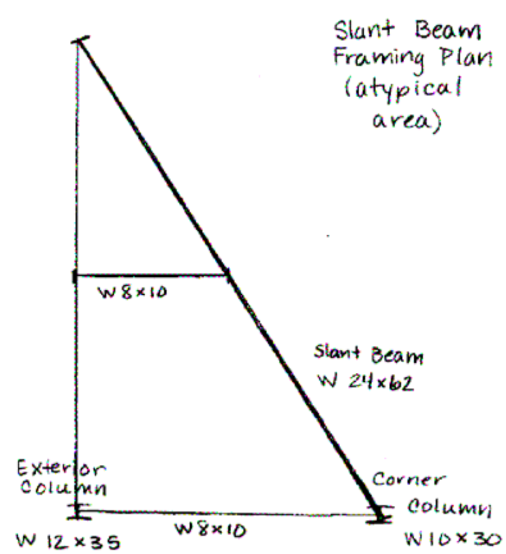

Figure 8: Framing Plan for Atypical Area (Slant Beam) in 34.5' x
27.75' Bay Size

4.1.2 Bay Size 69' x 27.75'

This section discuss the design of the 69' x 27.75' bay size. This floor layout did not require additional design for the lecture hall on the first floor. The larger girder and beam design of the typical layout span the entire length and width of the lecture hall. Table 7 summarizes the beam design for this floor plan.

Table 7: Beam Design for 69' x 27.75' Bay Size

Floor	Bay Size	Beam	Number of Filler Beams	Critical Design Moment (ft-kips)	Depth of Construction	# of Studs For Floor
First	69' x 27.75'	w16 x 45	3	294	16"	696
		w14 x 38	4	220	14"	768
		w12 x 35	5	177	12"	880
Second & Third	69' x 27.75'	w10 x 49	3	205	10"	744
		w12 x 30	4	153	12"	608
		w8 x 35	5	124	8"	880
Fourth	69' x 27.75'	w10 x 39	3	167	10"	600
		w12 x 26	4	125	12"	544
		w10 x 22	5	100	10"	560

Subsequently, girders were designed to run perpendicular to the beams. The girder summary for the layout is shown in Table 8.

Table 8: Girder Design for 69' x 27.75' Bay Size

Floor	Bay Size	Girder	# of Filler Beams	Critical Design Moment (ft-kips)	Depth of Construction	# of Studs For Floor
First	69' x 27.75'	w40 x 278	3	684	40"	1260
		w40 x 211	4	513	40"	936
		w33 x 221	5	411	33"	900
Second, Third, & Fourth	69' x 27.75'	w40 x 167	3	342	40"	804
		w36 x 135	4	257	36"	648
		w33 x 130	5	205	33"	600

The total depth of construction for the 69' x 27.75' bay size was determined based on a 6-inch MEP allowance between the beams and drop ceiling, which is illustrated in Figure 6. The depth summary is shown in Table 9.

Table 9: Depth of Construction for 69' x 27.75' Bay Size

Floor	Bay Size	Beam	Number of Filler Beams	Critical Design Moment (ft-kips)	Depth of Construction	# of Studs For Floor
First	69' x 27.75'	w16 x 45	3	294	16"	696
		w14 x 38	4	220	14"	768
		w12 x 35	5	177	12"	880
Second & Third	69' x 27.75'	w10 x 49	3	205	10"	744
		w12 x 30	4	153	12"	608
		w8 x 35	5	124	8"	880
Fourth	69' x 27.75'	w10 x 39	3	167	10"	600
		w12 x 26	4	125	12"	544
		w10 x 22	5	100	10"	560

This bay size was not chosen as a best option for the space based on the results of the beam and girder design. This layout was too severe too allow for vertical flexibility; thus, column design was not performed. The maximum depth of construction for any design with this bay size was 45", or almost 4 feet. The minimum possible depth of construction would be 38" for this bay size. With this design, the depth of the girders eliminates valuable vertical space, and there would not be sufficient clearance between floors to maintain the desired 15' floor height for the adjoining lab space.

4.1.3 Bay Size: 34.5' x 18.5'

Similar measures were taken to design the third layout of our steel structural alternatives. It began with designing the typical building areas. As displayed in Figure 9, the typical bay size was 34.5' x 18.5' except for the lecture hall. Due to its size and function, the lecture hall was designed as a 69' x 37' layout. For the rest of the building, on floors two through four, the typical bay size was also 34.5' x 18.5'.

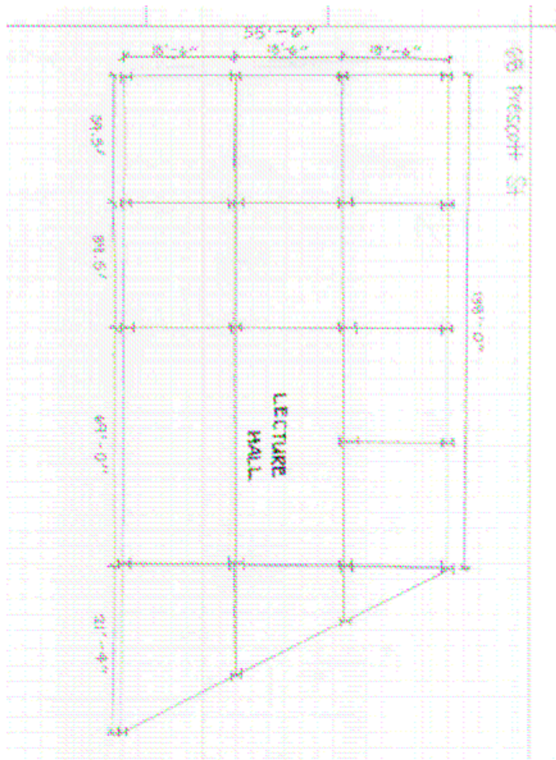

Figure 9: Layout for 34.5' x 18.5' Bay Size

Next, the beam and girder sizes were determined. The beams were designed first followed by the girders so that the girders would not only withstand the building's loading but the additional weight of the beams. Special considerations had to be made in order to accommodate the lecture hall due to its substantially larger size. Summaries of the beam and girder designs are shown in Tables 10 and 11.

Table 10: Beam Design for 34.5' x 18.5' Bay Size

Floor	Bay Size	Beam	Beam Spacing	Critical Design Moment (ft-kips)	Depth of Construction	# of Studs For Floor
First	34.5' x 18.5'	w 16 x 31	17.25'	194	16"	640
		w 12 x 16	11.5'	129	12"	480
		w 12 x 14	8.63'	97	12"	576
Lecture Hall	69' x 18.5'	w 30 x 90	23.'	1042	30"	672
		w 24 x 84	17.25'	785	24"	848
		w 24 x 68	13.8'	628	24"	860
Second & Third	34.5' x 18.5'	w 12 x 26	17.25'	134	12"	816
		w 12 x 19	11.5'	90	12"	864
		w 12 x 16	8.63'	67	12"	960
Fourth	34.5' x 18.5'	w 12 x 22	17.25'	109	12"	672
		w 12 x 26	11.5'	73	12"	1224
		w 10 x 22	8.63'	55	10"	1344

Table 11: Girder Design for 34.5' x 18.5' Bay Size

Floor	Bay Size	Girder	# of Filler Beams	Critical Design Moment (ft-kips)	Depth of Construction	# of Studs For Floor
First	34.5' x 18.5'	w 24 x 76	2	257	24"	1316
		w 24 x 55	3	171	24"	980
		w 21 x 48	4	128	21"	836
Lecture Hall	69' x 37'	w 27 x 84	3	342	27"	106
		w 24 x 76	4	257	24"	94
		w 24 x 62	5	205	24"	78
Second, Third & Fourth	34.5' x 18.5'	w 21 x 48	2	128	21"	960
		w 21 x 44	3	86	21"	896
		w 18 x 35	4	64	18"	704

Using our beam and girder layout, shown in Figure 6, we were able to determine the vertical space in the building. Table 12 shows the depth of construction unique to the 34.5’ x 18.5’ layout.

Table 12: Depth of Construction for 34.5' x 18.5' Bay Size

Floor	Bay Size	Number of Filler Beams	Beam	Girder	Beam & Girder Depth	Slab, MEP, & Suspended Ceiling Depth	Total Depth of Construction
First	34.5' x 18.5'	2	w16 x 31	w 24 x 76	24"	5" slab, no additional MEP	29"
		3	w12 x 16	w 24 x 55	24"	5" slab, no additional MEP	29"
		4	w12 x 14	w 21 x 48	21"	5" slab, no additional MEP	26"
	69' x 37' (Lecture Hall Design)	3	w30 x 90	w 27 x 84	30"	5" slab, 6" MEP & ceiling allowance	41"
		4	w24 x 84	w 24 x 76	24"	5" slab, 6" MEP & ceiling allowance	35"
		5	w24 x 68	w 24 x 62	24"	5" slab, 6" MEP & ceiling allowance	35"
Second & Third	34.5' x 18.5'	2	w12 x 26	w 21 x 48	21"	5" slab, no additional MEP	26"
		3	w12 x 19	w 21 x 44	21"	5" slab, no additional MEP	26"
		4	w12 x 16	w 18 x 35	18"	5" slab, no additional MEP	24"
Fourth	34.5' x 18.5'	2	w12 x 22	w 21 x 48	21"	5" slab, no additional MEP	26"
		3	w12 x 26	w 21 x 44	21"	5" slab, no additional MEP	26"
		4	w10 x 22	w 18 x 35	18"	5" slab, no additional MEP	23"

For the typical area, the final step was to design the columns. The columns were two-story continuous columns. Table 13 displays the chosen sizes for the interior, exterior, and corner columns.

Table 13: Column Design for 34.5' x 18.5' Bay Size

Floor	Bay Size	Column	Size	Pu (kips)	Area Req'd (in^2)
First & Second	34.5' x 18.5'	Interior	w 21 x 44	281	12.7
		Exterior	w 16 x 26	70	3.2
		Corner	w 16 x 26	140	6.4
Third & Fourth	34.5' x 18.5'	Interior	w 16 x 36	208	9.4
		Exterior	w 16 x 26	52	6.3
		Corner	w 16 x 26	104	7.3

The last step in the overall gravity load design was to design the atypical area of the masonry building. For this section of the building we employed slant beam design for each floor. This design is summarized in Table 14.

Table 14: Atypical Area (Slant Beam) Design

Floor	Slant Beam	Interior Beams	Length (ft)	Studs
First	w 24 x 68		59.5	
		w 8 x 10	21.3	
		w 8 x 10	14.2	
		w 8 x 10	7.1	128
Second & Third	w 21 x 50		59.5	
		w 8 x 10	21.3	
		w 8 x 10	14.2	
		w 8 x 10	7.1	212
Fourth	w 16 x 45		59.5	
		w 8 x 10	21.3	
		w 8 x 10	14.2	
		w 8 x 10	7.1	100

In conclusion, the steel structural alternative that offers the best balance between depth of construction and total weight of steel is the use of 3 filler beams in the 34.5' by 18.5' bay size. For this alternative the filler beams are W 12 x 16 for the first floor, W 12 x 19 for floors two and three and W 12 x 26 for the roof. A W 24 x 55 girder was chosen for the first floor and a W 21 x 44 was chosen for the second, third and fourth floors. The lecture hall area was designed as a 69' x 37' bay size. The beam chosen was W 24 x 84 and the girder was W 27 x 84.

The framing plan for this layout can be seen in Figure 10.

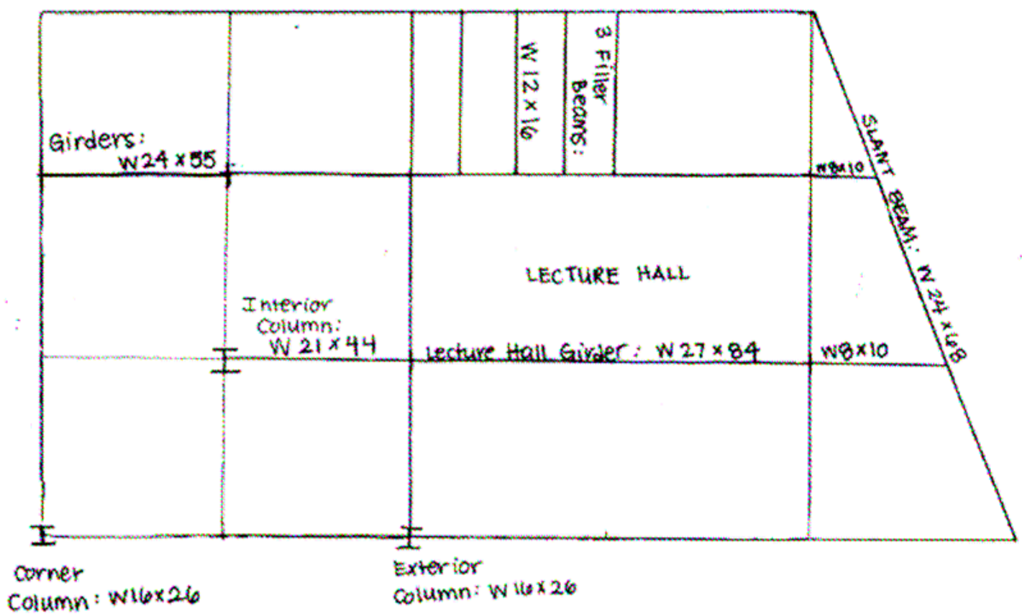

Figure 10: Framing Plan for Typical Areas in 34.5' x 18.5' Bay Size

4.2 Lateral Load Design Summary Tables

After determining the wind load based on the fastest wind speed, we applied the uniform load to both the short and long side. We utilized RISA 2-D Educational Version to determine our deflections. Due to the constraints of the computer model it was necessary to cut the long side in half and apply only half of the load to that system. This displacement occurred on the long side at the top of the facility. The wind forces applied to the building are shown in Table 15.

Table 15: Steel Wind Loads

Floor	Area (ft^2)	Conversion	Force (k-ft)
4	416.25	0.02	8.325
3	793.65	0.02	15.873
2	760.91	0.02	15.22
1	756.2	0.02	15.2

It is only necessary to apply one-half to three-fourths of the total uniform wind load. For the structural steel scenario, we applied the total load and our results given in

Appendix H show that the maximum deflection of our system is 3.14 inches as affected by wind loadings.

Similarly, seismic loads were applied to each floor shown in Table 16. The maximum deflection of the system was 0.264 inches. This displacement occurred on the long side at the top of the facility.

Table 16: Steel Seismic Loads

Floor	Total Weight (psf)	Fx (psf)
4	148.9	335
3	147.4	681
2	147.5	650.5
1	151.1	660

Investigating lateral loads is a very important step in steel design. High wind pressures applied to the sides of tall buildings can produce overturning moments. The axial strength of the columns are typically able to resist these moments, but the horizontal shears produced on each level may be sufficient in magnitude to cause the building to require special bracing or moment-resisting connections.[101]

[101] McCormac, Jack. Structural Steel Design. 621.

4.3 Cost Estimate

The cost estimate of the steel structural alternatives is dependent on the beam and girder choices. For the slant beam design, the building's atypical area, and column design cost estimates will be added to the cost estimate of the best two alternatives chosen from the steel design for a total cost estimate.

Subsequent to determining the total weight of steel required for the floor layouts, we calculated the total cost of steel based on $3500 per ton of steel.[102] For the slab allowance we used a cost of $116 per cubic yard of concrete, from the RS Means Handbook. The volume of concrete needed for the space was calculated based on the slab thickness and the total square footage of the building. This cost is calculated independently of the floor layout because the same slab thickness was used throughout each structural steel design. A cost of $3 per shear stud was used for the cost estimates. This value was determined based on previous undergraduate work in steel design.

[102] "Fabricated Steel per Ton." http://www.eng-tips.com/viewthread.cfm?qid=148002&page=1

4.3.1 Cost Estimate for 34.5' x 27.75' Bay Size with 4 filler beams

This section presents an aggregate cost summary for the 34.5' x 27.75' bay size, with a 69' x 36' layout for the lecture hall on the first floor. The design of the lecture hall added additional cost to the project that may not have been accrued if the space did not require the removal of a column. This column removal required a longer girder span; hence, both the depth and cost of the girder increased. Table 17 summarizes the total cost of constructing the new steel frame as well as the cost per square foot.

Table 17: Cost Analysis of Structural Alternative

Floor	Design Element	Total Cost	ft^2	$/$ft^2$
First	Beams & Girders (Including Lecture Hall)	$103,400.00	7,500	$13.79
	Columns	$35,800.00	7,500	$4.78
	Atypical[a]	$7,100.00	7,500	$0.95
	¾" Studs	$3,600.00	7,500	$0.48
Second	Beams & Girders	$48,400.00	7,500	$6.46
	Columns	$0 [b]	7,500	$0.00
	Atypical[a]	$6,300.00	7,500	$0.84
	¾" Studs	$2,900.00	7,500	$0.39
Third	Beams & Girders	$48,400.00	7,500	$6.46
	Columns	$27,900.00	7,500	$3.72
	Atypical[a]	$6,300.00	7,500	$0.84
	¾" Studs	$2,400.00	7,500	$0.32
Roof	Beams & Girders	$48,400.00	7,500	$6.46
	Columns	$0 [b]	7,500	$0.00
	Atypical[a]	$4,800.00	7,500	$0.64
	¾" Studs	$2,600.00	7,500	$0.35
All	Slab Allowance (5")	$54,000 [c]	30,000	$1.80
All	Gutting building (including asbestos abatement)	$300,000	30,000	$10.00
	Total Cost :	$702,300.00	Cost / ft^2 :	$60.00

[a] : The atypical area refers to the slant beam design portion of the building

[b] : The cost for columns for the second and roof has been included in the estimate for the first and third floor, respectively.

[c] : Slab Allowance: 5" slab covering a total area of 30,000 ft^2. The cost of concrete is $116/$yd^3$. Thus, (5"/12") x 30,000ft^2 x ($116/$yd^3$)[103]/(27$ft^3$/$yd^3$) = $54,000.

To gauge the economic feasibility of this steel frame design we consulted the RS Means cost estimating manuals. According to their text the reported costs for completed commercial 2 to 4 story projects, "…range from $51.35 to $198.95 per [square foot]."[104]

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[103] RS Means Building Construction Costs Data: 65th Annual Edition. 64.

[104] RS Means Square Foot Costs: 26th Annual Edition. 174.

As can be seen from the summarized cost of the steel frame, this design option is well within the average economic limits, at a cost of $60 per square foot.

We can discern a considerable difference in the cost between different levels of the building from Table 17. This variance can be explained by increased loading at the lower floors. On the first floor, the live load value was 100 psf for the lecture hall with moveable seating; also, there was additional cost associated with longer girder spans for this area. Thus, the cost of the beams and girders for this area was greater than for the second or third floors, where the required office live load is only 50 psf and girder span length was uniform.

Similar results can be seen in the column costs. The column cost for the first and second floors is nearly 30 percent greater than that of the upper floors. For the two-floor continuation in the column design we attributed the cost to the lower floor. Thus, in the second and fourth floors, the cost of the columns appears as zero in the cost estimate; however, this cost was not neglected.

4.3.2 Cost Estimate for 34.5' x 18.5' Bay Size with 3 filler beams

Figure 18 displays the cost analysis of the third design layout. In the first floor section, the lecture hall is noted. Due to its' size and function, the lecture hall had a significant impact on the overall cost of the design. In particular, this lecture hall boasts a girder, large enough in size that it can support the building with the removal of a column. In additional to the cost per floor, Figure 18 shows the cost per square foot.

Table 18: Cost Analysis of Structural Alternative

Floor	Design Element	Total Cost	ft^2	$\$/ft^2$
First	Beams & Girders (Including Lecture Hall)	$93,100.00	7,500	$12.42
	Columns	$34,000.00	7,500	$4.54
	Atypical[a]	$7,900.00	7,500	$1.06
	¾" Studs	$3,800.00	7,500	$0.51
Second	Beams & Girders	$75,600.00	7,500	$10.08
	Columns	$0 [b]	7,500	$0.00
	Atypical[a]	$6,000.00	7,500	$0.80
	¾" Studs	$2,900.00	7,500	$0.39
Third	Beams & Girders	$75,600.00	7,500	$10.08
	Columns	$32,900.00	7,500	$4.39
	Atypical[a]	$6,000.00	7,500	$0.80
	¾" Studs	$2,500.00	7,500	$0.34
Roof	Beams & Girders	$44,100.00	7,500	$5.88
	Columns	$0 [b]	7,500	$0.00
	Atypical[a]	$5,500.00	7,500	$0.74
	¾" Studs	$3,300.00	7,500	$0.44
All	Slab Allowance (5")	$54,000 [c]	30,000	$1.80
All	Gutting building (including asbestos abatement)	$300,000	30,000	$10.00
	Total Cost :	$747,200.00	Cost / ft^2 :	$65.00

[a] : The atypical area refers to the slant beam design portion of the building

[b] : The cost for columns for the second and roof has been included in the estimate for the first and third floor, respectively.

[c] : Slab Allowance: 5" slab covering a total area of 30,000 ft^2. The cost of concrete is $\$116/yd^3$. Thus, $(5"/12") \times 30{,}000 ft^2 \times (\$116/yd^3)/(27 ft^3/yd^3) = \$54{,}000$.

As Table 18 displays, $65 was the total cost per square foot of the building. According to the RS Means cost estimating manuals, $65 is well within the economic limits of building a two to four story building.

It is to be noted the significant cost difference between the four floors. The first floor is unique due to the presence of the lecture hall. Lecture hall occupancies calls for higher load considerations thus stronger support. The size of the lecture hall is also unique, its bay size is both double in length and width of the buildings' typical bay sizes. For functionality purposes, columns were strategically placed and an enlarged girder was chosen so that the students' utilizing the lecture hall could view the lecture material without any obstructions.

The second and third floor layouts are identical. They are designed to uphold simply the loading of a typical office space. It is also to be noted that the price of the columns are only included in the second floor and the roof sections of Table 18. The columns used for this building are two story continuous columns, thus there are not four separate column sizes for each floor. Overall, the design and cost of this particular layout was found to be typical according to the RS Means cost estimate manual.

5. Reinforced Concrete Design

The following chapter shows the results obtained from the structural design process of the reinforced concrete system. Various alternatives were investigated to determine the most flexible and economical layout. We chose Scenario 1, one-way slab with thickness of 5 inches, to be the most desirable option. We then applied lateral loads to this system to check for adequacy.

5.1 Gravity Load Summary Tables

According to Table 19, Scenario 1 was the most economical choice and still provided a flexible layout with bay sizes of 21'x18' and 21'x 19'. This scenario includes a 5 inch slab with 1#4 bar top steel and 1#4 bar bottom steel reinforcement. In addition, the beam size is 12-inches by 24-inches with 4#7 bars, and the girder dimension is 15-inches by 25-inches. It was the best option in terms of depth of construction, with the thinnest slab and smallest girder dimension as shown in Table 19. This was important to consider due to the addition of the link and laboratory facilities. The latter requires high ceilings, usually a minimum of 15 feet. The cost was also inexpensive in comparison to the other scenario at just under $33,000 per floor.

Table 19: Concrete Summary Table

	Bay Size			Slab Thickness	Moment	Reinforcement		Beam	Rebar	Girder	Concrete needed	Cost
	feet	feet	feet	inches	ft-kips/ft	Top steel	Bottom steel	inches		inches	yd3	
One way Slab												
Interior	11x18	11x19	11x18	5	2.68	1 #4	1 #4	12x24	4 #7	15x25	215	$32,600
Exterior	10x18	10x19	10x18	5	2.68	1 #4	1 #4	12x24	4 #7	15x25		
Interior	15x27.5	15x27.5		7.5	5.84	1 #5	1 #5	18x36	3 #5	18x30	370	$56,000
Exterior	15x27.5	15x27.5		7.5	5.84	1 #5	1 #5	18x36	3 #5	18x30		
Interior	15x20	15x15	15x20	6.43	5.96	1 #5	1 #5	15x30	4 #6	18x26	290	$43,800
Exterior	10x20	10x15	10x20	6.43	5.96	1 #5	1 #5	15x30	4 #6	18x26		
Continuous T-Beam					ft-kips							
Interior	10x18	11x19	11x18	5	84.3	5#5	N/A*	14x14	3 #5	18x30	218	$33,300
Exterior	11x18	10x19	10x18	5	84.3	4#6, 4#5	N/A*	14x14	3 #5	18x30		
Two Way Slab												
Interior	11x18	11x19	11x18	7.5	85.5	18#4	N/A*	N/A	N/A		193	$27,000
Exterior	10x18	10x19	10x18	7.5	85.5	15#4	N/A*	N/A	N/A			

* Steel reinforcement included in top steel reinforcement

5.2 Lateral Load Design Summary Tables

After determining the wind load based on the fastest wind speed, we applied the uniform load to both the short and long side. We utilized RISA 2-D Educational Version to determine our deflections. Due to the constraints of the computer model it was necessary to cut the long side in half and apply only half of the load to that system. It is only necessary to apply one-half to three-fourths of the total uniform wind load. The wind loads can be seen in Table 20.

Table 20: Concrete Wind Load

Floor	Area (ft^2)	Conversion	Force (k-ft)
4	416.25	0.02	8.325
3	793.65	0.02	15.8
2	760.91	0.02	15.2
1	756.2	0.02	15.1

For the reinforced concrete scenario, we applied the total load and our results given in Appendix H show that the maximum deflection of our system is 0.047 inches as affected by wind loadings. This displacement occurred on the long side at the top of the facility.

In the same respect as the wind load, we applied the total determined seismic load, shown in Table 21, to the system that resulted in a maximum deflection of our system of .014 inches as affected by seismic loadings. This displacement occurred on the long side at the top of the facility.

Table 21: Concrete Seismic Loads

Floor	Total Weight (psf)	Fx (psf)
4	3937	2289
3	3937	4578
2	3937	4376
1	3937	4309

As can be seen, the facility is sufficient for the lateral loadings applied. After completing lateral load design, we determined the cost of all reinforced concrete scenarios.

5.3 Cost Estimate

Table 22 shows a breakdown of the calculated costs of each concrete system. All costs are based on 4000 psi ready-mix concrete and include the labor, material, and equipment fees. Although, Scenario 5 is the most economical, we determined that Scenario 1 has a comparable cost and deemed it the most desirable choice. This choice was made weighing all aspects of the design; feasibility, economics, and flexibility of the layout within the structure.

Table 22: Cost Estimates of Concrete Systems

	Slab	Beams	Girders	Total	$/ft^2
Scenario 1	$19,100	$4,900	$8,600	**$32,600**	**$39.95**
Scenario 2	$27,700	$10,500	$17,800	**$56,000**	**$42.76**
Scenario 3	$23,700	$7,300	$12,800	**$43,800**	**$41.30**
Scenario 4	$19,000	$2,000	$12,300	**$33,300**	**$40.04**
Scenario 5	$27,000	$0	$0	**$27,000**	**$39.28**

The cost per cubic yard was determined to include the cost of demolition and renovation with each structural alternative. As previously mentioned, the cost of demolition was determined to be $300,000. We applied this price to the total cost per square foot.

Concrete framing systems differ from steel systems with respect to the ascending floors. Steel systems call for lighter members in the upper levels of the building. On the contrary, each floor design is identical within a concrete system. The beam, slab, and column sizes of concrete systems are chosen to allow reuse of the forms from floor to floor to minimize construction costs. This stands as an economical advantage of using concrete systems.

After completion of all structural alternative scenarios and their corresponding cost estimates, we performed a code analysis of the facility as it stands to determine if the building was in compliance with the current code.

6. Fire Safety Code Analysis

The following chapter outlines the egress systems of the recently restored masonry building of Gateway Park. A code analysis was performed on the building utilizing the latest version (2006) of the International Building Code. Our analysis focuses on the means of egress throughout this facility.

6.1 Facility Description, Classification, Processes

The building being reviewed is part of the new Gateway Park Expansion under ownership of the WBDC and WPI. The master plan of the Gateway Project is a mixed use project including space for retail, research, housing, and parking. The building under study is a masonry building built in the late 1800s. The Gateway Park Proposed Master Plan layout can be seen in Appendix E.

The original building has been renovated as a Mixed Occupancy. The building will be occupied by offices, graduate research areas, and a lecture hall.

6.2 Occupancy Classification

The renovated masonry building was classified as follows:

Table 23: Occupancy Classification

Building Use	Floor	IBC Code Classification
Lecture Hall	1st	Assembly (A-3)
Offices	2nd, 3rd, 4th	Business (B)
Mechanical	Basement	Incidental Use
Reception Area	1st	Business (B)

The building is a Mixed Occupancy, which is defined as a building consisting of two or more individually classified occupancies, per IBC Section 508. According to the Code, the building can be classified as either Non-separated Use or Separated Use. The building could be considered a Non-separated use because there are no requirements to distinguish between the two identified occupancies, in which we could view the building as strictly a Business Occupancy since it makes up more that 90 percent of total building area. On the other hand, the building could also be classified as a Separated Use. This is a result of treating the occupancies individually, in which each occupancy group meets the defined requirements for that corresponding section.

From IBC Table 508.3.3, the required fire separation between the A-3 occupancy group and the B occupancy class is 1 hour for buildings equipped with an automatic sprinkler system installed per 903.3.1.1.

From IBC Table 508.3.3, there is no required fire separation between the two Business uses.

For Separated Use the above required separation is applied and each fire area must comply with the height limitations based on the use of that space and the type of construction classification.

The occupancy diagram for the masonry building can be seen in Figure 11.

4	**Business**	
3	**Business**	
2	**Business**	
1	**Assembly – 3**	**Business**
	Basement (Incidental Use Area)	

Figure 11: Occupancy Diagram

6.3 General Building Heights and Areas

From IBC Table 503 the allowable height and building area, per floor for the two major occupancies area as follows:

A-3	TYPE I A	Number Stories: Unlimited	AREA: Unlimited
	TYPE I B	Number Stories: 11	AREA: Unlimited
	TYPE II A	Number Stories: 3	AREA: 15,500 ft^2
	TYPE II B	Number Stories: 2	AREA: 9,500 ft^2
	TYPE III A	Number Stories: 3	AREA: 14,000 ft^2
	TYPE III B	Number Stories: 2	AREA: 9,500 ft^2
	TYPE IV	Number Stories: 3	AREA: 15,000 ft^2
	TYPE V	Number Stories: 2	AREA: 11,500 ft^2
	TYPE V	Number Stories: 1	AREA: 6,000 ft^2

B	TYPE I A	Number Stories: Unlimited	AREA: Unlimited
	TYPE I B	Number Stories: 11	AREA: Unlimited
	TYPE II A	Number Stories: 5	AREA: 37,500 ft^2
	TYPE II B	Number Stories: 4	AREA: 23,000 ft^2
	TYPE III A	Number Stories: 5	AREA: 28,500 ft^2
	TYPE III B	Number Stories: 4	AREA: 19,000 ft^2
	TYPE IV	Number Stories: 5	AREA: 36,500 ft^2
	TYPE V	Number Stories: 3	AREA: 18,000 ft^2
	TYPE V	Number Stories: 2	AREA: 9,000 ft^2

As permitted by IBC Section 504.2, the area and height of a building can be increased when an automatic sprinkler system is installed. For buildings protected throughout with an approved sprinkler system the value for maximum height, specified in Table 503, may be increased by 20 feet and the maximum number of stories increased by one.

The plans indicate an assumption of 50 percent of the area to be open frontage. We applied this percentage only to the masonry building for the purpose of our project. The perimeter of this portion of the building is 413.33 ft. Applying this value to Equation 5-2 in Section 506.2 of the IBC, the allowable area increase due to frontage has been calculated to be 25 percent.

In addition, per Section 506.3 the automatic sprinkler system increase allows a 200 percent increase in area for multi-story buildings fully equipped with an approved automatic sprinkler system.

Table 24 reflects the actual floor area by occupancy:

Table 24: Floor Area by Occupancy

Floor	Occupancy	Actual Floor Area (ft^2)	Permissible Floor Area (ft^2)	Actual Stories	Permissible Stories
Basement	Incidental	8,325	-	-	-
1	A-3	1,444	30,875	1	3
1	B	6,881	61,750	1	4
2	B	8,325	61,750	4	4
3	B	8,325	61,750	4	4
4	B	8,325	61,750	4	4

The actual total height of the building is 66’-1” from the basement slab to the high point of roof.

6.4 Minimum Construction Type

Based on IBC Table 503 the minimum construction is Type IIIB. However, in order to qualify as a Type III building, the interior timber framing must be fire-retardant treated. In contrast, Type IV construction would permit the use of unprotected timber construction provided certain geometric restrictions are met and there are no concealed spaces. Since dropped ceilings have been installed for HVAC purposes within the building, the construction Type IV can not be applied. In the existing structure upright sprinklers have been installed in the drop ceilings. In addition, concrete floors have been placed over the existing wooden floors. As a result, we have decided to classify a construction type of IIIB for the code analysis.

6.5 Fire Resistance Rating

Table 25 reflects the required fire resistance rating for each building element found within a structure with Type IIIB construction. Although there is no required fire rating for the floor construction, the concrete slabs provide a 1-hour fire rating. Also, a spray fire proofing application was noted within the egress stairwells during a January 2007 site visit.

Table 25: Required Fire Resistance Ratings

Building Element	**Required Fire Resistance Rating (Hours)**
	Type IIIB Construction
Bearing Exterior Walls	2 (Table 601)
Nonbearing Exterior Walls	(Per Table 602 of IBC based on fire separation distance.) No rating required where distance is • 30’
Interior columns, girders, trusses	0 (Table 601)
Floor Construction	0 (Table 601)
Roof Construction	0 (Table 601)
Stair, Elevator Shaft	2 (Section 707.4)
Exit Access Corridors	1 (Table 1017.1)
Storage Rooms > 100 ft^2	1 (Table 508.2)

6.6 Exterior Wall Construction

Exterior walls of the building must be permitted by the type of building construction, per IBC Section 704.4. Construction Type IIIB requires the exterior shell to be composed of non-combustible materials. The building at hand was constructed with a masonry shell and therefore, meets this requirement.

Exterior wall ratings and openings must comply with IBC Table 602 and Table 704.8. If the fire separation distance is greater than 5 feet, the fire-resistance of the exterior wall must be rated for fire exposure from the inside. The fire separation distance is the distance measured from the building face to one of the following: the closest interior lot line; to the centerline of a street, an alley or public way; or to an imaginary line between two buildings on the property. In the first case, where the fire separation distance is measured from the face of the building to the closest interior lot line, the fire separation distance is zero as the building lies on the west lot line. However, in the other two cases, the fire separation distance exceeds 5 feet. Because the building is protected by an automatic sprinkler system, the exterior wall is rated for fire exposure from the inside and this requirement is met. Additionally, an opening, whether protected or unprotected, is not permitted on the exterior wall where the fire separation distance measures zero feet.

6.7 Floor Construction

The atrium in the link of the Gateway facility attached to the existing structure. Since it is not located directly within the existing structure the atrium is not being evaluated in this report. Therefore, there are no openings within the floor system of the existing structure. Thus, the floor slabs must meet item 4 in Table 25. As discussed previously there is no required fire rating but since concrete floor slabs have been placed over the wood, a 1-hour fire rating has been provided.

6.8 Interior Finish

Interior wall and ceiling finish must comply with IBC Table 803.5. For A-3 since the building is sprinklered throughout, Class B finish is required for corridors, exit enclosures and exit passageways. Class C is required in rooms and enclosed spaces. For B occupancy, Class C is required in corridors, rooms and all enclosed spaces and Class B is required in exit enclosures and passageways.

Interior Floor Finish must comply with Section 804. For all use groups excluding I-2 and I-3, interior finishes shall not be less than Class II.

6.9 Means of Egress

In order to analyze the structure's means of egress, it was necessary to include a portion of the newly constructed link. The existing structure would not be in compliance with the IBC if the link was not included. In the analysis provided to us by Consigli, the stairwell in the existing structure was not included as a means of egress. However, the two enclosed stairwells within the link are sufficient for egress travel for occupants of the upper floor levels.

The occupant load was determined using IBC Table 1004.1.1. The following table shows the calculated value of occupant load per floor area. Because the assembly occupancy contains fixed seating, the occupant load is based on the number of seats within the lecture hall. A business area allows 100 gross of floor area per occupant. Therefore, we calculated the occupant load by dividing the floor area by 100.

Table 26: Occupant Loads

Floor	Area	Floor Area (ft^2)	Floor Area per Occupant	Occupant Load
1 A	Assembly	1,444	-	97
1 B	Business	6,881	100 gross	69
2	Business	8,325	100 gross	83
3	Business	8,325	100 gross	83
4	Business	8,325	100 gross	83

The number of exits was determined in accordance with IBC Table 1019.1. This value is based on the total occupant load of each floor using the results from Table 26: Occupant Loads. The number of exits provided is sufficient throughout the building.

Table 27: Number of Exits Per Floor

Floor	Total Occupant Load	Required Number of Exits	Number of Exits Provided
1	166	2	4
2	83	2	2
3	83	2	2
4	83	2	2

The exit capacity was calculated using IBC Table 1005.1. A value of 0.15 inches is allowed per person for any exterior or stairwell door. A value of 0.2 inches is allowed per person within a stairwell. The total exit width provided was measured during a site visit at the facility. These values were multiplied by the exit allowance factors to determine the total exit capacity provided in persons. All aspects of the exit capacity are sufficient for providing egress capability for the determined total occupant load, as shown in Table 28.

Table 28: Exit Capacity

Floor	Total Occupant Load	Exit Allowance (in/persons)	Total Exit Width Provided (in)	Total Exit capacity Provided (persons)	Status
1	166	0.15 (door)	36 x 4	960**	In Compliance
2	83	0.2 (stairs) 0.15 (stair door)	43 35	**215** 233	In Compliance
3	83	0.2 (stairs) 0.15 (stair door)	49 35	**245** 233	In Compliance
4	83	0.2 (stairs) 0.15 (stair door)	43 35	**215** 233	In Compliance

**Assume one door is not accessible, with multiple means of egress, the loss of one means shall not reduce the available capacity to less than 50% of the required capacity per 1005.1.
Door width 36" x 4 doors /0.15/door = 240 people x 4 doors = 960 total exit capacity

6.9.1 Exit Access Travel Distance

The exit access travel distance is defined as the maximum distance an occupant would have to travel to reach an exit refuge. Per IBC Section 1016.1 and Table 1016.1 in the masonry building the following requirements are necessary. The two egress routes within the masonry building must lead to enclosed stairwells. Thus, the exit access travel distance can be measured from any point within the facility to the closest stairwell. For an Assembly group installed with a sprinkler system, the maximum travel distance is 250 feet. For the rest of the building, classified as Business Occupancy, the exit access travel distance shall be 300 feet from any point to the enclosed stairwells as a result of being equipped with a sprinkler system.

The measured maximum distance the occupants would have to travel was determined to be 96 feet, which is in compliance with the Code. The following layout illustrates this maximum travel distance. Figure 12 depicts the layout of floors 2, 3, and 4 within the building. The masonry building is outlined in blue. The green line demonstrates the maximum travel distance, of 96 feet, that an occupant would travel in following the proper egress route. Note that this is the most probable exit path for occupants of the offices located in this end of the corridor.

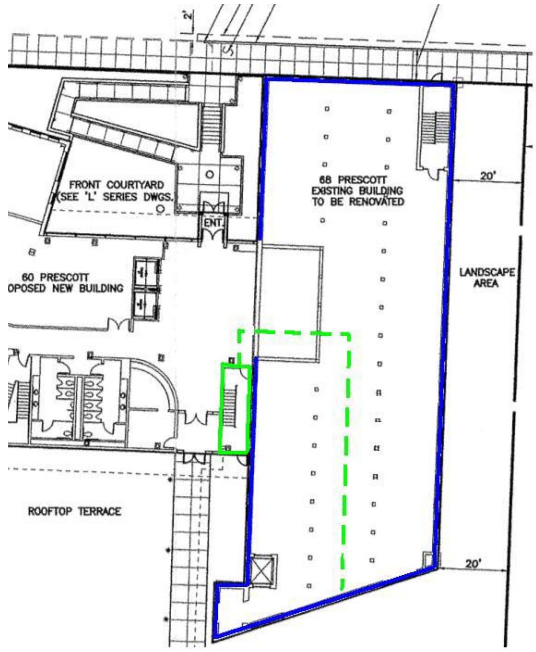

Figure 12: Greatest Travel Distance

6.9.2 Remoteness of Exits

Typically, two exits should be located at 1/2 the length of the maximum overall diagonal dimension of the building from each other. Per IBC 1015.2.1, Exception 2 allows for the distance between exits to be increased because the building is sprinklered throughout. This exception provides an increase of distance from 1/2 to 1/3 the length of the maximum overall diagonal dimension between the two exits.

6.9.3 Corridors

The minimum corridor width shall be determined in Section 1005.1 but not less than 44 inches (IBC Section 1017.2). The corridor width throughout the building was measured at 60 inches.

6.9.4 Exit Signs and Means of Egress Illumination

Exits signs are required for access to exits where the exit or the path of egress travel is not immediately visible to the occupants. Signs shall be located such that no point in an exit access corridor is more than 100 feet from the nearest visible exit sign.

Stairway exit signs are required to be tactile and adjacent to each door to an egress stairway in accordance with IBC section 1011.3.

There are specific exceptions such as exit signs are not required in rooms or areas that only require one exit or exit access. Main exterior exits do not require exit signs where approved by the building official.

The graphics required on the exit sign are delineated in the IBC section 1011.5.1 entitled Graphics.

Exits shall be illuminated either internally or externally with prescribed illumination intensity. Externally illuminated signs shall have an intensity of 5 foot-candles. All of the exit signs noted in the building were internally illuminated. There is an exception that allows self luminous exit signs that provide evenly illuminated letters with a prescribed luminance of 0.06 foot-lamberts. Tritium signs are one such self luminous sign that may be used.[11] Figures 13 and 14 are examples of typical signs.

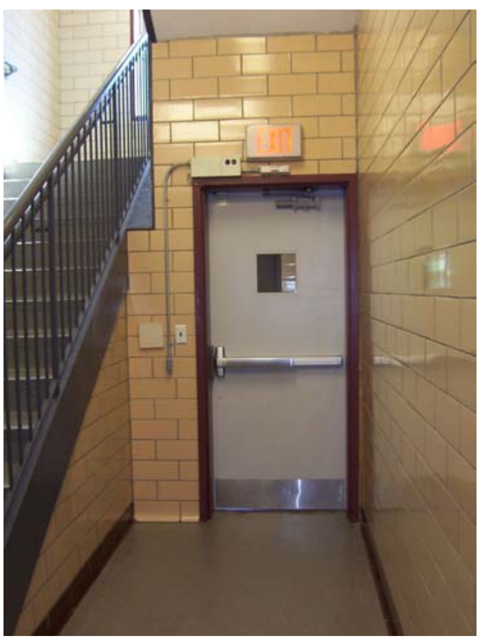

Figure 13: Example of Self Illuminated Sign (Possibly Tritium type)

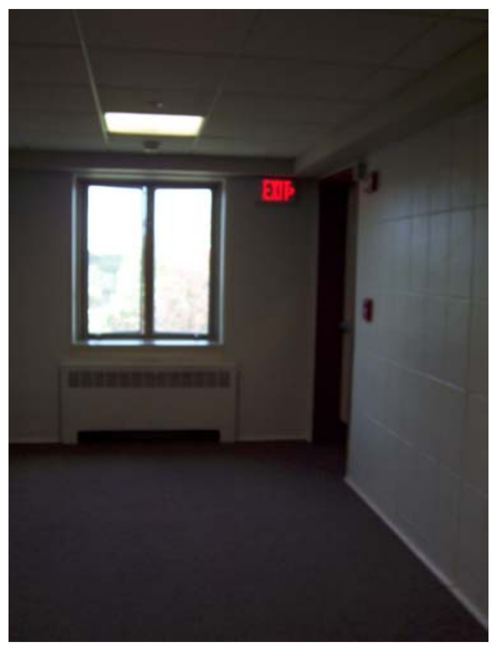

Figure 14: Example of Internally Illuminated Sign

Exit signs are required to be illuminated at all times. Exit signs shall be connected to a backup power supply to maintain illumination for at least 90 minutes in case of primary loss of power. An exception allows those signs that provide continuous illumination independent of external power sources (for 90 minutes during loss of primary power) are not required to be connected to an emergency electrical system.

