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Structural Design of WPI Recreation Center

A Major Qualifying Project Report

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Abstract

The objective of this MQP was to develop a sustainable design for the new WPI Recreation Center. This design included the structural steel members, trusses and connections, and a foundation including the footings, retaining wall and pool. LEED certification was integrated into the sustainable design and various software tools such as RISA 2D, AutoCAD and Revit were used for structural analysis and architectural design.

Capstone

The purpose of implementing Capstone Design criteria is to gain a better understanding of how both synthesis and analysis are involved in creating a well rounded final, or capstone, project. This model provides a path that helps to find the solution to a problem and make decisions that affect more than just a final report. A project that follows the Capstone Design requirements involves a decision making process that includes synthesis and analysis to develop alternative designs based on constraints that include but are not limited to technical, environmental, economic, social, political and sustainable.

This MQP specifically addresses the design and constructability of a new Athletic Facility to be built on the WPI campus. The problem presented to the group was to successfully develop a sustainable structural and foundation design for the new facility. To address this design challenge, the group created an overall building layout, performed design and analysis for both the foundation and structural framing, conducted basic cost estimating activities, and completed this final report of the design recommendation options.

The capstone design constraints had to be considered during the planning and design process. As different constraints were considered, alternative designs were developed. In order to address these real world constraints, this section of the report will review how they were considered during the design process. The eight principle constraints which the MQP considered were:

Economic

Addressed in this project are different factors affecting not only the economy of the WPI community, but also the City of Worcester. In order to better understand these factors the group has looked into cost estimates and considered the recent construction on the WPI campus. We were then able to gain an understanding of how this new and very expensive project may tie into expenditures of the institution and the lives of the surrounding community. The group must address whether or not there would be a worthwhile return on investment (i.e. benefitting quality of life, engaging more students and alumni, increasing recruitment, increasing donations and alumni giving etc). It had to be considered what the return would be on the investment.

One main factor kept in mind was how the construction would affect the state of the school while in session. The project will be in progress for multiple years. The construction may be disruptive to the student enrolled, and may take away from the overall aesthetics of the school for visitors. This negative aspect may be outweighed by the positive attraction to the school. Such a facility may cause more spectators to attend sporting events, stimulate heightened interest of fans, create a more competitive team and students may enjoy life on campus more. These are all reasons arousing more interest in the school itself, which can definitely influence the economy of WPI.

In the Worcester community, how will a private institution, such as WPI building a large recreational center, have an influence? The group has thought about the affects during construction and also after the facility is built. The economic upsides seem to outweigh any negative economic effects on the immediately surrounding community. Some examples of positive effects include: the creation of jobs for contractors, subcontractors, and others in the immediate community, the opportunity for community involvement in WPI sports, the creation of community workshops, classes, and sponsored use of the facility to bring in profits, and possibly even having sponsored events for the community to provide community service opportunities for the school.

Environmental

The group has also considered how the building of such a large structure and its permanent existence will affect the environment. We have looked into how a large scale construction project and the overall process of construction affect the surrounding environment. Much of the research we have performed on LEED has helped us to become more familiar with how to mitigate some of the issues. For example, a few issues involved in the process that can be somewhat or even fully counteracted are:

- Machinery/automobiles emissions and noise pollution vs. more efficient equipment
- Full waste vs. recycling construction waste
- Groundwater runoff/paving vs. green roof and grey water usage
- Project length vs. fast-track project scheduling

Also, we looked at how a building of this size and surface area would affect the surrounding environment. Again, for example:

- Paved area, covered surface area, etc.

- Roof drainage
- Repairs/maintenance
- Use by humans (waste, foot traffic, new outdoor facilities)

These aspects of the building's existence can also be neutralized by using green building practices and proper maintenance.

Sustainability

Considering environmental concerns, sustainability has been a main focus of investigation by the group. We have addressed different ways a design can work to alleviate some of the environmental impacts of such a large project. We have also examined ways that new technology can be incorporated into the design in order to use environmental elements to the advantage of the longevity and sustainability of the building itself. This topic has been selected as a particular topic of interest in the Capstone Design Experience and the incorporation of these new technologies has ultimately had a profound effect on the final building design. Some processes that we have found to have great potential for use in this design include:

- Green building practices (waste removal/recycling)
- Reuse of water/runoff
- Lower vehicle/machinery emissions

Also, we have addressed actual design elements and Green Building concepts that could be included in the final design such as:

- LEED Certification by the U.S. Green Building Council
- Green Roof Design to prevent premature/excess water runoff
- Curtain wall polarization or BIPV (Building Integrated Photovoltaics) to use solar energy to power a battery operated/aided mechanical/HVAC system or interior lighting system.

Constructability

Constructability has been looked at from multiple angles. Such a large construction undertaking relies on many factors. The success of the project is heavily dependent on the determination of whether it is more easily constructible or not. If there are a number of issues raised with the actual ability of an architectural work to be built, a project may not move forward or the contractors could have a hard time engineering a solution. This could result in a longer, more costly process. Some things considered in designing both the layout and structure include:

- Architectural design elements and the layout conforming to structural requirements.
- Community and political constraints (zoning, permitting, community objections).
- Access for construction (road/utility access and availability)
- Timeline (problems during school year, effects on the life of current students, etc.)
- Budget, cost (considering current state of economy)
    - Inflationary pressures may limit the use of more desirable aesthetic design choices at the cost of the chosen architect's vision.

Ethical Issues

This concept heavily focuses on the community members, both enrolled at WPI as students as well as those living in the surrounding communities. There are many avenues by which ethical issues may come to pass and they have many different ways of being viewed. As a group of engineering students, it was important for us to address this real world subject matter. The well-being of the young WPI community as well as the surrounding Worcester community that chooses to use the facility is placed in the hands of those that design the building. We have recognized in performing the various design activities the importance of safety factors and how the design of individual elements can affect the structure as a whole, and therefore all those that come into contact with it. It is this realization that has enlightened us to the importance of ethical issues faced by professional civil engineers. The well-being of the community comes first and foremost and in designing a structure that can positively affect the lives of so many, it is the job of the engineer to keep priorities straight and to construct the most structurally sound building possible, given economic constraints.

Health and Safety

The health and safety of community members, during construction and after the facility is built, should be explored. This issue greatly ties into the environmental impact and the idea of sustainability. On the positive side, the recreation center certainly promises the opportunity for better health and fitness following completion of the project. Negatively, the process of construction has potential to affect the area, but as discussed previously many of the following problems can be lessened:

- Emissions from machinery
- Waste products and water entering the surround areas water supply
- Noise pollution
- Night safety/secure construction site

Social

The social aspect of any design and construction project largely ties together health and safety, economic and environmental impact. It is in the society's perception of the project and its advantages or disadvantages where potential problems or praise lay. The community, both WPI and surrounding, must be heard and its approval sounded before a project such as this can be approached. The need of a new Athletic Facility is a great part of that and is something that has been discussed at length in the background of this report. The decision to move ahead with the design and construction of this project has pointed to a positive assessment of the pros and cons of the different factors faced in creating such a facility and all that comes with the process. It seems to be the conclusion of the WPI community that the positive effects of building this Recreation Center outweigh the negatives.

Political

A political aspect must be addressed as a part of the Capstone Design experience. The politics of the building industry are multi-faceted, especially in competitive school such as WPI. The main issue lies in where funds should be allocated. In building such a facility there are trade-offs, such as reducing the amount of money available to hire more faculty, to provide scholarships to more students, and to build more dorms. These being some negative effects, the positives may include building a better athletic program, reflecting well on the current administrations and increasing the visibility of the school.

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Executive Summary

Worcester Polytechnic Institute is currently in the beginning stages of the design and construction of a new Recreation Center. As the number of students attending WPI continues to grow, buildings are being erected to accommodate for the increased population and to improve the quality of life on campus. Such buildings also provide the school better recruiting opportunities and offer attractions for a wide range of prospective students.

The objective of this project was to create an initial building layout, a design scheme for steel members, trusses and connections and a foundation plan consisting of spread footings, a retaining wall and pool design. Each portion of design encountered unique challenges and issues, but ultimately a practical and structurally sound design was produced.

The first major step in the design process was to come up with an overall architectural building layout. Given a list of the desired facilities, the group was able to brainstorm different floor plan options and analyze the most practical placement of each activity space. The resulting five story building, incorporated into the side of the hill leading down to Alumni Field, provided a base for the placement of the members for the steel design.

The design for the structural steel portion of the building was performed using the LRFD design method and the AISC guides and criteria. To further aid in the process, a spreadsheet was formulated in which design loads and dimensions were used to yield the necessary member sizes for the frame. The final design consists of 900 horizontal members, over 200 wide-flanged lengths for columns and two styles of trusses of 6 ft. and 8 ft. depths. The connections for the frame are primarily bolted and thus sample calculations were prepared according to AISC methods as well. Upon conducting an overall frame analysis using the portal method, the steel skeleton was deemed stable.

The foundation design consisted of spread footings for the columns, retaining walls and a slab design for the pool, and a retaining wall on the East side of the building. Taking into consideration the soil conditions, and recommendations from the geotechnical report for allowable soil bearing pressures, three different size footings were designed for three categories of vertical loadings produced by the building frame. The selected design also consists of an independent, cantilevered retaining wall for the East wall. This procedure consisted of dimensioning and detailing the design, as well as incorporating the use of grouted tie-back anchors to resist excessive overturning and sliding loads. The third component of the foundation was the swimming and diving pool containment area. This included the design of cantilevered retaining walls for a range of depths and the design of a welded wire fabric reinforced slab-on-grade.

A cost estimation of the project was conducted by estimating the cost of raw materials and fabrication cost. The size and length of the steel members, the cubic yardage of 4000 psi concrete and the units of rebar were all incorporated into the estimate. The cost was calculated using average costs from *RS Means* and a percentage was added to both the steel members and rebar for fabrication. The approximate total cost for steel, concrete and fabrication equaled $1.6 million.

As with any project, the design process produced a number of challenges and roadblocks. For example, the first major challenge was encountered in the layout design. With limited architectural design experience, the group had difficulty in selecting the desired spaces, organizing them into a flowing, practical and functional building and also taking into consideration where critical design loads and the corresponding structural elements would be located as a result. Another challenge was faced in designing the trusses. Since these trusses spanned 100 ft. in length and supported large loads, it was a challenge limiting them to a maximum depth. This depth had to allow for ample clearance between the bottom of the truss and the floor below. Design of the retaining wall was also challenging because of its large height. This loading was redirected using the tie-back anchors and the size of the wall and footing components. These issues are just a selection from the many obstacles faced during the design process.

The group was able to achieve a sustainable design of the recreational center that would enhance the quality of life on campus and encourage sports and a healthy living. The new center would be an asset in recruiting new students to campus and to engaging the community in more activities that are not possible with current facilities. This project challenged the group to make decisions and seek alternative solutions every step of the way, including architectural design, structural design, constructability, cost, functional attributes, and sustainability. It was the process and the obstacles encountered that ensured a well-rounded project and learning experience.

1.0 INTRODUCTION

Sport, by definition, is a “physical activity that is governed by a set of rules or customs and often engaged in competitively” (Dictionary, 2008). Sports are largely recognized and encouraged at Worcester Polytechnic Institute (WPI), a University of Science and Technology. To WPI, sports give their students a chance to compete in competitive physical activities and develop active lifestyles. They are also a great way for students to meet new people, develop skills such as communication, confidence, leadership and team work. The number of student athletes has been increasing every year and now WPI must expand their facilities to accommodate this growth.

The design of the new Recreation Center will provides student athletes and faculty new workout facilities, basketball and racquetball courts, a pool, climbing wall, classrooms and treatment rooms. Having this state-of-the-art facility allows the athletes to train and compete in sport activities, developing higher levels of skill, team work, and overall enjoyment. It also allows students to continue living active lifestyles, and maintaining their health and physical well-being. The successful completion of construction for this new Recreation Center will bring generations of WPI students the benefit of having new-age facilities available for their use.

Our Major Qualifying Project (MQP) group was presented with the task of performing structural design activities for the new Recreational Center. The completion of this MQP satisfies the ABET Capstone Design experience and is the culmination of the group’s education at WPI. The final result is a sustainable design, which was reached through a process that encountered realistic constraints and obstacles. This design process included the creation of an initial footprint and floor plan, exploration into permitting, zoning and green building practices, the design of steel members, trusses, connections, and a foundation design consisting of spread footings, retaining walls and slab-on-grade. Once the final product was complete a cost estimate for the raw materials and fabrication was calculated.

The Recreation Center is meant to meet the necessities of WPI and the community. Promoting athletics and health within WPI’s community is an integral aspect of campus life, a characteristic that continues to be upheld as the population grows and changes. It is important for students to represent the campus community and to be involved in this process. This project addressed real-world design challenges and the task of creating a practical facility capable of housing the desired facilities. This exploration of the design procedure and final result is illustrated within the following report.

2.0 BACKGROUND

Athletics and fitness are a prominent aspect of education and the emphasis on activity is growing even more as the percentages of youth obesity rises. In early education and at the college level, the United States is continually pushing the importance of fitness and health knowledge. An integral part of the college curriculum is the facilities in which to learn and to exercise in order to achieve the best-rounded experience. Thus, in colleges like Worcester Polytechnic Institute, building state of the art centers for recreation and fitness are becoming an important and essential component of delivering a quality education.

Specifically concentrating on collegiate schools like WPI, one can see the effect of sporting activities on a community. Varsity sports and team related physical education can have a profound effect on the mood, involvement, and overall health of a campus. This is one reason why private education organizations are relying on the need for a variety of space and equipment for fitness education and involvement. There are a number of other influencing factors that may push an institution like WPI in the direction of a recreational building including housing the growing number of admitted students per year (Management, 2007), the need to stay competitive and attract a wider range of students, the necessity of providing a well-rounded education, and simply improving the overall quality of life on campus.

The design and construction endeavors on university campuses are each architecturally unique and are helping schools to stay competitive, attract scholars, gain recognition, and provide more space for learning, but even in their distinctiveness the process by which they are designed and erected is fundamentally the same. Most large-scale, campus construction projects include the following components: recognition of necessity, basic layout, detailed architectural design, foundation design, building structural design, preconstruction activities, and execution of construction. These are the main procedures and practices that will be discussed in the following sections.

2.1 Initial Layout

The first stages of any design project heavily depend on an idea, a building block, and in this case, a layout. After the client and the architect gauge what facilities are needed and desired they can begin to create a layout within the space provided. The footprint provides this surface area restriction and within this area the preferred facilities are placed. Only after this are the architect and structural design team able to really apply their practices to design around the specific space.

The WPI administration and community have voiced their opinions and it has been determined which facilities will be part of the new Recreation Center. A layout design was conducted from a list of desirable spaces and facilities and their required surface areas as shown in Table 1.

Table 1: List of Desired Facilities for Athletic Building

PROGRAM AREA	Quantity	Total Area
Public Spaces		(NSF)
Lobby	1	4,700
Hall of Fame/ Trophy Display	1	500
Reception Desk	1	450
Vending	1	100
Lounge	1	1,250
Public Restrooms	2 @ 300 SF	600
Activity Spaces		
4-Court Gymnasium	1	29,000
Elevated Jogging track	1	8,400
Gym Storage	4 @ 600 SF	2,400
Fitness Center	1	10,000
Fitness center storage	1	300
Free Weights	1	5,000
Free Weights Storage	1	300
Group Exercise/Multipurpose	3 @ 1200 SF	3,600
Group Exercise Storage	1	400
Classroom	2 @ 600 SF	1,200
Classroom Storage	2 @ 200 SF	400
Racquetball/Squash Courts	3 @ 840 SF	2,520
Climbing wall	1	1,200 (NIC)
Natatorium	1	19,100
Natatorium Storage	1	600
Pump Room	1	1,000
Natatorium Spectator Seating	1	2,800
Meet Management/Lifeguard	1	120
Indoor Rowing Tank	1	2,000
Support Spaces		
Men's General/Pool Locker	1	1,000
Women's General/Pool Locker	1	1,000
Storage/Collection	1	225
Loading	1	400

PROGRAM AREA	Quantity	Total Area
Training/Rehabilitation		
Office	2 @ 180 SF	360
Exam Room	1	150
Secure Storage	1	80
Bulk Storage	1	250
Taping/Treatment/Rehabilitation	1	3,000
Hydrotherapy	1	180
Ice Machine	1	100
Restroom	1	80
Administration/Coaches		
Reception/Secretarial	1	300
A.D.	1	200
A.D. Assistants	3 @ 150 SF	450
Offices	18 @ 150, 1 @ 200	2,900
Workroom	1	1,050
Video Editing Room	1	200
Storage	1	150
Secure Storage	1	100
Rest Rooms	1	150
Summary		
Public Spaces		7,600
Activity Spaces		89,140
Training Spaces		4,200
Administrative Space		5,500
Support Spaces		2,625
Total NSF		**109,065**
Total GSF		**~ 128,260**

With the facilities names and area being given, it was possible to create a layout providing a base for the coming structural design and analysis activities. This arrangement needed to be completed before any structural design could be started. In order to begin, zoning and coding also needed to be considered, the footprint and elevations needed to be determined and the establishment of the desired number of floors and relative major architectural elements was necessary. To create a layout for the new WPI Recreational Facility the following methods were utilized by the group:

- Obtaining the list of spaces (Table 1: List of Desired Facilities for Athletic Building).
- Consultation of building codes and zoning ordinances.
- Determination of general square footage footprint by using a combination of existing topographical/contour maps and mapping procedures using a hand-held GPS unit.