The means of egress shall also be illuminated per IBC section 1006.1. Anytime the building space is occupied, the means of egress, including the exit discharge shall be illuminated. The illumination level shall be at least 1 foot-candle at the floor level in accordance with IBC Section 1006.2. No illumination readings were taken within the building. However, no specific illumination concerns were noted.

One of the most important features of the means of egress illumination is for a loss of power supply; the means of egress shall automatically illuminate specific areas of the building. Those areas in the building include exit access corridors, passageways, and exit stairways. In addition those portion of the exterior exit discharge immediately adjacent to the exit discharge doorways.

6.9.5 Stairwells

Table 29 summarizes the stairwell requirements and the status of the actual construction. Per the IBC the following stairwell requirements should be met:

Table 29: Stairwell Requirements

Building Feature	IBC Requirement	Actual	Status
Stair Width	44” minimum	43”	Not in Compliance
Stairwell Headroom Clearance	80” minimum	100”	In Compliance
Stair Riser Height	7.75” maximum	7.75”	In Compliance
Stair Riser Depth	10” minimum	10.5”	In Compliance
Landing Width	> Stairway Width (43”)	60”	In Compliance

The only clear deficiency of the stairwells within the building is the width of the stairs. The measured width was an inch less than required. However, the landing widths were greater than the stairwell widths and the stairwells did not narrow in the path of egress.

The IBC also regulates handrails within stairwells. As per Section 1009.10, there are handrails on both sides of the stairs that extend horizontally at least 12" beyond the top riser and continue to slope for the depth of one tread beyond the bottom riser. All handrail requirements, as well as those listed in Table 30, were found to be in compliance with the IBC.

Table 30: Handrail Requirements

Building Feature	IBC Requirement	Actual	Status
Handrail Diameter	1.25" minimum 2" maximum	1.75"	In Compliance
Handrail Wall Clearance	1.5" minimum	1.5"	In Compliance
Handrail Height	34" minimum 38" maximum	37"	In Compliance

6.10 Conclusions

There are both strengths and weaknesses identified in the Egress features of the facility. The weaknesses are generally offset by the automatic sprinkler system that is installed throughout the facility. In general the egress features of the building appear to be substantially complete and in compliance with the IBC, however, there are some noted non-compliant areas that require attention.

7. Fire Protection Systems

This section discusses the active and passive fire protection systems installed in the masonry building. The provisions of Chapter 9 of the IBC were applied to determine the compliance of the building's active and passive fire protection systems. This chapter provides the reader with a basic understanding of the fire protection systems installed in the building. Additional information regarding the fire protection systems is provided in Appendices H, I and J.

7.1 Active Systems

The term Active Fire Protection Systems refers to those devices within a building that require power for their operation.[105] This power may be supplied manually, electrically or mechanically; however, without this power the system would not operate.[106] This section explores the active fire protection systems of the masonry building and investigates their compliance with the International Building Code.

It is important to note that all of the fire protection system components used within the building are UL listed and intended for use in fire protection service.[107] Additionally, the installation of the automatic sprinkler systems, standpipe, and hose systems must comply with the regulations of NFPA 13 and 14, respectively.[108]

7.1.1 Fixed Automatic Fire Protection Systems

The renovated masonry building is fully sprinklered. The majority of the building runs on a wet pipe system that covers office space, library stacks, and mechanical and electrical rooms.[109] There is a dry system installed to cover the emergency generator room; however, this room is located in the newly constructed building adjacent to the masonry building.[110] There are electrical and telephone/data closets on every floor that

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[105] NFPA. Fire Protection Handbook: Nineteenth Edition. 2-104.
[106] Ibid. 2-104.
[107] Gateway Park Project Fire Protection Specifications (Consigli). 4.
[108] Ibid. 1.
[109] Ibid. 5
[110] Ibid. 5.

Figure 15: Sprinkler Head in Electrical Closet

are equipped with both sprinklers and smoke detectors. A sprinkler head installed in the electrical room on the first floor can be seen in Figure 15.

The lecture hall located on the first floor, with an A-3 occupancy classification, is fully sprinklered. According to IBC Section 903.2.1.3, this space does not require an automatic sprinkler system because the floor area does not exceed 12,000 ft^2, has an occupancy load of less than 300 (97 people for lecture hall), and is located on the level of exit discharge.[111] Therefore, the installation of sprinklers in this space exceeds IBC requirements. In addition, there is a Business Occupancy located on the first floor.

The upper three floors of the masonry building are class B Occupancy and are sprinklered throughout. There are no requirements outlined in the IBC for the installation of automatic suppression system in Business Occupancies; thus, the installation of these systems exceeds the provisions set forth by the IBC.

The masonry building is connected to an atrium in the newly constructed building. This atrium space meets the IBC definition of an atrium listed in Section 404.1.1.[112] There is no structure to provide separation between the atrium and the first and second levels of the masonry building, and the open space of this area causes great concern in terms of the likelihood of fire spread. Therefore, although the atrium is not located directly in the masonry building, it is an important feature to mention. As with the rest of the building, the atrium is outfitted with an automatic sprinkler system. No additional systems were installed to meet the unique challenges of detecting fire in this space.

The basement in the masonry building is considered an incidental use area. The IBC requires a one-hour fire barrier between incidental use areas and business occupancies.[113] The stairwells leading from the basement area up to the ground floor were constructed with a 2-hour fire rating, and exceed the requirements of the IBC. No

[111] 2006 International Building Code. 174.
[112] Ibid. 41.
[113] Ibid. 81.

further protection, such an automatic suppression system is required in this area because although the basement is an underground structure, the floor level is not more than 30 feet below the lowest level of exit discharge. Therefore, the automatic sprinkler system requirements of underground buildings of Section 405.3 do not apply to this space.

Sprinkler Design and Placement

Most of the masonry building is considered a light hazard. However, the library stacks on the fourth floor and electrical rooms are Ordinary Hazard Group 2 (OH2). These different hazard designations require varying design areas and densities for the sprinkler systems. For additional information on design area and densities required for varying hazards please refer to Appendix K.

There are upright sprinklers in the plenum space in the ceiling. These sprinklers were installed to prevent ignition of the timber floor structure that is hidden by the drop-ceiling. In addition to these sprinklers, covered pendant sprinkler heads are also installed in the ceiling to protect the occupant area below. From both visual inspection and notes on the building plans we found the sprinkler heads to be centered in the ceiling tiles. Such is the case to create a uniformity that is aesthetically pleasing and consistent throughout the building. Also, there are sprinklers located throughout the building's stairwells, including upright sprinkler heads underneath the stair structure, as shown in Figure 16.

Figure 16: Upright Sprinkler Head in Stairwell

Piping and Standpipe System

The automatic sprinkler system in the building includes piping to carry water from the water supply to sprinkler heads, as well as the standpipe system. The piping for the entire system is required to be black steel of standard weight that meets ASTM A795 or ASTM A135 standards.[114] All of the pipes of 6-inch to 2.5-inch diameter are Schedule 10, while those pipes 2-inch diameter and smaller are Schedule 40 piping.[115]

[114] Fire Protection Specifications, Consigli. 6.
[115] Ibid. 6.

The building contains a standpipe system that is capable of, "…providing service to every part of the building while using a 100 foot hose with a 30 foot hose stream"[116]. The standpipe system is designed to provide a, "…500 gpm [gallon per minute] minimum from [the] first standpipe and 250 gpm from each additional standpipe…."[117]. There are 6-inch diameter standpipes in both stairwells in the masonry building that provide fire department connections. The standpipe installed in the stairwell at the front of the masonry building is shown in Figure 17.

Figure 17: Standpipe in Stairwell

For buildings with a floor level above 30 feet, the IBC provides a minimum requirement of installing a class III standpipe.[118] However, for buildings equipped throughout with an automatic sprinkler system, only a class I standpipe system is required. The fire protection specifications for the masonry building specify the standpipe system as class III. The standpipe system exceeds the requirements for this system component. The system was installed in accordance with NFPA 14, which also meets IBC requirements.[119]

There are fire department connections (FDC) located throughout the building, in both the stairwells and the hallways. As shown in Figure 18, the FDC's are well labeled for quick and easy access. Similar to the other components of the automatic suppression system, the fire department connections have been designed and installed to NFPA standards. These standards are the same ones cited in the IBC; hence, the fire department connections provided in the masonry building are up to code.

Figure 18: Fire Department Valve in Hallway

[116] Ibid. 5.
[117] Ibid. 5.
[118] 2006 International Building Code. 180
[119] Ibid. 180.

System Valves

There are backflow-preventer assemblies throughout the system to maintain proper directional flow of the water in the system. These backflow-preventers are, "…double check valve assembly with test cocks for field testing, two OS&Y valves with tamper switches."[120] The term OS&Y stands for an Outside Stem and Yolk valve. Theses valves are used, "…to quickly determine if the valve is in the open or closed position" and allow for the system to be shut down.[121] In the OS&Y valve the stem feature, which rises as the valve is opened, allows a person to visually determine if the valve is open or closed.[122] This feature is an important indicator that the system is ready to respond in the case of a fire emergency.

The double check valve installed in the system is made of bronze components and rubber facings to ensure that the seals are tight.[123] This valve is outfitted with pressure gauges so that system may be checked quickly.[124] The American Water Works Association (AWWA), an, "… international nonprofit scientific and educational society dedicated to the improvement of water quality and supply," requires a double check valve assembly when there is a fire department connection on the system.[125, 126] In addition to those already mentioned the masonry building has floor control valves installed on every building level. These assemblies are located in the stairwell at the front of the building that provides exit discharge onto Prescott Street. Floor control valves manage the flow of water through the piping. In the case that a control valve is closed the suppression system will not operate; thus, preventing the system from performing its function. Therefore, these assemblies are typically posted with signs indicating that they must be left open at all times.[127]

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[120] Fire Protection Specifications, Consigli. 6.
[121] NFPA. Fire Protection Handbook: Nineteenth Edition. 10-198.
[122] Ibid. 10-198.
[123] Ibid. 10-53.
[124] Ibid. 10-53.
[125] Ibid. 10-53.
[126] American Water Works Association. "Who We Are." http://www.awwa.org/About/
[127] NFPA. Fire Protection Handbook: Nineteenth Edition. 10-378.

7.1.2 Manual Systems

There are class ABC fire extinguishers in recessed fire cabinets on each floor. Class ABC signifies a dry chemical extinguisher that, "…is filled with monoammonium phosphate, a yellow powder that leaves a sticky residue that may be damaging to electrical appliances such as a computer."[128] These extinguishers are located in each hallway on the floors. The length of the hallway determines the number of extinguishers, and in the case of the masonry building there are two extinguishers per floor.

Portable Fire Extinguishers are addressed in Section 906 of the IBC, which states that such devices are to be provided in accordance with the International Fire Code.[129] The signs noting their location will also be installed in accordance with the *International* Fire Code, which is published by the International Code Council.

Figure 19: Manual Pull Station

There are manual pull stations located throughout the masonry building, an example of which is shown in Figure 19. The IBC states that manual fire alarm systems are not required for group B Occupancies with less than 1000 people above the lowest level of exit discharge.[130] In the masonry building, there are far less than 1000 people occupying the space above the exit discharge level. Thus, the manual fire alarm boxes located throughout the building exceed the requirements stated in Section 907.2.2 of the IBC.[131]

7.1.3 Detection and Alarm Systems

This section addresses the sprinkler system monitoring and alarms. This applicable section of the IBC provides standards for fire alarm control panels, fire alarm terminal cabinets, trouble signal transmission, and much more. Provisions are also made for horn-strobes and smoke detectors for occupant notification.

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[128] "Fire Extinguisher Types." http://www.fire-extinguisher101.com/index.html
[129] 2006 International Building Code. 182.
[130] Ibid. 182.
[131] Ibid. 182.

There is a fire alarm control panel. It is located in the adjacent new construction. We were not able to gain access to this portion of the building; therefore, the exact make and model of the fire alarm control panel are unknown. However, it is supplied with backup power from a generator. The length of time that the generator provides power for is also unknown.

Figure 20: Fire Alarm Terminal Cabinet

As shown in Figure 20, there is a fire alarm terminal cabinet on each floor of the old masonry building and each floor of the new building as well. All of the smoke detectors, horn strobes, fire dampers, and duct smokes located on any given floor are tied back to these boxes. On the inside of these terminal cabinets there are several red wires "terminated" with a screw. Each terminal cabinet is connected into the individual fire alarm communication box located on each floor, which in turn is tied back into the main fire alarm cabinet, which is then tied back to the Worcester Fire Department.

Each project is different depending on the Engineer/Architect/design. Some projects have only one fire alarm terminal cabinet in the entire building that all the smokes, horn strobes, etc. are tied back into, but in general terms, the process is to have one per floor.

There are combination horn-strobe alarms installed throughout the building. An example of these horn-strobe devices, which are typical for current fire alarm system standards, is shown in Figure 21. The device emits a bright flash as well as a loud horn sound to inform occupants of the building emergency or potential hazard. The masonry building is not installed with a voice communication system; thus, these devices are the primary means of alerting building occupants.

Figure 21: Horn-Strobe

The detection and alarm system in the building provides a direct signal to the Worcester Fire Department. Section 903.4.1 of the IBC mandates trouble signal transmission to an approved central station. This provision, also found in NFPA 72, it met by the masonry building's alarm system.

For Business Occupancies, Section 907 of the IBC requires smoke detectors be installed in any building with an occupant load of more than 100 persons above the lowest level of exit discharge.[132] There are there smoke detectors throughout the building. They are located in the corridors, offices, and stairwells. Thus, the masonry building meets the provisions of the Code. There are also smoke detectors located in throughout the lecture hall and the telephone/data closets on every floor.

The atrium is outfitted with sprinklers and smoke detectors. However, no additional systems were installed to combat the difficulties associated with detecting a fire in a large, open area such as this.

7.1.4 Smoke and Heat Ventilation

There are no automatic smoke control systems or other features to control smoke movement or ventilation installed in this building. Section 909 of the IBC does not require smoke or heat ventilation for the masonry building. Thus, the building is in compliance with the codes specifications.

Additionally, the atrium in the adjacent space is not installed with a smoke control system. However Section 404.4 of the IBC states that such a system is not required in the case of atriums that connect only two building levels, as is the case with the masonry building and adjacent building.[133]

7.1.5 Water Supply and Reliability

The system is maintained using water from the City of Worcester water supply. There are four hydrants within close proximity to the masonry building. Three of the hydrants are located along Prescott Street, and the fourth is located behind it. Appendix L shows the fire hydrant locations nearest the building.

Water supply requirements are listed in Section 903.3.5 of the IBC. The section cites the requirements of the automatic sprinkler code, NFPA 13. As previously mentioned, the Fire Protection Specifications for the project cite that the automatic sprinkler system must meet the provisions of NFPA 13; thus, the masonry building is also compliant with the IBC.

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[132] Ibid. 182.
[133] Ibid. 42.

7.2 Passive Systems

As defined by Fitzgerald, [134] passive fire defenses are the components within a building that remain fixed whether or not a fire exists. According to the 19th Edition of the NFPA Fire Protection Handbook, the three main goals of passive fire protection are controlling the rate of fire growth, compartmentation, and ensuring a safe emergency egress system.[135] This section includes a detailed description of each of these factors in regard to the masonry building.

7.2.1 Rate of Fire Growth

An important aspect of Passive Fire Protection is the rate of fire growth, which can be controlled by the interior finishes used in the facility. When the rate of fire growth is slow, there is more time for occupants to egress safely, and there is typically less property damage. When considering the rate of fire growth, the interior contents of the facility are not analyzed because they are not a fixed part of the building.

Materials form the basic components of a structure, and are combined to form products. For example, gypsum, paper, and glass fibers are combined to form sheets that result in the construction of fire-rated walls and ceiling assemblies. Assemblies are the combination of products, for example, wood-frame walls are the combination of wood studs and gypsum wallboard.[136]

The Fire Protection Handbook refers to non-combustible materials as materials that produce a negligible amount of heat when exposed to elements in a post-flashover fire. Concrete and steel are recognized by model building codes to be non-combustible and will prevent flame spread in the event of a fire. NFPA 101- *Life Safety Code* makes a distinction between noncombustible and limited combustible materials. NFPA 101-3.3.150.3 defines a non-combustible material as:

> A material that, in the form in which it is used and under the conditions anticipated, will not ignite, burn, support combustion, or release flammable vapors, when subjected to fire or heat. Materials that are reported as passing

[134] Fitzgerald, Robert. Building Fire Performance Analysis .12.
[135] NFPA. Fire Protection Handbook: Nineteenth Edition. 2-103.
[136] Ibid. 2-106.

ASTM E 136, *Standard Test Method for Behavior of Materials in a Vertical Tube Furnace at 750 Degrees C*, shall be considered noncombustible materials.[137]

It is important to realize that any material will burn at a high enough temperature but the intent is to use materials that will prevent the movement of fire into different compartments.

As described in the Fire Safety Analysis, the masonry building falls under the occupancy classification of A-3 for the lecture hall and B for the office space.[138] Table 508.3.3 of the IBC requires that firewalls have a fire resistant rating of 1-hour between these occupancy groups, for a sprinklered building.[139] For Type IIIB construction both interior and exterior bearing walls must have a fire-resistance rating of at least 2 hours.[140] Also, there is a required 2-hour fire rating for the ceiling above the electrical and telephone/data closets that are located on each floor.

Unprotected structural steel loses strength very rapidly in a fire. Mineral spray-on materials or gypsum board coverings usually protect steel beams and girders. The known materials used to construct the building were exterior brick masonry bearing walls and heavy timber beams and columns.

During the renovation of the building the timber beams were encased in gypsum wallboard. This addition improved the aesthetics of the space, as well as provided a fire barrier to the timber column. To create an open space on the first floor for the lecture hall, a large steel beam was added to the ceiling layout. Replacing a column and an old timber beam, the steel beam provided the space with greater flexibility and improved aesthetics. On the upper floors where the office space is located, the partitions between offices are constructed from gypsum wallboard.

A built up roof covering is defined by the IBC as "two or more layers of felt cemented together and surfaced with a cap sheet, mineral aggregate, smooth coating or similar surfacing material". This type of roof construction would be classified as Class A roof assemblies and according to the IBC are effective against severe fire test exposure.

[137] National Fire Protection Association. NFPA 101: Life Safety Code: 2006 Edition.
[138] 2006 International Building Code. 23,24.
[139] Ibid. 81
[140] Ibid. 87.

It is of typical practice for a building to comply with the IBC in regards to protecting itself from large fire growth rates. It is apparent that the masonry building of Gateway took adequate measures to fireproof its appropriate components.

7.2.2 Compartmentation

One important part of a passive fire protection system is containing the fire within a limited area, known as compartmentation. In order for successful compartmentation of a fire, the structural elements of a building should be designed to prevent structural failure. This reduces the risk of fire spread to adjacent areas of the building. Barriers also play a significant role in the compartmentation of smoke, fire, and heat. Not only do barriers prevent the spread of these components of a fire, but they also assist in some detection methods as they control and limit movement.

The renovated masonry building demonstrates both strength and weakness for compartmentation in the case of a fire. The building's exterior brick masonry bearing walls and gypsum walls throughout the building, are examples of adequate fire barriers.

The second through fourth floors are composed of office space and conference rooms. Each room has a fire-rated door with a door closure that prevents the spread of products of combustion. All common rooms and restrooms within these floors also have doors, leaving the corridors and the space leading to the connector building as the only "open" spaces. The open space in the masonry building opens into the atrium in both the first and second floors. In the event of a fire in any of the rooms on the third or fourth levels of the building, compartmentation is probable. However, the open space on the first and second floors may prevent compartmentation.

The main level of the building consists of a lecture hall and a large, open, common area with an atrium extending to the second level of the building. The lecture hall is adjacent to the atrium, but is enclosed and separated from this space by fire-rated, double doors. The atrium acts as a lobby for the masonry building, as well as the new construction, and would facilitate significant fire and smoke spread, neglecting active fire protection systems.

All stairwells throughout the building are enclosed and provide access through fire-rated doors with door closures. The enclosed stairwells will provide compartmentation in the instance of a fire in this location. On the other hand, if the fire is not within the stairwell, it will provide an area of refuge or a safe, smoke-free egress route for occupants.

The importance of compartmentation has become a more important issue in respect to protecting a building. Overall, the masonry building does have a strong Passive Fire Protection system despite minor weaknesses which, in general, are due to the design of the building, not failure to comply with the IBC. The Firestop Contractors International Association (FCIA) recognizes that it would be beneficial to collaborate with other groups to try and affect change in codes for compartmentation. This includes changes to fire doors and hardware, fire dampers, fireproofing, fire glass, the fire wall of gypsum, concrete or block, and others to learn more and educate about compartmentation, while promoting the industry's importance in fire and life safety.[141]

7.2.3 Emergency Egress

An essential component of Passive Fire Protection is ensuring proper emergency egress for building occupants. The emergency egress system includes exit access, exits, and exit discharge from the building. Defined corridors, enclosed stairwells, and exit doors are all components of egress that can be found in the masonry building. These elements are all built into the structure of the building to allow passage of occupants from a fire emergency to a place of safety. Using the proper regulations, the egress system has been designed to facilitate the exit of all building occupants. Please refer to the Fire Safety Code Analysis section of this report for a discussion of egress as it specifically applies to the masonry building.

[141] FCIA. http://www.fcia.org/index.htm

8. Risk Assessment

Risk assessment refers to the likelihood of adverse events.[142] For our risk assessment we identified possible scenarios that could occur in the masonry building and what their implications would be. This section is intended to investigate threats that exist in the building and the sequence of events that would lead to full room involvement (FRI). Those events include the presence of an ignition source, no automatic suppression, no manual suppression, and sufficient fuel and ventilation for the fire to grow. These possible events will be addressed through unfavorable scenarios occurring in the masonry building

It is important to note that in completing a risk assessment there may be several different objectives to accomplish. These objectives are dependent on the different points of view regarding the space under consideration. For example, the owner of a building is likely to have concerns that vary greatly from those of a building occupant. The building occupant may simply be concerned with proper egress design to ensure his or her safety in the event of a fire. However, a building owner would be concerned with life safety issues, as well as, the costs associated with smoke and fire damage. Beyond the costs of repairing the actual space, the building may have to be shut down for a period of time, which could lead to a loss of profit for the company. This example provides us with an awareness of the different factors that may come under consideration in a risk assessment.

8.1 Manual Suppression After Hours

Many current WPI facilities are occupied after normal business hours by both faculty and students. The building will primarily be used at the graduate level, typically graduate lectures and class meetings occur in the evening to accommodate part-time students who maintain a full-time job. Therefore, it is likely the lecture hall would be occupied on many occasions in the evening. Additionally, students especially, are known to work on research and homework at night, in some instances through the night. In the event that a fire occurred at night and occupants were in the facility it is possible they

[142] "Risk Assessment." http://dictionary.reference.com/browse/risk%20assessment

would sense the fire before the detection system, in turn, the fire would be smaller at the time of suppression.

If one were to base the probability of the building being occupied based on the code classification, business and assembly, it would be unlikely that occupants will be at the facility after normal business hours. Thus, if a fire occurred after hours, manual suppression is not a possibility. According to these assumptions there are two scenarios presented that would result from a fire occurring in a building with no occupants.

The first scenario investigates if no automatic suppression system were installed or failed to function. In this case the fire would be able to grow infinitely large until detection systems (if installed) notified the local fire department. This case would result in the greatest loss of property and damage. Currently, the measures in place to control the fuel loads when manual suppression is not an option would be the passive fire protection in place. In Type III construction the exterior of the building is made of non-combustible materials, however, the interior is typically constructed with combustible materials. In this case, the partitions appear to be constructed with gypsum wallboard. There is currently fire-rated gypsum wallboard on the market that if installed in the historic building, even with the absence of sprinklers the building, would have a higher probability of the fire being contained to one compartment.[143] In the case of Gateway Park, automatic suppression systems are installed throughout the facility.

On the other hand, if automatic suppression systems are installed, the fire would grow until the detection system detected the fire and activated the automatic suppression systems. It is likely that the fire would be limited to one compartment. Although the fire may grow larger than if it were detected by occupants it will still be relatively small and full room involvement likely would not happen due to sprinkler activating prior to this point. According to the National Fire Protection Association only one or two sprinkler heads were activated in 81 percent of the fires with wet pipe systems and 56 percent of the fires with dry pipe systems.[144]

The majority of the existing building is being used as office space for WPI faculty and their supporting staff. In such occupancies the main fuel loads would be desktop

[143] "Temple Inland."
[144] National Fire Protection Association, "Facts and Figures"

computers and paper products including student assignments, research, and faculty reference libraries. A computer could be damaged through fire and also through suppression of fire. It is probable that there is a large amount of valuable information stored on the computers and the loss of which would be devastating. Due to the nature of the rest of the facility (i.e. laboratory and research areas in the new construction) there is likely an abundance of important research stored in the facility. The loss of such research due to fire could potentially be devastating. However, sprinklers typically reduce the average property loss due to fire by one-half to two-thirds compared to where sprinklers are not present. [145]

8.2 Wet Versus Dry Pipe Automatic Suppression Systems

There are potential risks involved in installing different types of automatic suppression systems in a building. In the case of a wet pipe system versus a dry pipe system there are innumerable risk related issues. However, we focused on those associated with false alarms and time needed for water delivery using the different systems.

By definition, a wet pipe suppression system is that in which, "…the piping contains water at all times and in connected to water supply so that water discharges immediately…" once the sprinkler head activates.[146] In the case of a dry pipe system, there is no water in the piping. These systems also utilize automatic sprinkler heads; however, when they activate the drop in pressure then activates a dry pipe valve that allows water to enter the pipes.[147]

Wet and dry pipe systems are often used in much different ambient conditions. For instance, wet pipe systems are typical for commercial and residential spaces. On the other hand, dry pipe systems are used in warehouses or other facilities where there is a possibility of the pipes freezing due to lower ambient temperatures.

The possibility of a false alarm occurring would have very different results with a wet system or a dry system. In the case of a wet pipe system with a false alarm, a more substantial amount of damage could be caused due to the earlier release of water from the

[145] National Fire Protection Association. "Facts and Figures".
[146] Dubay, Christian. Automatic Sprinkler System Handbook. 141.
[147] Ibid. 143.

system. While there would be some water damage in the case of both types of systems, the delay of the water release in a dry pipe system would minimize damage to the effected area. Aside from a false alarm, there is also the possibility of knocking off a sprinkler head, perhaps during building maintenance. Similarly, in the case of a wet pipe system there would be significant water damage; however, there would be less damage with a dry pipe system.

Conversely, in the case of an actual fire emergency a wet pipe system would be more likely to minimize damage and costs. The quicker release of water would control or suppress the fire, as well as limit fire spread from the room of origin, and the only time lag associated with this type of system is that required to activate the sprinkler head. In the case of a dry pipe system there is an additional time lag from sprinkler activation to water release.

Based on these inherent differences there may be a substantial difference in the time required for water delivery between wet and dry pipe systems. The lag for dry systems would allow the fire to continue growing for some period of time. This fire growth could allow the fire to spread from the room of origin to other sections of the building, causing greater fire damage and possible life safety issues.

An important aspect of risk assessment is identifying the uncertainties that may present themselves in the proposed situation. Identifying these uncertainties creates a more comprehensive understanding of possible situations that may develop. In the case of an automatic sprinkler system there is uncertainty regarding sprinkler activation. It is possible for a sprinkler head to be defective. Both wet and dry pipe systems face this uncertainty.

Additional uncertainty can be attributed to the type of fire that develops and how the sprinkler head is designed to react. For a rapidly developing fire sprinkler heads are likely to perform as designed and control fire spread upon activation. However, in the case of a smoldering fire, or a slowly developing fire, a sprinkler would take significantly longer to activate. In these fires temperature rise would occur at a slower rate, which would effect the time to sprinkler activation.

In developing this risk assessment it was important to acknowledge the additional risk that may develop from situational uncertainties. It is evident that there is risk, as well as disadvantages and advantages, associated with both wet and dry pipe suppression systems.

8.3 Egress

One of the most important concerns in the event of a fire is life safety. To ensure the safety of the occupants within a building, the egress system of the facility must be properly designed. Upon speaking with representatives from the firm managing the Gateway Park Project, we were informed that one of the stairwells within the existing masonry building was not considered a route of egress. The building, however, has access to two main stairwells within the link of the buildings, both of which are sufficient for means of egress. Though this design meets the code, there are apparent risks that arise from the situation.

For the purpose of discussion, we will refer to the insufficient stairwell as Stairwell A. Although the stairwell is not considered part of the egress system, the avoidance of this path is not guaranteed. It is noted that in the event of a fire, building occupants will exit in the most familiar route traveled. It is assumed that most occupants will enter and exit the building through the main exits within the link. In this case, most will seek this route to exit the building if a fire occurs.

On the other hand, some persons will be prone to make use of the closest exit, which in some cases will be the aforementioned stairwell. Stairwell A is located directly off of the main corridor of the upper levels of the building. Many offices are situated off of this corridor. It is expected that many occupants of these rooms, would use this exit for simplicity and quickness in a hectic situation of a fire. Figure 22 shows Stairwell A outlined in red. The main corridor of the building is highlighted in yellow.

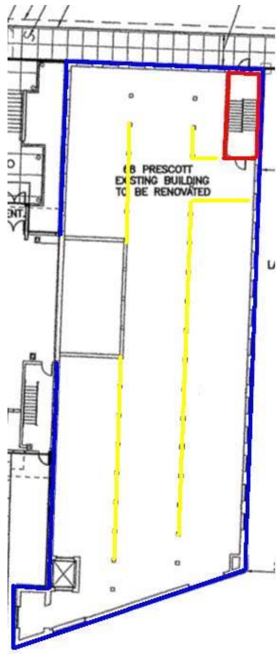

Figure 22: Stairwell A

During a walk through of the facility, Stairwell A was examined and measurements were taken. There were several features within the stairwell that were not in compliant with the Code. First, the width of the stairwell proved to become narrower in the direction of egress. In addition, the handrails were not located at a proper height. Insufficient qualities such as these could result in major problems in the event of a fire.

Because Stairwell A does not meet the standards of the Code, it should not be utilized as egress. Consequently, there are risks involved in maintaining a stairwell as such within a building. It is inevitable that Stairwell A will be used if there is a fire in the building. In the event that the stairwell is used, injuries and deaths could potentially occur. The behavior of occupants under the circumstances of a fire would cause additional risk to the use of the stairwell, as some may panic and exit in a disorderly manner. As a result, the likelihood of injuries may increase. Furthermore, if the fire was in a location that blocked access to one of the other exits, the possibility of occupants resorting to Stairwell A would be highly probable. The building owner should be concerned about the stairwell resulting in injuries and possible legal cases that could arise.

Our recommendation to avoid the occurrence of injuries or court settlements is to bring the stairwell up to Code. This may cause an initial cash outflow but it would decrease the risk within the egress system while increasing the safety of the occupants. Therefore, the building owner may save money in the future because there would not be a possibility that he or she be at fault for injuries occurring within the egress route during a fire.

8.4 Presence of Fuel

Nowadays, office facilities typically consist of "high-tech" equipment such as computers and plasma screen televisions. Unique to this masonry building, there is a lecture hall that will have a large, drop down screen, projectors and presentation equipment. All of these elements are essential for the facility to serve its daily purpose. On the other hand, if a fire were to occur, this equipment can cause catastrophic damage due to its flammability.

Other than heat and oxygen, fuel is the other component of creating a fire. The location of the fuel is a contributing factor of how large the fire can grow, how fast it can spread throughout the building and ultimately how much damage, both long and short term, it will cause the building. For example, a fire present in an electrical room could smolder, which would not be detected in a timely manner depending on its location relative to the fire detection systems. There could be significant property loss if a fire was present in another part of the building such as an office. In an office, there are different forms of fuels such as cubicles, chairs, waste paper baskets, file cabinets, computers, printers, fax machines and possibly plasma screen televisions. Important implications to consider in this situation: loss of data, loss of equipment and monetary damages. The process of replacing data is more difficult than replacing equipment. In the long run, losing data over equipment will be more detrimental monetarily. An emotional element needs to be factored in as well. The roll of a graduate student, particularly in the biological field, is to conduct research via experimentation. If their data, results and conclusions were lost, months, possibly years of their work would not exist anymore.

With the presence of various fuels in this building, risk management, the process of assessing risks of a facility and then forming strategies to manage them is imperative.

Strategies include transferring the risk to another party, avoiding the risk, reducing the negative effect of the risk, and accepting some or all of the consequences of a particular risk. An example of risk reduction would be the installation of appropriate protection (i.e. sprinklers) in vulnerable areas, such as the lecture hall with many sources of fuel. In the case of the laboratory space, extra precautions are to be made due to the immense amount of flammable chemicals and the value of the ongoing research experiments.

It is to the owner's benefit to recognize the hazard that lies in the presence of fuels in their building. Luckily, insurance companies have come to realize the importance of data loss and recovery in conjunction with property loss and recovery. There is a reason why there are insurance policies designed to keep companies afloat by compensating for lost revenues while either equipment is down or data is lost. From an owner and occupant point of view, the facility, the equipment in the facility and the data contained within the equipment is invaluable and worth taking many routes of precaution.

In conclusion, there are many possible scenarios that might occur when considering fire risk. The intent of this chapter was to show possible sequences of events that may be considered concerning life safety issues. The were four scenarios considered in relation to the Gateway Park development were manual suppression after hours, wet pipe versus dry pipe suppression systems, egress, and fuel loads. This chapter presented just a few of the many different scenarios that could occur and the implication of the events to both the occupant and the building owner.

9. Conclusions and Recommendations

The intent of this report was to provide an overview of structural alternatives within the Gateway Park Project facility. After investigating different construction materials and layouts, we determined the most desirable scenario for both the steel and reinforced concrete systems. Our analysis not only focused on the flexibility of the space but the cost estimates of each design as well. Lastly, a conclusion was drawn with respect to the fire protection systems within the building, giving specific attention on the egress system of the building. The following chapter summarizes our conclusions and recommendations for future study.

9.1 Structural Steel

While investigating the process of structural steel design, we were able to experiment with three different layouts: 34.5' x 27.75', 69' x 37.75' and 34.5' x 18.5'. From the gravity design of our structural steel alternatives we were able to recognize an important relationship between increased span and depth of construction. As the span covered by girder for bay areas become larger, the depth of construction also increases. With this increase in depth of construction vertical flexibility is adversely affected because there is a limiting height in the building. When the floor and ceiling assemblies increase the usable area decreases.

Also, in our alternative structural designs we completely gutted the building to introduce a new steel frame. One consequence of putting a new frame in the building is placing new 5-inch concrete slabs in place of the current wood floors of the masonry building that are uneven due to the effects of time and settling of the structure. Thus, the building would have level floor surfaces.

Testing the 34.5' x 27.75' bay size with 4 filler beams for lateral deflection with RISA we determined 3.14 inches of sway at the roof level. This exceeds the maximum allowable sway due to wind forces, and based on these results we can conclude that our structural steel alternative is under-designed. However, the maximum deflection due to seismic loading resulted as 0.264 inches, indicating our structural steel alternative is adequately designed or perhaps slightly over-designed.

It was important to consider the vertical flexibility of the space in our design to meet the desired 15’ floor height for the adjacent lab space. However, this vertical flexibility could also affect the future return in a resale of the building. If the floor height was diminished by large girders or beams the use of the space would be limited, which could in turn affect the resale value of the property. A cost analysis provided us with an important understanding of the relationship between design and economics.

In design, it is important to consider the flexibility of a space provided by the structural layout versus the economics of providing adequate steel members. Large, open floor plans may be more desirable for building occupants; however, this can detract from valuable vertical space. As span length for beams and girders increase, depth and cost increase as well.

9.2 Reinforced Concrete

After investigating different reinforced concrete systems, an analysis of each was performed. In determining the most desirable option for further study with respect to lateral loading, the following characteristics were considered; most flexibility, i.e. open space; system integration within vertical constraints; and lastly, the cost of each design. Based on these factors, the preferred option was Scenario 1: a one-way 5-inch slab with end bays of 21’ x 18’ and interior bays of 21’x 19’.

Our results show that the maximum lateral deflection of our system was 0.047 inches due to wind loadings. In respect to seismic loadings, our system resulted in a maximum deflection of 0.014 inches. These deflection values portray a system that is structurally intact even under extreme conditions. It is likely that these small deflection values exemplify an over-design of the concrete system. The lateral loading results are a reflection of the designed gravity system. Thus, smaller beams, girders, and columns may have been used for the gravity systems, from which larger deflections within reasonable limits may have resulted. Consequently, the cost of our system would have been decreased in the event that our system was designed with smaller members.

According to our calculated cost estimates, Scenario 5, the two-way slab was the most economical design with a cost of approximately $39 per square foot. However, in weighing all aspects of the design as previously mentioned, we selected Scenario 1 as the best option. This system had a comparable cost of $40 per square foot. We concluded that the difference in the cost would be minimal and this design is more desirable because it offers more flexibility and fits within the vertical limits.

The economics of construction have been known to make or break a project. In this regard, over designed systems are not desirable. In many instances, building owners are looking to cut back on cost any way possible. Cost reductions often lead to innovative design.

9.3 Structural Alternatives

It is necessary for a building owner to weigh advantages and disadvantages of a structural framing system based on what they desire within the space. The owner would need to consider the function of the space and decide the most important parameter of constructing the building. There are disadvantages and advantages of both the steel and reinforced concrete systems analyzed in this report.

The advantage of the steel system is that it provides the most flexibility within in the facility, with the most desirable bay size of 34.5’ x 27.75’. However, this system is not the most economical. It appears that the most desirable reinforced concrete system costs less than the aforementioned system. Additionally, steel systems require spray-on fireproofing to obtain the same rating that is inherent within a concrete slab of adequate thickness.

In conclusion, all the scenarios analyzed are sufficient to the objectives of the project- creating structural alternatives that provides more flexibility within the space with a reasonable cost associated with them. However, the most desirable situation would rely on the owner’s objectives and the feasibility of the construction challenges. First, the structural alternatives presented require completely gutting the interior of the building. During demolition of the interior, environmental concerns may arise due to toxic materials located in the facility, for example, asbestos. Also, typically when buildings are gutting the structural framework is kept in tact. However, we are replacing the structural skeleton, which would require bracing of the exterior masonry.

9.4 Fire Protection

The renovated masonry building of Gateway Park has been reviewed with respect to its active and passive fire protection systems. These have been shown to be a relatively strong point in the overall design features of the building. A fire alarm system that includes smoke detection is provided to allow for relatively early warning in the event of a fire.

Although we found discrepancies within the building with regards to the provisions of the 2006 IBC, we felt that it was necessary to investigate compliance with the Massachusetts State Building Code, as it is the acting jurisdiction. According to Massachusetts State Building Code Section 1014.3, the minimum stairwell width is 44 inches. The egress stairwell provided in the Gateway Park facility does not meet this requirement.

In addition, Stairwell A, shown in Figure 22, was not in compliance with either building code. The measured stairwell width and handrail heights do not fulfill the requirements of either code. Additionally, the stairwell width decreases in the direction of egress which is not permitted per the codes. However, this stairwell is not considered a means of egress in the facility. Although, the building owner has provided sufficient egress routes, we feel that there is still risk involved in maintaining a stairwell that does not meet building code requirements.

In conclusion, the egress system throughout the building notes some weaknesses but we feel the areas that were found to be non-compliant to the Code, have been offset by the fully installed automatic sprinkler system. In general, the egress system is sufficient for emergency exiting procedures.

The active fire protection systems installed in the masonry building were found to be in complete compliance with the IBC. As noted from the Fire Protection Specifications of the Gateway Park Project, the automatic sprinkler and standpipe systems for the masonry building were designed to comply with NFPA 13 and 14. The regulations of these NFPA standards are also cited throughout the IBC. Similarly, the alarm and detection systems comply with NFPA and IBC standards; thus, there are no components of the active fire protection systems that are deficient.

Whether or not the passive fire protection systems are compliant with the IBC is left up for interpretation. The element of passive fire protection in question is egress. Technically, "Stairwell A" of the building is not considered part of the egress system. One could argue that in a state of emergency, the occupants are likely to take the fastest way out. Depending on the location of the occupants at the time of the fire, Stairwell A could be their best escape route despite the fact that the stairwell should not be used as a means of egress.

It appears that compartmentation is not an issue with the aid of the building's exterior brick masonry bearing walls, gypsum walls and fire rated doors. During the renovation of the building the timber beams were encased in gypsum wallboard. This finish provided a fire barrier to the timber components. The only location where compartmentation is not likely is the open space in the building on the first and second floors which opens into a large atrium. Lack of compartmentation within the atrium would allow fire to spread to adjacent areas within the facility, likely resulting in a multi-story fire more quickly.

This project aimed to create a synergy between our undergraduate civil engineering capstone design project and our graduate level fire protection engineering courses. The structural alternatives that our group designed formed the basis for the project, and we tied in fire protection through building fire safety and active and passive fire protection systems analysis. In performing these analyses of the masonry building, we used the International Building Code, the standard for code knowledge used in our graduate classes. Thus, we were able to implement knowledge learned from our graduate classes into our undergraduate work.

9.5 Recommendations for Future Work

The Gateway Park Project is a great foundation for extensions for future projects. Civil Engineering is broad enough that engineers pursuing different areas of civil engineering could create a project pertaining to their field of study.

A main focus of this project is cost estimating. One of the important aspects of renovating the masonry building was the cost of gutting it while maintaining its structural stability. Additional precautions were taken to maintain stability while building the area for the lecture hall. Future studies could include performing detailed cost estimates of these tasks with their corresponding precautions.

As an extension of risk assessment, future work could be put towards finding the mathematical probabilities of the scenarios outlined by this project. Additionally, the laboratory space in the new part of the building creates a different set of scenarios. A risk assessment could be performed of the laboratory area of the newly constructed building.

A fire protection investigation could also be conducted with respect to the laboratory space in the newly constructed building. Issues such as installing appropriate equipment (i.e. wet vs. dry pipe suppression systems, heat or smoke detectors) or passive fire protection systems (i.e. compartmentation, fire rated doors) are important areas to investigate.

Nowadays during construction, it is not uncommon to encounter environmental issues while trying to complete a job. Gateway Park is built on a brownfield site. A brownfield site presents difficulty due to the presence or potential presence of a hazardous substance, pollutant, or contaminant. A future project could investigate the real world constraints of building on a brownfield site and also the related costs of doing so.

In conclusion, this project provides a strong basis for future study. Through our work other students will be able to further investigate the structural systems and fire protection of Gateway Park.