- Basic dimensions for each individual space were used in brainstorms and sketches of different possible arrangements.
    - To aid in this brainstorming process, the group went on a site visit to the Boston University Fitness and Recreation facility. Using insight gained at this other campus on the overall design aesthetic and practicality, the group was able to begin sketching.
- Sketches were used to come up with a more dimensionally accurate drawing.
    - This was done using AutoCAD. The general spaces were dimensioned and placed in each floor plan matching the area of the footprint of the building. The resulting drawings were then combined to create the overall floor plans and elevations of the proposed building. The floor plans and elevations we created can be seen in Appendix A.

2.2 Applicable Loads

As part of the design process, the loads on the structure must be determined. Even in the architectural planning of a building the designer must be mindful of structural challenges and the resulting costs. Especially in the new, sustainable wave of construction, the architectural aspect must take into consideration even more challenging elements. This means all types of loading possibilities for the potential edifice must be recognized. These loads must then be used during the structural design of the building, in both the sub grade foundation and above in the superstructure. The general loads that will be explored in this project include dead load, live load, wind load, snow load, seismic load, and other special loads related to this particular building. A brief explanation of each is as follows.

Dead load is the load created by those materials that are directly attached or built-in to the building itself. This includes any material making up the super structure, envelope, interior architectural elements, permanent equipment, etc.

Live load, also known as the occupancy load, is the downward force created by different types of occupancy. For example a building created for office use would have a different occupancy load than one meant for performances, such as auditoriums.

Wind load, **snow and seismic loads** are all variable by location. For a certain region these loads can be calculated and applied accordingly.

Special loads are variations or additions to the live load. This building is unique in that it will serve many different purposes and provide a multitude of activity spaces. For example, special spaces like weight rooms, pool and courts will be present. Green building options like green roof or shading might add to the roof load and curtain wall load as well. Mechanical system loads are also included in this category.

2.3 Structural Building Design

The base of a building's superstructure is its structural framing which enables it to stand and remain stable. This skeleton takes the dead and live loading and transfers it to the foundation which in turn disperses it into the ground. For this particular project, the WPI Recreational Facility, the group has chosen to explore steel member options for the building. Concrete members were not chosen for the primary building material because it does not have tensile strength that steel boasts and overall it is not as versatile a material. Aspects that have been considered as sub-categories in this structural design are the types and sizes of members, trusses and connections.

2.3.1 Steel Design

Structural steel was used in the first skyscraper, the Home Insurance Building, in Chicago in 1884-1885. This marked a turning point in modern-day construction. The need to expand upward coupled with innovations in light-weight, strong, iron-carbon compounds led to hot-rolled steel eventually becoming the norm in large and small construction projects worldwide.

Looking at recently built projects on the WPI campus, it has been observed that steel is the main structural material around which the buildings are erected. The skeleton of both the Bartlett Center and the new residence hall, East Hall, were constructed mostly, if not entirely, of steel girders, beams, and columns. Because the Athletic Facility is to be a larger undertaking than both of these buildings previously done, steel again seems like the most feasible and practical material. This is because it can be manufactured and prepped mainly offsite and is more cost and time efficient than options such as composite members in fast-track construction projects. There are many elements that make up an actual steel framed structure and it is those pieces that will be discussed in the following sections.

Material

Steel is an alloy made mainly of iron with a small percentage of carbon. Because carbon is the most cost-effective alloying agent to be combined with iron, it is most commonly used in the steel construction industry. The level or percentage of carbon that is infused into the iron is the determining factor of the grade that steel is given. This grade represents the strength properties (ductility, hardness, tensile strength, etc.) of the material. We have chosen to use a steel grade of 50 ksi for design purposes. This grade of steel is also commonly found in steel construction, along with grade 36 ksi. We have opted for the higher grade.

Beginning in the mid-19th century steel was mass-produced and used for many applications that wrought iron had been used for previously. The material used in construction is called rolled steel which is typically cut, shaped and drilled in steel fabrication shops and then shipped on-site. These different shapes and their functions will be explored further in a later section.

Structural Members

Steel members consist of horizontally spanning members and vertical members. Horizontally, the two main categories are beams and girders, which have very similar shapes and structures, but beams transfer loads to the girders. This results in girders being larger members. Columns, which are vertical, are the steel component that transfers the girder loads down to the foundation of the building (Ching & Adams, 2001).

Beams and Girders

There are four main rolled-steel shapes for beams and girders. One of the most-used in modern construction is the **wide-flange** (W) shaped beam. Other common shapes include the **I-beam** (S), the **channel** (C), or **structural tubing**. Because the main goal is to have the lightest weight material that can resist the largest amount of forces, another option for beams and girders are open-webbed members.

Joists or trusses are most commonly used to span longer spaces. In these cases they are usually a more practical option than a solid beam or girder as they use less material, but are engineered to resist the same tensile, compressive, shear, and bending forces as a full beam.

Columns

As stated above, the most commonly used column shape is the **wide-flange** (W). This is mainly because, along with its strength properties, it can also be connected to in two directions. Welding and bolting are also possible at each of its surfaces. Though this is most typical, other potential shapes include the **round pipe**, **rectangular or square tubing**. Also, like in the horizontal members, plates and shapes can be combined to create other alternatives. For example, **plates** can be welded into a similar W shaped wide-flange or a square tube. **Angles** can be welded as well to form such shapes as a **cruciform**.

Structural Steel Framing

Steel framing can be used in anything from a one-story building to the biggest of skyscrapers. This fitness center application is no different and typical steel framing practices should provide a great base from which the full structural design activities can stem. Characteristically, steel framed buildings are dimensioned in a rectangular grid and within this grid different options can be explored.

The building blocks for such a frame are the girder, beam and column. The horizontal beam transfers loading to a girder or directly to a column. Also horizontal, a girder transfers that loading to columns. These columns transfer all that loading to the foundation which then disperses it into the subsoil or bedrock. The organization of these main pieces can take the form of either a one-way beam system, two-way beam system or triple beam system. The one we have found to be most applicable to this design is the two-way beam system.

Two-Way Beam System

Considering a rectangular portion, in this system, columns run in two lines along the length. Connecting each column in the line is a girder and spanning the length between the two lines are the beams. Within this option, there are two ways in which the connections can be made. The first is framing the beams into the web of the girder. This method helps to minimize floor depth but still allows for some mechanical systems to be incorporated, but will most likely leave need for a drop ceiling for the larger components. The other option is layering the beams on top of the girder, thus providing more floor depth and space for utilities. Because lateral stability of the frame is important, the girders can be used to span the short axis of the building, therefore adding to this steadiness.

The frame dimensions are the next step in designing for the loading situation for a given building. In a one-way beam system the typical beam span range is 20 ft. to 32 ft. and beam spacing is 6 ft. to 15 ft. For larger spans, a more practical and economical option is the use of open web joists. Also made of steel, these truss-like structures replace the heavier steel beams of similar strength and spanning capabilities.