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Appendix A: Project Proposal

Introduction

The Gateway Project, located on Prescott Street in Worcester, MA, is a business venture undertaken by Worcester Polytechnic Institute (WPI) and the Worcester Business Development Corporation (WBDC). The project entails the cleanup of a brownfield site and renovation of an aged masonry building. Other aspects of the project include construction of new office space, laboratory facilities and a parking garage. The development of the 11.5-acre site is the beginning of the master plan for a mixed-use expansion to rejuvenate the 55-acre section of Worcester.

The scope of our work included researching historic masonry construction, performing a structural analysis of the existing masonry building, and designing structural alternatives. We concluded the structural alternatives portion of our project with a cost estimate to determine the feasibility of our suggested alternatives. Then, we proceeded to evaluating the fire protection systems and the building's egress system based on the 2006 International Building Code.

To understand the comprehensive nature of the Gateway Park Project, we conducted extensive research on the cleanup of this brownfield site. Some considerations that we found to be integral to this project were soil contamination from the past uses of the site and the effect of the contaminants on future uses of the site.

One objective of the Gateway Park renovation project was to structurally update the masonry building for office space and a lecture hall. This presented owners and builders with the challenge of bringing a late 1800's building into compliance with modern code criteria and redesigning the interior. Also, the building plans called for construction of a new brick building, to resemble the masonry building, as well as a connector building between the aforementioned structures. The demolition of a section of the wall of the old masonry building to provide a connection between buildings posed structural stability design concerns.

The masonry building currently contains the original timber structural framing system. The strength of timber is not as high as other construction materials, therefore requiring more columns which places constrains on the layout of the occupants office space. This report investigated the use of alternative structural systems within the existing building frame to provide a more open floor plan. Steel framing systems of varying bay sizes were developed, as well as, various concrete framing systems, including the use of one-way slabs, joist floor systems, and two way slabs. Considerations, such as depth of construction, were made to determine the feasibility of each alternative. We also developed cost estimates to determine which alternatives provided the most beneficial design while maintaining a reasonable cost.

Based on the building standards of life safety, property protection, and mission protection developed in the 2006 International Building Code, we performed a code analysis of the masonry building. We investigated the building's egress system, and analyzed the active and passive fire protection systems that were installed in the building. Subsequently, we developed methods to perform a risk assessment of the masonry building.

After completion of the various structural and fire protection alternatives and their associated cost estimates the best design scenario was determined.

Objectives

(1) Understanding historic masonry construction.

(2) Development of a structural analysis and design alternatives with a cost analysis.

(3) Developing a synergy with undergraduate education and FPE work.

Scope

The scope of work for this project will entail the following activities:

Discussion of what was necessary to bring building up to code

Inspections,

Code Analysis

Structural analysis of existing structure

1. Assess materials in building
    a. Timber

   b. Masonry
2. Assess constraints of floor plan
3. Gap between the old and new building

Concrete Alternatives

1. One way slab design
   a. Continuous T-Beam design
   b. Girder design
   c. Column design
2. Joist floor design
3. 2 way slab design
4. Compare:
   a. Flexibility within design (ie. Open floor plan with consideration of slab thickness-floor to ceiling height)
   b. Economics within design (which system will provide the best alternative but will not be too expensive)

Steel Alternatives

1. Bay Size 1
   a. Beam design
   b. Girder design
   c. Column design
   d. Combination metal decking and slab vs. basic slab
2. Bay Size 2
   a. Beam design 1
   b. Beam design 2
   c. Girder design
   d. Column design
   e. Combination metal decking and slab vs. basic slab
3. Compare:
   a. Flexibility

    b. Economics

Fire Protection

1. Risk Assessment
2. Analysis of what was put into the facility
    a. Active fire protection
        i. Fixed Automatic Fire Protection Systems
        ii. Manual Systems
        iii. Detection and Alarm Systems
        iv. Smoke and Heat Ventilation
        v. Water Supply and Reliability
    b. Passive fire protection
        i. Fire growth rate
        ii. Compartmentation
        iii. Emergency egress
    c. Egress System
3. Alternative system
4. Compare
    a. Occupant risk and safety
    b. Economics of what is in place vs. alternative

Understanding Masonry Construction

1. Historic Construction
    a. Building code
    b. Construction method
    c. Materials
2. Brownfield Development
    a. Regulations
    b. Problems associated with redevelopment
    c. Cleaning the area

Capstone Design

This MQP concerns the construction of the Gateway Park project off Route 290 in Worcester. This project addresses both new and old construction on a site with environmental challenges. This project addresses many real world constraints, which will be discussed in further detail.

Health and Safety

To ensure the safety of the buildings occupants, a risk assessment and fire protection analysis will be performed. Active and passive fire protection systems will be investigated and alternatives will be developed. The fire protection systems will be analyzed according to new structural designs.

Environmental

Gateway Park is a brownfields development and the project site was found to have contaminated soil from past industrial use. To develop this site a considerable amount of clean up was needed to remove the hazardous wastes and toxins found in the soil. The code requires a specific level of safety to be reach for different occupancies. In this case, the soil was thoroughly cleaned due to its intended office and research use.

Economic

From a structural standpoint, different alternatives will be developed for the interior design of the building. The alternative designs entail various concrete and steel frames. A cost estimate will be formed to compare and weigh the feasibility of each design alternative.

Social

The Master Plan for Gateway Park aims to revitalize the old industrial district in which it is located. The project will result in many important social implications for the City of Worcester and surrounding towns. By rebuilding the site with modern structures and facilities, the project will create new jobs, research opportunities and improved aesthetics in this rundown section of the city. Additionally, the area will bring a wider demographic to that part of the city.

Manufacturability

This project will present structural alternatives for concrete and steel within the existing exterior frame. The alternative designs are intended to provide a more open layout. Considerations will be made to avoid the creation of vertical constraints. Additionally, considerations will be made to accommodate the desired 15 feet of floor to ceiling space in the adjacent laboratory facility.

Schedule

A Term

Dates	Activity
Week 1	Organize MQP and meetings
Week 2	Background research
Week 3	Background research; Organization of project; Site visit
Week 4	Continued background & organization; Meeting with Steve Hebert (WPI V. President)
Week 5	Create schedule & list of activites for B Term; Meeting with Craig Blais (WBDC)
Week 6	Draft proposal
Week 7	Preliminary design calculations

B Term

Dates	Activity
Week 1	Structural analysis
Week 2	Continue structural analysis
Week 3	Work on draft of MQP report; Continue structural analysis
Week 4	Work on draft of MQP report; Continue structural analysis
Week 5	Develop alternatives using concrete and steel
Week 6	Work on draft of MQP report; Continue to develop alternatives
Week 7	Work on draft of MQP report; Continue to develop alternatives; Create schedule & activities for C Term

C Term

Dates	Activity
Week 1: Jan 15 – 21	Lateral Loading
Week 2: Jan 22 – 28	Egress code analysis; Passive & active systems analysis
Week 3: Jan 29 – Feb 4	Continue egress code analysis; Continue passive & active systems analysis
Week 4: Feb 5 – 11	Risk Assessment
Week 5: Feb 12 – 18	Continue Risk Assessment
Week 6: Feb 19 – 25	Tie up loose ends for completion of MQP Report
Week 7: Feb 26 – Mar 1	Prepare presentation for Project Presentation Day

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Craig Blais, Representative of Worcester Business Development Corporation (WBDC)

Steve Hebert, Vice President of WPI

Steve Johnson, Project Manager of Gateway Park for Consigli

Megan Lynch, Project Manager of Gateway Project for WBDC

Appendix B: Interview with Mr. Steve Hebert

Interview with Vice President, Steve Hebert
September 19, 2006
10:00 A.M.

Attendees:
V.P. Steve Hebert
Kaitlin McGillvray
Caitlin Ramig
Katie Strumolo
Kelly Thompson

1. How did this project develop?
    - David Forsburg (Worcester Building Development Corporation-WBDC) and Steve Hebert (WPI) thought the project was both interesting and possible for each party
    - WBDC
        a. Re-development
        b. Brownfield clean-up
        c. Taxed-based expansion
        d. Jobs
    - WPI
        a. Potential expansion of campus
        b. Research facility
        c. Investment for endowment
        d. Solution to "Dead Parking" issue
            1. Student that parks a car on Sunday and doesn't move it until Friday.
            2. Relieves parking hassle
    - WPI & WBDC
        a. Idea "meshed" conceptually
        b. November 1999, idea "meshed" between Steve Hebert and Dave Forsburg
        c. Had the support of WPI Board of Trustees & WBDC Board
        d. Gateway L.L.C. was created in December 1999
        - Four buildings were at Gateway purchased for $5.7 million
        - Approximately 11.5 acres
        - 50/50 partnership between WPI and WBDC
        - WPI is continuing to purchase land and buildings
        - Future purchases
            - Machine shop at Worcester Vocational School

2. Brownfields
   - Responsibility
     a. WBDC focuses on clean out

   - Old industrial buildings are not up to today's "performance level"
   - Asbestos etc.
   - Posed risks
     a. Hired risk firms to assess
     b. Required to clean out
     c. WPI & WBDC "accepted the risk"
   - Possible Solutions
     a. Clean and cap for parking
     b. Clean to build on
        i. EPA approved anything to be built except for a daycare
   - Garage
     a. Strategically placed
        i. Used to be an Electro-plating facility
     b. Garage property cost $5,000
     c. Clean-out cost $845,000
        i. Various elements dumped into Gateway Park in the 1930's and 1940's

3. Masonry Building
   - Built in late 1800's
   - Architect's decision to keep
   - Helps transition from the old area to the new building
   - Keeps traditional look
   - Architecturally and esthetically pleasing
   - Wooden timbers and floors
   - Substantially sound, structurally
   - Historical
     a. Consulted Massachusetts Historical Commission
     b. Required to outline how they were going to clean and what materials they intended on using
   - Removed columns to build 100-seat lecture hall

4. Layout
   - Old Masonry Building (68 Prescott St.)
     a. Office space
     b. Lecture Hall
   - New Building (60 Prescott St.)
     a. "Wet" Laboratory space
        1. Approximately 65,000 sq. ft.
   - Middle Section
     a. Lobby

    b. Elevators
    c. Conference rooms
- Parking
    a. 860 spaces
    b. Able to expand to 1300 spaces
- Available space
    a. Only 4,000 sq. ft. of the 125,000 sq. ft. is available

5. 65 vs. 11 acres
    - Gateway Park L.L.C is 11.5 acres
    - 65 acres incorporates Prescott St., Tuckerman Hall etc.

6. What Steve Hebert foresees
    - 45-60% of the development being life and health sciences
    - Hopes to attract corporations like Pfizer
    - High-end retailers/restaurants
        a. Sole Proprietor
        b. Legal Seafood
    - Building turnover
        a. April 1, 2007
    - Occupants move in
        a. May/June 2007
    - Grand Opening
        a. October 2007
    - Overall Gateway Project-Development approximately 2 years ahead of schedule
        a. Research facilities-60% Bio or life-science related (included Healthcare)
        b. Office space
        c. Housing-Graduate Students
        d. Retail
        e. Parking

7. Advantages for WPI
    - Frees up space in Goddard Laboratory and Salisbury
    - +$17 million of retro-fitting
    - Upgrading lab and office space
    - $10 million of the $17 million is committed

Appendix C: Interview with Mr. Craig Blais

Interview with WBDC Executive Vice President Craig Blais
September 26, 2006
10:00 A.M.

Attendees:
V.P. Steve Hebert
Craig Blais
Kaitlin McGillvray
Caitlin Ramig
Katie Strumolo
Kelly Thompson

How did the Worcester Business Development Council (WBDC) get involved in Brownfields?

- 1998 Massachusetts Brownfield Act
- EPA could keep up in evaluating sites
- Privatized whole process
- 1999= WBDC adopted Brownfields into strategic plan

Who evaluates the sites?

- LSP- License Site Professionals
  - Advise developers
  - Put together plans
- RAM- Risk Assessment Management
- RAO- Risk Assessment Outcome
- Audit on demand- developers want it, EPA doesn't

The Gateway district in the past was utilized in the steel plating operations, what are the main contaminants consistent with those operations?

- Arsenic
- Cyanide
- Led
- TCE

What did you find in the ground within the Gateway district?

- United States Steel in all of Gateway Park
- 60 Prescott Street
  - underground storage tanks, heating oil (not as contaminated)
- 75 Grove Street
  - underground storage of diesel fuel tank, leaking into groundwater
  - test monitoring wells, stopped source of what was removed

- 31 Garden Street
  - plateing company (where parking garage is located)
  - contaminated with arsenic, cyanide
- 68 Prescott Street
  - Asbestos removed in first phase
  - $118,000 to remove it
  - Cannot get demo permit until Asbestos removed
  - least contaminated, no storage tanks (were at 60 Prescott)

- Worcester has a high led concentration

What is done with the waste once you have cleaned it?

- Debris put in specifically designed landfill or incinerator
- The landfills are usually lined
- Vapor extractions – sit and dry out
- Soil management plan

What are the costs associated with the clean up?

- 1 acre cost between $5,000 and $750,000 to clean up, which included both phases
- Phase 1- historical analysis
- Phase 2- environmental insurance, limited subsurface testing

Appendix D: Field Notes from Site Visit

Field Notes obtained from site visit
September 27, 2006
10:00 A.M.

Attendees:
Project Manager Steve Johnson
Kaitlin McGillvray
Caitlin Ramig
Katie Strumolo
Kelly Thompson

Existing Masonry Building:

- Reappointing entire building
- 95% of the bricks in restored building the same
- Existing foundation remains untouched
- The wood stairs and approximately 90% of the wooden columns of old building will remain erect. However, new treads will be installed to "look nice".
- Lecture hall was constructed to hold approximately 100 people. Columns were removed for open space and ceiling beams were added for support.
- Fire proofing has been applied to exterior and around shafts of steel I-beams
- Concrete product was placed over wood floors.
- Dry walls around wood columns
- 95% of brick will be covered on interior
- Not as many contaminated soils, problems, etc. in this building compared to other former building that was demolished
- Fire safent (paste material) of 1- or 2-hour rating at most penetrations.

New Construction:

- An attachment of the old and new building referred to as the "link" is made of aluminum, glass, and a metal panel.

- There is an apparent gap between the old building and the newly constructed "link", approximately 1-ft, that was covered with an expansion joint.

- The floor to ceiling height of old building measured 13'-6". Typically, for laboratories, the height is 15'. However, to be consistent with the old building, they had to manage to design the lab space 1.5' shorter than desired.

- Fire safent spray along steel beams acting as installation for fire rating between buildings

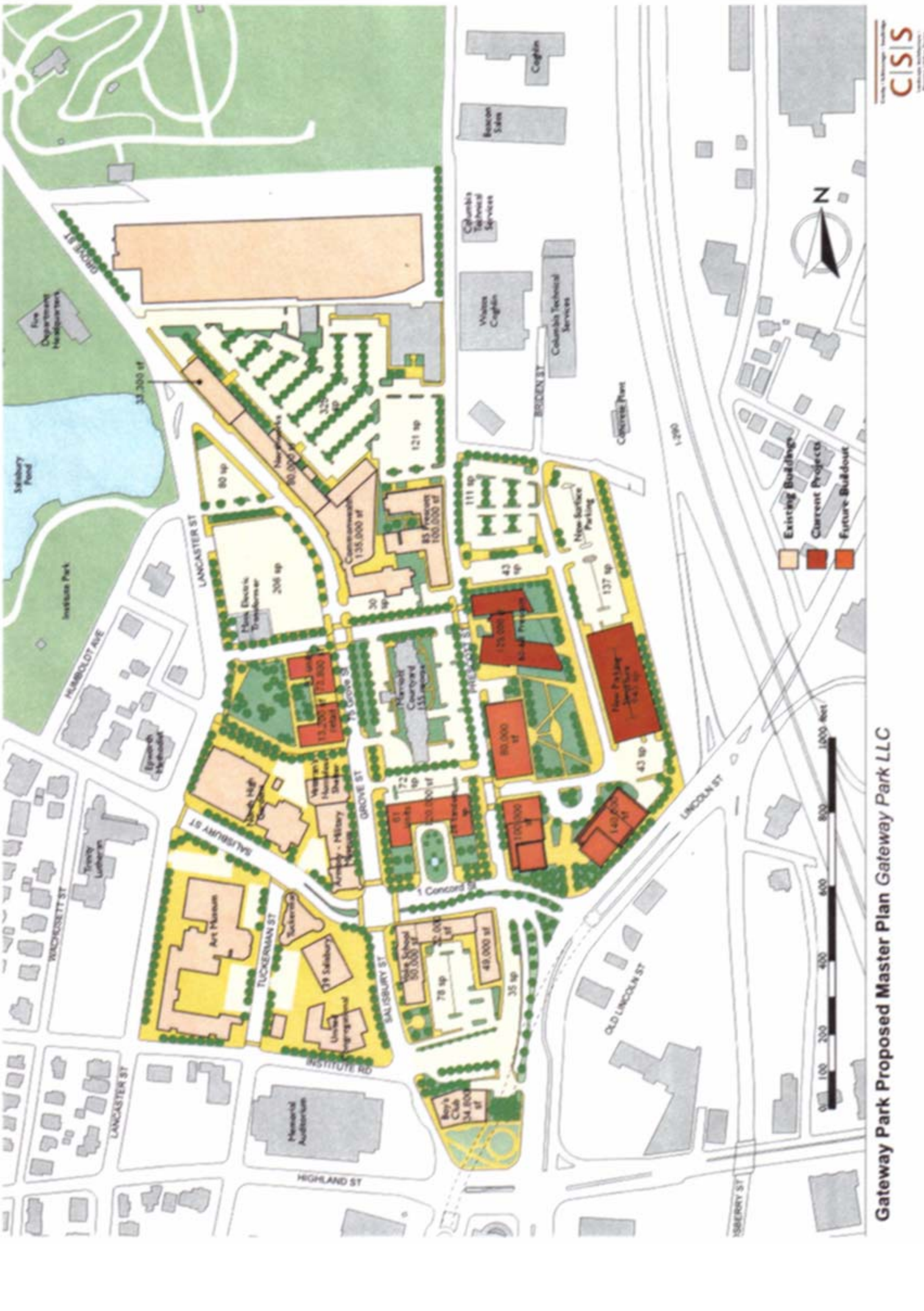
- 2-hr rating on walls of labs

- Type III B Construction

- Electrical room has 2 –hr rating on all walls except exterior

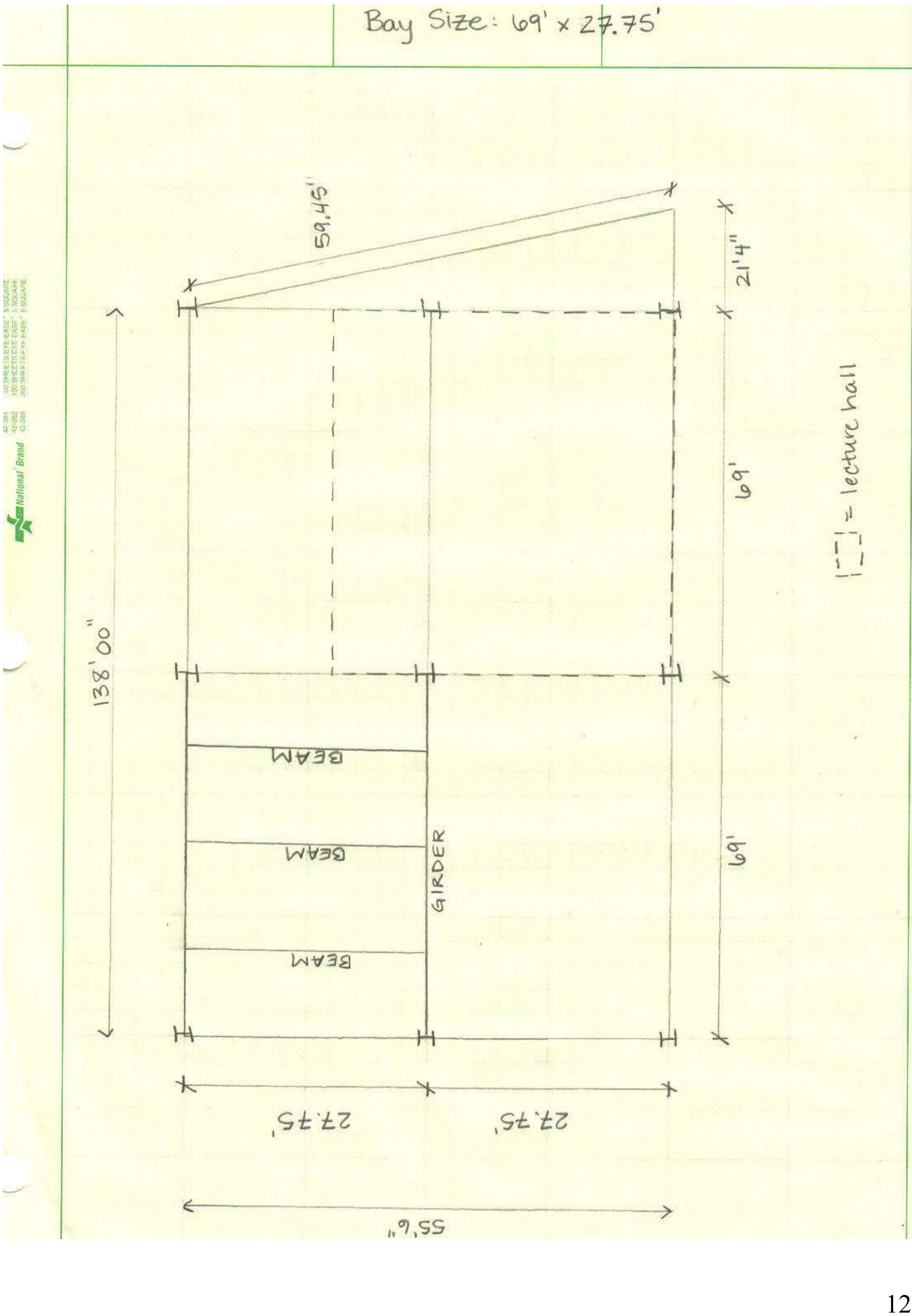
- Vapor barriers in new buildings but did not have apply in old buildings because of details in the brick layering it already had.

Appendix E: Master Plan of Gateway Park Project



Appendix F: Steel Design Hand Calculations

Bay Size [69'x 27.75'] and Floor Layout:



Beam Design Hand Calculations:

BEAM DESIGN

$\phi M_p \geq M_u$

$\phi Z F_y \geq M_u$

trib width = 23'
Length = 27.75'

required $Z \geq \dfrac{M_u}{(\phi = 0.9)(F_y = 50ksi)}$

First Floor

$w_u = 1.2D + 1.6L$

$= 1.2(83.4psf \times \frac{23'}{2}) + 1.6(100psf \times 23'/2) = 1151 + 719$

$= 2991^{lb}/_{ft} \approx 3\ ^{K}/_{ft}$

$M_u = \dfrac{w_u L^2}{8} = \dfrac{(3\ ^{K/ft})(27.75)^2}{8} = 289\ ft \cdot K$

$Z_x \geq \dfrac{(289\ ft \cdot k)}{(0.9)(50ksi)} \times (12 inches/1 foot)$

$Z_x \geq 77.0\ in^2$

Picked beam from (3- Design of Flex Members, p. 3-17?)

W 16×45 $\quad Z_x = 82.3 in^3$

W 12×53 $\quad Z_x = 77.9\ in^3$

Update w_u for W18×40 beam loading.

$w_u = 3000^{lb}/ft + 1.2(D = 45^{lb}/ft) = 3.1^{K}/ft$

$M_u = \dfrac{(3.1^K)(27.75')^2}{8} = 298\ ft \cdot K$

$Z_x \geq 79.6\ in^2 \qquad 79.6\ in^2 < 82.3\ in^3$

$\therefore$ W 16×45 OK ✓

Compact Section Criteria: W16×45 $\quad E = 29,000ksi,\ F_y = 50ksi \quad$ buckling?

Flange: $\dfrac{b_f}{2t_f} = 6.23 \quad \leq 0.38\sqrt{E/F_y} = 9.2 \quad$ no OK ✓

Web: $\dfrac{h}{t_w} = 41.1 \quad \leq 3.76\sqrt{E/F_y} = 90.5 \quad$ no OK ✓

Web & flange are compact $\therefore \phi M_n = \phi M_p$

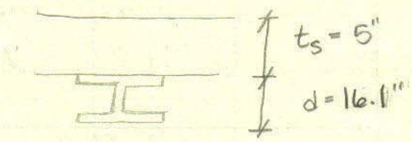
2/

Shear

$$V_u = \frac{W_u L}{8} = \frac{(3.1^k)(27.75')}{8} = 10.8^k$$

$$\phi V_n = \phi(0.6 F_y)\, d \cdot t_w = (0.9)(0.6)(50ksi)(16.1'')(0.345'')$$
$$= 150^k$$

$$150^K > 10.8^K \quad \therefore \text{ ok } ✓$$



$f'_c = 3.5$ ksi
span $L = 27.75'$
$S =$

$b_E \le S\,(12\text{ inches}/1\text{ foot}) = 23'(12) = 276''$
$b_E \le \frac{L}{4} = \frac{27.75'}{4}(12\text{ inches}/\text{foot}) = 83.25''$

] smallest value governs.

$$83.25'' < 276'' \quad \therefore \quad b_E = 83.25''$$

$$\frac{h}{t_w} \le 3.76\sqrt{E/F_y}$$

W16x45

$$41.1 \le 3.76\sqrt{29000/50} = 90.6$$

ok ✓

∴ check passes & we can use plastic capacity

For equilibrium of the section : C=T

$$0.85(f'_c)a \cdot b_E = A_s \cdot F_y$$

$$a = A_s \cdot F_y / (0.85)(f'_c)\, b_E = \frac{(13.3 in^2)(50 ksi)}{0.85(3.5 ksi)(83.25'')} = 2.69''$$

$$(a = 2.69'') \le (t_s = 5.0'') ✓$$

3/

Connections:

for buildings: stud $\phi = 3/4''$

(1) concrete crushing:

$$V_n = 0.85 (f'_c)\, t_s \cdot b_E$$
$$= 0.85 (3.5\text{ksi})(5'')(83.25'')$$
$$= 1238^K$$

(2) tension yielding of the steel section

$$V_n = A_s \cdot F_y = (13.3 \text{ in}^2)(50\text{ksi})$$
$$= 665^K$$

The limit state of tension yielding governs because the load capacity of the concrete crushing.

stud design:

"n" has to be sufficient to be sufficient to transfer $V_n = 665^K$

$$n \geq \frac{665^K}{Q_n} \qquad Q_n = \text{capacity of 1 shear stud.}$$

E_c - elastic modulus of concrete

$$E_c = w^{1.5}\sqrt{f'_c} = (145\text{pcf})^{1.5}\sqrt{3.5\text{ksi}}$$
$$= 3270 \text{ ksi}$$

A_{sc} - area of stud

$$A_{sc} = [\pi (3/4)^2]/4 = 0.4418 \text{ in}^2$$

$F_u = 60\text{ksi}$

$$Q_n = 0.5 (A_{sc})\sqrt{f'_c\, E_c}$$
$$= 0.5 (0.4418 \text{ in}^2)\sqrt{(3.5\text{ksi})(3270\text{ksi})}$$
$$= 23.6^K$$

$$Q_n \leq (A_{sc}) F_u = (0.4418 \text{ in}^2)(60\text{ksi}) = 26.5^K$$

4/

26.5^k = upper bound of stud strength

$23.6^k < 26.5^k$ ok✓

Number of Studs:

$$n \geq \frac{665^k}{Q_n} = \frac{665^k}{23.6^k} = 28.2$$

∴ use 29 studs.

Stud Spacing

minimum = $6d = 6(3/4") = 4.5"$

maximum = $8t_s = 8(5") = 40"$

$4.5" < \text{spacing} < 40"$

$$\text{spacing} = \frac{\left(\frac{27.75'}{2}\right) \times \left(\frac{12 \text{ inches}}{1 \text{ foot}}\right)}{29 \text{ studs}} = 5.74"$$

29 studs for each side = $29 \times 2 = 58$ studs

$$\left[W\,16 \times 45\ (58)\right]$$

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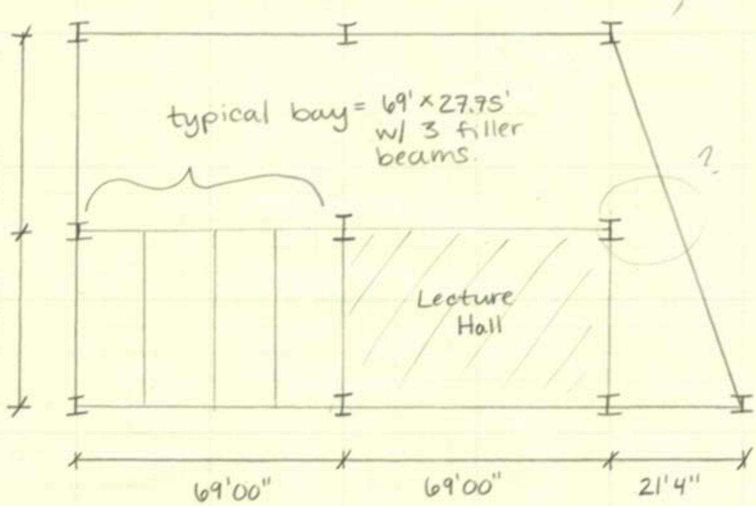
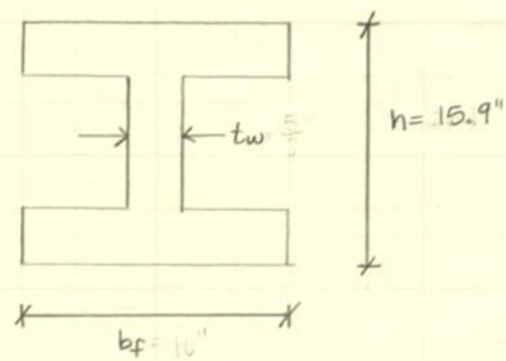
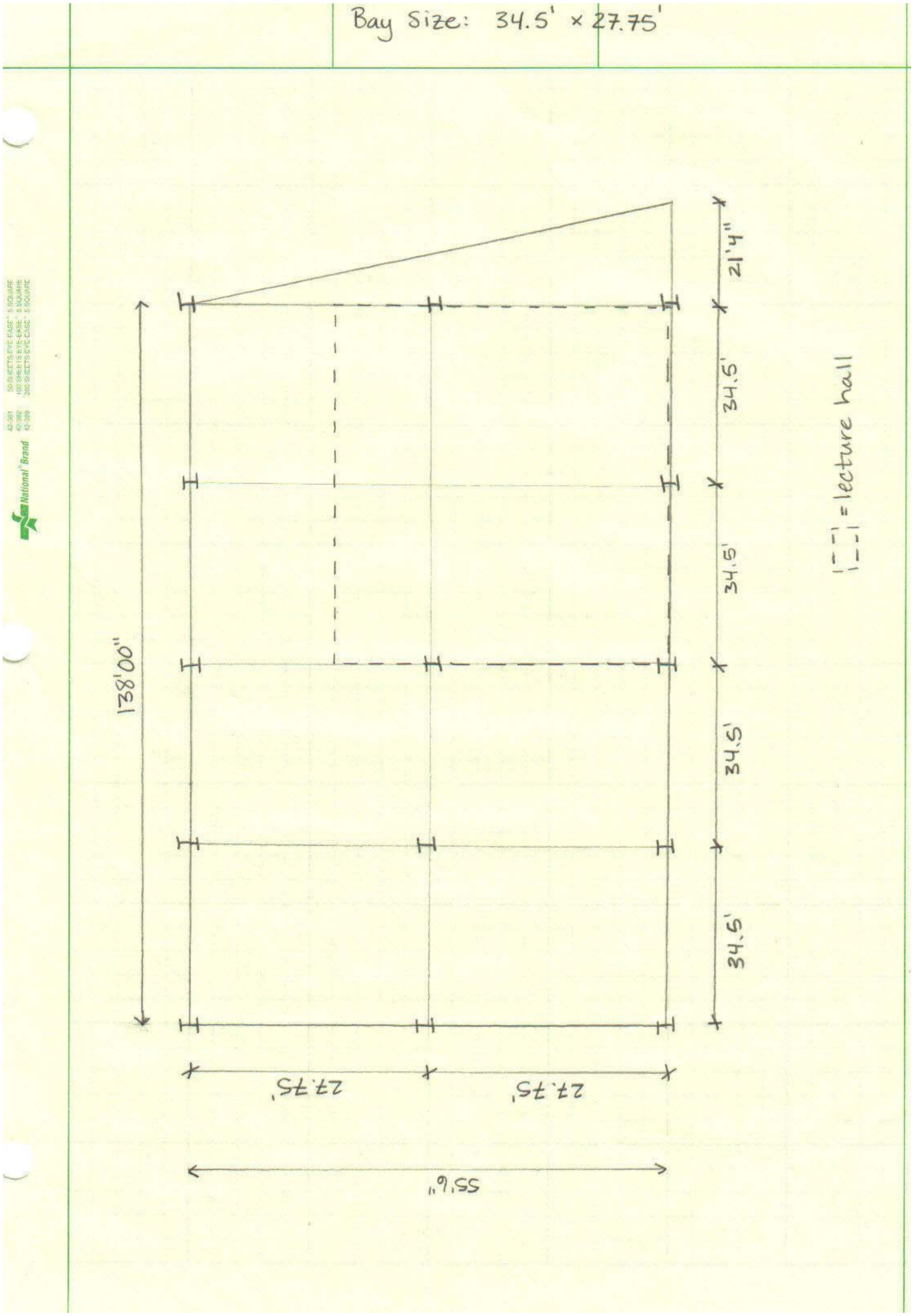
First Floor Plan



Beam: W16×31

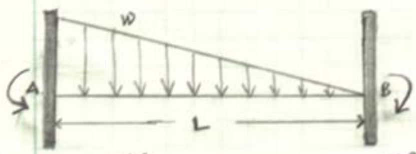


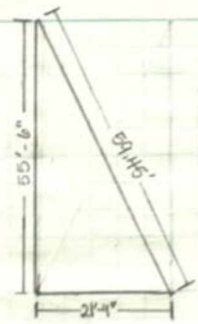
Bay Size [34.5’ x 27.75’] Layout:



Slant Beam Design Hand Calculations:

Fixed End Moments





$AB = \frac{wL^2}{20}$ $\quad BA = \frac{wL^2}{30} = 0$

- $w_u = (1.2D + 1.6L)(\text{Trib. width})$

$$= \frac{(1.2(83.4) + 1.6(100))\left(\frac{21.333'}{2}\right)}{1000}$$

$w_u = 2.77$ K/ft

- $M_u = \frac{w_u L^2}{20} = \frac{(2.77)(60')^2}{20}$ $\quad \underline{M_u = 498.6\ 'k}$

- Req'd $Z \geq \frac{M_u}{\phi_b F_y}$

$$Z \geq \frac{498.6}{(.9)(50)} \times 12''/' = 132.96 \text{ in}^3$$

Pick W24×68 $\quad Z_x = 160 \text{ in}^3$

Update w_u: $\quad DL = \frac{(1.2 \times 68)}{1000} = .066$ $\quad w_u = 2.77 + .0816 = 2.85$ K/ft

- $M_u = \frac{w_u L^2}{20} = \frac{(2.85)(60')^2}{20} = \underline{513.3\ 'k}$

- Req'd $Z > \frac{M_u}{\phi_b F_y} = \frac{513.3}{(.9)(50)} \times 12''/' = 136.88 \text{ in}^3$ ✓ $\quad$ 6.04 $\quad$ 43.6

Flange: $\frac{b_f}{2t_f} \leq .38\sqrt{\frac{E}{F_y}}$

$$6.04 \leq .38\sqrt{\frac{29{,}000}{50}} = 9.15$$

Web: $\frac{h}{t_w} \leq 3.76 \sqrt{\frac{E}{F_y}}$

$43.6 \leq 3.76 \sqrt{\frac{29{,}000}{50}} = 90.6$

Plastic Capacity: $\phi M_n = \phi Z_x F_y$

$\phi M_n = (.9)(160)(50)(1/12)$

$\phi M_n = 600$ ft·K

• $V_u = \frac{w_u L}{20} = \frac{(2.85)(60')}{20} = 8.55$ K

• $\phi V_n = \phi\,(.6 F_y) d t_w$ 23.7

$= .9\,(.6 \times 50)(23.7)(.415)$ .415

$\phi V_n = 265.6 > 8.55$ k ✓

• composite Design - connections

$b_E \leq \frac{L}{4} = \frac{60' \times 12''/'}{4} = 180''$

• $\frac{h}{t_w} = 43.6 \leq .376 \sqrt{\frac{E}{F_y}} = 90.6$

• EQUILIBRIUM

$a = \frac{A_s F_y}{.85 f'_c b_e} = \frac{(20.1)(50)}{.85(3.5)(180'')} = 1.88'' \leq 5.0''$

• For 3/4" ϕ stud

1. concrete crushing

$V_n = .85 f'_c (t_s)(b_E) = .85(3.5)(5'')(180'')$

$V_n = 2677.5$ k

2. Tension yielding of steel section

$V_n = A_s F_y = (20.1)(50\text{ksi})$

$V_n = 1005$ k

22-141 50 SHEETS
22-142 100 SHEETS
22-144 200 SHEETS
AMPAD

· STUD DESIGN

$$n \geq \frac{V_n}{Q_n}$$

$Q_n = .5 A_{sc} \sqrt{f'_c E_c}$ AND $Q_n \leq A_{sc} F_u$

$A_{sc} = \frac{\pi d^2}{4} = \frac{\pi (.75")^2}{4} = .4418 \text{ in}^2$

$F_u = 60$ ksi

$E_c = (w^{1.5}) \sqrt{f'_c} = (145)^{1.5} \sqrt{3.5} = 3267$ ksi

$Q_n = (.5)(.4418) \sqrt{(3.5)(3267)} = 23.6$ k AND $Q_n \leq (.4418)(60 \text{ ksi}) = 26.5$ k

$Q_n = 23.6$ k governs

$n = \frac{1005 \text{ k}}{23.6 \text{ k}} = 42.6 \rightarrow$ use 43 studs

· Minimum spacing = $6(d) = (6 \cdot .75) = 4.5"$
· Maximum spacing = $8(t_s) = (8 \times 5") = 40"$

$4.5" \leq \text{spacing} \leq 40"$

· spacing = $\frac{(\frac{60'}{2})(12"/')}{43} = 8.37$ "/stud √ O.K.

Appendix G: Concrete Hand Calculations

One way slab design hand calculations (Scenario 1)

1/6

Building Description

- 4 story office building w/ Lecture Hall
- Occupancy Category II (IBC Table 1604.05)
- Location: Worcester, Massachusetts
- Exterior masonry frame
- Interior concrete design
- Loads:
  - Live = 100 psf (Assembly w/ movable seats)
  - Wind = 90 mph (Fastest mile wind speed - IBC Fig 1609)
  - Snow = 50 psf (IBC 1608.02)

One Way Slab Design Calcs 2/6

Choose 5" slab for 2 hr fire rating

End bay:

$$\text{min } h = \frac{l}{24} \Rightarrow 5'' = \frac{l}{24}$$

$$l = 120'' = 10\text{ ft}$$

Interior bay: $\text{min } h = \frac{l}{28} \Rightarrow 5'' = \frac{l}{28}$

$$l = 140'' = 11.67\text{ ft.}$$

Therefore, use 5 in. slab assuming 3/4 in. clear cover and No. 4 bars

$$d = 5\text{ in} - \left(0.75 + \frac{0.5}{2}\right) = 4\text{ in.}$$

Compute trial unfactored load

$$W_D = \frac{5\text{ in}}{12\text{ in/ft}} \times 150\text{ pcf} = 62.5\text{ lb/ft}^2 \text{ of floor surface}$$

Assume additional dead loads as follows:

Floor cover: 0.5 psf
Mechanical equipment: 4 psf
Ceiling: 2 psf

$DL_{total} = 69$ psf

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Determining Load combinations
ACI Section 9.2.1

Eqn 1: $U = 1.4(D + F)$
$U = 1.4(69\ psf) = 96.6\ psf$

Eqn 2: $U = 1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$
$U = 1.2(69\ psf) + 1.6(100\ psf) + 0.5(50\ psf)$
$U = 267.8\ psf \longleftarrow$ Max Load

Eqn 3: $U = 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (1.0\ L \text{ or } 0.8\ W)$
$U = 1.2(69 psf) + 1.6(50\ psf) + 1.0(100\ psf)$
$U = 262.8\ psf$

Eqn 4: $U = 1.2D + 1.3W^* + 0.1L + 0.5(L_r \text{ or } S \text{ or } R)$
$U = 1.2(69\ psf) + 1.3(90 psf) + 0.1(100\ psf) + 0.5(50\ psf)$
?

*ACI Section 9.2.1 (b.) Where wind load W has not been reduced by a directionality factor, it shall be permitted to use 1.3 W in place of 1.6 W in equation (9-4) and (9-6)

$U = 234.8\ psf$

Eqn 5: $U = 1.2D + 1.0E + 1.0L + 0.2S$
$U = 1.2(69\ psf) + 1.0(25\ psf) + 1.0(100\ psf) + 0.2(50 psf)$
?
$U = 217.8\ psf$

Eqn 6: $U = 0.9D + 1.3W + 1.6H$
$U = 0.9(69) + 1.3(90)$
$U = 179.1\ psf$

Eqn 7: $U = 0.9D + 1.0E + 1.6H$
$U = 0.9(69\ psf) + 1.0(25\ psf)$
$U = 87.1\ psf$

Select strength-reduction factors

Assume slab is tension controlled

Ultimate tensile strain ≥ 0.005

$\therefore$ use $\phi = 0.90$

Check whether slab thickness is adequate for the moment

$3W_D = 3(62.5 \text{ psf}) = 187.5 \text{ psf} > W_L = 100 \text{ psf}$

$f'c = 3750 \text{ psi}, \; f_y = 60{,}000 \text{ psi}, \; \rho \leq 0.01$

$$\omega = \frac{\rho f_y}{f'c} = \frac{0.01 \times 60{,}000}{3{,}750} = 0.16$$

$$\phi K_n = \phi \left[f'c\, \omega (1 - 0.59\omega) \right] = 0.9 \left[3750 (0.16) (1 - 0.59(0.16)) \right]$$

$\phi K_n = 489$

Exterior face of first interior support:

$M_u = \frac{W_u l_n^2}{10}$, where ln for computation of negative moment at interior supports is the average of the clear spans of adjacent spans

$$l_n = \frac{(105'' + 126'')}{2} = 115.5'' / 12 \text{ in./ft} = 9.625 \text{ ft}$$

$$M_u = \frac{(267.8 \text{ lb/ft})(9.625)^2}{10} = 2480.92 \text{ ft-lb/ft. of width}$$
$= 2.48$ ft-kips/ft.

Second interior support:

$$M_u = \frac{W_u l_n^2}{11} = \frac{(267.8 \text{ lb/ft})(10.5)^2}{11} = 2684 \text{ ft-lb/ft of width}$$
$= 2.68$ ft-kips/ft

$\therefore$ Maximum $M_u = 2.68$ ft-kips/ft

$$bd^2 = \frac{Mu \times 12{,}000}{\phi Kn} = \frac{2.68 \times 12{,}000}{489} = 65.77 \text{ in.}^3$$

By designing 1 ft wide strip, $b = 12$ in.

$\therefore \; d = \sqrt{\frac{65.77}{12}} = 2.34$ in.

A minimum $d = 2.34$ in. keeps $\rho \le 0.01$. Because $d = 4$ in. from previous calculation (see pg. 2) exceeding the minimum 2.34 in. the slab is adequate for flexure.

Check to ensure slab thickness is adequate for shear.

Exterior face of first interior support:

$$Vu = \frac{1.15\, Wu\, \ell n}{2} = \frac{1.15\,(267.8)(105/12)}{2} = 1347 \text{ lb/ft of width}$$

Typical interior support:

$$Vu = \frac{(267.8 \text{ lb/ft}) \times (126/12) \text{ ft}}{2} = 1406 \text{ lb/ft of width}$$

$$\phi V_c = 0.75(2\sqrt{f'c}\; b_w d) = 0.75\left[2\sqrt{3750}\;(12)(4)\right] = 4409 \text{ lb/ft}$$

$\phi Vc = 4409$ lb/ft $>$ $Vu = 1406$ lb/ft

$\therefore$ Slab chosen is adequate for shear

Design of Reinforcement

End bay: $\ell_n = 105"$

Interior bay: $\ell_n = 126"$

For interior supports, ℓn equals average of adjacent spans

$\Longrightarrow \; \ell n = 115.5"$

$$A_s = \frac{Mu \times 12000}{\phi f y\, jd} = \frac{2.68 \times 12{,}000}{0.9\,(60{,}000)(0.925 \times 4 \text{ in.})} = 0.161 \text{ in}^2/\text{ft}$$

Assume $\phi = 0.90$ for tension controlled section and $jd = 0.925d$ for a slab

6/6

$$a = \frac{0.161 \times 60{,}000}{0.85 \times 3750 \times 12} = 0.253$$

$$jd = d - \frac{a}{2} = \left(4 - \frac{0.253}{2}\right) = 3.873 \Rightarrow j\!d = \frac{3.873}{4} = 0.968$$

compared to assumed value of .925

⟹ Recalculate A_s

$$A_s = \frac{M_u \times 12000}{\phi f_y jd} = \frac{2.68 \times 12000}{0.9\,(60{,}000)(3.873)} = 0.154 \ in^2/ft$$

$$A_{s(min)} = 0.0018\,bh = 0.0018\,(12\ in.)\,(5\ in.) = 0.108\ in^2/ft$$

According to ACI Section 7.6.4, max. spacing = 3h = 15 in.

Check reinforcement spacing for crack control

$$s = \frac{540}{f_s} - 2.5C_c \text{ but not more than } 12\left(\frac{36}{f_s}\right) \text{ in.} \quad \text{(ACI Eqn 10-5)}$$

Where f_s is stress in tension steel in ksi which can be taken as $0.6f_y = 36$ ksi and C_c is clear cover from the tension face of slab to surface of reinforcement = 0.75 in.

$$s = \frac{540}{0.6 \times 60} - 2.5(0.75) = 13.1 \text{ but not more than } 12\left(\frac{36}{36}\right) \text{in} = 12 \text{ in.}$$

12 in. < 15 in. ⟹ smaller (12 in.) overrides

Select top and bottom flexural steel

- See table on the following page

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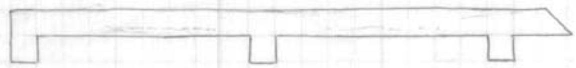


Line						
1.) l_n [ft]	8.75	8.75	9.625		10.5	10.5
2.) $W_u l_n^2$	20.5	20.5	24.81		29.52	29.52
3.) M coeff.	1/24	1/14	1/10	1/11	1/16	1/11
4.) M_u [ft-kip/ft]	0.854	1.46	2.48	2.26	1.85	2.68
5.) $A_{s(req)}$ [in²/ft]	~~0.049~~	~~0.084~~	0.142	0.130	~~0.106~~	0.154
6.) $A_{s(min)}$ [in²/ft]	0.108	0.108	~~0.108~~	~~0.108~~	0.108	~~0.108~~
7.) Choose Steel	No. 4 @ 12"	No. 4 @ 12"	No. 4 @ 12"	No. 4 @ 12"	No. 4 @ 12"	No. 4 @ 12"
8.) $A_{s(provided)}$	0.20	0.20	0.20	0.20	0.20	0.20

- Determine the shrinkage and temperature reinforcement

$A_s = 0.0018bh = 0.108$ in²/ft

Max. spacing = 5h if < 18 in. for shrinkage and temperature reinforcement

$5h = 5(5 \text{ in.}) = 25 \text{ in.} > 18 \text{ in.}$

∴ Max. spacing = 18 in.

Therefore, provide No. 4 bars at 15 in. o.c.

Rectangular Beam Design
Scenario 1

Design of Rectangular Beam 1/3

Scenario 1: Slab = 5"

1.) Estimate dead load of beam
Weight = 0.3 - 0.6 kip/ft

h = 8-10% of span ⇒ interior = 11'
exterior = 10'
span = 21'

$h = 1.68$ to 2.1 ft

$b = 0.5h = 0.84$ to 1.05

Assume: uniform service live load = 1.75 kip/ft
uniform superimposed service dead load = 1.00 kip/ft

$W_u = 1.2(1.0 + 0.5) + 1.6(1.75) = 4.6$ kip/ft

2.) Compute the factored moment

$$M_u = \frac{w_u l_n^2}{8} = \frac{4.6\text{ kip/ft }(9.625\text{ ft})^2}{8} = 53.3\text{ ft-kips}$$

3.) Compute b and d

$$\frac{M_u}{\phi k_n} = \frac{bd^2}{12{,}000}$$

$$k_n = f'_c\,\omega(1 - 0.59\omega)$$

$$\omega = \frac{\rho f_y}{f'_c}$$

Assume $\rho = 0.01 \Rightarrow \omega = \frac{0.01\,(60{,}000)}{3750} = 0.16$

$k_n = 543$

$\phi = 0.90$

$$\frac{bd^2}{12{,}000} = \frac{M_u}{\phi k_n}$$

$$\frac{bd^2}{12{,}000} = \frac{53.3}{0.9(543)}$$

$bd^2 = 1308.8 \text{ in}^3$

Try $1.5 \leq \frac{d}{b} \leq 2.0$

$\Rightarrow b = 12 \text{ in.}, \; h = 24 \text{ in.}$

Eqn 4-32d: $d \simeq h - 1.1 \text{ in} \Rightarrow d = 22.9 \text{ in.}$

Try rectangular cross section w/ $b = 12''$, $d = 22.9''$, $h = 24''$

4.) Check dead load and revise M_u

$$DL = (1.33 \times 3 \times 1)\ ft^3/ft \times (0.15 \ kip/ft^3) = 0.60 \ kip/ft$$

$\therefore$ Total load/ft $= 1.2(1.0 + 0.6) + 1.6(1.75) = 4.72$ kips/ft

$$M_u = \frac{4.72 \text{ kips/ft } (9.625)^2}{8} = 54.7 \text{ ft-kips}$$

5.) Compute Area of reinforcement

$jd = 0.875d = 20$ in.

$$A_s = \frac{M_u}{\phi f_y jd} = \frac{54.7 \text{ ft-kips} \times 12 \text{ in/ft}}{0.9\,(60\text{ksi})\,(20 \text{ in.})} = 0.61 \text{ in}^2$$

6.) Compute minimum reinforcement

$$A_{s,min} = \frac{3\sqrt{f'_c}}{f_y} b_w d = \frac{3\sqrt{3750}}{60{,}000}(12)(23) = 0.85 \text{ in}^2$$

but not less than $A_{s,min} = \frac{200\, b_w d}{f_y} = \frac{200(12)(23)}{60{,}000} = 0.92 \text{ in}^2$

7.) Select steel

$A_{s,min} > A_{s,req}$

$\therefore A_{s,min} = 0.92 \text{ in}^2$ governs

Try three No. 5 bars where $A_s = 0.93 \text{ in}^2$

$a = 1.75$, $c = 2.06$

8.) $\epsilon_t = 0.03 > 0.005$, the section is tension-controlled, $f_s = f_y$ and $\phi = 0.90$

9.) Compute M_n and ϕM_n

$M_n = A_s f_y (d - a/2) = 0.93 \times 60{,}000 \left(22.9 - 1.75/2\right)/12{,}000 = 102$

$\phi M_n = 91.8$ ft-kips $> M_u = 54.7$ ft-kips

$\therefore$ Design is ok

Use $b = 12$ in., $h = 24$ in, $d = 22.9$ in.

w/ $f'c = 3750$ psi and $f_y = 60{,}000$ psi and three No. 5 bars in one layer

Because second interior $l_n = 10.5$ ft, $M_u = \dfrac{4.72 \text{ kips/ft } (10.5)^2}{8}$

$M_u = 65$ ft-kips

$\therefore \phi M_n > M_u$ still so this beam design can be used throughout.

Scenario 2

Design of Rectangular Beams | 1/4

Design of a Beam when b and d are not known
(MacGregor, 4th Edition → Example 4-7)

Design Scenario 2

1.) Estimate dead load of beam

Weight of rectangular beam = 10-20% of load it must carry (ranging from 0.3-0.6 Kip/ft

h = 8-10% of span ⇒ 2.4 to 3.0 ft
b = 0.5h ⇒ 1.2 to 1.5 ft

Assume:

Uniform service live load = 1.75 Kip/ft

Uniform superimposed service dead load = 1.00 Kip/ft

Estimate beam weight at 0.5 kip/ft

$W_u = 1.2(1.0 + 0.5) + 1.6(1.75) = 4.6$ Kip/ft

2.) Compute the factored moment

$$M_u = \frac{W_u l_n^2}{8} = \frac{4.6 \text{ Kip/ft } (13.79 \text{ ft})^2}{8} = 109 \text{ ft-Kips}$$

3.) Compute b and d

$$\frac{M_u}{\phi k_n} = \frac{bd^2}{12{,}000}$$

$$k_n = f'c\,\omega(1 - 0.59\omega)$$

$$\omega = \frac{\rho f_y}{f'c}$$

Assume $\rho = 0.01 \Rightarrow \omega = \frac{0.01(60{,}000)}{3750} = 0.16$

$k_n = 3{,}750(0.16)[1 - 0.59(0.16)] = 543$

$\phi = 0.90$ for a beam

$$\frac{bd^2}{12{,}000} = \frac{M_u}{\emptyset K_n}$$

$$\frac{bd^2}{12{,}000} = \frac{109}{0.9(543)}$$

$bd^2 = 2676.5 \text{ in}^3$

Try $1.5 \le \frac{d}{b} \le 2.0$

$\Rightarrow b = 18 \text{ in}, \; h = 36 \text{ in}$

(Eqn 4-32d)

For one-way slab spans over 12 ft, $d \simeq h - 1.1 \text{ in} \Rightarrow d = 36 - 1.1 = 34.9''$

Try rectangular cross section with $b = 18$, $h = 36''$, and $d = 34.9$ in.

4.) Check dead load and revise M_u

For $b = 18$ in and $h = 36$ in., the self-weight per foot

$$= (1.33 \times 3 \times 1) \text{ ft}^3/\text{ft} \times 0.15 \text{ kip/ft}^3 = 0.60 \text{ kip/ft}$$

$\rightarrow$ Total load/ft $= 1.2(1.0 + 0.6) + 1.6(1.75) = 4.72$ Kips/ft

$$\therefore M_u = \frac{4.72 \text{ Kips/ft } (13.79)^2}{8} = 112 \text{ ft-kips}$$

5.) Compute the area of reinforcement A_s

Assume that $jd = \left(d - \frac{a}{2}\right) = 0.875d = 30.5$ in.

$$A_s = \frac{M_u}{\emptyset f_y jd} = \frac{112 \text{ ft-Kips} \times 12 \text{ in/ft}}{0.9\,(60 \text{ ksi})(30.5 \text{ in.})} = 0.82 \text{ in.}^2$$

6.) Minimum Reinforcement

$$A_{s,min} = \frac{3\sqrt{f'c}}{fy} b_w d = \frac{3\sqrt{3750}}{60{,}000}(18)(35) = 1.93 \text{ in}^2$$

but not less than $A_{s,min} = \frac{200 b_w d}{fy} = \frac{200(18)(35)}{60{,}000} = 2.1$

7.) Select Steel

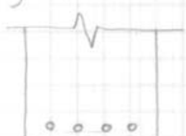
$A_{s,min} > A_{s,req.}$

$\therefore A_{s,min} = 2.1 \text{ in}^2$ governs

Possible reinforcement choices:

5 No. 6 bars $\rightarrow A_s = 2.20 \text{ in}^2$
4 No. 7 bars $\rightarrow A_s = 2.40 \text{ in}^2$
3 No. 8 bars $\rightarrow A_s = 2.37 \text{ in}^2$
2 No. 9 bars $\rightarrow A_s = 2.00 \text{ in}^2$

Try 4 No. 7 bars where $A_s = 2.40 \text{ in}^2$



8.) Compute ϵ_t and check whether $f_s = f_y$ and whether the section is tension controlled

$$a = \frac{A_s f_y}{0.85 f'c} = \frac{2.40 \text{ in}^2 (60 \text{ ksi})}{0.85 (37.5 \text{ ksi})} = 4.52$$

$$c = a/\beta_1 = 4.52/0.85 = 5.32$$

From the strain distribution and similar triangles

$$\epsilon_t = 0.003\left(\frac{d - c}{c}\right) = 0.003\left(\frac{34.9 - 5.32}{5.32}\right) = 0.017$$

4/4

Because $\epsilon_t = 0.017$ exceeds 0.005, the section is tension-controlled, $f_s = f_y$, and $\phi = 0.90$

9.) Compute M_n and ϕM_n

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

$$M_n = 2.40 \times 60,000 \left(34.9 - \frac{4.52}{2}\right) / 12,000$$

$$M_n = 392 \text{ ft-kips}$$

$$\phi M_n = 0.9(392) = 353 \text{ ft-kips}$$

Since $\phi M_n = 353$ ft-kips $> M_u = 112$ ft-kips, the design is OK.

∴ Use $b = 18$ in., $h = 36$ in., with $f'_c = 3750$ psi, $f_y = 60,000$ psi and four No. 7 bars in one layer.

Because second interior $l_n = 13.83$ ft, $\phi M_n > M_u$ still so this beam design can be used throughout.

Scenario 3

Design of Rectangular Beam | 1/3

Scenario 3

- slab = 10"
- interior span = 15'
- exterior span = 10'

} span = 25'

h = 8 - 10% of span ⇒ h = 2.0 to 2.5 ft

b = 0.5 h ⇒ 1.0 to 1.25 ft.

Assume uniform service live load = 1.75 kip/ft
& uniform superimposed service dead load = 1.00 kip/ft

W_u = 4.6 kip/ft

2.) Compute factored Moment

$$M_u = \frac{4.6 \text{ kip/ft } (11.33 \text{ ft})^2}{8} = 73.8 \text{ ft-kips}$$

3.) Compute b and d

Assume $\rho = 0.01$

$\therefore$ $\omega = 0.16$

$k_n = 543$

$\phi = 0.90$

$$\frac{bd^2}{12{,}000} = \frac{73.8}{0.9(543)}$$

$bd^2 = 1812 \text{ in}^3$

Try $1.5 \leq \frac{d}{b} \leq 2.0$

⇒ b = 15 in, h = 30 in.

$d \simeq h - 1.1 = 30 - 1.1 = 28.9"$

Try rectangular cross section w/ b = 15", d = 28.9", h = 30"

4.) Check dead load and revise Mu

$DL = 0.6$ kip/ft

$\therefore$ Total load = 4.72 kip/ft

$$M_u = \frac{4.72 \text{ kips/ft } (11.33 \text{ ft})^2}{8} = 75.7 \text{ ft-kips}$$

5.) Compute Area of reinforcement

$jd = 0.875d = 25.3$ in.

$$A_s = \frac{M_u}{\phi f_y jd} = \frac{75.7 \text{ ft-kips} \times 12 \text{ in/ft}}{0.9 (60\text{ksi}) (25.3 \text{ in})} = 0.665 \text{in}^2$$

6.) Compute minimum reinforcement

$$A_{s,min} = \frac{3\sqrt{f'c}}{f_y} b_w d = \frac{3\sqrt{3750}}{60,000} (15")(29") = 1.33 \text{ in}^2$$

but not less than $A_{s,min} = \dfrac{200\, b_w d}{f_y} = \dfrac{200 (15")(29")}{60,000} = 1.45 \text{ in}^2$

7.) Select steel

$A_{s,min} > A_{s,req}$

$\therefore A_{s,min} = 1.45$ in² governs

Try four No. 6 bars where $A_s = 1.76$ in²

8.) Compute ϵ_t and check whether $f_s = f_y$

$$a = \frac{A_s f_y}{0.85 f'c} = \frac{1.76 (60)}{0.85 (37.5)} = 3.31 \text{ in.}$$

$$c = a/\beta_1 = 3.31/0.85 = 3.9 \text{ in.}$$

$$\epsilon_t = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{28.9" - 3.9"}{3.9"}\right) = 0.019$$

3/3

Since $\epsilon_t = 0.019 > 0.005$, the section is tension controlled, $f_s = f_y$, and $\phi = 0.90$

9.) compute M_n and ϕM_n

$$M_n = A_s f_y (d - a/2)$$
$$= 1.76 \times 60{,}000 \,(28.9" - 3.31/2) / 12{,}000$$
$$= 240 \text{ ft-kips}$$

$$\phi M_n = 0.9(240) = 216 \text{ ft-kips}$$

Because $\phi M_n = 216$ ft-kips $> M_u = 75.7$ ft-kips, design is ok

Use $b = 15$ in, $h = 30$ in, $d = 28.9$ in. with $f'c = 3750$ psi and $f_y = 60{,}000$ psi and four No. 6 bars in one layer

Second interior: $\ell_n = 13.83$ ft

$$M_u = \frac{4.72 \text{ kips/ft } (13.83)^2}{8} = 113 \text{ ft-kips}$$

$\therefore$ $\phi M_n > M_u$ still so this beam design can be used throughout

Girder Design

Design of a Continuous Girder | 1/3

Designing a three-span continuous girder (G1-G2-G1)

Girders carry 2/3 of full column bay area

$R = 0.08\,(A - 150)$

$2/3\,A = 2/3(825) = 550$

$R = 0.08\,(550 - 150) = 32\%$

Design Live Load = 100 psf

Concentrated loads:

Live Load: $100\ psf \times 15ft \times 27.5\ ft = 41{,}250\ lbs$

Dead Load: $69\ psf \times 15ft \times 27.5\ ft = 28{,}463\ lbs$

Beam Weight: $18'' \times 36'' \times \frac{150}{144} \times 27.5\ ft = 18{,}563\ lbs$

Total = 88,276 lbs

or 88 kips

Assume girder section is 18 in. wide and 30 in. in overall height, the unit distributed load is

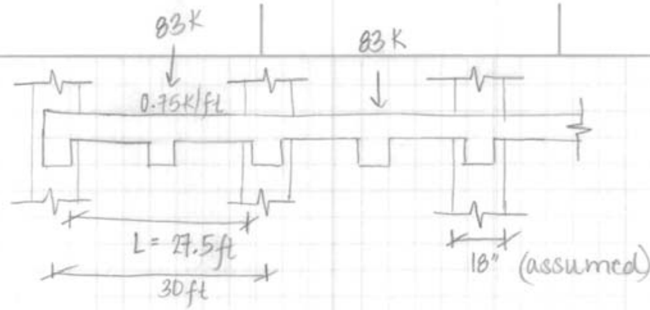
Girder weight: $18'' \times 30'' \times \frac{150}{144} = 563\ lb/ft$

Superimposed Load: $124\ psf \times 1.5\ ft = 186\ lb/ft$

Total = 749 lb/ft

or 0.75 kips/ft

2/3



Total $W = 0.75\,k/ft\,(27.5) + 83\,k$
$= 104\,k$

Moment factors					
C =	1/16	1/11	1/10	1/11	1/16

Moment k-ft				
CWL = 185	269	296	185	

jd =

As,req [in²]

Top

Bottom

Design Moment = 2/3 (296) = 197 k-ft

Required $bd^2 = \frac{M}{R} = \frac{197 \times 12}{0.32} = 7387.5\ in.^3$

If $b = 18\ in.$, $d = \sqrt{\frac{7387.5}{18}} = 20.3\ in.$

∴ The assumed 18 in. by 30 in. section is adequate for flexure.

Check shear

Consider critical condition at the interior ends of the exterior spans

$$V_{max} = 1.15 \times \left(\frac{83}{2}\right) = 47.73K$$

Use max shear value = 47 K for design

Shear stress for stirrup spacing:

$$v = \frac{V}{bd} = \frac{47{,}000}{18 \times 20.3} = 129 \text{ psi}$$

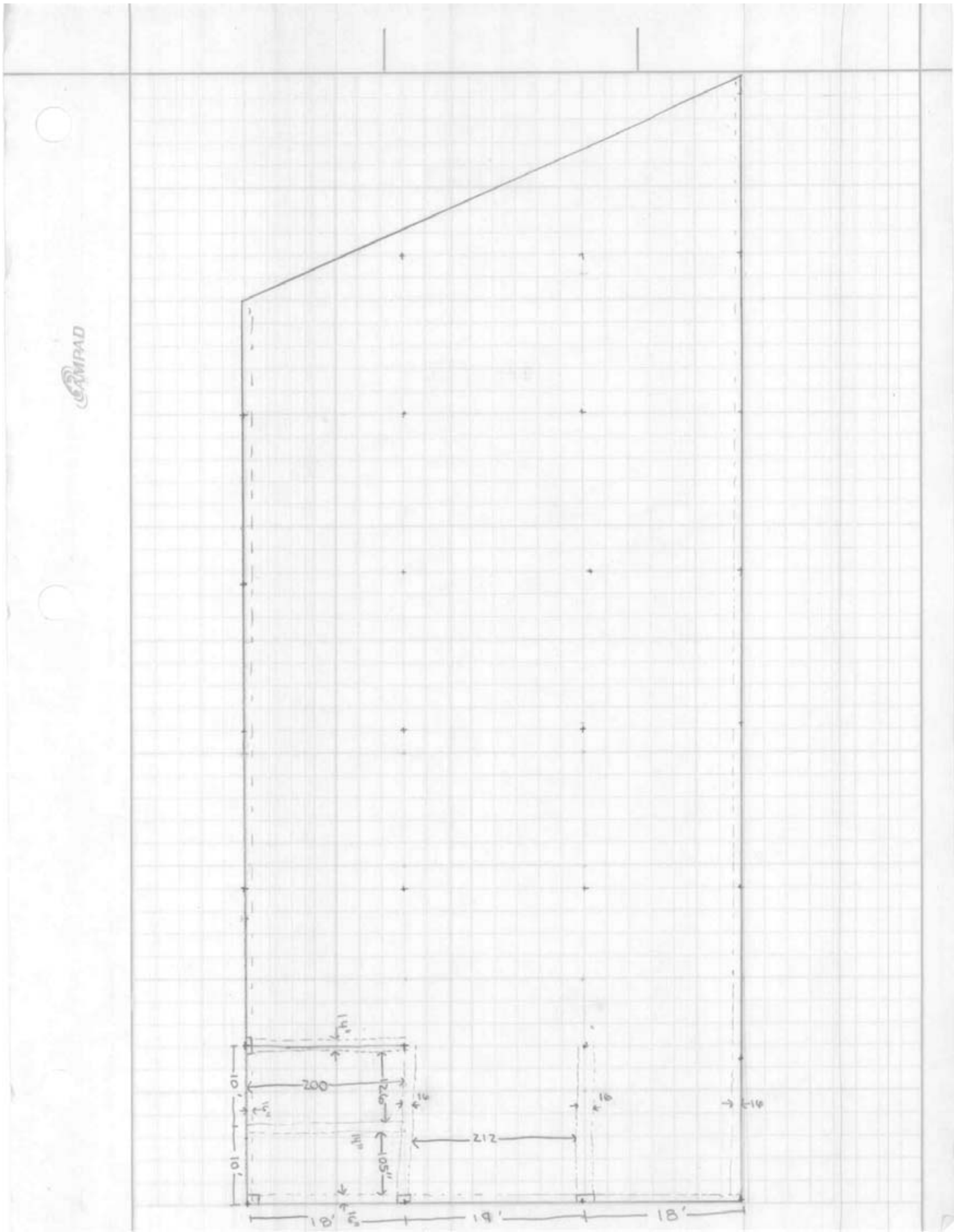
$$\sigma_{allow} = 1.1\sqrt{f'c} = 1.1\sqrt{3750} = 67.4 \text{ psi}$$

$\Rightarrow$ 129 - 67 = 62 psi (stress to be taken by the stirrups)

Try No. 4 U-stirrup

$$\text{spacing}, s = \frac{A_v f_v}{v' b} = \frac{2(0.20) \times 20{,}000}{62 \times 18} = 7.17 \text{ in.}$$

Scenario 4:



1/

Design of a Continuous T Beam

B3-B4-B3

$f'c = 3750$ psi
$fy = 60{,}000$ psi
$= 40{,}000$ psi stirrups

$A_T = \frac{212\ (105/2 + 14 + 126/2)}{12 \times 12} = 191\ ft^2$

Reduced live load

① $L = L_0\left(0.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right) = 100\left(0.25 + \frac{15}{\sqrt{2(191)}}\right) = 50.58$

Max. Neg. Moment

$A_T = \frac{(105/2 + 14 + 126/2) \times (212 + 16 + 200)}{12 \times 12} = 385\ ft^2$

② $L = 100\left(.25 + \frac{15}{\sqrt{2(385)}}\right) = 79.06$

Max. Pos. Moment

$A_T = \frac{(105/2 + 14 + 126/2)200}{12 \times 12} = 179.86$

③ $L = 100\left(.25 + \frac{15}{\sqrt{2(180)}}\right) = 104.1$

Summary of unfactored reduced live loads

For negative moment at exterior end, positive moment at midspan, and shear in beam

B3 = ① 50.58

For negative moment at support B = 79.06 ②

For positive moment at midspan of B4 = 104.1 ③

Select Method of Analysis

a) there are two or more spans

b) 212/128 = 1.66 > 1.2

c) loads uniformly distributed

d)

e)

Select the strength reduction factors

Slab
Floor, ceiling, mechanical
$\rangle$ 69 psf = DL

Live load

$W_U = 1.2(69.0) + 1.6(79.06) = 209.3$ psf

2/

$\left(\frac{105}{2} + 14 + \frac{126}{2}\right) = 129.5\text{ in} = 10.79\text{ ft}$

$10.79\text{ft}(209.3\text{ psf}) = 2.26$ kips per foot

1) The factored DL of the stem is taken as 10-20% of the factored loads on the beam, this gives

= .23 to .45 kip/ft

2) The overall depth of beam h is taken to be 8 to 10% of ln and bw is taken to be 0.5h. This gives the overall h as 10 to 13 in with the stem extending 17 - 21 in below the slab, and gives bw as 5 - 7 in. The factored load from stems of such sizes ranges from to kip/ft.

As a first trial, assume the factored weight of the stem to be 0.60 kip/ft then

Total trial load per foot $= 2.26 + .600 = 2.86$ kips/ft

Choose the actual size of the beam stem.

Calculate the minimum depth based on deflections.

Beam B3 is one-end continuous $f_y = 60{,}000$ psi and normal-density concrete

$\min. h = \frac{\ell}{18.5}$

where ℓ shall be taken as the span center to center of supports.

$\min h. = \frac{212 + 16\text{ in}}{18.5} = 12.3$

$\text{moment} = \frac{w_u \ell_n^2}{10}$

$\ell_n = \frac{212 + 200}{2} = 206\text{ in} = 17.17\text{ ft}$

$M_u = \frac{2.86(17.17^2)}{10} = 84.3\text{ ft-kips}$

p - tension controlled limit 0.0169 (A-5)

$\phi K_n = 616$ (A-3)

31

$$\frac{bd^2}{12{,}000} = \frac{M_u}{\phi K_n} = \frac{84.3}{616} = 0.1374$$

$bd^2 = 1642$

Possible choice

$b = 10$ $d = 12.8$

$b = 12$ $d = 11.7$

$b = 14$ $d = 10.8$

USE

$h \simeq d + 2.5 = 11 + 2.5 = 13.5$

Try a 14 in wide by 14 in deep extending 9 in below the slab.

Check shear capacity of the T-beam

$V_u = \phi(V_c + V_s)$

Maximum shear of beam loads at interior end of B3

$$V_u = 1.15\frac{w_u l_n}{2} = 1.15\frac{(2.86)(17.17)}{2} = 28.24 \text{ kips}$$

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

$= 2\sqrt{(3150)}(7)(11.5) = 9.86$ kips

min. bw for bar area

$V_s = 8\sqrt{f'c}\, b_w d$

$= 8\sqrt{3150}(7)(11.5) = 39.44$ kips

$\phi V_n = .75(9.86 + 39.44)$

$= 37.0$

Summary use

$b = 14$

$h = 14$ (9 in below the slab)

$d = 11.5$, assuming one layer of steel at all sections

4/

Compute the DL of the stem and recompute the total load per foot

Weight per foot of the stem below the slab $= \frac{14 \times 9 \times 14}{1728} \times 0.15 = .153$ kip/ft

Total DL of B3-B4-B3

$w_D = .069$ ksf $\times 10.79$ ft $+ .153$ kip/ft
$= .900$ kip/ft

LL for B3-B4-B3

Beam B3,

$w_L = .0558$ ksf $\times 10.79 = .602$ kip/ft

Negative moment at B

$w_L = .07906$ ksf $\times 10.79 = .853$ kip/ft

Positive moment for B4

$w_L = .1041$ ksf $\times 10.79 = 1.12$ kip/ft

Summary of factored loads on B3-B4-B3

B3 $w_U = 1.2(.90) + 1.6(.602) = 2.04$ kip/ft
B $w_U = 1.2(.90) + 1.6(.853) = 2.44$ kip/ft
B4 $w_U = 1.2(.90) + 1.6(1.12) = 2.87$ kip/ft

Calculate the flange width for the positive-moment regions

$.25 l_n$ (based on shorter span) $= .25(128) = 32$ in

$b_w + 2($

$b_w + \frac{126}{2} + \frac{105}{2} = 14 + \frac{126''}{2} + \frac{105''}{2} = 129.5$

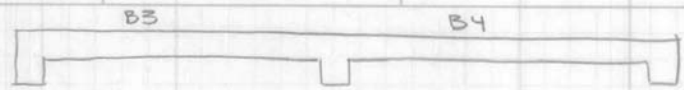
Therefore, the effective flange width is 32 in

Compute the beam moments

ratio of clear spans $= 212/200 = 1.06 < 1.20$

1.20 = upper limit on the use of the moment coefficients

5/



Line							
1) ℓ_n, ft	17.67	17.67		17.17	16.67		17.17
2) w_u, kip/ft	2.04	2.04		2.44	2.87		2.44
3) $w_u \ell_n^2$	637	637		719	798		719
4) C_m	1/24	1/14	1/10	1/11	1/16	1/11	1/10
5) $M_u = C_m w_u \ell_n^2$ (ft-kips)	-26.5	45.5		-71.9	50.0		-71.9

Design the flexural reinforcement

Compute the area of steel required at the point of maximum negative moment

$$A_s = \frac{M_u \text{ (ft-kips)}}{\phi f_y j d} \qquad \text{assume } j = .875 \quad \phi = .90$$

$$= \frac{71.9 \times 12000}{.9(60{,}000)(.875 \times 11.5)} = 1.59 \text{ in}^2$$

$$a = \frac{1.59 \times 60{,}000}{.85 \times 3750 \times 14} = 2.14 \text{ in}$$

$$\frac{a}{d} = \frac{a}{d_t} = \frac{2.14}{11.5} = .186 < \frac{a_b}{d} = .503$$

$$\frac{a}{d} < \frac{a_{TCL}}{d_t} = .319 \rightarrow \text{tension controlled}, \ \phi = .9$$

Now actual As value

$$A_s = \frac{M_u \times 12{,}000}{.9(60{,}000)(11.5 - .186/2)} = .019 M_u$$

6/

Line				
5) Mu (ft-Kips)	-26.6	45.5	-71.9	50.0
6) As coeffic.	.019	.020	.019	.020
7) As, required (in²)	.505	.915	1.36	1.0
8) As > As min	NO (use As min = .537	YES	YES	YES
9) Bars selected	2 no 5	3 no 5	4 no 6	4 no 5
10) As provided	.62	.93	1.55	1.24
11) bw OK				

Compute the area of steel required at the point of maximum positive moment

assume $j = .95$

$$A_s = \frac{50 \times 12000}{.9(60000)(.95 \times 11.5)} = 1.02$$

$$a = \frac{A_s f_y}{.85 f'_c b} = \frac{1.02(60000)}{.85(3750)(50)} = .384 < 6 = h_f$$

$b = 50$ = effective flange width

$$\frac{a}{d} = \frac{a}{d_t} = \frac{.384}{11.5} = .033 \qquad \phi = .9$$

$$A_s = \frac{M_u(12000)}{.9(60{,}000)(11.5 - .384/2)} = .020 M_u$$

Calculate the minimum reinforcement

$$A_{s,min} = \frac{3\sqrt{f'_c}}{f_y} b_w d \geq \frac{200 b_w d}{f_y}$$

$$\frac{3\sqrt{3750}}{60{,}000}(14 \times 11.5) \geq \frac{200(14)(11.5)}{60{,}000}$$

$$.493 \geq .537$$

7/

Calculate the area of steel and select bars
Done on page 6!

Check the distribution of reinforcement
positive moment region
3 no5 bars (middle of span A-B)

C_c = 1.5in cover + .375 in stirrups = 1.875 in

max spacing

$$s = \frac{540}{f_s} - 2.5C_c \leq 12\left(\frac{36}{F_y}\right)$$

$f_s = .6f_y$ (f_y = 60 ksi)

s = 10.3

negative moment region

effective flange width 200/10 = 20 in

5 no 5 bars

$$S = \frac{540}{.6 \times 60} - 2.5 \times$$

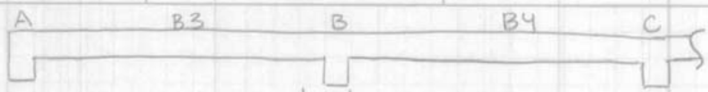
C_c =

Design the shear reinforcement

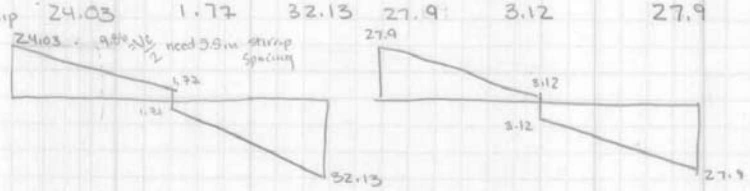
A B3 B B4 C

(next page)

8/



	A	B3	B	B	B4	C
ln, ft		17.67			16.67	
Wu kip/ft		2.04			2.87	
WLu kip/ft		.602			1.12	
$\frac{W_u l_n}{2}$		18.02			23.9	
Cv@ support and midspan	1.0	or $.25 \times \frac{.602}{2.04} = .074$	1.15	1.0	$\frac{.25 \times 1.12}{2.87} = .098$	1.0
$V_u = C_v\left(\frac{w_u l_n}{2}\right)$	18.02	1.33	24.1	20.9	2.34	20.9
$V_n = \frac{V_u}{\phi}$, kip	24.03	1.77	32.13	27.9	3.12	27.9



Exterior end of B3

$$\frac{V_u}{\phi} \text{ at } d = 24.03 - \frac{11.5}{100}(24.03 - 1.77)$$

$$= 21.44$$

ACI section 11.5.5.1 requires stirrups if $V_u > \phi V_c/2$

$$V_c = 2\sqrt{f'c}\, bwd$$
$$= 2\sqrt{3750}\,(14)\left(\frac{11.5}{1000}\right) = 19.72 \text{ kips}$$

$$\frac{V_c}{2} = \frac{19.72}{2} = 9.86$$

21.44 > 9.86 so stirrups are required

STIRRUPS - come back

9

Try No 3 grade double leg stirrups with a 90° hook enclosing a No 4 stirrup support bar. The maximum stirrup spacing is the smaller of

$$\frac{d}{2} = \frac{11.5}{2} = 5.75$$

$$S = \frac{A_v f_y}{50 b_w} = 12.57$$

$$S = \frac{A_v f_y}{.75 \sqrt{f'c}\, b_w} = 13.69$$

Use 5 in as the maximum spacing

$$S = \frac{A_v f_y d}{\frac{V_u}{\phi} - V_c} = \frac{.22 \times 40000 \times 11.5}{(21.44 - 19.72) \times 1000} = 57.8$$

S = 8 in

$$\frac{V_u}{\phi} = \frac{A_v f_y d}{S} + V_c = \frac{.22 \times 40 \times 11.5}{8} + 19.72$$

$$= 32.37 \text{ Kips}$$

$$x = \frac{24.03 - 32.37}{24.03 - 1.72} \times 100 =$$

Check the development lengths and design bar cutoffs
d = 11.5

$12d_b$ = 10.5 for no5, 10.5 for no6

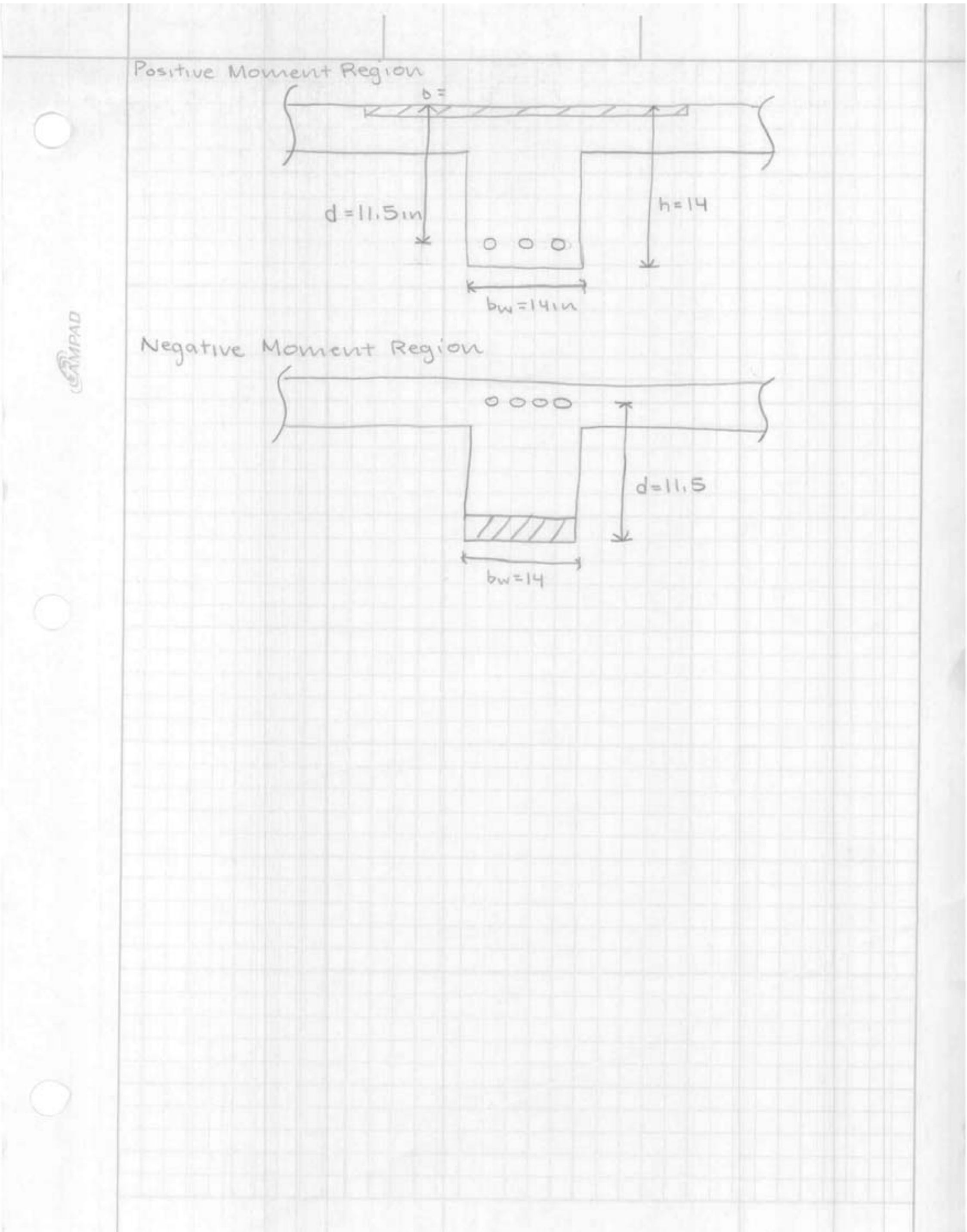
$\frac{ln}{16} = \frac{200}{16} = 12.5$ for B3

$= \frac{212}{16} = 13.25$ for B4

d always exceeds 12db
ln/16 exceeds d in both cases

Select cut offs for positive moment steel in B3

Spacing and confinement case


Positive Moment Region
b =
d = 11.5in
h = 14
b_w = 14in
Negative Moment Region
d = 11.5
b_w = 14

Girder Design:

pg 167

Girder Design
- Continuous T-Beam

Live load: 100psf

Every third beam is carried directly by columns

$R = .08(A-150)$ baysize 21×19

$\frac{2}{3}A = \frac{2}{3}(399) = 266$

$R = .08(266-150)$
$= 10\%$

Design Live load = 100psf

Concentrated loads

LL: $100\text{psf} \times 11' \times 19' = 20900$ lbs

DL: $110\text{psf} \times 11' \times 19' = 22{,}990$ lbs

Beam wt: $14 \times 9 \times \frac{150}{144} \times 19 = 2494$ lb

46384 lb

round to 46 K

Assume girder section that is 18in wide and 30in overall height

Girder weight: $18 \times 30 \times \frac{150}{144} = 563$ lb/ft

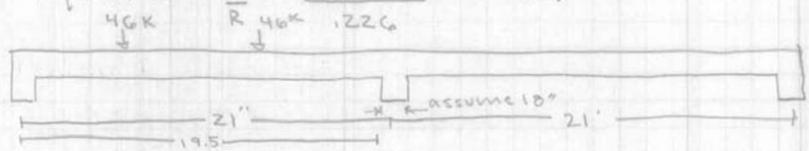
superimposed load: $124\text{psf} \times 1.5\text{ft} = 186$ lb/ft

Total unit load: 749 lb/ft

round to .75 K/ft

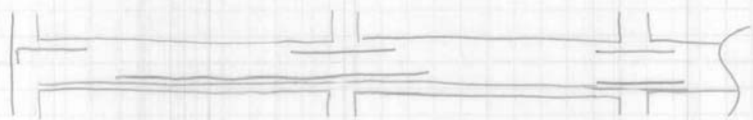
Reasonable limit → balanced moment capacity is approximately 2/3 that of the total required resisting moment

design $M = \frac{2}{3} \times 209 = 140$ k-ft required $bd^2 = \frac{M}{R} = \frac{140 \times 12}{.226} = 7434 \, in^3$



1/16	1/11	1/10	1/11	1/16	= C
131	190	209		131	= CWL
					= jd
					= Required As - in²

Bar Selection



if b = 18in

$d = \sqrt{\frac{7434}{18}} = 20.3$

Adequate for flexure (18in×30in)