Steel Connections

Crucial to the stability of any steel framed structure are the connections between the members and between the members and foundation. These connections are most commonly created using components like steel tees, plates and angles. Also involved in connecting the members is the bolting, welding, or riveting of these smaller elements to each constituent. It is more than just the rigidity of a connection that determines which should be used; other factors include economic feasibility, efficiency of erection, and aesthetic necessity. After looking at these aspects, the design always falls back on the structural capacity. The American Institute of Steel Construction's (AISC's) *Steel Construction Manual* should be referenced to find the desired properties of the connecting elements and the requirements for each type of connection. There are two main fields of connections; one regarding beam and girder connections and the other addressing column and base connections. Furthermore, these two types are broken down into whether they are moment connections, shear connections, or semi-rigid connections. This determination aids in the selection and design process of the most suitable connection (Ching & Adams, 2001).

Steel Beam Connections

There are a great number of ways beams, girders, and columns can be connected. Here, each type (moment, shear, semi-rigid) will be explored with options described.

Moment Connections or Rigid-Frame Connections (American Institute of Steel Construction, 2005) must be capable of maintaining the original angle at which it was built. This is possible when it is constructed to resist moment under loading. Moment connections are typically formed using plates or bolts and welding them to the beam flange/supporting column. Other common practices include the use of web stiffeners, back bars, seat angles, and tab plates.

Shear Connections or Simple Frame Connections (American Institute of Steel Construction, 2005) must only be constructed to resist shear. This means they are capable of rotating under the gravity loads to which they are subjected. When using such connections, the use of diagonal bracing is typical because the structure needs to maintain lateral stability. This type of connection is dominated by the simple bolting or welding of angles or tab plates to the beam web and column.

Semi-Rigid (American Institute of Steel Construction, 2005) provide connection strength with some, but not total moment-resisting capability.

Steel Column Connections

Now that the manner in which beams and girders are normally connected to columns has been explored, it is also important to investigate the types of connections critical to column design. There are two main areas at which a column must transfer its load to another element. The first is where a smaller column is spliced on top of a larger one. The second is where the column meets the foundation or footing (McCormac, 2008).

Column Splices occur when one column must be connected to another. This usually occurs either where there are columns of different size and strength needed at different levels or where a height is simply too long for a single member to be feasible. Where the nominal size does change it is common practice for a butt plate to be welded to the end of each column, thus reassigning the load. In similar or same sized members bolted connections can be used. In this case a back plate can substitute for any difference in flange thickness. Finally, a welded butt connection is simply welding the two ends together. In order to maintain alignment before and during welding a plate is used.

The main component of a **Column Base** is the thick steel base plate. This allows for the load from the column to be distributed evenly to the concrete foundation. This plate is usually leveled first on non-shrinking grout and leveling nuts are necessary where the column exceeds a certain size. The column is then welded to the base plate. In some cases stiffener plates and anchor bolts are also used (Ching & Adams, 2001).

Steel Structure and Foundation Design

The steel framed structure is the most widely used construction practice in modern industry for large scale projects. The basics explained above have been assessed to come up with the most appropriate design. This design will ultimately transfer the building loads to the foundation, the details of which are discussed in the following section.

2.4 Foundation Design

Building foundations are plainly defined as structures that transfer loads from the superstructure to the ground beneath. Otherwise known as the "substructure," the foundation is usually built either mostly or entirely underground. Foundations must not only support the building above, but also anchor it in the varying types of soil, strength of bedrock, and water conditions below grade. This structure must also be large enough to accommodate the overall edifice above it and be able to distribute the loads sufficiently.

2.4.1 Material

Modern foundation construction is dominated by either reinforced concrete or concrete masonry units (CMU). We decided on reinforced concrete because of the ample size of the foundation structures and for constructability reasons. One governing part of the design process is to choose the correct mix and reinforcement materials. Throughout the design of the foundation we have used the following:

Spread Footings and Retaining Wall Structures:

- 50 ksi reinforcing steel
- 4 ksi concrete

Pool Slab on Grade:

- 60 ksi WWF
- 4 ksi Concrete

Tiebacks

- 150 ksi steel

2.4.2 Loading and Settlement

The loads that specifically affect the foundation include the vertical loads (dead and live) transferred from the columns in the superstructure and others that the building is subjected to like wind-induced sliding, uplift, and overturning. The foundation must also be resistant to reasonable seismic activity and the walls must provide sufficient opposition to the pressures of the surrounding soil and groundwater.

One of the greatest challenges that must be considered in the design and construction of a substantial foundation is the idea of settlement. Under loading, a sub-grade substance, mainly soil, would tend to consolidate, resulting in settlement downward. This consolidation can either occur quickly, in small amounts, or over longer period of time on a bigger scale, all depending on the type of soil and its moisture condition. To avoid as much settlement as possible, a foundation must be designed so that the system distributes the load as evenly as possible per unit area. When this is not done or unexpected settlement occurs in some areas and not others, it is called differential settlement and can cause cracks in the foundation and warps in the building frame itself.

Settlement, sliding, uplift, and overturning can all be resisted if a proper foundation is designed to not only fit the building above it, but also to effectively reassign the load from above to the ground. A determining factor in the efficiency of such a scheme is the actual type of foundation system and its competence in reacting to the conditions for which it is made.

2.4.3 Foundation Systems

Foundation is a relatively broad term. There are many possible arrangements that can transfer the loads of a superstructure to the ground below. One example is the generic basement which consists mainly of load bearing walls that support the building as well as hold back the earth. On a smaller scale, crawl spaces accomplish much of the same thing and are usually used just as access to utilities. Slab-on-grade is concrete placed directly on the earth's surface. However, this practice is usually just used in smaller buildings or dwellings. Because the building, in this case a collegiate athletic facility, is a rather large undertaking, certain types of shallow or deep foundations and retaining walls will be explored.

Shallow Foundations

Because the soil type most commonly encountered in the Worcester area and specifically on the WPI campus is glacial till, a relatively stable soil, a shallow or spread foundation should be sufficient (Haley & Aldrich, 2008). These are used when the bearing capacity of the subsoil is adequate near the surface where the building is to be erected. If this were not the case, deep foundations would need to be further investigated and piles that extend down to bedrock or soil with sufficient bearing capacity would have to be used. According to *Building Construction*

Illustrated: Third Edition, by Francis D.K. Ching and Cassandra Adams, the following are the main factors in the selection of a foundation system (2001):

- Pattern and magnitude of building loads
- Subsurface and groundwater conditions
- Topography of the site
- Impact on adjacent properties
- Building code requirements
- Construction method and risk

Footings

After taking these factors into consideration and looking into the foundation design of similar facilities built on the WPI campus, it is assumed that a shallow foundation will be used. Types of footings that could potentially be utilized can be found in Table 2.

Table 2: Footing/Foundation Types

Footing/Foundation Type	Characteristics/Purpose
Strip Footing	Forms the base of each foundation wall and runs along the entire length
Continuous Footing	Supports a row of columns or piers
Spread/Isolated Footing	Supports unconnected columns
Grade Beam	Transfers loads from load bearing wall to individual footings or mini-piles
Stepped Footing	Strip footings on changing levels to accommodate a sloped grade
Cantilever/Strap Footing	An isolated footing connected by tie-beam to another footing for balance
Combined Footing	A footing for a perimeter wall is extended to support interior column load
Mat/Raft Foundation	Thick, reinforced slab as a single footing for numerous columns or whole building

The size of footing depends heavily on the load that is vertically applied as well as on the bearing capacity of the subsoil. In the case that soil bearing capacity is low and the resulting footing size makes up more than 50% of the entire footprint of the building, it is assumed that a single slab mat or raft foundation would be a more practical and economical option. After performing some sample foundation designs based on loading from the columns and on an allowable bearing pressure of 8 ksf (Haley & Aldrich, 2008) we found that such a mat footing would not be necessary. It would be more practical and economical to use simple spread footings.

In the formation of foundation footings, some strict guidelines must be followed. In the New England area, it is important that the bottom of the footings be located at least 12 in below the frost line. Shallow foundation footings must also be constructed of concrete with a minimum compressive strength of 2500 psi at 28 days of procurement, have longitudinal temperature reinforcement, and sometimes have tensile reinforcement depending on if the footing projects more than half the wall thickness. There must be a minimum 6 in. of concrete above steel reinforcement and 3 in. minimum clearance between reinforcing bars and concrete surface.