Maximum Shear force

$V = 1.15 \times \frac{107}{2} = 61.53 \, K$

Reduced slightly for the critical design at d distance, use 60K

Maximum shear stress for stirrup spacing

$v = \frac{V}{bd} = \frac{60{,}000}{18 \times 20.3} = 164 \, psi$

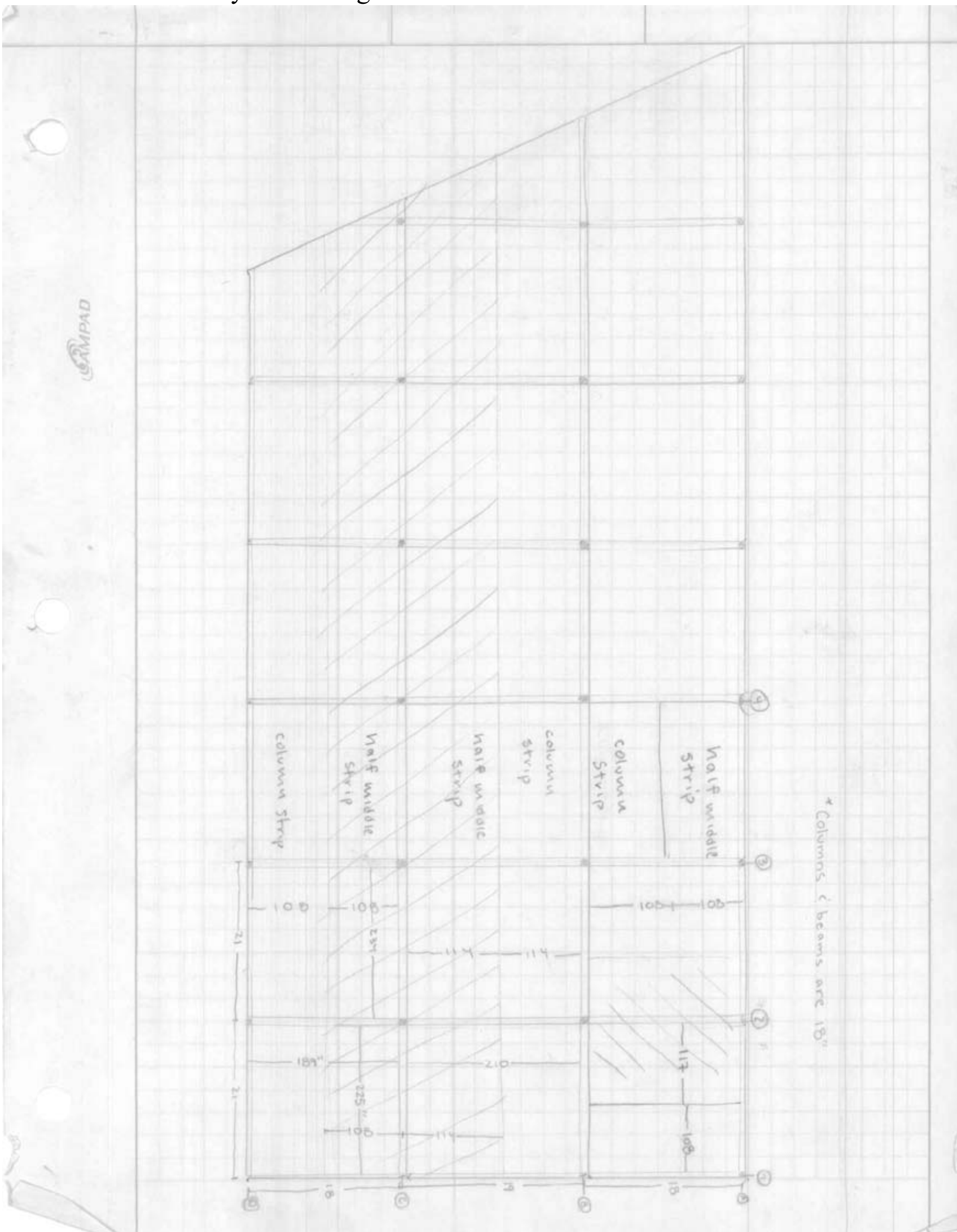
With an allowable stress of 60 psi $(1.1\sqrt{f'_c})$ leaves 104 psi to be taken by the stirrups

Use No 4. U stirrup
required spacing

$$s = \frac{A_v f_v}{v'b} = \frac{2(.20)\times 20{,}000}{104 \times 18} = 4.27\text{ in}$$

Would require average spacing of about 4.5 in for the stirrups in the end third of the girder spans

Scenario 5- Two Way Slab Design:



1/

1) Direct design method if:
- min of 3 consecutive spans each way ✓
- successive span lengths differ by not more than 1/3 of the longer span ≥ .667 : 18/19 = .947 ✓
- All loads are uniformly distributed gravity loads

Estimate slab thickness

Panel 1-2-A-B (corner):
$h = \frac{189}{33} = 5.73$

Panel 1-2-B-C (edge)
$h = \frac{210}{33} = 6.36$

Panel-A-B-1-2
$h = \frac{225}{33} = 6.82$

Panel-A-B-2-3
$h = \frac{234}{33} = 7.09$

Try h=7.25

Width is 18.75 ft from center line of column 1 to column 2
line of zero shear is at .44×18.75=8.25' from line 1 and 10.5 from line 2
Edge distance is 4+6in=.833 ft
length is 15.75 ft from center line A to centerline B
line of zero shear is at .44×15.75=6.93
Edge distance is 4+6in=.833 ft

Span 2-3 = same as above

Span B1-B2

Width is 18.75' column 1 to 2
line of zero shear = 8.25 from 1 & 10.5 from 2
Width is 17.5' from B to C
line of zero shear is .44×17.5=7.7'
Edge distance is 4+6 in=.833 ft

$w_u = 1.4\left(\frac{7.25}{12}\times150+25\right)+1.7(\underset{\text{Liveload}}{40}) = 238.6$

Average d ≅ 7.25 - 1.4 in ≅ 5.85 in

2/

Column B2

Assume line of zeroshear at .44 l

$b_o = 2\left(12 + 2\cdot\frac{d}{2} + 19 + 2\frac{d}{2}\right) = 85.4\,in$

$V_u = 238.6\left[(8.25+7.7)(10.5+9.8) - \left(\frac{17.85 \times 24.85}{144}\right)\right]$

$= 76.5$ kips

$\phi V_c = .85\left(2 + \frac{4}{\beta_c}\right)\sqrt{f'c}\; b_o d$

$= .85\left(2 + \frac{4}{19/12}\right)\sqrt{3750}\,(85.4) \times 585$

$= 117{,}704$ lb

$\phi V_c = .85\left(\frac{\alpha_s d}{b_o} + 2\right)\sqrt{f'c}\; b_o d$

$= .85\left(\frac{40 \times 5.85}{85.4} + 2\right)\sqrt{3750}(85.4)(5.85)$

$= 123262$ lb

$\phi V_c = \phi 4\sqrt{f'c}\; b_o d$

$= .85(4)\sqrt{3750}\,(85.4)(5.85)$

$= 104{,}018 = 104{,}000$ lb

Therefore, $\phi V_c = 104{,}000$ lb

$\phi V_c > V_u$ thickness ok for shear

$1.2 \times 76.5 \times 1000 = 91800 < \phi V_c$ ✓

Column B1

$b_o = 18.68 + 24.85 + 18.68 = 62.2\,in$

$V_u = 238.6\left[(8.25+7.7)(10.5+.833) - \left(\frac{18.68 + 24.85}{144}\right)\right]$

$= 43.1$ kips

$\phi V_c = .85\left(2 + \frac{4}{19/12}\right)\sqrt{3750}\,(62.2)(5.85)$

$= 85{,}700$ lb

$\phi V_c = .85\left(\frac{30 \times 5.85}{62.2} + 2\right)\sqrt{3750} \times 62.2 \times 5.85$

$= 91,442\ lb$

$\phi V_c = .85(4\sqrt{3750} \times 62.2 \times 5.85)$

$= 75760\ lb$

$\phi V_c = 75,800$

$\phi V_c = 1.76$ times V_u

if ϕV_c is less than 1.8 to 2 times V_u at exterior column might be inadequate for shear

Increase thickness to 7.5 in

Compute final W_u

$W_u = 1.4\left(\frac{7.5}{12} \times 150 + 25\right) + 1.7(40)$

$= 234.2 = 235\ psf$

→ Compute α for exterior beams.
since there are no edge beams $\alpha = 0$

Table 1)
Compute the moments in a slab strip along column line 2

	A2			B2		C2			D2
l_1 (ft)		18			19			18	
l_n (ft)		15.75			17.5			15.75	
l_2 (ft)		19.1			19.1			19.1	
w_u (ksf)		.235			.235			.235	
$M_o = \frac{w_u l_2 l_n^2}{8}$		139.2			171.8			139.2	
moment coefficient	-.26	.52	-.70	-.65	.35	-.65	-.70	.52	-.26
− & + moment	-36.2	72.4	-97.4	-111.7	60.1	-111.7	-97.4	72.4	-36.2
Σ of column moments	36.2			28.6		28.6			36.2

Table 2 4/

Calculation of neg & pos. moments for East-West Slabstrip

	A1			B1			C1		D1
l_1(ft)		18			19			18	
l_n(ft)		15.75			17.5			15.75	
l_2 edge		9			9.5			9	
w_u		.235			.235			.235	
M_o		65.6			85.5			65.6	
Moment coefficient	-.26	.52	-.70	-.65	.35	-.65	-.70	.52	-.26
- & + moments	-17.1	34.1	-45.9	-55.6	29.9	-55.6	-45.9	34.1	-17.1
column moments	17.1		19.4			19.4			17.1
Wall load		.42			.42			.42	
Wall M_o									
- & + moments									
Column moments									
total column moments									

Table 3)
Calculation of neg & pos. moments for North-South Slab Strip B

5/

	B1		B2			B3
l_1 (ft)		21			21	
l_n (ft)		19.5			19.5	
l_2 (ft)		17.5			17.5	
w_u (ksf)		.235			.235	
Mo		195.5			195.5	
moment coefficient	-.26	.52	-.70	-.65	.35	-.65
- & + moments	-50.8	101.7	-136.9	-127.1	68.4	127.1
column moments	50.8					

Table 4)
Calculation of neg & pos moments for North-South slab Strip A

	A1		A2			A3
l_1 (ft)		21			21	
l_n (ft)		19.5			19.5	
l_2 (ft)		9			9	
w_u		.235			.235	
Mo		100.5			100.5	
Moment coefficient	-.26	-.52	-.70	-.65	.35	-.65
- & + moment	-26.1	52.3	-70.4	-65.3	35.2	-65.3
Column moment						
Wall load		.42			.42	
Wall Mo						
- & pos moments						

6/

	A1	A2	A3
column moments			
Total column moments			

$$M_{col} = .07\left[(1.4 \times 94 + .5 \times 1.7 \times 40) \times 21 \times 17.5^2\right] - \left[(1.4 \times 94) \times 21 \times 19.1^2\right]$$

$$= 4.0 \text{ ft-kips}$$

Compute the moments in the slab strip along column line

Table 5)

Division of moment to column & Middle strips: East-West strips

	Column Strip	Middle Strip	Column Strip	Middle Strip	Edge Column
Exterior Negative Moments					
1) Slab moment	-36.2		-36.2		-17.1
2) Moment coefficients	0.0 1.0 0.0	0.0 0	1.0	0.0 0 0.0	1.0
3) Moment to column & middle	1.0		1.0		1.0
4) Wall moment	-36.2		-36.2		-17.1 -3.7
5) Total moment	-36.2	0	-36.2	0	-36.2
6) As required					
7) Min As	1.49	1.49	1.49	1.49	1.35
8) Choose steel	10#4	8#4	10#4	8#4	7#4
9) As provided	2.0	1.6	2.0	1.6	1.4
End Span Positive Moments					
1) Slab moment	72.4		72.4		34.1
2) moment coefficients	.2 .6 .2	.2	.6	.2 .4	.6
3) moment to columns & middle	14.5 43.4 14.5	14.5	43.4	14.5 13.6	20.5
4) Wall moment	43.4	29	43.4	28.1	7.4 27.9
5) Total moment					
6) As required					
7) Min As	1.49	1.49	1.49	1.49	1.35
8) Choose steel					
9) As provided					

First Interior Negative Moments

1) Slab Moment	-111.7			-111.7			-55.6
2) Moment coefficient	.75	.125	.125	.75	.125	.125	.75
3) moment to column; middle	-83.8	-14.0	-14.0	-83.8	-14.0	-7.0	-41.7
4) Wall moment							-11.5
5) Total moment	-83.8	-28		-83.8	-21		-53.2
6) As required							
7) Min As	1.49	1.49		1.49	1.49		1.35
8) Choose steel							
9) As provided							

Interior Positive Moments

1) Slab moment	60.1			60.1			29.9
2) Moment coefficient	.6	.2	.2	.6	.2	.4	.6
3) Moment to column;	36.1	12.0	12.0	36.1	12.0	12.0	17.9
4) Wall Moment							6.1
5) Total Moment	36.1	24.0		36.1	24.0		24.0
6) As Required							
7) Min As	1.49	1.49		1.49	1.44		1.35
8) Choose Steel							
9) As provided							

9/

Compute the moments in the slabstrip along column line B
Done in table

Compute the moments in the slab strip along column lin A
Done in table

Distribute the negative & positive moments to the column & middle strips and design the reinforcement strips spanning East-West

Column strips extend 54in or 57 in on each side of the column lines

Total width of the column strip is 111 in = 9.25 ft
Edge strip has a width of 54 + 10 = 5.33 ft

Divide the moments between the column & middle strip and design the reinforcement

$$A_s = \frac{M_u}{\phi f_y j d}$$

The largest moments in the panel occur at support B2
Assume no 5 bars

$$d = 7.5\,in - \frac{3}{4}\,in - \frac{1}{2}\,\text{bar diameter}$$
$$= 6.44\,in$$

$$A_{s(reqd)} = \frac{85.5 \times 12000}{.9 \times 40,000 \times .925 \times 6.44} = 4.78$$

$$a = \frac{A_s f_y}{.85 f'c\, b} = \frac{4.78 \times 40,000}{.85 \times 3750\,(8.25 \times 12)} = .588$$

$$\frac{a}{d} = \frac{.588}{6.44} = .091$$

Check strain ϵ_1 in the extreme tension layer of the steel)

$$\epsilon_1 = \frac{(d_t - c)}{c} \times .003$$

$$d_t = d =$$

$$c = \frac{a}{B_1} = \frac{.091}{.85} = .107$$

$$\epsilon_1 = \frac{(6.44 - .107)}{.107} \times .003 = .178$$

10/

since ε_t = exceeds .005 the slab is tension controlled and ϕ = .90

compute jd and the constant for computing A_s

$$jd = d - \frac{a}{2} = 6.44 - \frac{.588}{2} = 6.15$$

$$A_s = \frac{M_u \times 12000}{.9 \times 40{,}000 \times 6.15}$$

A_s = .0542 M_u (ft-kips)

Edge column strip

$A_s(min)$ = .002bh (Grade 40 reinforcement)

= .002 (7.5 × 12) 7.5 = 1.35

minimum number of bar spaces

$$= \frac{7.5 \times 12}{14} = 6.43$$

minimum number of bar = 7

Other strips

$A_s(min)$ = .002bh

= .002(8.25 × 12) × 7.5

= 1.49

minimum number of bars = 7

Distribute the negative and positive moments to the column and middle strips and design the reinforcement strips spanning North & South

Divide the slab strips into middle and column strips (see figure)

Divide the moments between the column and middle strips and design the reinforcement

Assuming No 5 bars

$d \cong 7.5 - .75 - 1.5(.625) = 5.81$

Compute a trial A_s

largest M_u =

$$A_s(reqd) = \frac{101.7 \times 12000}{.9 \times 40{,}000 \times .925 \times 5.81} = 6.31$$

11/

$$a = \frac{A_s f_y}{.85 f'_c b} = \frac{.631 \times 40{,}000}{.85 \times 3750\,(8.25 \times 12)} = .80$$

$$\frac{a}{d} = \frac{.80}{5.81} = .138$$

copy table 5 $\Leftarrow$

---

compute $jd = d - \frac{a}{2} = 5.81 - \frac{.80}{2} = 5.41$

$$A_s(in^2) = \frac{M_u \times 12000}{.9 \times 40000 \times 5.41}$$

Therefore $A_s(in^2) = .061\ M_u(ft\text{-}kips)$

between A & B

$A_s(min) = .002\,(8.75 \times 12)\,7.5 = 1.58$

minimum number of bar spaces = 7.5

minimum number of bars = 8

between B & C

$A_s(min) = .002(8.75 \times 12) \times 7.5 = 7.5$

minimum number of bars = 8

Check the shear at the exterior columns for combined shear and moment transfer

Locate the critical shear perimeter

$d_{avg} = 5.75 \qquad \frac{d}{2} = 2.88$

$b_1 = 17.85 \qquad b_2 = 24.85$

Locate the centroid of the shear perimeter

$$y_{AB} = \frac{2 b_1 d_{avg}\, b/2}{2 b_1 d_{avg} + b_2 d_{avg}}$$

$$= \frac{2(17.85)(5.75)(17.85/2)}{2(17.85)(5.75) + 24.85(5.75)} = 5.26$$

12/

Therefore,

$c_{AB} = y_{AB} = 5.26$

$c_{CD} = b_1 - y_{AB} = 12.54$

For moments about the W-W axis

$c_{CB} = c_{AD} = \frac{b_2}{2} = \frac{24.85}{2} = 12.4$

Compute the shear and the moments about the centroid of the shear perimeter

$V_u = .235 \times \left[(8.25+7.7)(10.5+.833) - \frac{17.85 \times 24.85}{144}\right]$

$= 41.8$

$V_{uw} = \left(\frac{17.85 \cdot 24.85}{12}\right) \times 1.4(.3 \text{ kips/ft}) = 6.6 \text{ kips}$

Total $V_u = V_u + V_{uw}$

$= 41.8 + 6.6 = 48.4$

Z-Z Axis

$M_0 = 195.5$

$.3M_0 = 58.7$ ← moment transferred to the columns

compute ϕV_c ; $V_u/\phi V_c$

$b_0 = 2 \times 17.85 + 24.85 = 60.55$

$\phi V_c = \phi 4\sqrt{f'_c}\, b_o d$

$= .85(4)\sqrt{3750} \times 60.55 \times 5.75$

≈ 72.5 kips

$V_u/\phi V_c = .668$

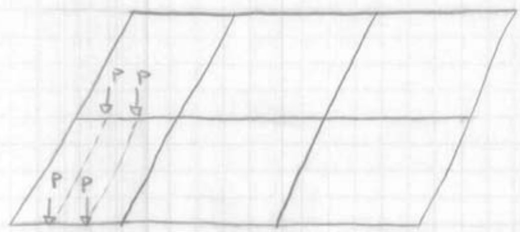
Determine the fraction of the moment transferred by flexure γ_f Moment about the Z-Z axis

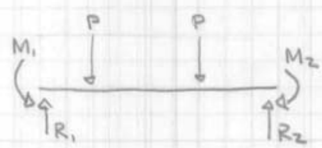
$\gamma_f = \frac{1}{1 + \frac{2}{3}\sqrt{b_1/b_2}} = \frac{1}{1 + \frac{2}{3}\sqrt{\frac{17.85}{24.85}}} = .639$

$\frac{V_u}{\phi V_c} < .75 \quad \gamma_f = 1.0$

Column Design

Moment entry for FRAME
Short side

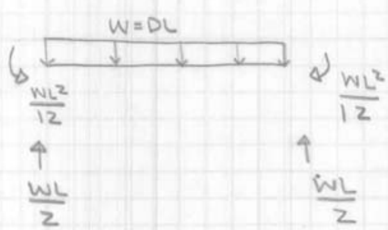


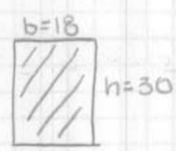


$M_1 = M_2 = (0.2222)\ P \times L$
$= (0.2222)(46 \times 18)$
$= 183.8$

OR/ $.2222(46 \times 19)$
$= 194.03$

$R_1 = P = 46$



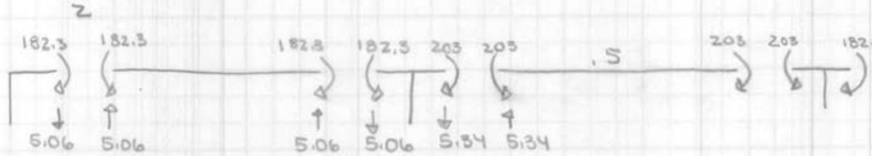


$W_0 = \frac{18}{12} \times \frac{30}{12} \times \frac{150}{1000}$
$= .5625\ K/ft$

$\frac{WL^2}{12} = \left[\frac{.5625(18^2)}{12}\right] = 15.19 \times 12$
$= 182.3$

$16.9 \times 12 = 203$

$\frac{WL}{2} = 5.06\ K$



MQP Problem 1

NUMBER OF NODES = 20
NUMBER OF ELEMENTS = 28
NUMBER OF MATERIALS = 1
NUMBER OF SUPPORT JOINTS = 4
NUMBER OF LOADED JOINTS = 4

NODAL DATA

NODE	X	Y	Code Restraints		
1	0.000	670.000	0	0	0
2	360.000	670.000	0	0	0
3	444.000	670.000	0	0	0
4	660.000	670.000	0	0	0
5	0.000	491.000	0	0	0
6	360.000	491.000	0	0	0
7	444.000	491.000	0	0	0
8	660.000	491.000	0	0	0
9	0.000	327.000	0	0	0
10	360.000	327.000	0	0	0
11	444.000	327.000	0	0	0
12	660.000	327.000	0	0	0
13	0.000	162.000	0	0	0
14	360.000	162.000	0	0	0
15	444.000	162.000	0	0	0
16	660.000	162.000	0	0	0
17	0.000	0.000	1	1	1
18	360.000	0.000	1	1	1
19	444.000	0.000	1	1	1
20	660.000	0.000	1	1	1

ELEMENT DATA

ELEMENT	J1	J2	AX	IZ	E
1	1	2	540.000	486000.000	3150.0
2	2	3	540.000	486000.000	3150.0
3	3	4	540.000	486000.000	3150.0
4	5	6	540.000	486000.000	3150.0
5	6	7	540.000	486000.000	3150.0
6	7	8	540.000	486000.000	3150.0
7	9	10	540.000	486000.000	3150.0
8	10	11	540.000	486000.000	3150.0
9	11	12	540.000	486000.000	3150.0
10	13	14	540.000	486000.000	3150.0
11	14	15	540.000	486000.000	3150.0
12	15	16	540.000	486000.000	3150.0
13	1	5	324.000	104976.000	3150.0
14	2	6	324.000	104976.000	3150.0
15	3	7	324.000	104976.000	3150.0
16	4	8	324.000	104976.000	3150.0
17	5	9	324.000	104976.000	3150.0
18	6	10	324.000	104976.000	3150.0
19	7	11	324.000	104976.000	3150.0
20	8	12	324.000	104976.000	3150.0
21	9	13	324.000	104976.000	3150.0

22	10	14	324.000	104976.000	3150.0
23	11	15	324.000	104976.000	3150.0
24	12	16	324.000	104976.000	3150.0
25	13	17	324.000	104976.000	3150.0
26	14	18	324.000	104976.000	3150.0
27	15	19	324.000	104976.000	3150.0
28	16	20	324.000	104976.000	3150.0

NODAL LOADS

NODE	WX	WY	MZ
1	20.000	0.000	0.00
5	20.000	0.000	0.00
9	20.000	0.000	0.00
13	20.000	0.000	0.00

NODAL DISPLACEMENTS - global axis

NODE	X-DISP	Y-DISP	Z-ROT
1	0.10365	0.01070	-0.00004
2	0.10066	0.00374	-0.00005
3	0.10025	-0.00088	-0.00006
4	0.09980	-0.01356	-0.00007
5	0.08560	0.01009	-0.00007
6	0.08215	0.00415	-0.00006
7	0.08162	-0.00113	-0.00007
8	0.08102	-0.01311	-0.00008
9	0.06198	0.00840	-0.00009
10	0.05842	0.00427	-0.00007
11	0.05788	-0.00130	-0.00007
12	0.05726	-0.01137	-0.00009
13	0.03062	0.00511	-0.00012
14	0.02789	0.00325	-0.00006
15	0.02746	-0.00116	-0.00006
16	0.02685	-0.00721	-0.00009
17	0.00000	0.00000	0.00000
18	0.00000	0.00000	0.00000
19	0.00000	0.00000	0.00000
20	0.00000	0.00000	0.00000

ELEMENT FORCES in Local Axis - x,y,z

ELEMENT	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT-Z
1	1	14.153	-3.516	-580.90
1	2	-14.153	3.516	-684.71
2	2	8.327	-1.142	140.71
2	3	-8.327	1.142	-236.65
3	3	3.522	-2.550	-210.60
3	4	-3.522	2.550	-340.16
4	5	16.325	-6.998	-1288.20
4	6	-16.325	6.998	-1231.06
5	6	10.666	-8.623	-226.99
5	7	-10.666	8.623	-497.38

6	7	4.769	-8.308	-802.54
6	8	-4.769	8.308	-991.88
7	9	16.793	-9.787	-1841.04
7	10	-16.793	9.787	-1682.14
8	10	11.077	-16.847	-644.76
8	11	-11.077	16.847	-770.40
9	11	4.878	-14.866	-1467.27
9	12	-4.878	14.866	-1743.77
10	13	12.891	-11.919	-2369.52
10	14	-12.891	11.919	-1921.44
11	14	8.816	-26.107	-1109.04
11	15	-8.816	26.107	-1083.92
12	15	4.796	-19.692	-1902.84
12	16	-4.796	19.692	-2350.62
13	1	-3.516	5.847	580.90
13	5	3.516	-5.847	465.78
14	2	2.373	5.826	544.01
14	6	-2.373	-5.826	498.80
15	3	-1.408	4.805	447.25
15	7	1.408	-4.805	412.89
16	4	2.550	3.522	340.16
16	8	-2.550	-3.522	290.22
17	5	-10.514	9.522	822.42
17	9	10.514	-9.522	739.24
18	6	0.748	11.485	959.26
18	10	-0.748	-11.485	924.32
19	7	-1.092	10.702	887.03
19	11	1.092	-10.702	868.11
20	8	10.857	8.290	701.67
20	12	-10.857	-8.290	657.95
21	9	-20.300	12.729	1101.81
21	13	20.300	-12.729	998.54
22	10	-6.312	17.201	1402.57
22	14	6.312	-17.201	1435.58
23	11	0.889	16.901	1369.56
23	15	-0.889	-16.901	1419.15
24	12	25.723	13.168	1085.82
24	16	-25.723	-13.168	1086.98
25	13	-32.220	19.839	1370.98
25	17	32.220	-19.839	1842.90
26	14	-20.500	21.275	1594.90
26	18	20.500	-21.275	1851.73
27	15	7.304	20.921	1567.61
27	19	-7.304	-20.921	1821.63
28	16	45.415	17.965	1263.64
28	20	-45.415	-17.965	1646.62

SUPPORT REACTIONS

NODE	RX	RY	MZ
17	-19.839	-32.220	1842.90
18	-21.275	-20.500	1851.73
19	-20.921	7.304	1821.63
20	-17.965	45.415	1646.62

MQP Problem 1 live

NUMBER OF NODES = 20
NUMBER OF ELEMENTS = 28
NUMBER OF MATERIALS = 1
NUMBER OF SUPPORT JOINTS = 4
NUMBER OF LOADED JOINTS = 16

NODAL DATA

NODE	X	Y	Code Restraints		
1	0.000	670.000	0	0	0
2	360.000	670.000	0	0	0
3	444.000	670.000	0	0	0
4	660.000	670.000	0	0	0
5	0.000	491.000	0	0	0
6	360.000	491.000	0	0	0
7	444.000	491.000	0	0	0
8	660.000	491.000	0	0	0
9	0.000	327.000	0	0	0
10	360.000	327.000	0	0	0
11	444.000	327.000	0	0	0
12	660.000	327.000	0	0	0
13	0.000	162.000	0	0	0
14	360.000	162.000	0	0	0
15	444.000	162.000	0	0	0
16	660.000	162.000	0	0	0
17	0.000	0.000	1	1	1
18	360.000	0.000	1	1	1
19	444.000	0.000	1	1	1
20	660.000	0.000	1	1	1

ELEMENT DATA

ELEMENT	J1	J2	AX	IZ	E
1	1	2	540.000	486000.000	3150.0
2	2	3	540.000	486000.000	3150.0
3	3	4	540.000	486000.000	3150.0
4	5	6	540.000	486000.000	3150.0
5	6	7	540.000	486000.000	3150.0
6	7	8	540.000	486000.000	3150.0
7	9	10	540.000	486000.000	3150.0
8	10	11	540.000	486000.000	3150.0
9	11	12	540.000	486000.000	3150.0
10	13	14	540.000	486000.000	3150.0
11	14	15	540.000	486000.000	3150.0
12	15	16	540.000	486000.000	3150.0
13	1	5	324.000	104976.000	3150.0
14	2	6	324.000	104976.000	3150.0
15	3	7	324.000	104976.000	3150.0
16	4	8	324.000	104976.000	3150.0
17	5	9	324.000	104976.000	3150.0
18	6	10	324.000	104976.000	3150.0
19	7	11	324.000	104976.000	3150.0
20	8	12	324.000	104976.000	3150.0

21	9	13	324.000	104976.000	3150.0
22	10	14	324.000	104976.000	3150.0
23	11	15	324.000	104976.000	3150.0
24	12	16	324.000	104976.000	3150.0
25	13	17	324.000	104976.000	3150.0
26	14	18	324.000	104976.000	3150.0
27	15	19	324.000	104976.000	3150.0
28	16	20	324.000	104976.000	3150.0

NODAL LOADS

NODE	WX	WY	MZ
1	0.000	-46.000	-2328.40
2	0.000	-46.000	2328.40
3	0.000	-46.000	-2328.40
4	0.000	-46.000	2328.40
5	0.000	-46.000	-2328.40
6	0.000	-46.000	2328.40
7	0.000	-46.000	-2328.40
8	0.000	-46.000	2328.40
9	0.000	-46.000	-2328.40
10	0.000	-46.000	2328.40
11	0.000	-46.000	-2328.40
12	0.000	-46.000	2328.40
13	0.000	-46.000	-2328.40
14	0.000	-46.000	2328.40
15	0.000	-46.000	-2328.40
16	0.000	-46.000	2328.40

NODAL DISPLACEMENTS - global axis

NODE	X-DISP	Y-DISP	Z-ROT
1	0.00090	-0.07187	-0.00013
2	-0.00143	-0.07797	0.00005
3	-0.00172	-0.07698	-0.00003
4	-0.00294	-0.07065	0.00011
5	-0.00076	-0.06427	-0.00007
6	-0.00073	-0.06918	0.00004
7	-0.00073	-0.06846	-0.00002
8	-0.00073	-0.06328	0.00007
9	-0.00030	-0.04999	-0.00007
10	-0.00017	-0.05369	0.00004
11	-0.00012	-0.05315	-0.00002
12	-0.00006	-0.04924	0.00007
13	-0.00073	-0.02831	-0.00008
14	0.00008	-0.03048	0.00004
15	0.00015	-0.03017	-0.00003
16	0.00060	-0.02787	0.00007
17	0.00000	0.00000	0.00000
18	0.00000	0.00000	0.00000
19	0.00000	0.00000	0.00000
20	0.00000	0.00000	0.00000

ELEMENT FORCES in Local Axis - x,y,z

ELEMENT	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT-Z
1	1	11.034	-2.681	-1242.05
1	2	-11.034	2.681	276.84
2	2	5.839	1.424	1558.71
2	3	-5.839	-1.424	-1439.10
3	3	9.627	4.004	-536.29
3	4	-9.627	-4.004	1401.07
4	5	-0.168	-0.431	-545.23
4	6	0.168	0.431	390.17
5	6	-0.086	-0.101	1080.96
5	7	0.086	0.101	-1089.43
6	7	0.066	0.599	-598.20
6	8	-0.066	-0.599	727.63
7	9	-0.616	-0.825	-606.91
7	10	0.616	0.825	309.91
8	10	-0.964	0.303	1154.41
8	11	0.964	-0.303	-1128.96
9	11	-0.514	1.220	-521.46
9	12	0.514	-1.220	785.00
10	13	-3.828	-1.681	-795.93
10	14	3.828	1.681	190.86
11	14	-1.473	0.773	1348.81
11	15	1.473	-0.773	-1283.85
12	15	-3.577	2.618	-417.16
12	16	3.577	-2.618	982.56
13	1	43.319	-11.034	-1086.35
13	5	-43.319	11.034	-888.74
14	2	50.105	5.195	492.85
14	6	-50.105	-5.195	436.98
15	3	48.580	-3.788	-353.00
15	7	-48.580	3.788	-325.00
16	4	41.996	9.627	927.33
16	8	-41.996	-9.627	795.93
17	5	88.888	-10.866	-894.44
17	9	-88.888	10.866	-887.66
18	6	96.435	5.113	420.29
18	10	-96.435	-5.113	418.25
19	7	95.280	-3.939	-315.77
19	11	-95.280	3.939	-330.30
20	8	87.397	9.693	804.84
20	12	-87.397	-9.693	784.79
21	9	134.063	-10.250	-833.83
21	13	-134.063	10.250	-857.42
22	10	143.563	5.461	445.83
22	14	-143.563	-5.461	455.22
23	11	142.197	-4.390	-347.68
23	15	-142.197	4.390	-376.69
24	12	132.177	9.179	758.61
24	16	-132.177	-9.179	755.96
25	13	178.382	-6.422	-675.05
25	17	-178.382	6.422	-365.28
26	14	192.017	3.106	333.52
26	18	-192.017	-3.106	169.63
27	15	190.041	-2.287	-250.70
27	19	-190.041	2.287	-119.72

28	16	175.559	5.603	589.88
28	20	-175.559	-5.603	317.73

SUPPORT REACTIONS

NODE	RX	RY	MZ
17	6.422	178.382	-365.28
18	-3.106	192.017	169.63
19	2.287	190.041	-119.72
20	-5.603	175.559	317.73

MQP Problem

NUMBER OF NODES = 20
NUMBER OF ELEMENTS = 28
NUMBER OF MATERIALS = 1
NUMBER OF SUPPORT JOINTS = 4
NUMBER OF LOADED JOINTS = 16

NODAL DATA

NODE	X	Y	Code Restraints		
1	0.000	670.000	0	0	0
2	360.000	670.000	0	0	0
3	444.000	670.000	0	0	0
4	660.000	670.000	0	0	0
5	0.000	491.000	0	0	0
6	360.000	491.000	0	0	0
7	444.000	491.000	0	0	0
8	660.000	491.000	0	0	0
9	0.000	327.000	0	0	0
10	360.000	327.000	0	0	0
11	444.000	327.000	0	0	0
12	660.000	327.000	0	0	0
13	0.000	162.000	0	0	0
14	360.000	162.000	0	0	0
15	444.000	162.000	0	0	0
16	660.000	162.000	0	0	0
17	0.000	0.000	1	1	1
18	360.000	0.000	1	1	1
19	444.000	0.000	1	1	1
20	660.000	0.000	1	1	1

ELEMENT DATA

ELEMENT	J1	J2	AX	IZ	E
1	1	2	540.000	486000.000	3150.0
2	2	3	540.000	486000.000	3150.0
3	3	4	540.000	486000.000	3150.0
4	5	6	540.000	486000.000	3150.0
5	6	7	540.000	486000.000	3150.0
6	7	8	540.000	486000.000	3150.0
7	9	10	540.000	486000.000	3150.0
8	10	11	540.000	486000.000	3150.0
9	11	12	540.000	486000.000	3150.0
10	13	14	540.000	486000.000	3150.0
11	14	15	540.000	486000.000	3150.0
12	15	16	540.000	486000.000	3150.0
13	1	5	324.000	104976.000	3150.0
14	2	6	324.000	104976.000	3150.0
15	3	7	324.000	104976.000	3150.0
16	4	8	324.000	104976.000	3150.0
17	5	9	324.000	104976.000	3150.0
18	6	10	324.000	104976.000	3150.0

19	7	11	324.000	104976.000	3150.0
20	8	12	324.000	104976.000	3150.0
21	9	13	324.000	104976.000	3150.0
22	10	14	324.000	104976.000	3150.0
23	11	15	324.000	104976.000	3150.0
24	12	16	324.000	104976.000	3150.0
25	13	17	324.000	104976.000	3150.0
26	14	18	324.000	104976.000	3150.0
27	15	19	324.000	104976.000	3150.0
28	16	20	324.000	104976.000	3150.0

NODAL LOADS

NODE	WX	WY	MZ
1	0.000	-5.060	-182.30
2	0.000	-10.400	-20.70
3	0.000	-10.400	20.70
4	0.000	-5.060	182.30
5	0.000	-5.060	-182.30
6	0.000	-10.400	-20.70
7	0.000	-10.400	20.70
8	0.000	-5.060	182.30
9	0.000	-5.060	-182.30
10	0.000	-10.400	-20.70
11	0.000	-10.400	20.70
12	0.000	-5.060	182.30
13	0.000	-5.060	-182.30
14	0.000	-10.400	-20.70
15	0.000	-10.400	20.70
16	0.000	-5.060	182.30

NODAL DISPLACEMENTS - global axis

NODE	X-DISP	Y-DISP	Z-ROT
1	-0.00123	-0.00952	-0.00002
2	-0.00169	-0.01527	0.00000
3	-0.00182	-0.01485	0.00001
4	-0.00207	-0.01035	0.00003
5	-0.00109	-0.00851	-0.00001
6	-0.00105	-0.01358	0.00000
7	-0.00105	-0.01321	0.00001
8	-0.00103	-0.00927	0.00002
9	-0.00060	-0.00659	-0.00001
10	-0.00051	-0.01059	0.00000
11	-0.00048	-0.01030	0.00001
12	-0.00043	-0.00716	0.00001
13	-0.00029	-0.00370	-0.00001
14	-0.00014	-0.00605	0.00000
15	-0.00009	-0.00589	0.00000
16	0.00000	-0.00400	0.00001
17	0.00000	0.00000	0.00000
18	0.00000	0.00000	0.00000
19	0.00000	0.00000	0.00000
20	0.00000	0.00000	0.00000

ELEMENT FORCES in Local Axis - x,y,z

ELEMENT	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT-Z
1	1	2.171	0.689	31.48
1	2	-2.171	-0.689	216.60
2	2	2.655	-0.068	-194.41
2	3	-2.655	0.068	188.69
3	3	1.985	-1.111	-229.97
3	4	-1.985	1.111	-9.98
4	5	-0.161	1.152	160.56
4	6	0.161	-1.152	254.03
5	6	-0.098	-0.241	-185.82
5	7	0.098	0.241	165.59
6	7	-0.115	-1.868	-263.35
6	8	0.115	1.868	-140.08
7	9	-0.435	0.854	113.09
7	10	0.435	-0.854	194.34
8	10	-0.619	-0.127	-140.27
8	11	0.619	0.127	129.56
9	11	-0.393	-1.393	-205.46
9	12	0.393	1.393	-95.41
10	13	-0.708	0.352	30.23
10	14	0.708	-0.352	96.32
11	14	-0.931	-0.019	-75.65
11	15	0.931	0.019	74.03
12	15	-0.683	-0.589	-110.43
12	16	0.683	0.589	-16.74
13	1	5.749	-2.171	-213.78
13	5	-5.749	2.171	-174.80
14	2	9.643	-0.484	-42.89
14	6	-9.643	0.484	-43.73
15	3	9.357	0.670	61.98
15	7	-9.357	-0.670	57.93
16	4	6.171	1.985	192.28
16	8	-6.171	-1.985	163.01
17	5	11.961	-2.010	-168.06
17	9	-11.961	2.010	-161.59
18	6	18.650	-0.547	-45.18
18	10	-18.650	0.547	-44.51
19	7	18.130	0.687	60.53
19	11	-18.130	-0.687	52.18
20	8	13.099	1.870	159.37
20	12	-13.099	-1.870	147.26
21	9	17.875	-1.575	-133.79
21	13	-17.875	1.575	-126.08
22	10	28.069	-0.363	-30.26
22	14	-28.069	0.363	-29.70
23	11	27.265	0.462	44.41
23	15	-27.265	-0.462	31.77
24	12	19.552	1.477	130.46
24	16	-19.552	-1.477	113.19
25	13	23.286	-0.867	-86.45
25	17	-23.286	0.867	-54.05
26	14	38.098	-0.140	-11.68
26	18	-38.098	0.140	-11.00

27	15	37.095	0.213	25.33
27	19	-37.095	-0.213	9.24
28	16	25.200	0.794	85.85
28	20	-25.200	-0.794	42.77

SUPPORT REACTIONS

NODE	RX	RY	MZ
17	0.867	23.286	-54.05
18	0.140	38.098	-11.00
19	-0.213	37.095	9.24
20	-0.794	25.200	42.77

Interior Column (Short)

$P_u = .75\,(1.2(38.1) + 1.6(192.0) + 1.6(20.5)) = 289.3$

$M_u = .75\,(1.2(0) + 1.6(0) + 1.6(1595)) = 1914$

$V_u = .75\,(1.2(0) + 1.6(0) + 1.6(21.3)) = 25.6$

$F_y = 60k$
$f'c = 3.75^k$
$P_t = 0.015$

18×18

$$\gamma = \frac{18 - 2(1.5 + .375 + .5)}{18} = .736$$

Interaction Diagram

$$\frac{\phi P_n}{A_g} = \frac{P_u}{A_g} = \frac{289.3}{18 \times 18} = 0.893$$

$$\frac{\phi M_n}{A_g} = \frac{M_u}{A_g \times h} = \frac{1914}{18 \times 18 \times 18} = 0.328$$

$p_t = 0.01$

Select the reinforcement

$A_{st} = p_t \times A_g = 0.01\,(18 \times 18) = 3.24$

six no 7 bars $A_s = 3.60\,in^2$

Check max load capacity

$\phi P_{n,max} = .85 \times .7\,[.85 \times 3(18 \times 18) - 3.60) + 60 \times 3.6]$

$= 614.65 > 289.3$

Design Lap slices

$$L_u = \frac{f_y \times \alpha \times \beta \times \lambda}{20\sqrt{f'c} \times d_b} = \frac{60{,}000 \times 1 \times 1 \times 1}{20\sqrt{3750} \times (.875)} = 56$$

Select ties

16 long bar diameter = 16(1.128) = 18.05"
48 tie diameters = 48(3/8) = 18"
least diameter of coil = 18"
Use Δ = 18" ties

Shear V_u

$$V_c = 2\left(\frac{L + N_u}{2000 A_g}\right)\sqrt{f'c}\, b_w \times d =$$

$$2\left(\frac{1 + 50{,}000}{2000(18 \times 18)}\right)\sqrt{3750} \times 18 \times 15.56$$

$$= 85.76$$

$V_u = 25.6 < .50 V_c = 42.9$

Use No 3 ties @ 18" on center

Appendix H: Lateral Loading

Steel

WIND LOADING

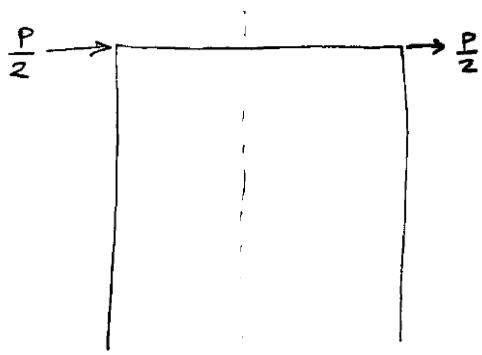
IBC - 90 mph = 970 kPa

Conversion Factor 4.7888026E01

$$\frac{970\,\text{kPa}}{4.788026\text{E}01} = 20.26\ \frac{\text{lb}}{\text{ft}^2}$$

$$= .02\ \text{k/ft}$$

Floor	Area		Force
4	416.25	.02	8.325
3	793.65	.02	15.873
2	760.91	.02	15.22
1	756.2	.02	15.2