2.4.4 Retaining Walls

Because the proposed site for the new Recreation Center is encompassing a large sloped area, the inclusion of a set of large retaining walls will be needed in the foundation design. A retaining wall is used to hold back the earth on the uphill side of a slope change and is needed when the slope of the grade exceeds the soil's angle of repose. The average angle of repose assumed for most soils is 33˚.The pressure that builds up against the resisting retaining wall grows proportionally to its maximum at the lowest portion of the wall.

In general, the pressure against the wall "may be assumed to be acting through the centroid of the triangular distribution pattern, one third of the distance above the base of the wall" (Ching & Adams, 2001). In certain cases where the grade continues to increase above the retaining wall, the extra loading is called the surcharge. In this design a surcharge will need to be added to account for the extra loading caused by the lobby area of the building. Also adding to these loads may be the vehicular traffic on the paved surface above the soil retained by the wall.

There are three main failures that form the basic challenges in designing retaining walls: overturning, horizontal sliding, and settling.

Overturning

Any pressure in excess could potentially overturn a retaining wall about its toe. To keep this from occurring, the design has to take into consideration the following statement: The resisting moment constructed into the wall along with the help of the soil presence on its heel should work to offset the effects of an overturning moment. This may require the assistance of additional tie-back supports extending into the soil.

Horizontal Sliding

Similarly, to lessen the likelihood of sliding, the wall weight and the amount of friction must be greater than the thrust created by soil pressure on the other side of the wall. Another way to aid in the resistance to sliding is the use of a key beneath the lowermost part of the footing as well as the same tie-back supports.

Settling

It is important for a retaining wall to undergo minimal settling because a number of failures could occur. These include the two mentioned above as well as the spilling over of earth above the wall which creates pressure against areas not meant to withstand it. To keep this from occurring, the downward forces cannot be greater than the bearing capacity of the supporting soil.

Types of Reinforced Concrete Retaining Walls

Because there are numerous variables in building conditions, such as slope of grade and soil type, there are different types of retaining walls that can be constructed for different applications. Figure 1 depicts only a few of the options for retaining wall construction, not all will end up being applicable to such a building as the Recreation Center.

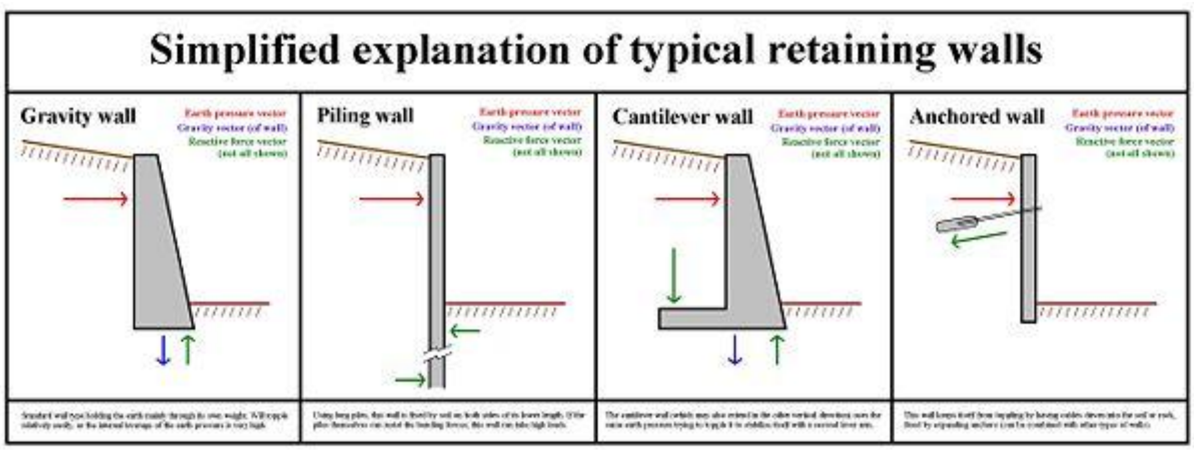

Figure 1: Types of Retaining Walls

A **Gravity Wall** is one that is able to resist sliding and overturning simply by how much it weighs and what its volume is. The hill leading down to alumni field is so large, the size of the building itself is substantial and gravity walls are typically only used in structures less than 10 feet high. Thus, this option would not be considered in the overall foundation design.

Similarly, a simple cantilever or **T-Type Cantilevered Wall** can usually only be used for walls up to 20 feet. Depending on the building design and desired depth, this could be employed except for the fact that two complete floors are to be incorporated into the space below the top.

There can be a combination, fitting these height requirements and desired architectural design, as well. A combination of **Anchored Walls** and cantilevered may be a plausible option, providing enough support for the proposed height of the retaining wall. This relies on the anchoring capabilities of the surrounding sub-grade soils or bedrock.

Because the soil is relatively stable in the WPI campus and surrounding areas, there is reason to avoid using **Piles** as a primary method of foundation and retaining wall design. It is a more involved construction process and if it can be avoided, that will speed up the project and result in a more cost effective design. Thus, the other type of reinforced concrete retaining wall adequate for such a strong grade slope and high wall is the **Counterfort Wall**. This uses a T-type cantilever structure in addition to triangle shaped cross walls. This stiffens the walls while providing additional weight for the base. These assisting walls should be spaced on center at half the wall-height. This may also prove to be a challenge because these walls would have to be rather large and may cut into crucial circulation space. The actual design process will take each of these options and some variations to come up with a reasonable and adequate retaining structure.

Other practices kept in mind during the design of the retaining wall structures are that the footing should be at least 2 ft. below the grade level or 12 in. below frost line, a drainage system and gravel backfill should be present at the base to alleviate water pressure, temperature steel for walls more than 10 in. thick, and steel reinforcement with 2 in. clearance in the walls and 3 in. in the footing.

2.4.5 Pool Design

The new facility will boast a pool of competition size for both swimming and diving. Though the pool in our layout is not full Olympic size, it will be designed in accordance with NCAA standards for pool dimensions. There will also be a moveable bulkhead for easy conversion to international competition lengths. The preferred requirements for the swimming section are that the "racing course should be 75 ft. –1 in. in length by at least 60 ft. in width, providing for not less than eight 7 ft. lanes with additional width outside lanes 1 and 8. A minimum water depth of 7 feet is desirable for competition." For diving, "facility may be separated from or incorporated with the swimming pool (Garland)."

Typical in-ground pools, of both residential and competition size, use “Shotcrete” or “Gunite” construction. This concrete producing pressure hoses to compact and place the concrete around reinforcement. The large scale of this particular collegiate pool requires more than simply spraying the concrete on grade. A foundation type holding area must be designed to stabilize the pool walls against the extreme earth pressure.

2.4.6 Overall Design Selection

The selection of design elements, both of the steel structure and concrete foundation, depends primarily on the applicable loads previously listed. Because of new technology and building options, these loads must now not only include the occupation, service, structural and architectural loads, but also loading from building incorporated green technologies. This concept is to be discussed in the coming section.

2.5 Green Building Options and Design Impact

In this first decade of the 21st Century the world is becoming more and more aware of the problems facing Mother Nature; problems we have created by simply using the earth as our place of residence. This ignorance of the long term effects is finally catching up and is catching the attention of human kind across the world. This is the major reason why, in recent years, the idea of “Green” building has become such an important development in the industry. This concept betters the efficiency by which a newly constructed or renovated building utilizes the resources available, both during construction and throughout its life. The overall purpose of this is to have a lasting impact on the environment and the human life it helps to sustain. A measure of how well a construction project accomplishes this is through an assessment by the Leadership in Energy and Environmental Design (LEED) Green Building Rating System designed by the U.S. Green Building Council. While this project is mainly structurally focused, it should be noted that there are a few aspects of our project that can assist in acquiring LEED Certification points through the structural design.

2.5.1 LEED Certification

Many private and public institutions are now, since 1998, striving to accomplish a LEED Certification as proof of their devotion to lowering the environmental impact of their buildings. This model inspires builders in their design practices and also provides healthy competition. This has proven very effective in leading the way towards a more effectively sustainable attitude in other industries as well.

The rating system for new construction and renovations specifically address the following major fields: sustainable building sites, water efficiency, energy and atmosphere, materials and resources, quality of the indoor environment, and innovation in design process. The ratings are performed by a board that grades the building and the manner in which construction was executed all on a points-based system. To be certified the project can receive anywhere from 26 to 32 points, for a level of Silver-33 to 38 points, for Gold- 39 to 51 points and lastly, for Platinum- 52-69 points (Council, 2008). The breakdown is as follows:

Sustainable Construction Site – 14 possible points

Water Efficiency – 5 possible points

Energy and atmosphere – 17 possible points

Materials and resources – 13 possible points

Indoor environmental quality – 15 possible points

Innovation and design process – 5 possible points

2.5.2 New Technologies

In order for the industry to take large steps in the Green Building arena and gain such certification, new technologies are quickly being discovered as new ways to lessen environmental impact. Along with site sustainable solutions, such as a great amount of material and waste recycling and lower machinery emissions, building-incorporated technologies are a great way to take advantage of available resources to the fullest. For example, there are technologies that allow the control and use of rainwater runoff, there are now ways to block sunlight or even harvest it for use, and there is new equipment, such as more environmentally friendly chillers, providing for more energy efficient ways to provide occupant comfort.

Green Roof

Used on the recently constructed East Hall on the WPI campus, a green roof is a great way to regulate runoff to the surrounding areas as well as utilize portions of the grey water for sustainable use. The basic design concept behind a green roof system is the use of beds of earth matter and vegetation. These beds absorb a large amount of rainwater and instead of it simply escaping via gutters and spouts onto the area below, it can be harnessed for use or released slowly over time, to avoid over saturation of the ground below. This oversaturation can cause many problems, not only for any surrounding structures and sub-grade construction, but in an urban environment, can release toxins from paced areas into the water cycle. The use of this grey water can reduce the overall use of public water supply as well have a great long term effect on the environment. An example can be seen below in Figure 2.

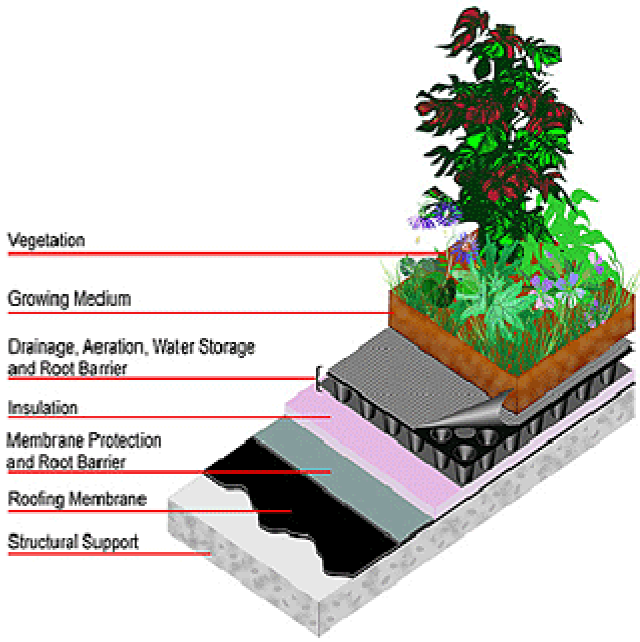

Figure 2: Green Roof Anatomy[1]

BIPV and Solar Shades

Another very popular set of new technologies are those specifically concerning the sun's rays. Building Integrated Photovoltaic (BIPV) is a newly popular use of solar cells. This practice can take the form of many different applications, including using solar voltaic roof tiles, integrating very thin solar cells into the actual glazing of windows and curtain walls, and even simply attaching larger solar panels on the exterior or incorporating them into the masonry. The power captured by these units can transfer and store energy in a large battery. This energy can later be used to power parts of the building lighting or mechanical system among other things. A second example of new technology using the sun is exterior solar shades. These are now being

[1] http://www.countyflatroofing.com/images/green-roof-2.gif

used worldwide on buildings that have extensive exterior glass curtain walls. The shades, much larger than interior blinds, automatically follow the path of the sun, either blocking or letting in as much sun as is desired according to the interior temperature controls. What's more, these shades can tie entirely into the mechanical system, further helping to control the interior climate in a more energy efficient way. Another, more simple option, would be the polarization of the curtain wall itself. These are just a few very interesting new tools that can help the building industry work towards a lower environmental impact, and if desired, a much coveted LEED Certification (Strong, 2008).

2.5.3 Design Impact

The use of Green Building practices are present at all levels and stages of projects sites devoted to their use. Because of this, these concepts will come up many times later in the development of this project, specifically in the project management and control as well was cost estimating procedures. While these options are being considered for construction and maintenance of the building, it is important to start with the structural design stages to avoid further conflicts with construction. This loading in particular will be a key to determine the accurate weight the structural frame must support. Thus, the group will decide on the technology to be used and will design the structure accordingly.

2.6 Preconstruction Activities

The preconstruction process is a multi-facetted, organized plan of attack starting from the design stages through the execution of construction. Excluding the design phase, parts of which have been outlined above, the other major activities must be controlled simultaneously in a way that allows the project to move forward smoothly and cost-effectively. The ultimate goal for a project is to be completed on-time, under budget, and to the best level of quality possible. Three of these major processes are the permitting and zone approval, cost-estimation, and project management and control. These processes continue into the actual construction of a project and therefore must be organized and prepared under constrictions of money and time.

2.6.1 Permitting and Zoning

Permitting

Before construction of a building is started, all the necessary permits must be acquired. Permits are designed to insure that all regulations and requirements are met in the initial stages of building to provide health and safety to the community members as well as those involved in the project. All cities have created their own set of guidelines for obtaining the mandatory permits, however most cities follow a similar basic step by step procedure:

1. Submit Application with Drawings and Fees
2. Obtain Permit
3. Obtain Sub-trade Permits
4. Inspections
5. Receive Certificate of Occupancy

The city of Worcester follows steps related to these, but requires individuals to go through different departments within the city to get a permit. There are eight different departments, four of which will be contacted for permits concerning the new Athletic Facility. These four departments: the Department of Public Works and Parks, the Fire Department, the Inspectional Services Department, and the Planning and Regulatory Services, all have different requirements and types of permits necessary to start our project.

Appendix B lists out the steps to obtain the permits (Worcester, 2008).

Zoning

As the years go by, the once state-of-the-art facilities become insufficient for modern day use. Unless these facilities become part of the historic district, some degree of reconstruction will take place to recapture its functionality. While innovative design is encouraged, a set of guidelines must be followed in order to "promote the health, safety and general welfare of the public and to contribute to the implementation of the City's ongoing comprehensive planning process" as stated in the City of Worcester Zoning Ordinance which is more simply referred to as the Zoning Ordinance. To accomplish this goal, new constructions or renovations must:

1. Creates and maintains conditions under which people and their environment can fulfill the social, economic and other needs of present and future generations;
2. Facilitates the adequate and economic provision of transportation, water supply, drainage, sewerage, schools, parks, open spaces, light and other public requirements;
3. Encourages the creation and preservation of housing of such type, size and cost suitable for meeting the current and future needs of the City;

4. Protects against: overcrowding of land; air and water pollution; use of land incompatible with nearby uses; undue intensity of noise; danger and congestion in travel and transportation; and loss of life, health, or property from fire, flood, panic or other dangers;
5. Protects natural resources as well as the scenic and aesthetic qualities of the community;
6. Promotes the preservation of historically/architecturally significant land uses.

With a set of rules to be followed, detailed in the Zoning Ordinance, all designs permitted by the city would be confined to the list of objectives above and thus together achieving the greater goal of further development of the City of Worcester.

As a university, Worcester Polytechnic Institute, under the zoning district of "Institutional District," falls under the category of "Institutional, Educational" or labeled as "IN-S" (See Appendix C). With a classification of educational institutions, City of Worcester mandates that all structures are required to be placed fifty feet away from the property line[2]. In terms of structures constructed between fifty-one and one hundred feet, the limitation of the abutting[3] zoning district shall be in effect. Beyond one hundred feet, no height limitation is in place.

The purpose of the Zoning Board of Appeals (ZBA) is to provide the ability for work to be done outside the confines of the zoning ordinance subjected to approval. Their duties include appeals, special permits, variances, and pre-existing nonconforming uses (Worcester, 2008).