SEISMIC LOADS

· WEIGHT OF STEEL : 84.5' x 27.75'

BEAMS + GIRDERS

1ST FLOOR BEAMS + GIRDERS → 23088 lbs + 21390 lbs = 44,478 lbs

2ND + 3RD FLOOR → (2)16,872 lbs + (2)10,764 lbs = 65,272 lbs

4TH FLOOR → 16,872 lbs + 10,764 lbs = 27,636 lbs

127,386 lbs

1ST FLOOR: 5.3 psf 2ND + 3RD : 3.3 psf 4TH : 3.3 psf

COLUMNS

1ST + 2ND FLOOR → Interior : 7,380 lbs
Exterior : 9,450 lbs
Corner : 3,600 lbs
20,430 lbs = 1.2 psf

3RD + 4TH FLOOR → Interior : 6,000 lbs
Exterior : 8,100 lbs
Corner : 1,800 lbs
15,900 lbs = .95 psf

CONCRETE SLAB

· 4.5 in. slab Area = 56' x 138' = 7728'
Atypical = 1/2 (56')(21.33) = 597.33'
Total Area = 8325.33'

8325 ft² (.375')(150 lb/ft³) = 468,280 lbs
LOAD = 56.2 psf / Floor
TOTAL = 225 psf

APPENDIX B - CE 3010 BOOK

Masonry (per 4in. of thickness)

BRICK (8in.) = 38.0 psf × 2 = 76.0 psf

GYPSUM = 5.0 psf

WINDOWS = 8.0 psf

1ST FLOOR TOTAL = 5.3 psf
.6 psf
56.2 psf
76.0 psf
5.0 psf
8.0 psf
151.1 psf

2ND FLOOR TOTAL = 1.7 psf
.6 psf
56.2 psf
76.0 psf
5.0 psf
8.0 psf
147.5 psf

3RD FLOOR TOTAL = 1.7 psf
.48 psf
56.2 psf
76.0 psf
5.0 psf
8.0 psf
147.4 psf

4TH FLOOR TOTAL = 3.3 psf
.48 psf
56.2 psf
76.0 psf
5.0 psf
8.0 psf
148.9 psf

•Seismic Base Shear

$V = C_s W$

$C_s = \frac{1.2 A_v S}{R T^{2/3}}$ → $A_v = 1.2g$ → $1.2(32.2\ ft/s^2) = 38.6\ ft/s^2$

$S = 1.2$ $T = .613$

$R = 4.5$

$T = C_T H_n^{3/4}$

$T = (.03)(55.83')^{3/4}$

$T = .613$

$C_s = \frac{(1.2)(38.6)(1.2)}{(4.5)(.613)^{2/3}} = 17.1$

$V = (17.1)(594.9\ psf)$

$V = 10164.24\ psf$

• Vertical Distribution of Seismic Forces $F_x = C_{vx} V$

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

1st FLOOR: $w_x = 151\ psf$

$h_x = 13.67'$

period	k
.5	1
.613	1.06
2.5	2

TribArea: $\frac{13.5 + 13.75}{2} = 13.63'$

$C_{vx} = \frac{(151)(13.67)^{1.06}}{(595)(49.14')^{1.06}} = .065$

$\sum W_i h_i$ =
151 × 13.67 = 2064.1
148 × 13.71 = 2029.1
147 × 14.3 = 2102.1
149 × 7.46 = 1111.5

595 49.14 7306.8

$F_x = C_{vx} V$

$F_x = (.065)(10164.24\ psf)$

$F_x = 660\ psf$

2ND FLOOR   Trib. Area = $\frac{13.75 \times 13.67}{2}$ = 13.71

$h_x = 13.71'$

$W_x = 148$

$$C_{vx} = \frac{148\,(13.71)^{1.06}}{(595)(49.14)^{1.06}} = .064$$

$F_x = (.064)(10164.24 \text{ psf})$

$F_x = 650.5$ psf

3RD FLOOR   Trib. Area $\frac{13.67 + 14.92}{2} = 14.3$

$W_x = 147$

$h_x = 14.3'$

$$C_{vx} = \frac{(147)(14.3)^{1.06}}{(595)(49.14)^{1.06}} = .067$$

$F_x = .067\,(10164.24) = 681$ psf

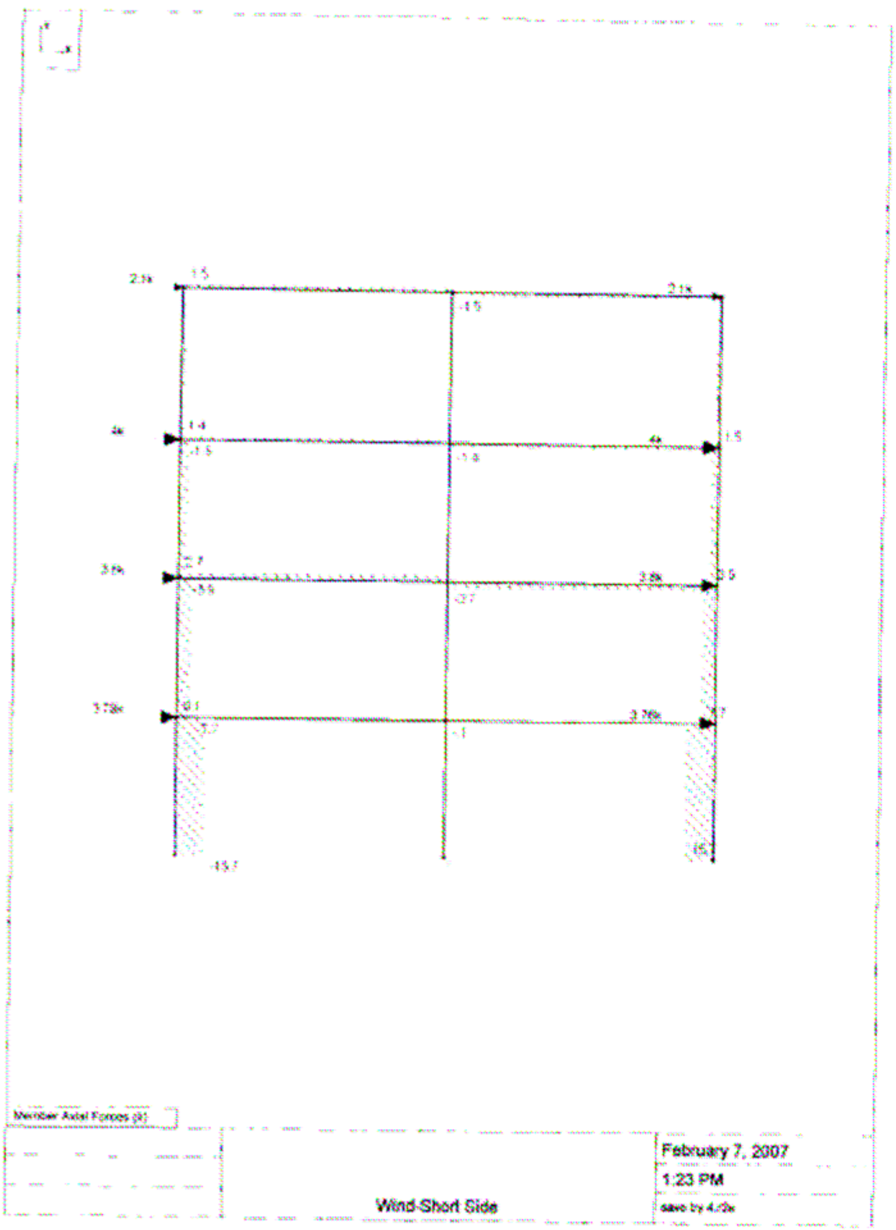
4TH FLOOR $\frac{14.92}{2} = 7.46$

$W_x = 149$

$h_x = 7.46$

$$C_{vx} = \frac{(149)(7.46)^{1.06}}{(595)(49.14)^{1.06}} = .033$$

$F_x = .033\,(10164.24) = 335$ psf



Member Axial Forces (k)
February 7, 2007
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Wind-Short Side

Designer :

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Joint Displacements

Joint Label	X Translation (in)	Y Translation (in)	Rotation (radians)
N1	0	0	-3.679e-3
N2	0	0	-3.855e-3
N3	0	0	-3.679e-3
N4	.525	.002	-2.355e-3
N5	.524	0	-2.002e-3
N6	.525	-.002	-2.355e-3
N7	.883	.003	-1.738e-3
N8	.88	0	-1.677e-3
N9	.883	-.003	-1.738e-3
N10	1.134	.004	-1.149e-3
N11	1.133	0	-1.085e-3
N12	1.134	-.004	-1.149e-3
N13	1.306	.004	-7.127e-4
N14	1.304	0	-6.167e-4
N15	1.306	-.004	-7.127e-4

Reactions

Joint Label	X Force (k)	Y Force (k)	Moment (k-ft)
N1	-8.05	-15.701	0
N2	-11.26	0	0
N3	-8.05	15.701	0
Totals:	-27.36	0	

Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
M1	1	-15.701	8.05	0
	2	-15.701	8.05	27.17
	3	-15.701	8.05	54.339
	4	-15.701	8.05	81.509
	5	-15.701	8.05	108.678
M2	1	-7.724	4.338	-4.985
	2	-7.724	4.338	9.927
	3	-7.724	4.338	24.839
	4	-7.724	4.338	39.751
	5	-7.724	4.338	54.662
M3	1	15.701	8.05	0
	2	15.701	8.05	27.17
	3	15.701	8.05	54.339
	4	15.701	8.05	81.509
	5	15.701	8.05	108.678
M4	1	7.724	4.338	-4.985
	2	7.724	4.338	9.927

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Designer :

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Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
	3	7.724	4.338	24.839
	4	7.724	4.338	39.751
	5	7.724	4.338	54.662
M5	1	0	11.28	0
	2	0	11.28	38.001
	3	0	11.28	76.002
	4	0	11.28	114.003
	5	0	11.28	152.003
M6	1	0	11.124	-63.359
	2	0	11.124	-25.12
	3	0	11.124	13.119
	4	0	11.124	51.358
	5	0	11.124	89.597
M7	1	-3.93	3.245	1.702
	2	-3.93	3.245	12.792
	3	-3.93	3.245	23.881
	4	-3.93	3.245	34.971
	5	-3.93	3.245	46.061
M8	1	-1.459	.62	11.448
	2	-1.459	.62	13.771
	3	-1.459	.62	16.095
	4	-1.459	.62	18.418
	5	-1.459	.62	20.742
M9	1	3.93	3.245	1.702
	2	3.93	3.245	12.792
	3	3.93	3.245	23.881
	4	3.93	3.245	34.971
	5	3.93	3.245	46.061
M10	1	1.459	.62	11.448
	2	1.459	.62	13.771
	3	1.459	.62	16.095
	4	1.459	.62	18.418
	5	1.459	.62	20.742
M11	1	0	5.71	-15.057
	2	0	5.71	4.457
	3	0	5.71	23.972
	4	0	5.71	43.486
	5	0	5.71	63
M12	1	0	2.961	-4.912
	2	0	2.961	6.191
	3	0	2.961	17.294
	4	0	2.961	28.397
	5	0	2.961	39.5
M13	1	.068	-7.976	113.663
	2	.068	-7.976	58.327
	3	.068	-7.976	2.991
	4	.068	-7.976	-52.345
	5	.068	-7.976	-107.681
M14	1	-.068	-7.976	107.681
	2	-.068	-7.976	52.345
	3	-.068	-7.976	-2.991
	4	-.068	-7.976	-58.327
	5	-.068	-7.976	-113.663

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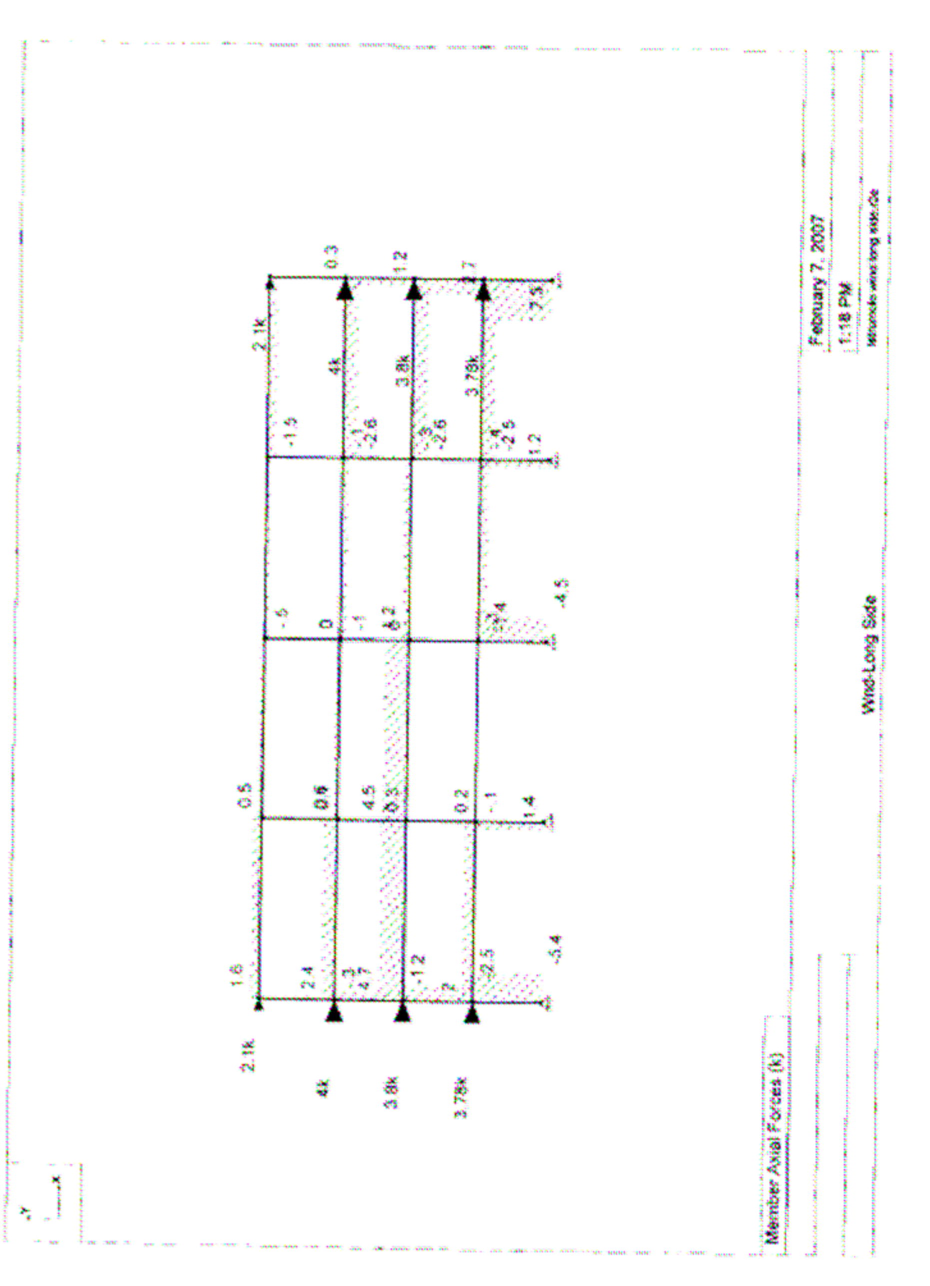
Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
M15	1	2.707	-3.794	52.96
	2	2.707	-3.794	26.638
	3	2.707	-3.794	.317
	4	2.707	-3.794	-26.005
	5	2.707	-3.794	-52.327
M16	1	-2.707	-3.794	52.327
	2	-2.707	-3.794	26.005
	3	-2.707	-3.794	-.317
	4	-2.707	-3.794	-26.638
	5	-2.707	-3.794	-52.96
M17	1	1.375	-2.471	34.613
	2	1.375	-2.471	17.47
	3	1.375	-2.471	.328
	4	1.375	-2.471	-16.814
	5	1.375	-2.471	-33.956
M18	1	-1.375	-2.471	33.956
	2	-1.375	-2.471	16.814
	3	-1.375	-2.471	-.328
	4	-1.375	-2.471	-17.47
	5	-1.375	-2.471	-34.613
M19	1	1.48	-1.459	20.742
	2	1.48	-1.459	10.619
	3	1.48	-1.459	.496
	4	1.48	-1.459	-9.627
	5	1.48	-1.459	-19.75
M20	1	-1.48	-1.459	19.75
	2	-1.48	-1.459	9.827
	3	-1.48	-1.459	-.496
	4	-1.48	-1.459	-10.619
	5	-1.48	-1.459	-20.742


February 7, 2007
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Wind-Long Side
Member Axial Forces (k)

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Joint Displacements

Joint Label	X Translation (in)	Y Translation (in)	Rotation (radians)
N1	0	0	-1.095e-2
N2	0	0	-1.092e-2
N3	0	0	-1.204e-2
N4	0	0	-1.224e-2
N5	0	0	-1.214e-2
N6	1.335	.006	-2.813e-3
N7	1.332	0	-2.826e-3
N8	1.332	.003	-5.864e-4
N9	1.333	0	-2.041e-4
N10	1.334	-.009	-4.128e-4
N11	1.898	.009	-2.855e-3
N12	1.889	0	-2.254e-3
N13	1.881	.003	-2.845e-3
N14	1.879	0	-2.858e-3
N15	1.884	-.012	-3.345e-3
N16	2.72	.011	-2.408e-3
N17	2.715	-.001	-1.419e-3
N18	2.714	.003	-1.524e-3
N19	2.716	0	-1.387e-3
N20	2.721	-.014	-2.382e-3
N21	3.142	.012	-7.99e-4
N22	3.139	-.001	-4.586e-4
N23	3.138	.003	-5.01e-4
N24	3.139	0	-4.7e-4
N25	3.142	-.014	-7.997e-4

Reactions

Joint Label	X Force (k)	Y Force (k)	Moment (k-ft)
N1	-3.058	-5.394	0
N2	-5.099	1.387	0
N3	-7.216	-4.479	0
N4	-7.58	1.176	0
N5	-4.407	7.33	0
Totals:	-27.36	0	

Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
M1	1	-5.394	3.058	0
	2	-5.394	3.058	10.321
	3	-5.394	3.058	20.643
	4	-5.394	3.058	30.964
	5	-5.394	3.058	41.286

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Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
M2	1	-2.492	1.256	-8.736
	2	-2.492	1.256	-4.419
	3	-2.492	1.256	-.103
	4	-2.492	1.256	4.214
	5	-2.492	1.256	8.531
M3	1	7.33	4.407	0
	2	7.33	4.407	14.874
	3	7.33	4.407	29.747
	4	7.33	4.407	44.621
	5	7.33	4.407	59.495
M4	1	2.749	3.159	-29.023
	2	2.749	3.159	-18.162
	3	2.749	3.159	-7.302
	4	2.749	3.159	3.558
	5	2.749	3.159	14.419
M5	1	1.367	5.099	0
	2	1.367	5.099	17.21
	3	1.367	5.099	34.419
	4	1.367	5.099	51.629
	5	1.367	5.099	68.839
M6	1	.24	3.045	-18.549
	2	.24	3.045	-8.081
	3	.24	3.045	2.387
	4	.24	3.045	12.854
	5	.24	3.045	23.322
M7	1	-4.479	7.216	0
	2	-4.479	7.216	24.353
	3	-4.479	7.216	48.706
	4	-4.479	7.216	73.059
	5	-4.479	7.216	97.412
M8	1	-.121	5.864	-49.741
	2	-.121	5.864	-29.583
	3	-.121	5.864	-9.426
	4	-.121	5.864	10.731
	5	-.121	5.864	30.889
M9	1	1.176	7.58	0
	2	1.176	7.58	25.582
	3	1.176	7.58	51.164
	4	1.176	7.58	76.746
	5	1.176	7.58	102.328
M10	1	-.376	6.476	-55.598
	2	-.376	6.476	-33.338
	3	-.376	6.476	-11.078
	4	-.376	6.476	11.182
	5	-.376	6.476	33.442
M11	1	.254	2.85	-18.634
	2	.254	2.85	-8.893
	3	.254	2.85	.848
	4	.254	2.85	10.588
	5	.254	2.85	20.329
M12	1	.054	1.049	-6.974
	2	.054	1.049	-3.042
	3	.054	1.049	.89

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Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
	4	.054	1.049	4.622
	5	.054	1.049	8.754
M13	1	.021	2.579	-16.289
	2	.021	2.579	-7.474
	3	.021	2.579	1.34
	4	.021	2.579	10.155
	5	.021	2.579	18.969
M14	1	.005	.994	-6.51
	2	.005	.994	-2.782
	3	.005	.994	.946
	4	.005	.994	4.675
	5	.005	.994	8.403
M15	1	-.261	2.656	-16.66
	2	-.261	2.656	-7.583
	3	-.261	2.656	1.493
	4	-.261	2.656	10.57
	5	-.261	2.656	19.647
M16	1	-.061	1.053	-7.049
	2	-.061	1.053	-3.1
	3	-.061	1.053	.848
	4	-.061	1.053	4.797
	5	-.061	1.053	8.745
M17	1	-1.234	2.12	-14.035
	2	-1.234	2.12	-6.791
	3	-1.234	2.12	.454
	4	-1.234	2.12	7.698
	5	-1.234	2.12	14.943
M18	1	-.297	.549	-2.632
	2	-.297	.549	-.572
	3	-.297	.549	1.488
	4	-.297	.549	3.548
	5	-.297	.549	5.608
M19	1	1.22	1.995	-12.657
	2	1.22	1.995	-5.839
	3	1.22	1.995	.978
	4	1.22	1.995	7.795
	5	1.22	1.995	14.612
M20	1	.299	.555	-2.699
	2	.299	.555	-.618
	3	.299	.555	1.464
	4	.299	.555	3.545
	5	.299	.555	5.626
M21	1	-1.428	-6.133	123.197
	2	-1.428	-6.133	70.3
	3	-1.428	-6.133	17.404
	4	-1.428	-6.133	-35.493
	5	-1.428	-6.133	-88.39
M22	1	-2.532	-4.581	69.537
	2	-2.532	-4.581	30.023
	3	-2.532	-4.581	-9.49
	4	-2.532	-4.581	-49.004
	5	-2.532	-4.581	-88.517
M23	1	1.978	-2.902	50.022

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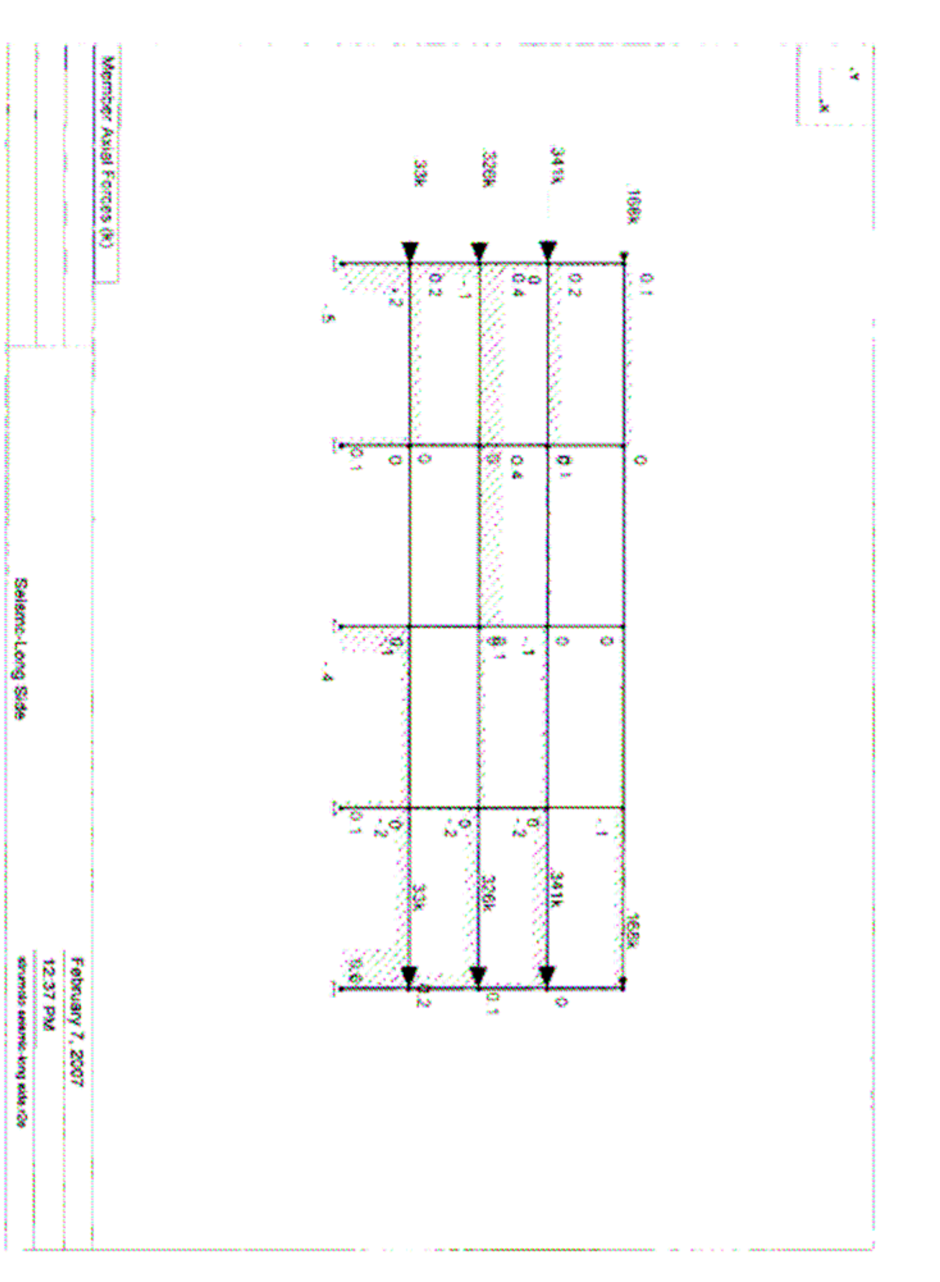
Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
	2	1.978	-2.902	24.992
	3	1.978	-2.902	-.037
	4	1.978	-2.902	-25.067
	5	1.975	-2.902	-50.097
M24	1	-.076	-1.775	37.291
	2	-.076	-1.775	21.979
	3	-.078	-1.775	6.667
	4	-.076	-1.775	-8.645
	5	-.076	-1.775	-23.956
M25	1	4.664	-1.258	22.566
	2	4.664	-1.258	11.712
	3	4.664	-1.258	.859
	4	4.664	-1.258	-9.995
	5	4.664	-1.258	-20.849
M26	1	4.469	-1.273	21.107
	2	4.469	-1.273	10.132
	3	4.469	-1.273	-.844
	4	4.469	-1.273	-11.82
	5	4.469	-1.273	-22.796
M27	1	1.184	-1.415	24.382
	2	1.184	-1.415	12.181
	3	1.184	-1.415	-.019
	4	1.184	-1.415	-12.22
	5	1.184	-1.415	-24.42
M28	1	-2.635	-1.529	25.682
	2	-2.635	-1.529	12.493
	3	-2.635	-1.529	-.697
	4	-2.635	-1.529	-13.886
	5	-2.635	-1.529	-27.075
M29	1	2.43	-.937	17.575
	2	2.43	-.937	9.494
	3	2.43	-.937	1.414
	4	2.43	-.937	-6.667
	5	2.43	-.937	-14.747
M30	1	.628	-.737	12.556
	2	.628	-.737	6.203
	3	.628	-.737	-.15
	4	.628	-.737	-6.503
	5	.628	-.737	-12.856
M31	1	-.957	-.72	12.624
	2	-.957	-.72	6.41
	3	-.957	-.72	.196
	4	-.957	-.72	-6.017
	5	-.957	-.72	-12.231
M32	1	-2.56	-.921	14.465
	2	-2.56	-.921	6.521
	3	-2.56	-.921	-1.423
	4	-2.56	-.921	-9.367
	5	-2.56	-.921	-17.311
M33	1	1.551	-.297	5.608
	2	1.551	-.297	3.049
	3	1.551	-.297	.49
	4	1.551	-.297	-2.07

RISA-2D Educational Version 1.0 [C:\...\Katie\Desktop\strumolo wind long side.r2e] Page 4


Member Axial Forces (k)
Seismic-Long Side
February 7, 2007
12:37 PM

Designer :

Checked By:_____

Joint Displacements

Joint Label	X Translation (in)	Y Translation (in)	Rotation (radians)
N1	0	0	-9.323e-4
N2	0	0	-9.295e-4
N3	0	0	-1.025e-3
N4	0	0	-1.041e-3
N5	0	0	-1.033e-3
N6	.114	0	-2.387e-4
N7	.113	0	-2.397e-4
N8	.113	0	-4.974e-5
N9	.113	0	-1.73e-5
N10	.113	0	-3.499e-5
N11	.161	0	-2.394e-4
N12	.16	0	-1.891e-4
N13	.16	0	-2.391e-4
N14	.159	0	-2.403e-4
N15	.16	0	-2.811e-4
N16	.23	0	-1.99e-4
N17	.229	0	-1.173e-4
N18	.229	0	-1.259e-4
N19	.229	0	-1.146e-4
N20	.23	-.001	-1.958e-4
N21	.264	0	-6.409e-5
N22	.264	0	-3.697e-5
N23	.264	0	-4.038e-5
N24	.264	0	-3.811e-5
N25	.264	-.001	-8.415e-5

Reactions

Joint Label	X Force (k)	Y Force (k)	Moment (k-ft)
N1	-.261	-.453	0
N2	-.435	.115	0
N3	-.614	-.38	0
N4	-.645	.1	0
N5	-.375	.617	0
Totals:	-2.329	0	

Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
M1	1	-.453	.261	0
	2	-.453	.261	.879
	3	-.453	.261	1.759
	4	-.453	.261	2.638
	5	-.453	.261	3.518

Designer :

Checked By:_____

Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
M2	1	-.207	.105	-.726
	2	-.207	.105	-.364
	3	-.207	.105	-.002
	4	-.207	.105	.36
	5	-.207	.105	.723
M3	1	.617	.375	0
	2	.617	.375	1.266
	3	.617	.375	2.531
	4	.617	.375	3.797
	5	.617	.375	5.062
M4	1	.229	.267	-2.448
	2	.229	.267	-1.53
	3	.229	.267	-.613
	4	.229	.267	.305
	5	.229	.267	1.223
M5	1	.115	.435	0
	2	.115	.435	1.467
	3	.115	.435	2.933
	4	.115	.435	4.4
	5	.115	.435	5.866
M6	1	.019	.256	-1.546
	2	.019	.256	-.667
	3	.019	.256	.211
	4	.019	.256	1.089
	5	.019	.256	1.968
M7	1	-.38	.614	0
	2	-.38	.614	2.072
	3	-.38	.614	4.145
	4	-.38	.614	6.217
	5	-.38	.614	8.289
M8	1	-.01	.495	-4.191
	2	-.01	.495	-2.491
	3	-.01	.495	-.791
	4	-.01	.495	.91
	5	-.01	.495	2.61
M9	1	.1	.645	0
	2	.1	.645	2.177
	3	.1	.645	4.353
	4	.1	.645	6.53
	5	.1	.645	8.706
M10	1	-.031	.547	-4.688
	2	-.031	.547	-2.81
	3	-.031	.547	-.931
	4	-.031	.547	.948
	5	-.031	.547	2.826
M11	1	.021	.238	-1.553
	2	.021	.238	-.74
	3	.021	.238	.073
	4	.021	.238	.886
	5	.021	.238	1.699
M12	1	.004	.084	-.558
	2	.004	.084	-.242
	3	.004	.084	.074

Designer :

Checked By:_____

Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
	4	.004	.084	.39
	5	.004	.084	.706
M13	1	.002	.215	-1.355
	2	.002	.215	-.62
	3	.002	.215	.115
	4	.002	.215	.85
	5	.002	.215	1.585
M14	1	0	.08	-.52
	2	0	.08	-.221
	3	0	.08	.079
	4	0	.08	.379
	5	0	.08	.678
M15	1	-.021	.221	-1.386
	2	-.021	.221	-.629
	3	-.021	.221	.128
	4	-.021	.221	.884
	5	-.021	.221	1.641
M16	1	-.005	.085	-.564
	2	-.005	.085	-.247
	3	-.005	.085	.071
	4	-.005	.085	.388
	5	-.005	.085	.705
M17	1	-.101	.177	-1.17
	2	-.101	.177	-.564
	3	-.101	.177	.041
	4	-.101	.177	.646
	5	-.101	.177	1.252
M18	1	-.024	.043	-.2
	2	-.024	.043	-.038
	3	-.024	.043	.125
	4	-.024	.043	.287
	5	-.024	.043	.45
M19	1	.1	.167	-1.053
	2	.1	.167	-.484
	3	.1	.167	.086
	4	.1	.167	.655
	5	.1	.167	1.224
M20	1	.024	.044	-.206
	2	.024	.044	-.042
	3	.024	.044	.123
	4	.024	.044	.287
	5	.024	.044	.452
M21	1	-.124	-.52	10.449
	2	-.124	-.52	5.963
	3	-.124	-.52	1.477
	4	-.124	-.52	-3.009
	5	-.124	-.52	-7.495
M22	1	-.222	-.389	5.899
	2	-.222	-.389	2.547
	3	-.222	-.389	-.805
	4	-.222	-.389	-4.158
	5	-.222	-.389	-7.51
M23	1	.175	-.246	4.244

Designer :

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Member Section Forces

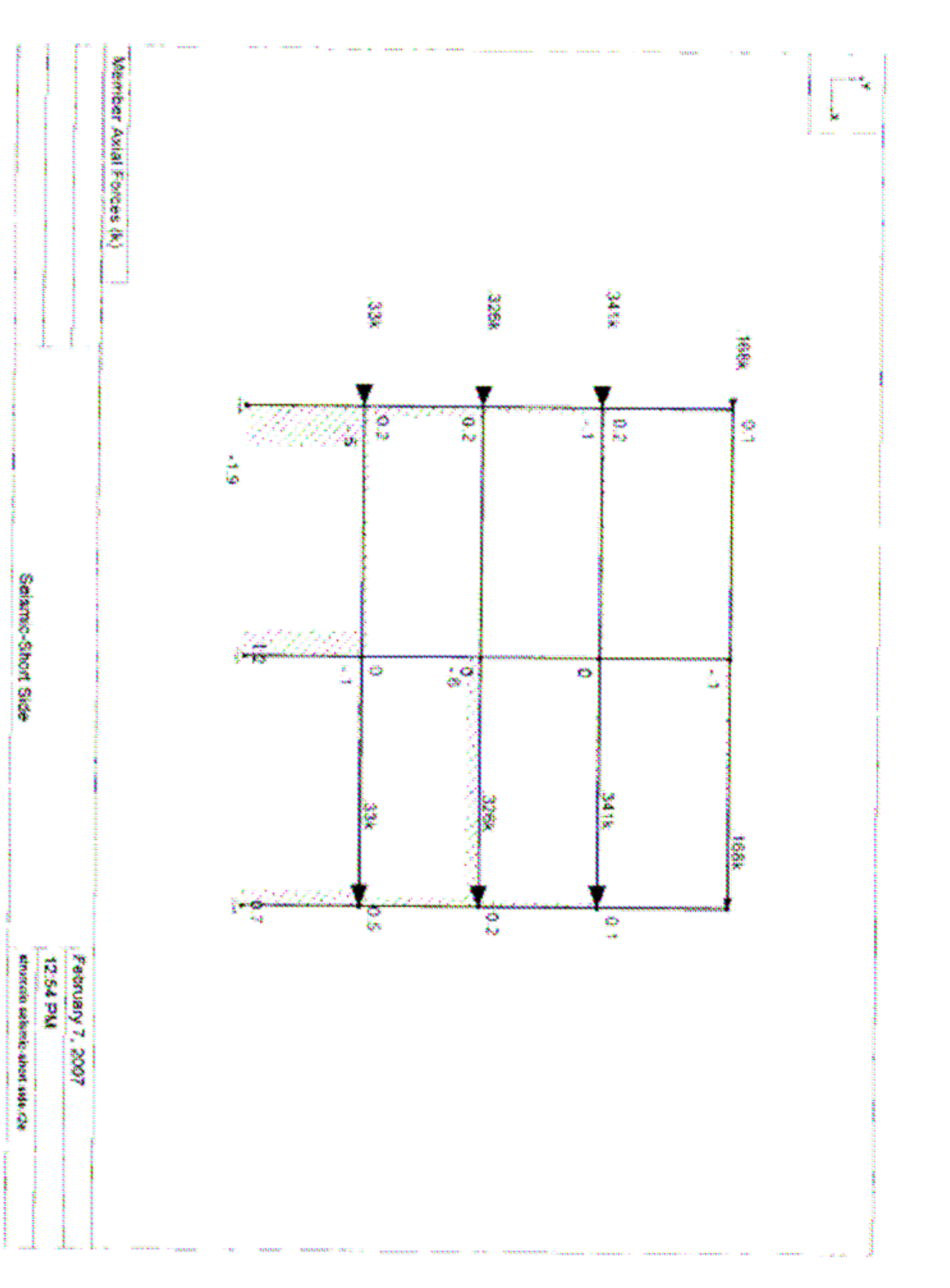
Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
	2	.175	-.246	2.12
	3	.175	-.246	-.003
	4	.175	-.246	-2.126
	5	.175	-.246	-4.249
M24	1	-.004	-.151	3.163
	2	-.004	-.151	1.864
	3	-.004	-.151	.565
	4	-.004	-.151	-.733
	5	-.004	-.151	-2.032
M25	1	.397	-.106	1.893
	2	.397	-.106	.982
	3	.397	-.106	.072
	4	.397	-.106	-.838
	5	.397	-.106	-1.749
M26	1	.38	-.107	1.772
	2	.38	-.107	.85
	3	.38	-.107	-.072
	4	.38	-.107	-.993
	5	.38	-.107	-1.915
M27	1	.1	-.119	2.05
	2	.1	-.119	1.024
	3	.1	-.119	-.002
	4	.1	-.119	-1.027
	5	.1	-.119	-2.053
M28	1	-.225	-.129	2.159
	2	-.225	-.129	1.05
	3	-.225	-.129	-.058
	4	-.225	-.129	-1.167
	5	-.225	-.129	-2.275
M29	1	.207	-.077	1.452
	2	.207	-.077	.785
	3	.207	-.077	.117
	4	.207	-.077	-.551
	5	.207	-.077	-1.219
M30	1	.054	-.061	1.038
	2	.054	-.061	.513
	3	.054	-.061	-.012
	4	.054	-.061	-.537
	5	.054	-.061	-1.062
M31	1	-.082	-.06	1.043
	2	-.082	-.06	.529
	3	-.082	-.06	.016
	4	-.082	-.06	-.497
	5	-.082	-.06	-1.01
M32	1	-.218	-.076	1.195
	2	-.218	-.076	.539
	3	-.218	-.076	-.118
	4	-.218	-.076	-.774
	5	-.218	-.076	-1.43
M33	1	.125	-.024	.45
	2	.125	-.024	.244
	3	.125	-.024	.039
	4	.125	-.024	-.167

Designer :

Checked By:_____

Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
	5	.125	-.024	-.372
M34	1	.04	-.02	.334
	2	.04	-.02	.164
	3	.04	-.02	-.005
	4	.04	-.02	-.174
	5	.04	-.02	-.344
M35	1	-.04	-.019	.335
	2	-.04	-.019	.169
	3	-.04	-.019	.003
	4	-.04	-.019	-.163
	5	-.04	-.019	-.328
M36	1	-.124	-.024	.377
	2	-.124	-.024	.17
	3	-.124	-.024	-.037
	4	-.124	-.024	-.244
	5	-.124	-.024	-.452


Member Axial Forces (k)
Seismic-Short Side
February 7, 2007
12:54 PM
341k
326k
33k
168k

Joint Loads/Enforced Displacements

Joint Label	[L]oad or [D]isplacement	Direction	Magnitude (k, k-ft, in, rad)
N4	L	X	.33
N6	L	X	.33
N7	L	X	.326
N9	L	X	.326
N10	L	X	.341
N12	L	X	.341
N13	L	X	.168
N15	L	X	.168

Joint Displacements

Joint Label	X Translation (in)	Y Translation (in)	Rotation (radians)
N1	0	0	-6.576e-4
N2	0	0	-6.257e-4
N3	0	0	-5.398e-4
N4	.073	0	-2.937e-5
N5	.073	0	-9.19e-5
N6	.073	0	-2.65e-4
N7	.115	0	-3.054e-4
N8	.115	0	-2.814e-4
N9	.116	0	-2.496e-4
N10	.164	0	-2.188e-4
N11	.164	0	-1.946e-4
N12	.164	0	-2.277e-4
N13	.197	0	-1.229e-4
N14	.197	0	-9.933e-5
N15	.197	0	-1.201e-4

Designer :

Checked By:______

Reactions

Joint Label	X Force (k)	Y Force (k)	Moment (k-ft)
N1	-.741	-1.905	0
N2	-1.262	1.169	0
N3	-.324	.736	0
Totals:	-2.328	0	

Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
M13	1	.211	-1.44	16.446
	2	.211	-1.44	6.453
	3	.211	-1.44	-3.54
	4	.211	-1.44	-13.533
	5	.211	-1.44	-23.526
M14	1	-.057	-.286	3.324
	2	-.057	-.286	1.342
	3	-.057	-.286	-.641
	4	-.057	-.286	-2.623
	5	-.057	-.286	-4.606
M15	1	-.046	-.224	3.15
	2	-.046	-.224	1.597
	3	-.046	-.224	.043
	4	-.046	-.224	-1.511
	5	-.046	-.224	-3.065
M16	1	-.637	-.204	2.886
	2	-.637	-.204	1.471
	3	-.637	-.204	.057
	4	-.637	-.204	-1.358
	5	-.637	-.204	-2.773
M17	1	.159	-.157	2.224
	2	.159	-.157	1.133
	3	.159	-.157	.043
	4	.159	-.157	-1.047
	5	.159	-.157	-2.138
M18	1	-.036	-.162	2.19
	2	-.036	-.162	1.065

RISA-2D Educational Version 1.0 [C:\...\Katie\Desktop\strumolo seismic-short side.r2e] Page 2

Designer :