2.6.2 Cost Estimating

Cost estimating spans a project from the original request for plans (RFP) to the completion of a final budget. Simply stated it is an actual estimation of the cost of materials and labor for a given construction undertaking. In the beginning stages, cost estimating serves as a projected budget, is continued as value engineering and change orders are brought forth or requested by the architect or client, and then it is used to come up with the final cost of the

[2] The property line shall be defined as the exterior boundary line of the Institutional property, as recorded on the deed, which separates the Institutional property from adjacent properties or streets.

[3] Abutting – Having a common border with, or being separated from such common border by a right of way, alley or easement.

project. For this project it will be most beneficial to understand the initial stages of the process, the actual cost estimation of materials used.

This MQP mainly focuses on the structural design of the foundation and superstructure for the new facility. In such a case, cost estimating will serve to give an idea of what the amount of materials, whether concrete, steel reinforcement, or hot-rolled steel, will cost overall. The method for doing this will be further discussed in the methodology section.

2.7 Execution

Because this project focuses mainly on the structural design aspects of a large project, WPI Recreation Center, there will not be much focus on the actual execution of construction, besides the practice of some cost estimating skills. Though this is the case, it is important to recognize that the construction of such a project is a culmination of all the things previously discussed in the background section. No foundation would be constructed if the footings were not first designed, the materials chosen, procured, and the labor scheduled. No superstructure would be built to house and support the new facility if no members were analyzed, designed, procured, an connections were not designed and welded, labor not scheduled and steel not fireproofed and prepped for fit-out. All these aspects, design, management and construction all work in a balance to best produce a creation first envisioned by the client and architect.

3.0 METHODOLOGY

3.1 Loads

3.1.1 Dead Loads

Similar to the live load, the dead load values used were found in the *ASCE loading manual* as well as in the *Mass Code*. Other calculations were used to apply this information to the geometry/quantity of each material used.

Because a majority of the building will have exposed ceilings and duct work, and because the allowance for duct work is negligible, we have not included these values in the design. The dead load that has been included is as follows.

<u>Concrete:</u> (density = 150 pcf)

- Interior slab = 4" thick x 150 pcf → 50 psf
- Lounge roof slab = 7" thick x 150 pcf → 87.5 psf **
- Lobby roof slab = 5"thick x 150 pcf → 62.5 psf

**Lounge roof slab will be changed to 5 in.

<u>Beam Weight:</u> Depends on member design and selection.

<u>Exterior Curtain Wall:</u> (based on the following assumptions)

- All members designed for masonry curtain wall
- All curtain walls are one wythe deep, running bond.
- 7 bricks per square foot with a weight of ~6 lbs. each
- The area carried per member is the height of wall above the girder multiplied by the span.
- The wall load is transferred uniformly along the center of the girder
- Final load ≈ 42 psf

3.1.2 Live Loads

To obtain the values for live load we referenced both the *ASCE Minimum Design Loads for Buildings and Other Structures* as well as the *Massachusetts Code*. The following is an outline of the Live loads used in the design of the horizontal members of the facility.

Interior:

- Gymnasium/Fitness Center Space: 100 psf *
- Corridor: 100 psf
- Office Spaces: 50 psf

*This loading was used in the majority of analysis for interior supporting members. The main building was conservatively designed entirely using this load.

Exterior/Roof:

- Green Roof: 115 psf

Since the construction of a green roof did not require structural calculations, this section instead discusses how a green roof was put together. Starting from the bottom, an insulating layer, waterproof membrane layer, root barrier layer, drainage layer, water retention layer, filter fabric layer, soil mix layer, and plant layer together made up the eco-roof design. The insulating layer prevents heat transfer between the interior and exterior of the building. The waterproof membrane layer would stop liquid from penetrating the roof and creating moisture damage to the interior of the building, including the structural members. The root barrier layer limits the growth of the roots from the plants above so that the waterproof members below would not be under stress. The drainage layer carries away the excess water to avoid ponding, which might have exceeded the weight restriction for the structural frame. The water retention layer provides a collection zone of water for the plants on top. The filter fabric layer prevents fine particles from being washed out of the roofing system, which ensured overall efficiency with the roof. The soil mix layer was specially created for eco-roofs. Its lightweight material reduces loading on the roof structure. Lastly, the selection of plants as the top layer stabilizes the lower layers of the eco-roof (Kibert, 2007). At the same time, it also creates an environmentally friendly atmosphere for the building. An extensive green roof, designed to be low maintenance, is usually not accessible for the public since they range from 15 psf to 50 psf for design dead loads. On the contrary, an intensive system is designed to create an actual roof garden on top of buildings. Since their design dead loads range from 80 psf to 150 psf, they provide accessibility for the public. For our structural design, a roof system of 15 inches, including all the parts mentioned above, was chosen. This meant that the dead load would translate into 115 psf.

3.1.3 Snow Load

The following procedure is for snow load, in Worcester, on a flat roof (American Society of Civil Engineers, 2006).

1. Snow Load = P_f = 0.7 $C_eC_tCIP_g$
2. Determine ground snow load- P_g (State Publications and Regulations Division, 2008)
    a. 55lb.ft^2 (900)
3. Table 7.2 to solve for C_e
    a. Determined that -terrain C
    b. Determined that -fully exposed
    c. Therefore C_e=0.9
4. Solve for thermal factor
    a. C_t = 1.0
5. Solve for importance factor I
    a. Using Table 7.4 and 1.1 → Category III
    b. I = 1.1
6. Consider partial loading → n/a
7. Consider snow drifts on lower roof if hc/hb ≥ 0.2 → n/a
8. Consider roof projections → n/a
9. Consider ponding
10. Ultimate Snow Load = P_f = 0.7 x 0.9 x 1.0 x 1.1 x 55lb/ft^2

3.1.4 Wind Load

On top of live and dead loads supported by girders and columns, structural members are also exposed to natural factors like wind load. Depending on the height of the building, the effect of wind can become a major contributor when dealing with load combinations. All of the factors and the design process are detailed in Appendix D and the zones are explained by ASCE figure 6-17 (American Society of Civil Engineers, 2006).

3.1.5 Seismic Load

- From table 1-1 ((American Society of Civil Engineers, 2006), occupancy for athletic facility is category 3
- S_{DS}= design spectral response acceleration parameter at short periods
  - S_{DS}= (2/3)*S_{MS} = (2/3)*0.25 = 0.167
  - S_{MS}= Fa*Ss = (1.0)*(0.25) = 0.25
    - Fa = site coefficient
    - Ss = mapped MCE Spectral response acceleration at short periods
  - USA is site class B
    - From Figure 22-1, Ss = 25% seismic activity, which is 0.25
    - From table 11.4-1, when Ss = 0.25, Fa = 1.0
- S_{D1} = design spectral response acceleration parameter at a period of 1s
  - S_{D1}= (2/3)*S_{M1} = (2/3)*0.06 = 0.04
  - S_{M1}= Fv*S1 = (1.0)*(0.06) = 0.06
    - Fa = site coefficient
    - S1 = mapped MCE Spectral response acceleration at short periods
  - USA is site class B
    - From Figure 22-2, S1= 6% seismic activity, which is 0.06
    - From table 11.4-2, when S1 = 0.06, Fv = 1.0
- From table 11.6-1, seismic design for athletic facility is category B when $S_{DS} \geq 0.167$
  - Redundancy factor rho (p) = 1.0 due to seismic design category B
- From table 12.2-1, athletic facility will be an ordinary steel moment frame
  - Overstrength factor (Ωo) = 3
- Determined load combinations for strength design (LRFD)
  - $(1.2 + 0.2S_{DS})D + pQ_E + L + 0.2S$
  - $(0.9 – 0.2S_{DS})D + pQ_E + 1.6H$
- Determined load combinations for strength design with overstrength factor (LRFD)
  - $(1.2 + 0.2S_{DS})D + \Omega o\ Q_E + L + 0.2S$
  - $(0.9 – 0.2S_{DS})D + \Omega o\ Q_E + 1.6H$
- Q_E = effect of horizontal seismic forces from V
  - V = Cs*W = 0.06*W
    - Cs = seismic response coefficient
    - W = effective seismic weight
  - Cs = S_{DS}/(R/I) = 0.167/2.8 = 0.06
    - R = response modification factor = 3.5 from table 12.2-1
    - I = occupancy importance factor = 1.25 from table 11.5-1
  - Cs value must not exceed Cs = (S_{D1}*TL)/(T^2*(R/I)) = (0.04*6)/((18)^2*2.8) = 2.65
    - TL = long period transition period = 6 from Figure 22-15