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Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
	3	-.036	-.162	-.059
	4	-.036	-.162	-1.183
	5	-.036	-.162	-2.307
M19	1	.099	-.084	1.203
	2	.099	-.084	.622
	3	.099	-.084	.042
	4	.099	-.084	-.538
	5	.099	-.084	-1.119
M20	1	-.11	-.084	1.129
	2	-.11	-.084	.546
	3	-.11	-.084	-.037
	4	-.11	-.084	-.62
	5	-.11	-.084	-1.203
M9	1	-1.905	.741	0
	2	-1.905	.741	2.502
	3	-1.905	.741	5.004
	4	-1.905	.741	7.507
	5	-1.905	.741	10.009
M10	1	-.465	.622	-6.438
	2	-.465	.622	-4.299
	3	-.465	.622	-2.159
	4	-.465	.622	-.02
	5	-.465	.622	2.119
M11	1	-.241	.251	-1.031
	2	-.241	.251	-.175
	3	-.241	.251	.682
	4	-.241	.251	1.538
	5	-.241	.251	2.395
M12	1	-.084	.069	.171
	2	-.084	.069	.429
	3	-.084	.069	.687
	4	-.084	.069	.945
	5	-.084	.069	1.203
M13B	1	.736	.324	0
	2	.736	.324	1.094
	3	.736	.324	2.189
	4	.736	.324	3.283

Designer :

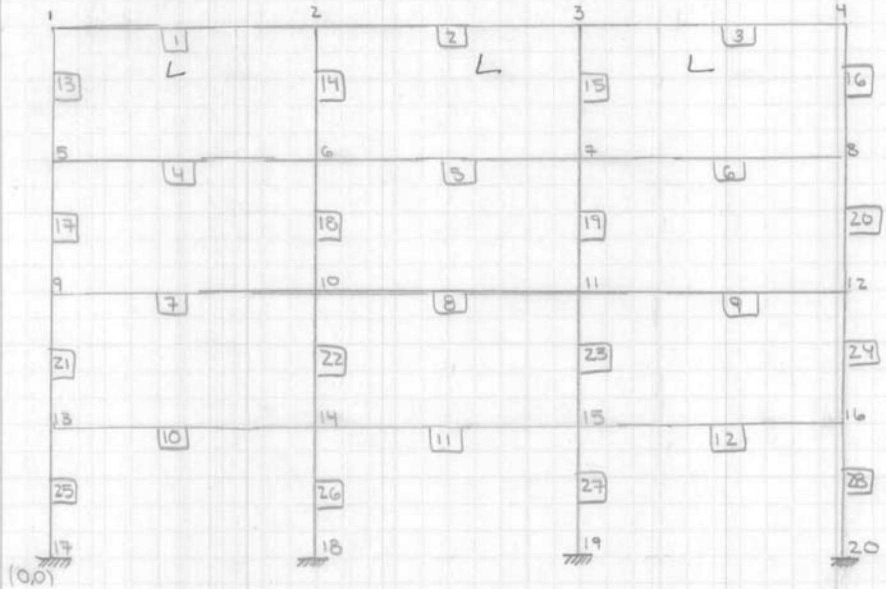
Checked By:_____

Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
	5	.736	.324	4.377
M14B	1	.45	.051	-.229
	2	.45	.051	-.054
	3	.45	.051	.121
	4	.45	.051	.296
	5	.45	.051	.471
M15A	1	.246	.362	-2.303
	2	.246	.362	-1.065
	3	.246	.362	.172
	4	.246	.362	1.409
	5	.246	.362	2.647
M16B	1	.084	.058	.339
	2	.084	.058	.555
	3	.084	.058	.771
	4	.084	.058	.987
	5	.084	.058	1.203
M17A	1	-.005	.404	-2.081
	2	-.005	.404	-.699
	3	-.005	.404	.683
	4	-.005	.404	2.064
	5	-.005	.404	3.446
M18B	1	0	.209	-.882
	2	0	.209	-.099
	3	0	.209	.683
	4	0	.209	1.465
	5	0	.209	2.248
M19A	1	1.169	1.262	0
	2	1.169	1.262	4.261
	3	1.169	1.262	8.521
	4	1.169	1.262	12.782
	5	1.169	1.262	17.042
M20A	1	.015	.995	-9.808
	2	.015	.995	-6.389
	3	.015	.995	-2.969
	4	.015	.995	.451
	5	.015	.995	3.87

Concrete

Short



Coordinates

1 0,670

2 216,670

3 444 670

4 660,670

5 0,491

6 216,491

7 444,491

8 660,491

9 0 327

10 216,327

11 444,327

12 660 ,327

13 0 162

14 216 ,162

15 444,162

16 660,162

17 0,0

18 216,0

19 444,0

20 660,0

216|12 = 18

444|12 = 37

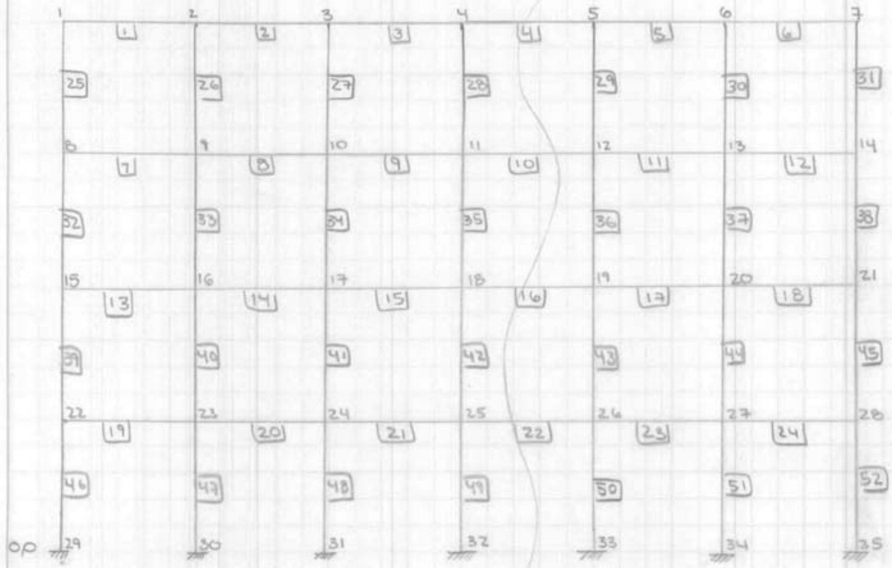
660|12 = 55

670|12 = 55.8

491|12 = 40.9

327|12 = 27.3

162|12 = 13.5



Coordinates

1 0,670

2 252,670 21

3 504,670 42

4 756,670 63

5 1008,670 84

6 1260,670 105

7 1512,670 126

8 0,491

9 252,491

10 504,491

11 756,491

12 1008,491

13 1260,491

14 1512,491

15 0,327

16 252,327

17 504,327

18 756,327

19 1008,327

20 1260,327

21 1512,327

22 0,162

23 252,162

24 504,162

25 756,162

26 1008,162

27 1260,162

28 1512,162

29 0,0

30 252,0

31 504,0

32 756,0

33 1008,0

34 1260,0

35 1512,0

Continuous T-Beam

Moment of Interia

$I = bh^3$

$= 14 \times (11.5)^3$

$= 21{,}292$

Area

$A = bh$

$= 14 \times 11.5$

$= 161$

Girder

Moment of Inertia

$I = bh^3$

$= 18 \times 30^3$

$= 486{,}000 \text{ in}^4$

$A = bh$

$= 18 \times 30$

$= 540 \text{ in}^2$

Column

Moment of Inertia

$I = 18 \times (18)^3$

$= 104{,}976$

$A = bh$

$= 18 \times 18$

$= 324$

Two-Way Slab

Column

Moment of Inertia

$I = bh^3$

$= 18 \times 18^3$

$= 104.976$

$A = bh$

$= 18 \times 18$

$= 324$

One Way

Girder

Moment of Inertia

$I = bh^3$

$= 15 \times 25^3$

$= 234375 \text{ in}^4$

$A = bh$

$= 15 \times 25$

$= 375$

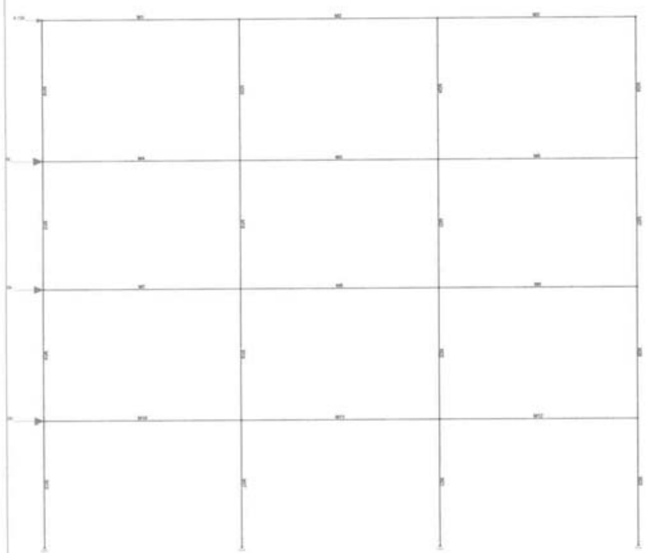
90 mph (IBC) = 970 Pa

conversion factor 4.7888026 E 01

970 Pa / 4.78880 26 E01 = 20.26 lb/ft^2

= 0.02 K/ft

Floor	Area		force
4	416.25	0.02	8.325
3	793.65	0.02	15.873
2	760.91	0.02	15.22
1	756.2	0.02	15.12



	Long Symmetry	February 1, 2007
		9:21 PM
		long symmetry.r2e

Designer :

February 1, 2007
9:21 PM
Checked By:_____

Joint Displacements

Joint Label	X Translation (in)	Y Translation (in)	Rotation (radians)
N1	0	0	-3.017e-4
N2	0	0	-3.054e-4
N3	0	0	-3.046e-4
N4	0	0	-2.993e-4
N5	.033	0	-1.338e-5
N6	.033	0	-3.636e-6
N7	.033	0	-3.645e-6
N8	.033	0	-1.331e-5
N9	.041	0	-6.955e-6
N10	.04	0	-1.999e-6
N11	.04	0	-2.009e-6
N12	.04	0	-6.964e-6
N13	.045	0	-4.562e-6
N14	.045	0	-1.334e-6
N15	.045	0	-1.335e-6
N16	.044	0	-4.681e-6
N17	.047	0	-2.707e-6
N18	.047	0	-7.536e-7
N19	.047	0	-7.556e-7
N20	.047	0	-2.849e-6

Reactions

Joint Label	X Force (k)	Y Force (k)	Moment (k-ft)
N1	-6.674	-14.879	0
N2	-6.987	3.427	0
N3	-6.968	-3.459	0
N4	-6.621	14.912	0
Totals:	-27.25	0	

Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
M1	1	3.337	-.184	6.321
	2	3.337	-.184	5.356
	3	3.337	-.184	4.391
	4	3.337	-.184	3.425
	5	3.337	-.184	2.46
M2	1	2.143	-1.09	11.438
	2	2.143	-1.09	5.717
	3	2.143	-1.09	-.005
	4	2.143	-1.09	-5.726
	5	2.143	-1.09	-11.447
M3	1	.923	-.231	-2.277
	2	.923	-.231	-3.491
	3	.923	-.231	-4.706
	4	.923	-.231	-5.92
	5	.923	-.231	-7.134
M4	1	6.013	-1.677	24.869

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Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
	2	6.013	-1.677	16.062
	3	6.013	-1.677	7.255
	4	6.013	-1.677	-1.552
	5	6.013	-1.677	-10.358
M5	1	3.899	-2.006	21.055
	2	3.899	-2.006	10.526
	3	3.899	-2.006	-.003
	4	3.899	-2.006	-10.532
	5	3.899	-2.006	-21.061
M6	1	1.813	-1.719	10.529
	2	1.813	-1.719	1.504
	3	1.813	-1.719	-7.521
	4	1.813	-1.719	-16.547
	5	1.813	-1.719	-25.572
M7	1	5.908	-3.698	49.964
	2	5.908	-3.698	30.552
	3	5.908	-3.698	11.14
	4	5.908	-3.698	-8.272
	5	5.908	-3.698	-27.684
M8	1	3.808	-3.076	32.279
	2	3.808	-3.076	16.128
	3	3.808	-3.076	-.023
	4	3.808	-3.076	-16.174
	5	3.808	-3.076	-32.325
M9	1	1.708	-3.689	27.602
	2	1.708	-3.689	8.232
	3	1.708	-3.689	-11.137
	4	1.708	-3.689	-30.506
	5	1.708	-3.689	-49.876
M10	1	5.318	-9.321	119.774
	2	5.318	-9.321	70.841
	3	5.318	-9.321	21.908
	4	5.318	-9.321	-27.025
	5	5.318	-9.321	-75.958
M11	1	3.74	-5.281	55.429
	2	3.74	-5.281	27.704
	3	3.74	-5.281	-.021
	4	3.74	-5.281	-27.747
	5	3.74	-5.281	-55.472
M12	1	2.178	-9.272	75.643
	2	2.178	-9.272	26.963
	3	2.178	-9.272	-21.716
	4	2.178	-9.272	-70.396
	5	2.178	-9.272	-119.076
M13	1	-14.879	6.674	0
	2	-14.879	6.674	22.526
	3	-14.879	6.674	45.053
	4	-14.879	6.674	67.579
	5	-14.879	6.674	90.105
M14	1	-5.559	4.442	-29.669
	2	-5.559	4.442	-14.343
	3	-5.559	4.442	.983
	4	-5.559	4.442	16.308
	5	-5.559	4.442	31.634
M15	1	-1.861	2.75	-18.33
	2	-1.861	2.75	-8.98
	3	-1.861	2.75	.371

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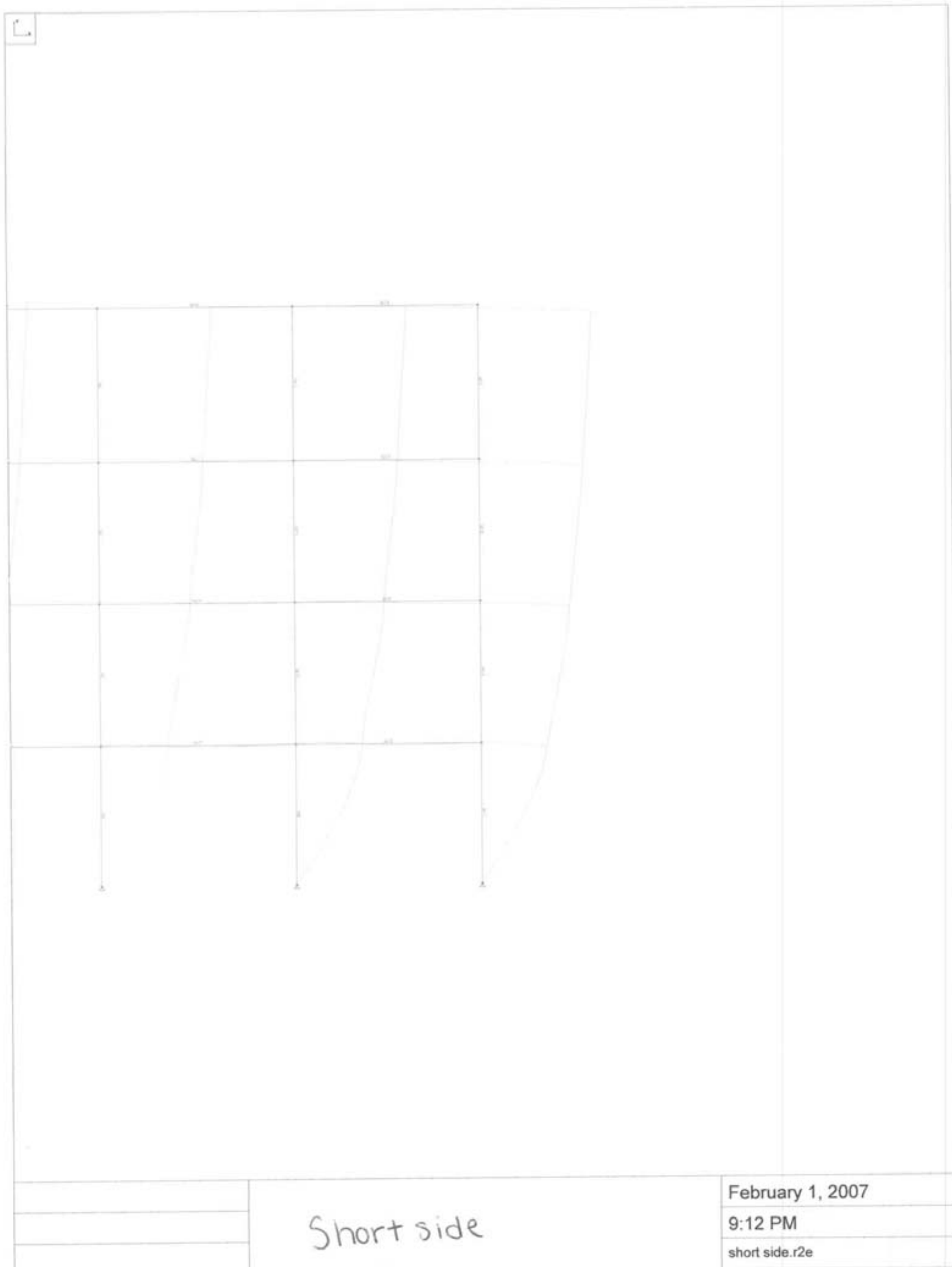
Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
	4	-1.861	2.75	9.722
	5	-1.861	2.75	19.073
M16	1	-.184	.813	-5.796
	2	-.184	.813	-2.767
	3	-.184	.813	.263
	4	-.184	.813	3.292
	5	-.184	.813	6.321
M17	1	3.427	6.987	0
	2	3.427	6.987	23.58
	3	3.427	6.987	47.159
	4	3.427	6.987	70.739
	5	3.427	6.987	94.319
M18	1	-.613	5.409	-37.069
	2	-.613	5.409	-18.409
	3	-.613	5.409	.25
	4	-.613	5.409	18.91
	5	-.613	5.409	37.569
M19	1	-1.234	3.308	-22.394
	2	-1.234	3.308	-11.145
	3	-1.234	3.308	.103
	4	-1.234	3.308	11.352
	5	-1.234	3.308	22.6
M20	1	-.906	1.194	-8.814
	2	-.906	1.194	-4.366
	3	-.906	1.194	.082
	4	-.906	1.194	4.53
	5	-.906	1.194	8.978
M21	1	-3.459	6.968	0
	2	-3.459	6.968	23.516
	3	-3.459	6.968	47.032
	4	-3.459	6.968	70.548
	5	-3.459	6.968	94.063
M22	1	.532	5.406	-37.051
	2	.532	5.406	-18.401
	3	.532	5.406	.25
	4	.532	5.406	18.901
	5	.532	5.406	37.552
M23	1	1.145	3.306	-22.375
	2	1.145	3.306	-11.135
	3	1.145	3.306	.105
	4	1.145	3.306	11.344
	5	1.145	3.306	22.584
M24	1	.858	1.22	-9.006
	2	.858	1.22	-4.462
	3	.858	1.22	.082
	4	.858	1.22	4.626
	5	.858	1.22	9.17
M25	1	14.912	6.621	0
	2	14.912	6.621	22.347
	3	14.912	6.621	44.694
	4	14.912	6.621	67.041
	5	14.912	6.621	89.388
M26	1	5.64	4.443	-29.688
	2	5.64	4.443	-14.359
	3	5.64	4.443	.97
	4	5.64	4.443	16.299
	5	5.64	4.443	31.628

Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
M27	1	1.95	2.736	-18.248
	2	1.95	2.736	-8.947
	3	1.95	2.736	.354
	4	1.95	2.736	9.655
	5	1.95	2.736	18.956
M28	1	.231	.923	-6.616
	2	.231	.923	-3.178
	3	.231	.923	.259
	4	.231	.923	3.697
	5	.231	.923	7.134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Joint Displacements

Joint Label	X Translation (in)	Y Translation (in)	Rotation (radians)
N1	0	0	-7.215e-5
N2	0	0	-7.49e-5
N3	0	0	-7.358e-5
N4	0	0	-6.851e-5
N5	.009	0	-1.812e-5
N6	.009	0	-9.374e-6
N7	.008	0	-9.301e-6
N8	.008	0	-1.743e-5
N9	.012	0	-9.388e-6
N10	.012	0	-6.228e-6
N11	.011	0	-6.23e-6
N12	.011	0	-9.39e-6
N13	.014	0	-6.191e-6
N14	.013	0	-4.37e-6
N15	.013	0	-4.476e-6
N16	.013	0	-6.583e-6
N17	.015	0	-3.889e-6
N18	.014	0	-2.874e-6
N19	.014	0	-2.953e-6
N20	.014	0	-4.668e-6

Reactions

Joint Label	X Force (k)	Y Force (k)	Moment (k-ft)
N1	-12.535	-31.713	0
N2	-15.203	.455	0
N3	-14.912	-.272	0
N4	-11.85	31.53	0
Totals:	-54.5	0	

Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
M1	1	-31.713	12.535	0
	2	-31.713	12.535	42.305
	3	-31.713	12.535	84.611
	4	-31.713	12.535	126.916
	5	-31.713	12.535	169.221
M2	1	-12.17	6.227	-29.59
	2	-12.17	6.227	-8.107
	3	-12.17	6.227	13.375
	4	-12.17	6.227	34.858
	5	-12.17	6.227	56.34
M3	1	-4.125	4.312	-24.353
	2	-4.125	4.312	-9.691
	3	-4.125	4.312	4.97
	4	-4.125	4.312	19.632
	5	-4.125	4.312	34.293
M4	1	-.581	.758	-2.379

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Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
	2	-.581	.758	.443
	3	-.581	.758	3.265
	4	-.581	.758	6.087
	5	-.581	.758	8.909
M5	1	.455	15.203	0
	2	.455	15.203	51.309
	3	.455	15.203	102.619
	4	.455	15.203	153.928
	5	.455	15.203	205.237
M6	1	-4.35	13.438	-87.9
	2	-4.35	13.438	-41.54
	3	-4.35	13.438	4.82
	4	-4.35	13.438	51.18
	5	-4.35	13.438	97.539
M7	1	-3.538	7.884	-50.725
	2	-3.538	7.884	-23.919
	3	-3.538	7.884	2.888
	4	-3.538	7.884	29.694
	5	-3.538	7.884	56.5
M8	1	-1.844	2.987	-20.131
	2	-1.844	2.987	-9.004
	3	-1.844	2.987	2.123
	4	-1.844	2.987	13.25
	5	-1.844	2.987	24.377
M9	1	-.272	14.912	0
	2	-.272	14.912	50.327
	3	-.272	14.912	100.655
	4	-.272	14.912	150.982
	5	-.272	14.912	201.31
M10	1	3.914	13.366	-87.521
	2	3.914	13.366	-41.408
	3	3.914	13.366	4.704
	4	3.914	13.366	50.817
	5	3.914	13.366	96.929
M11	1	3.033	7.86	-50.723
	2	3.033	7.86	-23.998
	3	3.033	7.86	2.728
	4	3.033	7.86	29.453
	5	3.033	7.86	56.178
M12	1	1.558	3.27	-22.199
	2	1.558	3.27	-10.019
	3	1.558	3.27	2.161
	4	1.558	3.27	14.341
	5	1.558	3.27	26.521
M13	1	31.53	11.85	0
	2	31.53	11.85	39.995
	3	31.53	11.85	79.991
	4	31.53	11.85	119.986
	5	31.53	11.85	159.982
M14	1	12.606	6.37	-31.64
	2	12.606	6.37	-9.665
	3	12.606	6.37	12.311
	4	12.606	6.37	34.286
	5	12.606	6.37	56.261
M15	1	4.63	4.143	-23.81
	2	4.63	4.143	-9.723
	3	4.63	4.143	4.364

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Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
	4	4.63	4.143	18.451
	5	4.63	4.143	32.538
M16	1	.866	1.285	-6.861
	2	.866	1.285	-2.072
	3	.866	1.285	2.716
	4	.866	1.285	7.504
	5	.866	1.285	12.293
M17	1	7.542	-.581	8.909
	2	7.542	-.581	6.297
	3	7.542	-.581	3.684
	4	7.542	-.581	1.071
	5	7.542	-.581	-1.542
M18	1	4.555	-2.424	22.836
	2	4.555	-2.424	11.32
	3	4.555	-2.424	-.195
	4	4.555	-2.424	-11.71
	5	4.555	-2.424	-23.226
M19	1	1.285	-.866	3.295
	2	1.285	-.866	-.602
	3	1.285	-.866	-4.499
	4	1.285	-.866	-8.396
	5	1.285	-.866	-12.293
M20	1	12.345	-3.544	36.672
	2	12.345	-3.544	20.722
	3	12.345	-3.544	4.773
	4	12.345	-3.544	-11.177
	5	12.345	-3.544	-27.127
M21	1	7.448	-5.238	49.504
	2	7.448	-5.238	24.621
	3	7.448	-5.238	-.261
	4	7.448	-5.238	-25.144
	5	7.448	-5.238	-50.027
M22	1	2.858	-3.764	28.35
	2	2.858	-3.764	11.413
	3	2.858	-3.764	-5.524
	4	2.858	-3.764	-22.462
	5	2.858	-3.764	-39.399
M23	1	13.285	-8.045	80.693
	2	13.285	-8.045	44.489
	3	13.285	-8.045	8.285
	4	13.285	-8.045	-27.919
	5	13.285	-8.045	-64.122
M24	1	7.732	-8.858	84.142
	2	7.732	-8.858	42.068
	3	7.732	-8.858	-.006
	4	7.732	-8.858	-42.079
	5	7.732	-8.858	-84.153
M25	1	2.226	-7.976	63.499
	2	2.226	-7.976	27.607
	3	2.226	-7.976	-8.286
	4	2.226	-7.976	-44.178
	5	2.226	-7.976	-80.071
M26	1	8.792	-19.542	198.811
	2	8.792	-19.542	110.87
	3	8.792	-19.542	22.929
	4	8.792	-19.542	-65.011
	5	8.792	-19.542	-152.952

Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
M27	1	7.027	-14.737	140.185
	2	7.027	-14.737	70.184
	3	7.027	-14.737	.182
	4	7.027	-14.737	-69.819
	5	7.027	-14.737	-139.821
M28	1	5.481	-18.924	149.01
	2	5.481	-18.924	63.852
	3	5.481	-18.924	-21.306
	4	5.481	-18.924	-106.463
	5	5.481	-18.924	-191.621

Seismic Loading Factor

Seismic hazard exposure Group II
Group I-2 with an occupant load greater than 50, not having surgery or emergency treatment facilities

Seismic Performance category - C
"Not limited"

Table 1612.4.8 Group II
Allowable Story Drift : .020 h_{sx} ← story height below level x

Seismic Base Shear
$V = C_s W$

$$C_s = \frac{1.2 A_v S}{RT^{2/3}} = \frac{1.2(1.2g)(1.2)}{(4.5)(.613)^{2/3}} = 17.1$$

$T = C_T H_n^{3/4} = .03(55.83')^{3/4} = .613$

W= 3848psf (see attached pages)

V= 17.1 (3937) = 67323 psf

Vertical distribution of seismic forces

$F_x = C_{vx} V$ $\qquad C_{vx} = \dfrac{W_x h_x^K}{\sum_{i=1}^{n} w_i h_i^K}$

period	K
.5	1
.613	$\boxed{1.06}$
2.5	2

Floor 1:

$W_x = 3937$

$h_x = 13.67$

$\Sigma W_i h_i = 3937(13.67) = 53819$
$+3937(13.71) = 53976$
$+3937(14.3) = 56299$
$+3937(7.46) = 29370$
15748 $\quad$ 49.14 $\quad$ 193464

Trib Area

$\frac{13.5 + 13.75}{2} = 13.63'$

$$C_{vx} = \frac{3937(13.67)^{1.06}}{15748(49.14)^{1.06}} = \frac{62962}{977567} = .064$$

$F_x = C_{vx} V = .064(67323) = 4309$

Floor 2

Trib Area

$\frac{13.75+13.67}{2}=13.71'$

$W_x = 3937$

$h_x = 13.71$

$C_{vx} = \frac{3937(13.71)^{1.06}}{15748(49.14)^{1.06}} = \frac{63158}{977567} = .065$

$F_x = .064(67323) = 4376$

Floor 3

Trib Area

$\frac{13.67+14.92}{2} = 14.3$

$W_x = 3937$

$h_x = 14.3$

$C_{vx} = \frac{3937(14.3)^{1.06}}{15748(49.14)^{1.06}} = \frac{66042}{977567} = .068$

$F_x = .068(67323) = 4578$

Floor 4

$\frac{14.92}{2} = 7.46$

$W_x = 3937$

$h_x = 7.46$

$C_{vx} = \frac{3937(7.46)^{1.06}}{15748(49.14)^{1.06}} = \frac{33134}{977567} = .034$

$F_x = .034(67323) = 2289$

Total Weight of building

5 in slab Area = $56' \times 138' = 7728'$

$\frac{1}{2}(56')(21'4'') = 597'$

Total: $8325\ ft^2$

$8325\ ft^2 (.42\ ft)\left(\frac{150\ lb}{ft^3}\right) = 524\ K$

2096 K (4 floors)

Beam

$\frac{12''}{12'} \times \frac{24''}{12'} \times 18' = 36\ ft^3 \times 14 = 504$

$\frac{12''}{12'} \times \frac{24''}{12} \times 19 = 38\ ft^3 \times 7 = 266$

Total volume $= 730\ ft^3 \left(\frac{150\ lb}{ft^3}\right) = 110\ K$

440 K (4 floors)

Girder

$\frac{15''}{12'} \times \frac{25''}{12'} \times 21' = 54.7 \times 26 = 1422.2\ ft^3$

$1422.2\ ft^3 \left(\frac{150\ lb}{ft^3}\right) = 213\ kpf$

852 k (4 floor)

Column

$\frac{18''}{12'} \times \frac{18''}{12} \times 13.5 = 30.4 \times 30 = 912\ ft^2$

$912\ ft^3 \left(\frac{150\ lb}{ft^3}\right) = 137\ K$

$\frac{18}{12} \times \frac{18}{12} \times 13.75 = 30.9 \times 30 = 927\ ft^2$

$927\ ft^3 \left(\frac{150\ lb}{ft^3}\right) = 139\ K$

$\frac{18}{12} \times \frac{18}{12} \times 13.67 = 30.8 \times 30 = 924\ ft^2$

$924\ ft^3 \left(\frac{150\ lb}{ft^3}\right) = 139\ K$

$\frac{18}{12} \times \frac{18}{12} \times 14.9 = 33.5 \times 30 = 1005.8$

$1005.8 \left(\frac{150\ lb}{ft^3}\right) = 151\ kpf$

Slab: $\frac{524{,}000\,lb}{8325\,ft^2} = 63 \frac{psf}{floor}$ 4 floor = 252 psf

Beam: $\frac{110{,}000\,lb}{730\,ft^2} = 150 \frac{psf}{floor}$ 4 floor = 600 psf

Girder: $\frac{852{,}000}{1422.2} = 599 \frac{psf}{floor}$ 4 floor = 2396 psf

Column = $\frac{137{,}000}{912} = 150$ psf = 600 psf

$= \frac{139{,}000}{927} = 150$ psf

$= \frac{151{,}000}{1006} = 150$ psf

Total weight = 3848 psf

-OTHER MATERIALS-

per floor

Masonry Walls

estimated thickness = 8 in

Load = 38 psf / 4 in

so 76 psf

Gypsum wallboard (1 in)

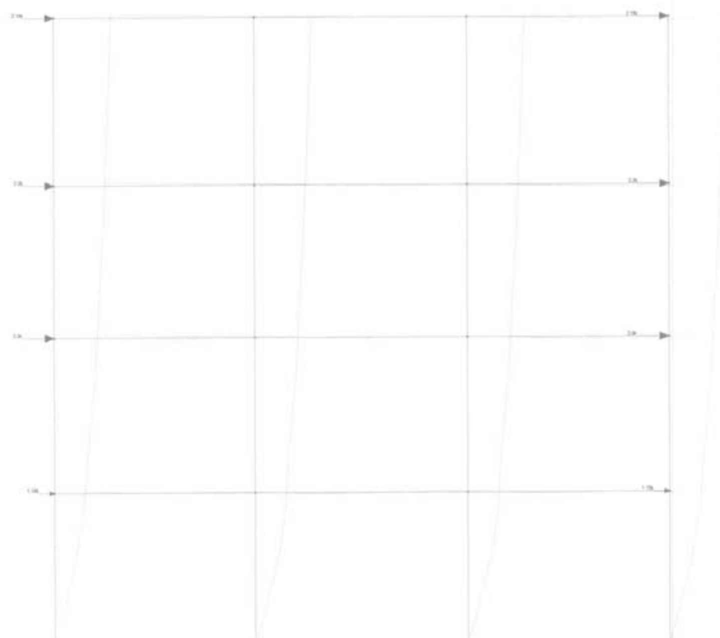
Load = 5 psf

Windows

Load = 8 psf

Total: 89 psf

Total per floor = 3848 + 89 = 3937 psf



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Joint Displacements

Joint Label	X Translation (in)	Y Translation (in)	Rotation (radians)
N1	0	0	-2.031e-5
N2	0	0	-2.16e-5
N3	0	0	-2.16e-5
N4	0	0	-2.031e-5
N5	.002	0	-5.417e-6
N6	.002	0	-2.866e-6
N7	.002	0	-2.865e-6
N8	.002	0	-5.416e-6
N9	.003	0	-3.247e-6
N10	.003	0	-2.119e-6
N11	.003	0	-2.117e-6
N12	.003	0	-3.243e-6
N13	.004	0	-2.441e-6
N14	.004	0	-1.639e-6
N15	.004	0	-1.639e-6
N16	.004	0	-2.424e-6
N17	.005	0	-1.66e-6
N18	.005	0	-1.096e-6
N19	.005	0	-1.09e-6
N20	.005	0	-1.768e-6

Reactions

Joint Label	X Force (k)	Y Force (k)	Moment (k-ft)
N1	-3.454	-10.483	0
N2	-4.345	-.011	0
N3	-4.346	.026	0
N4	-3.455	10.469	0
Totals:	-15.6	0	

Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
M1	1	-10.483	3.454	0
	2	-10.483	3.454	11.658
	3	-10.483	3.454	23.317
	4	-10.483	3.454	34.975
	5	-10.483	3.454	46.634
M2	1	-4.705	2.23	-12.063
	2	-4.705	2.23	-4.37
	3	-4.705	2.23	3.324
	4	-4.705	2.23	11.018
	5	-4.705	2.23	18.712
M3	1	-1.961	1.498	-8.935
	2	-1.961	1.498	-3.842
	3	-1.961	1.498	1.252
	4	-1.961	1.498	6.346
	5	-1.961	1.498	11.439
M4	1	-.471	.696	-4.074

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Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
	2	-.471	.696	-1.482
	3	-.471	.696	1.109
	4	-.471	.696	3.7
	5	-.471	.696	6.291
M5	1	-.011	4.345	0
	2	-.011	4.345	14.665
	3	-.011	4.345	29.33
	4	-.011	4.345	43.995
	5	-.011	4.345	58.66
M6	1	-1.298	4.418	-29.337
	2	-1.298	4.418	-14.097
	3	-1.298	4.418	1.144
	4	-1.298	4.418	16.385
	5	-1.298	4.418	31.625
M7	1	-1.039	2.85	-18.634
	2	-1.039	2.85	-8.944
	3	-1.039	2.85	.747
	4	-1.039	2.85	10.437
	5	-1.039	2.85	20.127
M8	1	-.53	1.469	-10.175
	2	-.53	1.469	-4.702
	3	-.53	1.469	.771
	4	-.53	1.469	6.244
	5	-.53	1.469	11.718
M9	1	.026	4.346	0
	2	.026	4.346	14.666
	3	.026	4.346	29.333
	4	.026	4.346	43.999
	5	.026	4.346	58.665
M10	1	1.315	4.42	-29.349
	2	1.315	4.42	-14.101
	3	1.315	4.42	1.147
	4	1.315	4.42	16.395
	5	1.315	4.42	31.643
M11	1	1.06	2.849	-18.633
	2	1.06	2.849	-8.946
	3	1.06	2.849	.742
	4	1.06	2.849	10.43
	5	1.06	2.849	20.118
M12	1	.55	1.477	-10.223
	2	.55	1.477	-4.722
	3	.55	1.477	.779
	4	.55	1.477	6.28
	5	.55	1.477	11.78
M13	1	10.469	3.455	0
	2	10.469	3.455	11.66
	3	10.469	3.455	23.321
	4	10.469	3.455	34.981
	5	10.469	3.455	46.641
M14	1	4.687	2.233	-12.077
	2	4.687	2.233	-4.375
	3	4.687	2.233	3.328
	4	4.687	2.233	11.031
	5	4.687	2.233	18.733
M15	1	1.941	1.502	-8.943
	2	1.941	1.502	-3.835
	3	1.941	1.502	1.274

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Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
	4	1.941	1.502	6.382
	5	1.941	1.502	11.49
M16	1	.451	.658	-3.973
	2	.451	.658	-1.521
	3	.451	.658	.931
	4	.451	.658	3.384
	5	.451	.658	5.836
M17	1	1.454	-.471	6.291
	2	1.454	-.471	4.17
	3	1.454	-.471	2.049
	4	1.454	-.471	-.073
	5	1.454	-.471	-2.194
M18	1	-.015	-1.001	9.524
	2	-.015	-1.001	4.768
	3	-.015	-1.001	.013
	4	-.015	-1.001	-4.742
	5	-.015	-1.001	-9.497
M19	1	-1.492	-.451	2.284
	2	-1.492	-.451	.254
	3	-1.492	-.451	-1.776
	4	-1.492	-.451	-3.806
	5	-1.492	-.451	-5.836
M20	1	1.397	-1.49	15.513
	2	1.397	-1.49	8.808
	3	1.397	-1.49	2.104
	4	1.397	-1.49	-4.601
	5	1.397	-1.49	-11.306
M21	1	.017	-2	18.996
	2	.017	-2	9.498
	3	.017	-2	0
	4	.017	-2	-9.497
	5	.017	-2	-18.995
M22	1	-1.356	-1.489	11.346
	2	-1.356	-1.489	4.644
	3	-1.356	-1.489	-2.059
	4	-1.356	-1.489	-8.761
	5	-1.356	-1.489	-15.464
M23	1	1.568	-2.743	27.647
	2	1.568	-2.743	15.302
	3	1.568	-2.743	2.957
	4	1.568	-2.743	-9.389
	5	1.568	-2.743	-21.734
M24	1	0	-3.002	28.525
	2	0	-3.002	14.266
	3	0	-3.002	.007
	4	0	-3.002	-14.251
	5	0	-3.002	-28.51
M25	1	-1.57	-2.747	21.766
	2	-1.57	-2.747	9.406
	3	-1.57	-2.747	-2.955
	4	-1.57	-2.747	-15.316
	5	-1.57	-2.747	-27.676
M26	1	-.074	-5.779	58.697
	2	-.074	-5.779	32.693
	3	-.074	-5.779	6.688
	4	-.074	-5.779	-19.316
	5	-.074	-5.779	-45.32

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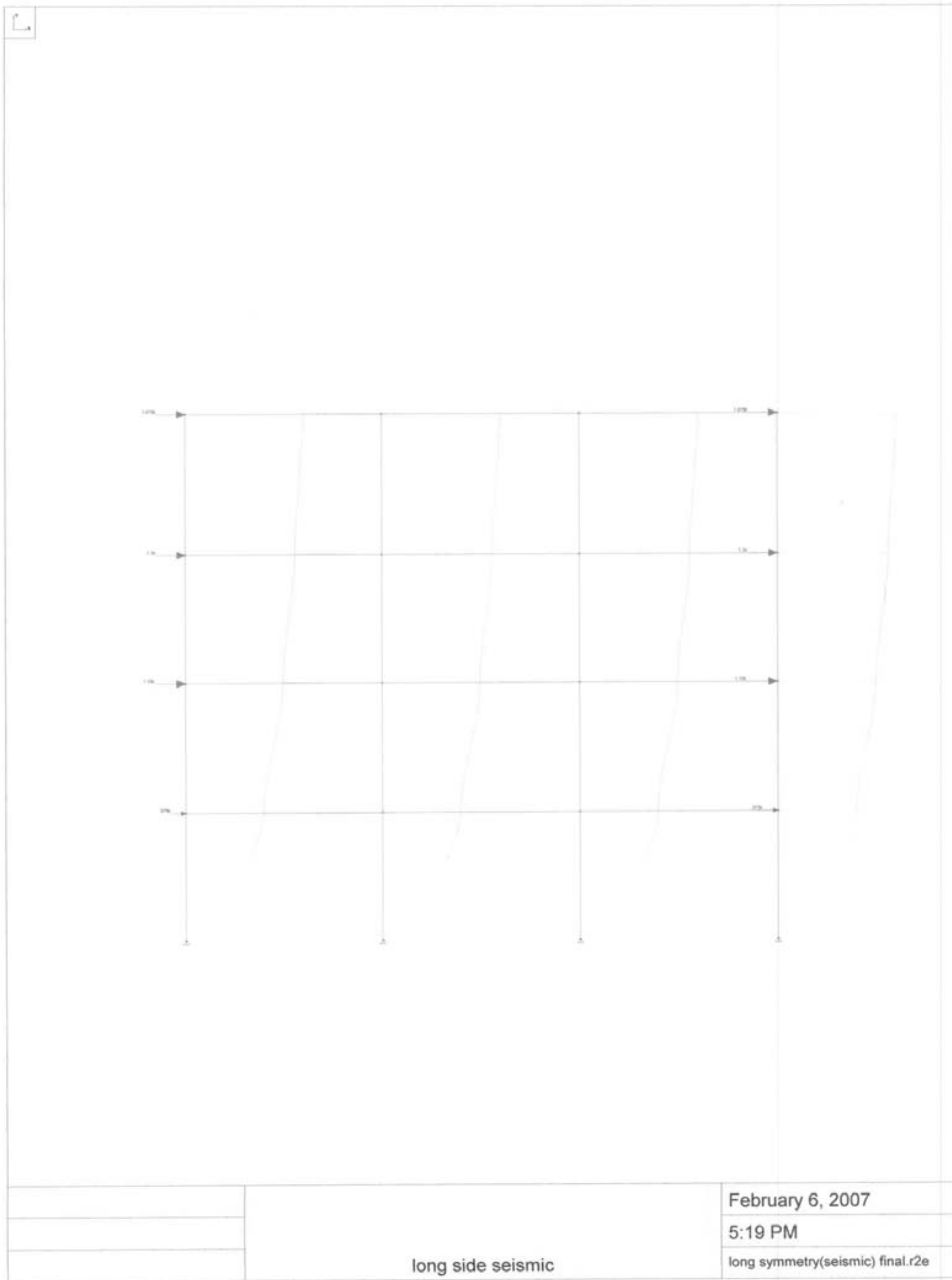
Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
M27	1	-.002	-4.492	42.676
	2	-.002	-4.492	21.339
	3	-.002	-4.492	.002
	4	-.002	-4.492	-21.335
	5	-.002	-4.492	-42.671
M28	1	.072	-5.781	45.343
	2	.072	-5.781	19.327
	3	.072	-5.781	-6.688
	4	.072	-5.781	-32.703
	5	.072	-5.781	-58.719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Joint Displacements

Joint Label	X Translation (in)	Y Translation (in)	Rotation (radians)
N1	0	0	-8.609e-5
N2	0	0	-8.752e-5
N3	0	0	-8.752e-5
N4	0	0	-8.609e-5
N5	.01	0	-4.048e-6
N6	.01	0	-1.104e-6
N7	.01	0	-1.105e-6
N8	.01	0	-4.048e-6
N9	.012	0	-2.407e-6
N10	.012	0	-6.895e-7
N11	.012	0	-6.904e-7
N12	.012	0	-2.408e-6
N13	.013	0	-1.769e-6
N14	.013	0	-5.073e-7
N15	.013	0	-5.064e-7
N16	.013	0	-1.778e-6
N17	.014	0	-1.126e-6
N18	.014	0	-2.971e-7
N19	.014	0	-2.962e-7
N20	.014	0	-1.136e-6

Reactions

Joint Label	X Force (k)	Y Force (k)	Moment (k-ft)
N1	-1.899	-4.93	0
N2	-2.001	1.067	0
N3	-2.001	-1.072	0
N4	-1.899	4.934	0
Totals:	-7.8	0	

Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
M1	1	.603	-.166	3.609
	2	.603	-.166	2.736
	3	.603	-.166	1.863
	4	.603	-.166	.99
	5	.603	-.166	.118
M2	1	.003	-.439	4.613
	2	.003	-.439	2.307
	3	.003	-.439	.002
	4	.003	-.439	-2.304
	5	.003	-.439	-4.609
M3	1	-.596	-.169	-.114
	2	-.596	-.169	-1
	3	-.596	-.169	-1.887
	4	-.596	-.169	-2.773
	5	-.596	-.169	-3.66
M4	1	.587	-.704	10.225

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Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
	2	.587	-.704	6.531
	3	.587	-.704	2.836
	4	.587	-.704	-.859
	5	.587	-.704	-4.554
M5	1	-.004	-.762	8.001
	2	-.004	-.762	4.002
	3	-.004	-.762	.002
	4	-.004	-.762	-3.998
	5	-.004	-.762	-7.998
M6	1	-.594	-.706	4.558
	2	-.594	-.706	.851
	3	-.594	-.706	-2.857
	4	-.594	-.706	-6.565
	5	-.594	-.706	-10.273
M7	1	.624	-1.278	17.283
	2	.624	-1.278	10.571
	3	.624	-1.278	3.86
	4	.624	-1.278	-2.851
	5	.624	-1.278	-9.562
M8	1	0	-1.055	11.077
	2	0	-1.055	5.538
	3	0	-1.055	-.002
	4	0	-1.055	-5.541
	5	0	-1.055	-11.081
M9	1	-.623	-1.278	9.557
	2	-.623	-1.278	2.848
	3	-.623	-1.278	-3.861
	4	-.623	-1.278	-10.57
	5	-.623	-1.278	-17.28
M10	1	.187	-2.782	35.821
	2	.187	-2.782	21.218
	3	.187	-2.782	6.616
	4	.187	-2.782	-7.987
	5	.187	-2.782	-22.59
M11	1	0	-1.607	16.868
	2	0	-1.607	8.433
	3	0	-1.607	0
	4	0	-1.607	-8.435
	5	0	-1.607	-16.869
M12	1	-.187	-2.781	22.588
	2	-.187	-2.781	7.986
	3	-.187	-2.781	-6.616
	4	-.187	-2.781	-21.218
	5	-.187	-2.781	-35.82
M13	1	-4.93	1.899	0
	2	-4.93	1.899	6.41
	3	-4.93	1.899	12.821
	4	-4.93	1.899	19.231
	5	-4.93	1.899	25.641
M14	1	-2.148	1.512	-10.18
	2	-2.148	1.512	-4.965
	3	-2.148	1.512	.251
	4	-2.148	1.512	5.466
	5	-2.148	1.512	10.682
M15	1	-.87	.985	-6.601
	2	-.87	.985	-3.251
	3	-.87	.985	.099

Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
	4	-.87	.985	3.449
	5	-.87	.985	6.799
M16	1	-.166	.472	-3.426
	2	-.166	.472	-1.668
	3	-.166	.472	.091
	4	-.166	.472	1.85
	5	-.166	.472	3.609
M17	1	1.067	2.001	0
	2	1.067	2.001	6.752
	3	1.067	2.001	13.504
	4	1.067	2.001	20.257
	5	1.067	2.001	27.009
M18	1	-.108	1.813	-12.449
	2	-.108	1.813	-6.193
	3	-.108	1.813	.063
	4	-.108	1.813	6.32
	5	-.108	1.813	12.576
M19	1	-.331	1.19	-8.063
	2	-.331	1.19	-4.017
	3	-.331	1.19	.028
	4	-.331	1.19	4.074
	5	-.331	1.19	8.12
M20	1	-.273	.599	-4.436
	2	-.273	.599	-2.203
	3	-.273	.599	.03
	4	-.273	.599	2.263
	5	-.273	.599	4.495
M21	1	-1.072	2.001	0
	2	-1.072	2.001	6.752
	3	-1.072	2.001	13.504
	4	-1.072	2.001	20.256
	5	-1.072	2.001	27.009
M22	1	.103	1.813	-12.449
	2	.103	1.813	-6.193
	3	.103	1.813	.063
	4	.103	1.813	6.319
	5	.103	1.813	12.575
M23	1	.326	1.19	-8.063
	2	.326	1.19	-4.017
	3	.326	1.19	.029
	4	.326	1.19	4.074
	5	.326	1.19	8.12
M24	1	.27	.599	-4.436
	2	.27	.599	-2.203
	3	.27	.599	.03
	4	.27	.599	2.262
	5	.27	.599	4.495
M25	1	4.934	1.899	0
	2	4.934	1.899	6.41
	3	4.934	1.899	12.821
	4	4.934	1.899	19.231
	5	4.934	1.899	25.641
M26	1	2.153	1.512	-10.179
	2	2.153	1.512	-4.964
	3	2.153	1.512	.251
	4	2.153	1.512	5.466
	5	2.153	1.512	10.681

Appendix I: Fire Protection Specifications

TABLE OF CONTENTS

SECTION 15300
FIRE PROTECTION TRADE WORK

Section 15300 - Fire Protection Trade Work

PART 1 - GENERAL

1.1. INTENT

A. It is the intent of this section to establish the scope of work for fire protection systems. Specifications governing the fire protection work are included under the FIRE PROTECTION sections, under the BASIC MATERIALS sections, and under other sections detailing general requirements. Refer also to:

1. GENERAL CONDITIONS and SUPPLEMENTARY CONDITIONS
2. Section 15010, General Requirements for Mechanical Work
3. Section 15050, Basic Materials & Methods - Mechanical
4. Section 15160, Basic Materials - Vibration Isolation and Seismic Restraint for Mechanical

B. SECTION 15050 has a detailed description on understanding how the general BASIC MATERIALS Sections are to be used in conjunction with the project-specific TRADE sections. Refer to SECTION 15050, paragraph on INTENT.

C. Fire Protection System shall meet Code, Owner's insurance company requirements, and following:

1. NFPA #13, Installation of Sprinkler Systems
2. NFPA #14, Installation of Standpipe and Hose Systems
3. NFPA #24, Installation of Private Fire Service Mains and Their Appurtenances

D. FM approved equipment shall be used where FM approved equipment is available for the intended application.

1.2. SCOPE

A. Provide labor, materials, equipment, services and transportation for complete and operational fire protection systems, as described on Contract Documents, including but not limited to following:

*1. Fire protection water service from five feet outside building wall, into the building

1. Wet standpipe systems
2. Wet sprinkler system
3. Fire department connections
4. Fire valve cabinets
5. Flushing connections
6. Vents
7. Pressure gauges
8. Backflow preventers
9. Painting as required under SECTION 15050, BASIC MATERIALS & METHODS - MECHANICAL
10. Sleeves, escutcheons, seals, waterproofing, firestopping, and similar devices
11. Hangers, anchors, guides, bases, and other supports

12. Access panels and access doors
13. System identification, including valve tags
14. Noise and vibration control
15. Seismic restraints, including equipment bolts and welding
16. Cleaning, lubrication, testing, balancing and adjusting
17. Coordination drawings
18. Record drawings
19. Operating and maintenance manuals
20. Instructions

B. Contract Drawings do NOT show complete system layout; only mains, risers and typical arrangements are shown. Provide additional branches, fittings, sprinkler heads, components, etc. as necessary to meet intent of Contract Documents.

1.3. SUBMITTALS

A. Submit for review shop drawings for each item of material, equipment, and system component furnished or installed as part of the work of this Section. Shop drawing requirements are specified under SECTION 15010, GENERAL REQUIREMENTS FOR MECHANICAL WORK and under DIVISION 1.

B. Shop drawings shall identify system and intended use of pipe, valve, etc.

C. Shop drawings shall include motor efficiency data for three-phase motors 1 HP and larger.

D. Submit system layout drawings for review, showing location and function of each piece of equipment, each sprinkler head, and each alarm.

1. Prior to submittal of drawings:

a. First: Obtain Architect's approval for revisions to layouts shown on Contract Drawings.

b. Second: Obtain approval of system layout drawings from Owner's insurance company, ISO or Industrial Risk Insurers or Factory Mutual, and local Fire Marshal.

c. Third: Drawings and calculations shall be sealed by a Professional Engineer currently registered in the State where the project is located.

2. Finished ceilings are intended to present uniform coordinated appearance with regard to location and spacing of light fixtures, air system diffusers and grilles, and sprinkler heads.

3. Carefully review Reflected Ceiling Plans before preparing sprinkler system layout drawings.

4. Where necessary to accomplish desired appearance and where necessary to meet Code, additional sprinkler heads (beyond minimum number) may be required.

E. Furnish written certificate of inspection to Architect advising that materials and installation are in full accordance with manufacturer's procedures and recommendations, and operate properly and safely.

F. Obtain Certificates of Review of the systems from local fire official, and submit to Architect for record.

G. Submit test results for review.

H. Submit hydrant flow test report, as outlined in paragraph 3.1.

I. In addition to items described under SECTION 15050, BASIC MATERIALS & METHODS - MECHANICAL, operations & maintenance manual shall include:

1. Copy of NFPA #25, Inspection, Testing, and Maintenance of Water-Based Fire Protection Systems.
2. Field test report.
3. Certifications required in paragraphs 1.3.E and 1.3.F.
4. Test results.

1.4. <u>COORDINATION DRAWINGS</u>

A. Before materials are purchased or work is begun, prepare Coordination Drawings showing size and location of mechanical pipes, ducts, equipment and appurtenances, relative to work of other trades.

B. Submit for review coordination CAD drawings with accompanying electronic files. Hard copy of drawings shall be signed by following trades: sheet metal, plumbing, fire protection, electrical and other HVAC trades. Electronic (CAD) drawing files shall be produced with each trade represented by separate layers.

C. Preliminary coordination drawings shall be prepared as follows:

1. First: Sheet metal trade shall prepare coordination drawings, minimum 1/4"=1' scale, to be used as composite construction floor plans for coordination of trades. Plans shall show floor and ductwork layouts in detail, including ceiling heights, duct heights and sizes (including insulation), registers and diffusers, and light fixtures.

2. Second: As part of work of SECTION 15300, fire protection trade shall draw fire protection piping, etc., on coordination drawings prepared by sheet metal trade.

3. Third: As part of DIVISION 16, ELECTRICAL WORK, electrical trade shall draw electrical distribution conduits, wires, panels, and other electrical work which must be coordinated with other trades, on coordination drawings which have been prepared by fire protection trade.

4. Fourth: As part of work of SECTION 15400, plumbing trade shall draw waste piping, vent piping, water piping, risers and other plumbing work which must be coordinated with other trades, on coordination drawings which have been prepared by electrical trade.

5. Fifth: As part of work of SECTION 15500, HVAC trades shall draw HVAC piping work which must be coordinated with other trades; on coordination drawings which have been prepared by plumbing trade.

6. Each trade shall use a different color code.

D. Coordination Meeting and Drawing Revisions

1. Sixth: Contractor shall hold a coordination meeting with sheet metal, HVAC, fire protection, electrical and plumbing trades and shall resolve conflicts between trades. Coordination drawings are to assist in identifying trade conflicts.

2. Seventh: Sheet metal trade shall revise coordination drawings to reflect revisions to the various trade work (including sheet metal, HVAC, fire protection, electrical and plumbing trades), as determined by coordination meeting.

3. Eighth: Sheet metal, HVAC, fire protection, electrical and plumbing trades shall sign the revised coordination drawings as indication of their acceptance of the construction layout shown thereon.

E. Sheet metal trade shall submit the revised coordination drawings to Architect for review.

F. Coordination Drawings are for Contractor's and Engineer's use during construction and shall not be construed as replacing shop, "as-built" or record drawings required elsewhere in the Contract Documents.

PART 2 - PRODUCTS

2.1. GENERAL PRODUCT REQUIREMENTS

A. Fire protection system components shall be UL listed and shall be for fire protection service.

B. Unless otherwise specified, sprinkler system equipment shall be by Central, Grinnell, Reliable, Automatic, Star or Viking, acceptable equivalent to equipment models named herein.

C. Unless otherwise specified, standpipe system equipment shall be by Seco, Standard or Potter-Roemer.

D. Unless otherwise specified, supervisory equipment shall be by Potter Electric Signal, Potter-Roemer, Grinnell, Reliable or Gamewell.

2.2. FIRE PROTECTION SYSTEMS - DESIGN CRITERIA

A. Sprinkler systems shall be hydraulically balanced. Design shall include minimum of 10 psi safety margin in hydraulic calculations to accommodate pipe deterioration due to age and service pressure reduction. Design parameters shown in table are minimum requirements; meet Insurance Company requirements also. Pipe velocity in sprinkler system shall not exceed 26 FPS.

SPRINKLER SYSTEM DESIGN CRITERIA				
System	Area Served	Hazard Group	Minimum Design Density	
			gpm/sf over remote area	Combined inside & outside hose stream allowance
Wet sprinkler	General office areas	Light hazard	0.10 gpm/sf 1,500 sf	100 gpm
		Ordinary hazard Group 1	0.15 gpm/sf 2,000 sf	250 gpm
Wet sprinkler	Library stack	Ordinary hazard Group 2	0.20 gpm/sf 1,500 sf	250 gpm
Wet sprinkler	Mechanical and electrical rooms	Ordinary hazard Group 2	0.19 gpm/sf 2,000 sf	250 gpm
Dry	Emergency generator room, loading dock, unheated spaces excluding attic spaces	Ordinary hazard Group 2	0.19 gpm/sf 2,600 sf	250 gpm

B. Fire standpipe system shall be Class III system providing service to every part of building while using 100-foot hose with 30-foot hose stream. System shall be designed to provide minimum 500 gpm from first standpipe and 250 gpm from each additional standpipe required, up to a total of 1250 gpm maximum.

C. Standpipe supply shall be sufficient to maintain 65 psi at topmost outlet, with outlet discharging at capacity. Refer to NFPA #14 and authority having jurisdiction for other requirements.

2.3 PIPING FOR INTERIOR FIRE PROTECTION WATER SYSTEMS

A. Every length of pipe shall be marked with manufacturer's name or trademark and ANSI or ASTM type and grade.

B. Pipe shall be standard weight, black steel meeting ASTM A795 or ASTM A135. On deluge, preaction, dry pipe and water gong piping systems, pipe shall be hot dip galvanized, inside and outside. Zinc-electroplating method will NOT be acceptable.

1. Pipe 2" and smaller shall be Schedule 40.
2. Pipe 2-1/2" to 6" shall be Schedule 10.

C. Fittings shall be screwed type, or roll-grooved joint by Victaulic or acceptable equivalent; approved for fire protection systems; meeting NFPA 13 or NFPA 14; as follows:

1. Screwed, 5" and smaller: 125 lb. cast iron, standard weight, meeting ANSI B16.4.

2. Screwed, 6" and larger, or where otherwise approved or required by Owner's insurance company: ASA 150# or 250# flanged steel, malleable iron or heavy weight cast iron; meeting ANSI B16.3, B16.4 and B16.5.

3. Grooved joint, 2" and larger: Use with roll-grooved joint couplings by Victaulic or acceptable equivalent.

4. Fittings on deluge, preaction, dry pipe and water gong piping systems shall be hot dip galvanized, inside and outside. Zinc-electroplating method will NOT be acceptable.

D. Joints:

1. Screwed joints shall be made up with acceptable pipe joint compound or joint tape.

2. Flanged joints shall be made up with non-asbestos gaskets. Unless otherwise specified below, joints shall meet requirements of SECTION 15050, BASIC MATERIALS & METHODS - MECHANICAL, paragraph on BLACK STEEL PIPE, Grooved Joint Fittings.

   a. Unless otherwise specified below, unions and flanges shall meet requirements of SECTION 15050, BASIC MATERIALS & METHODS - MECHANICAL, paragraph on UNIONS AND FLANGES.

   b. Flanged unions shall be bolted type with ASA 250# flanges.

   c. Unions shall be 300 lb. malleable iron with brass-to-iron or brass-to-brass ground joint.

   d. Gaskets shall be type and material approved for fire protection. Gaskets shall be placed inside bolt circle.

3. Grooved joint couplings shall be specifically approved for fire protection duty; by UL, FM, NFPA 13 and NFPA 14. Couplings and gaskets used in dry pipe systems shall be listed for dry pipe service.

   a. For systems up to and including 175 psi:

      (1) On long straight runs, on piping in mechanical rooms, and on piping in fire pump rooms: Couplings shall be Victaulic "Rigid Style 005 Firelock", with Grade E Type A gaskets and Victaulic "Firelock" fittings; or acceptable equivalent.

      (2) For wet systems, for other locations: Couplings shall be Victaulic "Flexible Style 75" with Grade E Type A gaskets; or acceptable equivalent.

      (3) For dry systems, for other locations: Couplings shall be Victaulic "Flexible Style 75" with "Flush-seal" gaskets; or acceptable equivalent.

4. Welded joints may be used on piping 4" and larger where and as approved by Code and authorities having jurisdiction.

E. Pipe nipples shall be same quality, material and class as pipe system. Close nipples are NOT permitted in fire protection systems.

2.4. <u>VALVES FOR UNDERGROUND FIRE PROTECTION SYSTEM</u>

A. General: Exterior valves on water and fire protection systems shall be by ITT Grinnell (Kennedy Division), Mueller or Stockham.

B. Gate valves 3" and larger, for use on underground lines: AWWA, UL/FM, iron body, double disc gate valves with mechanical joint ends and joint accessories: Kennedy/Grinnell #71X.

1. Where required to connect to equipment or pipe with flanged ends, use Kennedy #703X with one end flanged and the other end mechanical joint.

2. Provide two-piece sliding type valve boxes: Kennedy #120 with #122 valve wrench. Covers shall be marked "WATER".

C. Post-indicator valve assemblies for sectional control of fire protection piping: mechanical joint indicator post type control valve with flanged bonnet, iron body, bronze mounted, solid wedge, parallel seat, and non-rising stem; designed for 175 psi working pressure, hydrostatically tested to 350 psi. Nibco #M-609 gate valve with Nibco #NIP-1, #NIP-1A or #NIP-2 vertical indicator post or wall post.

2.5. <u>GENERAL SERVICE VALVES FOR ABOVE-GRADE INDOOR FIRE PROTECTION SYSTEMS</u>

A. General: Valves shall be rated at minimum of 175 psi non-shock.

B. Gate Valves

1. Gate valves 2" and smaller: 175# class, bronze body, OS&Y, solid wedge disc: Crane #459.

2. Gate valves 2-1/2" and larger: 175# class, iron body, flanged, OS&Y, bronze trim: Crane #467, or Mueller #A-2073-6.

C. Ball Valves

1. Ball valves 1" and smaller: 600# WOG, full port, brass two-piece body, with TFE Teflon seats and seals, brass ball, UL/FM approved for fire protection: Jamesbury "Clincher" #21-1100-TT-0-22.

2. Ball valves 1-1/4" to 2": 400# WOG, full port, brass two-piece body, with TFE Teflon seats and seals, brass ball, UL/FM approved for fire protection: Jamesbury "Clincher" #21-1100-TT-0-22.

D. Check Valves

1. Check valves 2" and smaller: 300# WOG, Bronze body, screw-in cap, screwed ends: Crane #141, or Milwaukee #510T.

2. Check valves 3" to 8": 175#, iron body, swing check, rubber-faced clapper, brass seat, bolted cover, flanged ends: Viking #G-1. When used for siamese piping, valve shall be tapped for automatic ball-drip connection.

*2. Check valves 2-1/2": 175# iron body, swing check, rubber-faced clapper, brass seat, bolted cover, grooved ends: Viking #D-1. When used for siamese piping, valve shall be tapped for automatic ball-drip connection.

*E. Check Valves: Victaulic "Series 710" dual disc, spring loaded, 250#, two-piece body with drain connections.

E. Butterfly Valves

1. Butterfly valves up to 2" size, with built-in tamper switch: Bronze body, full port, slow close, 175# WOG. Milwaukee #BB-SCS01 with threaded ends, Milwaukee #BB-SCS02 with threaded ends, Milwaukee #BBVSCS01 with grooved ends, or Milwaukee #BBVSCS02 with grooved ends.

2. Butterfly valves 2-1/2" and larger, normally closed: 175 psi, grooved ends, polymer coated, ductile iron body with rubber-faced ductile iron disc, internal supervisory switch: Grinnell "Gruvlok" #7700FP(FD). Switch shall be field wired for normally closed position.

3. Butterfly valves 2-1/2" to 6", normally open: Ductile iron body, 316 stainless steel disc, EPDM seat, wafer style, 316 stainless steel stem with bronze bushings, UL/FM approved, 175 psig rating: ITT Grinnell #WD-8272-4-FD. Valve shall have gear operator and two built-in tamper switches.

F. Ball-Drip Valves

1. Automatic ball-drip valves: Grinnell #F789, or Potter-Roemer "Series 5980".

2.6. BACKFLOW PREVENTERS

A. Provide double check valve assembly with test cocks for field testing, two OS&Y gate valves with tamper switches: Hersey #2 or Watts #709DCYRW.

B. At each backflow preventer provide placard indicating pressure readings at each test port.

2.7. SPRINKLER HEADS

A. Temperature ratings of heads in machine rooms, mechanical rooms, electrical rooms, attic spaces, ceiling spaces, crawl spaces, and tunnels shall be 212°F. Heads near high temperature equipment and piping shall have temperature rating as required by NFPA 13.

B. For exposed pipe installations in non-finished areas, heads shall be brass, with 165°F fusible links: Reliable Model G.

C. For installation in finished ceilings, heads shall be factory painted white, adjustable, concealed flush type: Reliable Model G1.

D. Horizontal sidewall heads in finished areas shall be bright chrome plated, with matching escutcheon: Reliable Model G/HSW1.

E. Upright sidewall heads in finished areas shall be bright chrome plated: Reliable Model G.

F. In areas of possible high-intensity fires, provide large drop sprinkler heads, UL listed as extra large orifice, with "K" factor of 11.2 and double deflector: Viking Model A high challenge type.

G. Dry pendent heads shall be bright chrome-plated, with matching escutcheon; Reliable Model G3.

H. Dry pendent recessed heads shall be bright chrome-plated, with recessed escutcheon; Reliable Model G3.

I. Dry pendent concealed heads shall be factory painted white, Reliable Model G3A.

J. In areas where heads are subject to physical damage, provide sprinkler guard assembly over head: Reliable Model C-1 or Reliable Model C-2 for dry heads. This shall include but not be limited to:

1. Heads in elevator shafts
2. Heads under lower rakes of stairways
3. Heads in electrical rooms
4. Heads in machine rooms
5. Heads in boiler rooms and other mechanical rooms
6. Heads installed 7'-0" or less above finished floor

K. When pendent sprinkler heads are installed adjacent to vertical obstructions such as surface-mounted light fixtures, provide matching two-piece extension escutcheons: Reliable Model HB.

L. Furnish minimum six spares of each style and type of head; furnish sprinkler wrench and storage cabinets. Furnish additional quantities if required by Code. Storage cabinet shall be steel with red finish: Potter-Roemer #6162.

2.8. INSPECTOR'S TEST CONNECTIONS

A. Inspector's test connection shall be 300 psi maximum working pressure, one-piece ductile iron body, meeting ASTM A536, 1-1/4" inlet and outlet connections; with bronze fitted test and drain valves, acrylic sight glass, 1/2" aluminum orifice inserts which give flow equal to one sprinkler head; Victaulic "Testmaster Style 718" or acceptable equivalent.

*A. Inspector's test connection shall be bronze body, chrome-plated bronze ball, 1" NPT, female threaded, UL and FM approved: Potter-Roemer #6166-70 or AGF Manufacturing #1000.

2.9. SIAMESE CONNECTIONS

A. Siamese connections shall be polished chrome-plated or polished brass, as selected by Architect; with pin lugs, caps, snoots and chains; as follows:

1. Flush wall-mounted type, three-way: Potter-Roemer #5830 6"x 2-1/2" x 2-1/2" x 2-1/2"

B. Siamese connections for sprinkler system shall have matching escutcheon labelled "AUTO-SPKR".

C. Siamese connection for standpipe system shall have matching escutcheon labelled "STANDPIPE":

D. Siamese connection for combined sprinkler/standpipe system shall have matching escutcheon labeled "SPKR/STANDPIPE":

2.10. SUPERVISORY EQUIPMENT AND ALARMS

A. Provide NEMA enclosures for electrical equipment.

B. Building exterior: Provide mechanical water motor and alarm gongs with rust-resistant weatherproof housings: Reliable Model C or acceptable equivalent.

1. For sprinkler system, labeled "SPRINKLER SYSTEM".
2. For standpipe system, labeled "STANDPIPE SYSTEM".

C. Alarm check valves

1. Provide alarm check valve for entering water service for wet sprinkler/standpipe system. Alarm check valve shall be Reliable Model E with retard chamber, alarm pressure switch, valves, piping and accessories.

2. Alarm pressure switch shall be electric, double pole, double throw switch suitable for variable pressure water suppy, with separate contacts for local alarm bell and for connection to fire alarm system: Reliable Model J. Contacts shall be located on retard chamber.

D. Provide (wet pipe) sprinkler excess pressure pump kit with pump, supervisory panel, piping, fittings, valves and trim: Gamewell #M2700-13 "Watchman Kit".

1. Electric Sprinkler Alarm Pressure Switches (wet sprinkler)

a. Provide normally open contacts: Gamewell #RM-47465, 120 V, single phase.

b. Provide low pressure switch: Gamewell #RM-47951, 120 V, single phase.

E. Supervisory Low Water Pressure Switch

1. Provide low water pressure switch to monitor pressure in service pipe to building, to determine position of street valve: single pole, double throw, mercury tube type switch; Gamewell #RM-43270A, 120 V, single phase.

F. Water Flow Alarm Switch

1. Provide water flow alarm switches for each zone of the wet sprinkler system.

2. Water flow alarm switch shall be UL and FM approved, surge-free, rated for 250 psig, with 0-90 second time-delay mechanism to prevent false alarms, with N.O. and N.C. contacts: Gamewell #8304, Potter Electric Signal #VSR-F (2" to 8" sizes) or Potter Electric Signal #VS-SP (1" and 1-1/2" sizes).

3. In looped systems (when sprinkler system is connected between two or three supply/standpipe risers): provide System Sensor "Model WFD" waterflow detector with minimum 4-10 gpm activation rate and up to 70 seconds retard adjustment.

G. Shut-off valve supervision:

1. Provide tamper switch on each OS&Y valve, to monitor open or closed position of valve, as indicated on Drawings. Switch shall have tamperproof cover and two sets of Form C (SPDT) contacts: Potter Electric Signal #OSYSU-A1 or #OSYSU-A2 as required.

* Switch shall have tamperproof cover and one set of Form C (SPDT) contacts: Potter Electric Signal #OSYS-B.

2. Provide weatherproof, tamperproof switch on each indicator post and each butterfly valve; to monitor open or closed position of valves, as indicated on Drawings. Switch shall have tamperproof cover, 1/2" NPT tapped hole, and one set of Form C (SPDT) contacts: Potter Electric Signal #PCVS.

3. In areas where valves are subject to unusual conditions, such as valve pits and non-rising stem gate valves, provide plug type tamper switch with SPDT contacts, normally closed, and tamperproof cover: Potter Electric Signal #PTS-B.

2.11. DRY PIPE VALVE AND TRIMMINGS (DRY SPRINKLER SYSTEM)

A. Provide dry pipe valve and trimmings: Reliable Model D flanged inlet/flanged outlet size, as required by hydraulic calculations. Trimmings shall include: priming, test, drain, air relief valve, retard chamber water gong and piping, air and water pressure gauges, electrical alarm switch, and air maintenance device.

B. Provide double pole, double throw pressure switch for remote alarm circuits.

2.12. AIR COMPRESSOR (DRY SPRINKLER SYSTEM)

A. Provide strap-on type automatic tankless, motor-driven air compressor and accessories to maintain dry pipe pressure. Compressor shall be air cooled, single stage, oil-less, meeting NFPA 13, with minimum capacity of 1.4 cfm at 50 psig: Reliable Model A, Viking Model E1, or acceptable equivalent by Gast.

B. Provide 120 V, single phase, 60 Hz, 1/4 HP motor.

C. Provide automatic air make-up devices to regulate and maintain air pressure at required level.

D. Accessories shall include:

1. Flywheel governor-operated mechanical unloader
2. Check valve
3. Moisture unloader
4. Relief valve
5. Magnetic starter
6. High pressure safety cutoff switch set for 5 psig above operating pressure of air maintenance package control switch

E. Provide low air pressure trouble switch.

F. Disconnect and wiring shall be provided as part of DIVISION 16, ELECTRICAL WORK.

G. Air compressor shall be listed for fire protection service.

2.13. FIRE VALVE CABINET (STANDPIPE SYSTEM)

A. Provide flush recessed, fire valve cabinet with pressure reducing valve, 20 gauge steel door, 18 gauge frame, and 20 gauge box: Potter-Roemer #1830-A.

B. Cabinet shall have following accessories:

1. Identifying decal which reads "For Fire Department Use Only"
2. 2-1/2" chrome-plated valve: Potter-Roemer #4023
3. Chrome-plated brass cap and chain: Potter-Roemer #4625.
4. Split ring escutcheon: Potter-Roemer #4723.
5. 5/8" projecting trim
6. Full glass door with double strength glass

7. Interior finish of white enamel.
8. Exterior white prime coat and exterior finish of color as selected by Architect
9. Cabinet box, dimensions 24"x24"x10"
10. 2-1/2" x 1-1/2" reducer with cap and chain

2.14 EXPOSED FIRE VALVES (STANDPIPE SYSTEM)

A. Provide exposed fire valve, including:

1. Identifying decal which reads "For Fire Department Use Only"
2. 2-1/2" chrome plated valve: Potter-Roemer #4023
3. 2-1/2"x 1-1/2" reducer with cap and chain: Potter-Roemer #4625

2.15 ISOLATION & SEISMIC RESTRAINT SCHEDULE

A. Seismic restraint shall be provided on fire protection piping systems at turns of more than 4 feet and throughout entire run; where piping is supported by hangers longer than 6", as measured from top of pipe to bottom of supporting structure. Restraint type and maximum spacing of restraints shall be as follows:

PIPE SEISMIC RESTRAINT SCHEDULE (Refer to SECTION 15150)				
			Maximum Spacing between Seismic Restraints	
Piping	Pipe Size	Seismic Restraint Type	Transverse	Longitudinal
Fire protection mains and crossmains	All	SR-C	40'-0"	80'-0"
		SR-SB	40'-0"	80'-0"
Sprinkler branch piping	All	SR-H	End of branch pipe	
	2-1/2" & larger	SR-C	40'-0"	80'-0"
		SR-SB	40'-0"	80'-0"
Fire protection riser	All	SR-C	Top of riser	
		SR-SB	Top of riser	

B. Seismic restraint on risers shall have four-way bracing. Seismic restraint on equipment and other piping shall have two-way bracing.

C. Seismic restraint shall be provided on fire protection equipment. Restraint device types shall be as follows:

EQUIPMENT SEISMIC RESTRAINT SCHEDULE (Refer also to SECTION 15160)	
Base-Mounted Equipment	Seismic Restraint Type
Floor-mounted equipment, if not specified elsewhere	SR-B

PART 3 - EXECUTION

3.1. HYDRANT FLOW TEST

A. Obtain or conduct a three-point hydrant flow test in cooperation with local hydraulic company to determine the water supply available. Flow test shall have been performed within twelve months of contract date. Hydrant flow test shall meet NFPA 13 and NFPA 291.

B. Flow test report shall include following data:

1. Static pressure
2. Residual pressure
3. Gallons per minute flowing during test
4. Hydrant numbers and locations
5. Hydrant datum elevations, adjusted to datum used on Site Drawings

3.2. INSTALLATION OF FIRE PROTECTION SERVICE

A. Provide OS&Y gate valve and backflow preventer on fire protection water service into building.

3.3. INSTALLATION OF FIRE PROTECTION SYSTEMS

A. If concealed space above ceiling exceeds 6" from ceiling to non-fireproofed wood construction and is not completely filled with insulation, provide sprinklers in concealed space.

B. Use hydraulic design programs to determine pipe sizing and arrangement for main feeder and cross-main distribution systems.

C. Number of zones and areas served by zones established on Contract Drawings shall NOT be altered without prior approval from Architect; if alterations are proposed, they shall be made with due regard for zone alarms, future service and maintenance.

D. In areas with suspended ceilings, locate sprinkler heads to fall in center of ceiling panels (using arm-over or swing-joint piping arrangement) and to form coordinated uniform pattern with light fixtures, air supply or return diffusers, registers, etc. Install piping concealed within spaces above suspended ceilings. Provide necessary offsets in branch pipes to accomplish desired results. Coordinate work closely with ceiling installer.

E. On dry pipe, deluge and preaction systems. Horizontal connections shall be made above centerline of horizontal mains.

F. Sprinkler heads shall NOT be installed until branch lines to heads have been cleaned.

G. Provide valve supervisory device on main control valve and on riser control valves. Provide drain valves at bottom of standpipe system.

H. When installing dry sprinkler heads using arm-over piping arrangements, install plugged tee at top of drop pipe. Installation of elbow in lieu of plugged tee shall NOT be allowed.

I. Provide pressure gauges on water supply main and as shown on Drawings. Provide pressure gauges, test tees and inspection valves at top of each standpipe and each combination sprinkler/standpipe main vertical feed.

J. For each wet sprinkler zone, provide inspector's test connections at each floor. For dry systems, provide inspector's test connections at end of most remote branch line. Test connection shall be accessible, maximum 7 feet above floor, piped to drain riser or to exterior of building whichever is indicated on Drawings.

K. Provide flanged connections in fire pump discharge. Pipe as required for installation of hose valve test header for fire pump test.

L. Install backflow preventers so that test ports are fully accessible, facing the front side. Provide minimum 12" clearance for test hose attachment. Where possible, backflow preventer shall be located 36" minimum, 66" maximum above finished floor, unless otherwise required by local authorities.

M. Provide NFPA- and FM-approved seismic expansion loops at building seismic joints.

3.4. PIPING SUPPORTS

A. Horizontal piping shall be supported throughout entire run. Provide hangers or supports within 2'-0" of elbows, as required to support heavy groups of fittings and valves, and as practical and necessary for attachment to building structure.

B. Hanger rod sizes and maximum spacing of hangers and supports shall be as follows. Where double rod hangers are used, rod sizes for pipe over 2" may be reduced one size.

Fire Protection System Pipe/Size	Spacing	Rod Size
Sizes and spacing of hangers and hanger rods shall be in accordance with NFPA 13, NFPA 14, Owner's insurance agency (e.g., FM or IRI) and manufacturer's recommendations.		

C. Arm-over and swing-joint piping arrangements longer than 12", subject to pressures greater than 100 psi, shall be supported. Arm-over and swing-joint piping arrangements longer than 24" shall be supported.

D. As part of renovation project in existing buildings, replace existing hanger rod beam clamps.

3.5. EQUIPMENT INSTALLATION

A. Coordinate installation with work on Fire Alarm System under DIVISION 16, ELECTRICAL WORK. Connections to building fire alarm system shall be part of work of DIVISION 16, ELECTRICAL WORK.

B. Install valves in fire valve cabinets with outlets horizontal, looking out.

C. Cabinet for spare sprinkler heads shall be located as directed by Architect.

1. Upon completion, inspect installation. Furnish written notice of inspection to Architect, 48 hours prior to inspection.

2. Furnish written certification of proper installation, to Architect.

D. Locations of water gong, siamese connection, fire pump test header, and similar apparatus on exterior walls shall be coordinated with Architect.

3.6. IDENTIFICATION

A. Refer to SECTION 15050, BASIC MATERIALS & METHODS - MECHANICAL, paragraph IDENTIFICATION.

B. Provide permanently marked, weatherproof signs of metal or rigid plastic ; secured with corrosion-resistant wire, chain or other approved means; displaying signage information required by NFPA. Signs shall be placed at following locations:

1. Main wet alarm valve
2. Main dry alarm valve
3. Supplying the corresponding hydraulic designed area

C. On exterior siamese connections, provide label to indicate system demand pressure, in psi.

D. Valves concealed in ceilings shall be identified with ceiling markers, Brady "Valve Finder Ceiling Tacks" or acceptable equivalent. Ink, crayon and similar marked identification will NOT be acceptable.

3.7. CLEANING FIRE PROTECTION PIPING SYSTEMS

A. Before operating piping systems: Flush with water and drain systems until all traces of dirt, construction debris, weld, scale, slag, corrosion material, oil, grease, etc., are removed from piping systems. Underground piping including service pipe to building shall be flushed according to NFPA 25 and NFPA 24.

B. Provide necessary drain and fill connections, temporary piping connections, and equipment required for flushing and cleaning. Install spool pieces in place of automatic control valves for duration of flushing.

C. Sprinkler heads shall NOT be installed during any portion of cleaning.

3.8. TESTING

A. Each system shall be functionally tested as required by fire department and Owner's insurance company. Tests shall include:

1. Testing of valves, equipment and accessories for proper operation.

2. Setting and adjusting of pressure switches and controls.

3. Testing of backflow preventers. As part of test, provide placard on each valve being tested. Placard shall indicate test date and pressures resulting from test.

B. Test waterflow alarm devices under operating conditions.

C. Operation of control valves (opening and closing) shall be verified under system water pressure.

D. Refer to SECTION 15050, BASIC MATERIALS & METHODS - MECHANICAL, for general testing requirements. Tests specified therein shall apply, except as follows:

1. Pressure testing of fire protection system shall be at test pressure at 200 psig.

3.9. FIRE PROTECTION DURING CONSTRUCTION

A. Provide temporary fire protection during construction, including temporary fittings, etc., until such time as permanent system can be activated.

END OF SECTION

Appendix J: Fire Protection Systems Collected Data

This Appendix contains information that was collected verbally during the site visit in January. All of the information was provided by the Project Manager for Consigli Construction Company, Steve Johnson, and is listed below.

Fixed Automatic Fire Protection Systems:

-Sprinkler system: wet or dry? **Wet**.

-Are there areas sprinklers in the electrical rooms and life safety closets? **Yes, as well as smoke detectors.**

-Are there sprinklers in stairwells? **Yes.**

-Are the sprinklers pendant, sidewall or covered pendant? **Covered Pendant**

-Are there standpipes? What is their diameter and locations? **Yes, on every floor in the stairwells. 6" diameter**

-Are there backflow control valves? **Yes, throughout the system.**

-Is there a fire department connection? Where? **Yes, in the stairwells.**

-Are there floor control valves? **Yes, in the stairwells.**

-Where is the test drain location? **In the stairwell.**

Manual Systems:

-Are there wall mounted fire extinguishers? Where are they located? **Yes, in cabinets in the hallways.**

-How many per floor? **Depends on distance, not number per floor. There is one in each hallway.**

-What class extinguishers are they? **ABC**

-Are there manual pull stations? Where? **Yes, throughout the building and stairwells.**

Detection and Alarm Systems:

-Is there a fire alarm control panel? Type? **Yes, however, we could not access it. The door to the room was locked.**

-Is it supplied with backup power? For how long? **Yes, by an emergency generator for an unknown period of time.**

-Where is it located? **Not in masonry building.**

-Are there smoke detectors? **Yes.**

-Are there alarm horns, strobes, or combination horn-strobe? **Combination horn strobes and indicators.**

-Is there a voice communication system? **No.**

-Does the system send a direct signal to the Worcester Fire Dept? **Yes.**

Smoke and Heat Ventilation:

-Are there any automatic smoke control systems or features? **No.**

Water Supply and Reliability:

-Is the water supplied by city of Worcester or a private supply? **City of Worcester.**

Additional information gathered:

-**Each floor is approximately 7500 square feet**

-**Each floor has a little fire alarm panel in the electrical room.**

-The shaft in the center of the building contains HVAC that runs from units on the roof.

-The cost of gutting the masonry building was $300,000 including the asbestos abatement.

-Total project cost: $30 million

Appendix K: Active Fire Protection Systems

This information presented in this appendix is intended to supplement that shown in the Active Fire Protection Systems section of the report.

Sprinkler Design

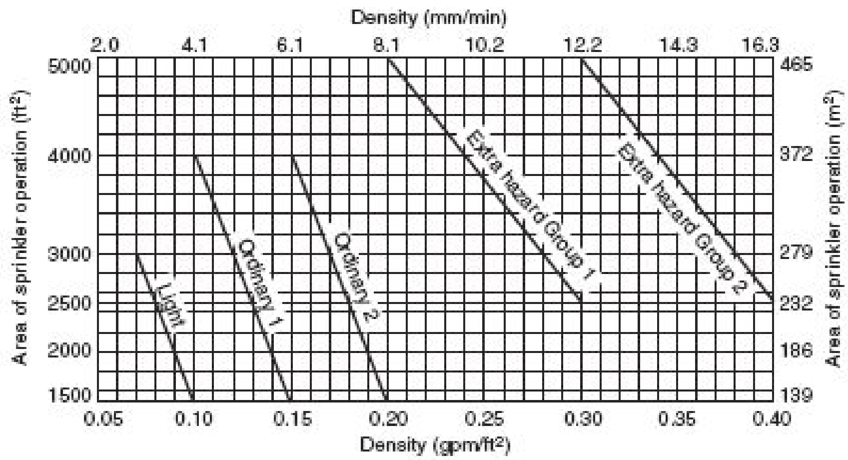
The design area and density information for the sprinklers in the Gateway Park Project is listed in the table below. This information was gathered from the Fire Protection Specifications shown in Appendix H.

Figure 23: Sprinkler System Design Criteria from Fire Protection Specifications

SPRINKLER SYSTEM DESIGN CRITERIA				
System	Area Served	Hazard Group	Minimum Design Density	
			gpm/sf over remote area	Combined inside & outside hose stream allowance
Wet sprinkler	General office areas	Light hazard	0.10 gpm/sf 1,500 sf	100 gpm
		Ordinary hazard Group 1	0.15 gpm/sf 2,000 sf	250 gpm
Wet sprinkler	Library stack	Ordinary hazard Group 2	0.20 gpm/sf 1,500 sf	250 gpm
Wet sprinkler	Mechanical and electrical rooms	Ordinary hazard Group 2	0.19 gpm/sf 2,000 sf	250 gpm
Dry	Emergency generator room, loading dock, unheated spaces excluding attic spaces	Ordinary hazard Group 2	0.19 gpm/sf 2,500 sf	250 gpm

The required area and density of water to design the different hazard classifications of the building were determined using NFPA 13. This document provides standards for the installation of automatic sprinkler systems. In chapter 11 the requirements for differing hazard classifications are outlined in the Density/Area Curve Graph. This graph can be seen in Figure 24.

Figure 24: Density/Area Curves From NFPA 13[148]



[148] National Fire Protection Association. NFPA 13: Standard for the Installation of Sprinkler Systems. 13-115.

Appendix L: Fire Hydrant Locations