- T = fundamental period of the structure = Ct*hn^x = 0.0724*(82*12)^(0.8) = 18
    - Ct and x = approximate period parameters = 0.0724 and 0.8

Load Combinations using Strength Design (LRFD)

1. 1.4(D+F)
2. 1.2(D+F+T) + 1.6(L+H) + 0.5(Lr or S or R)
3. 1.2D + 1.6(Lr or S or R) + (L or 0.8W)
4. 1.2D + 1.6W + L + 0.5(Lr or S or R)
5. $(1.2 + 0.2S_{DS})D + pQ_E + L + 0.2S$ or $(1.2 + 0.2S_{DS})D + \Omega o\ Q_E + L + 0.2S$
6. 0.9D + 1.6W + 1.6H
7. $(0.9 – 0.2S_{DS})D + pQ_E + 1.6H$ or $(0.9 – 0.2S_{DS})D + \Omega o\ Q_E + 1.6H$

3.1.6 Roof Loads

With the other load calculation complete, the roof load can then be determine using the combination of the relevant calculated loads (live load, dead load, wind load, snow load, and roof live load). After studying which of the loads will play a part in the roof design and when all of the different load combinations are accounted for, the equation which yields the highest force will be the critical load case for roof load. In our particular case, because of the different wind load zones on the roof, our roof load is divided into 3 different parts. Refer to wind load for the location of each zone (American Society of Civil Engineers, 2006).

Zone 1 = 1.2(D) + 1.6(S) + 0.8 (W) = 182.91 psf

Zone 2 = 1.2(D) + 1.6(S) + 0.8 (W) = 196.03 psf

Zone 3 = 1.2(D) + 1.6(W) + L + 0.5(S) = 210.27 psf

3.2 Structural Member Design

3.2.1 Beams

Determining Beam Size

To design the beams within our structure, we started by designing a sample girder with beams spaced at multiple distances to determine the appropriate spacing of beams per bay. With a majority of the bays in our building being 25 x 30 ft. we discovered that a beam spacing of 5 ft. was best. Other values that we had to fill in to solve for the max load were:

The density of concrete: 150 lb/ft3

Our beam span (L): varies

Live Load: varies

Slab thickness: 7 in. roof, 5 in. for the green roof & 4 in. everywhere else

Once these values were determined we solved for the Slab weight:

Slab weight= Concrete density * Spacing * Slab thickness

After solving for the slab weight we estimated a value for the weight of the beam and added that to the slab weight to find W_d. The WL was determined by multiplying the live load by the tributary width which was equal to the spacing. With these values multiple load combinations were used to determine the governing Wu value, which for all of the beams was:

Wu = 1.6WL + 1.2Wd

After Wu was solved for we were able to solve for the max moment:

$$\frac{Wu \ * \ L2}{8} = M_u$$

Using the steel Manual with 50ksi steel, a beam size was selected where the ΦbMpx > Mu. Once the beam was selected the process above was repeated with the actual beam weight to check that ΦbMpx remained greater than Mu (McCormac, 2008).

Check for shear

To check shear the following equation was used:

$$V_u = W_u * \frac{L}{2}$$

Checking that $\Phi_v V_n$ from the steel manual was > Vu

Check for Deflection

To check deflection we solved for the max permissible deflection which is equal to:

$\frac{L}{240}$ for members supporting the roof and $\frac{L}{360}$ for all other members

The actual deflection was solved for by using the expression below:

$$\frac{5W_u * L^4}{384 * E * I}$$

A majority of our beams failed the deflection test because the actual deflection was greater than the permissible deflection. As a result we used the following equation to solve for the required I_x value needed to meet the deflection requirements:

$$\frac{\text{Actual Deflection}}{\text{Max Permissible Deflection} * I_x}$$

Once a new beam was selected the above process was repeated to check the moment and shear. Appendix E displays a sample of a beam calculation (McCormac, 2008).

3.2.2 Girders

Interior Girder Design

As with the beam design procedure, we started with a sample calculation. From this we created a comprehensive spreadsheet to design for the girder sizes based off the known information.

Girders carrying beam loads and uniform dead load:

First, the beam loads supported by the girder had to be turned into point loads. To do this we took the Wu from the appropriate bay/beams already designed and multiplied that by the tributary area the girder was meant to support. The tributary area varied in different zones where some areas were open to the floors below. In such a case, only one side, or half, the tributary area was used to find the point loads. These point loads were then multiplied by the number of points at which they met the girder and then added to the factored dead load.

The next step was incorporating the dead load. This uniform load, for the interior girders, was simply the weight of the girder itself.

Thus, we were able to find the maximum moment using the resulting Fy_1 and Fy_2 values by summing the areas under the shear curve. This then allowed us to determine a sample beam size.

The next steps were the shear and deflection checks. See the beam design section above as the girder tests follow the same procedure.

Girders not carrying any point loads:

We found that some girders acted like beams. These members followed the same procedure for determining the size of beams. Appendix F shows a sample girder calculation (McCormac, 2008).

Exterior Girder Design

In designing for the exterior girders both of the above cases were found: girders supported dead load and point loads, as well as girders that act as beams. The main difference in design for these members was that one must take the dead load of the exterior curtain wall into account and must also check the effects of wind load on that wall bay.

To design the exterior girders supporting interior point loads as well as the curtain wall, the procedure was relatively the same. Because it was assumed that the curtain wall area above the member was transferred uniformly to the girder, it could be treated as an addition to the dead load provided by the beam weight. To check for wind load, the strength of the 50 ksi steel was compared to the total shear added to the stresses applied to the bottom flange of the girder by the wind load (Fw).

The other exterior girders were broken down into two categories; those that carried the curtain wall and one half their tributary area of slab weight and those that solely supported their self weight and exterior curtain wall. Both, in the instance, were calculated like beams, but also included the check for wind load. The design of the girders that carried only their self weight and curtain wall were controlled mainly by deflection (McCormac, 2008).

3.2.3 Columns

Once the beams and girders had been selected, the next step in the structural design was to calculate proper sizes for columns to support the weight expected to be experienced. By first determining the maximum shear transferred to each column of each floor from beams and girders, we obtained a basic limitation for the minimum allowable strength each column must endure. After finding out the minimum strength needed, the next step was to choose an appropriate W-shape section that would satisfy the requirement, with consideration of the effective height. Lastly to ensure the design is proper, frame analysis was done to double check axial and bending stress exerted on each column. The steps for column design are shown below and the calculations can be seen in Appendix G (McCormac, 2008).

1. Determine Axial Load P_u from all girders being supported by the column in question
2. Find estimate required area by: $P_u / \phi_c F_{cr}$, assume $\phi_c F_{cr} = 37.5$ to start.
3. Select trial column based on estimated required area.
4. Look up r_y from AISC Table 1-1 for selected column.
5. Calculate (KL/r_y) to find true $\phi_c F_{cr}$ value.
6. Calculate available strength $\phi_c P_n$, where $\phi_c P_n = \phi_c F_{cr}$ x Cross section Area of column (from AISC Table 1-1)
7. Compare $\phi_c P_n$ to P_u to verify weather column selected can support predetermined loads.

3.2.4 Trusses

Select Sample Truss Configuration for the Roof

In order to determine what type of truss to design for our building's roof, we had to first select a configuration that would support our roof load combination. We began researching different styled trusses and decided on a flat roof truss known as the Warren Configuration (See Figure 3).

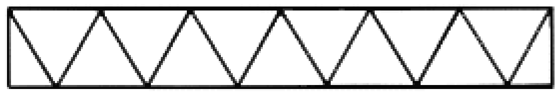

Figure 3: Basic Roof Truss Design

Solve Member Forces using Method of Joints

Once our truss configuration was chosen, we began solving for the member forces using the method of joints (see Appendix H). We solved the reaction forces of the entire truss first by applying the roof loads as distributed loads. This was followed by solving the unknown forces at each joint, taking into account the roof loads at each individual joint. Once the unknown forces had a value, we were able to solve for all the member forces of the truss (Hibbeler, 1985).

Solve Member Forces using RISA 2D

Once the truss was designed using the method of joints, we checked our values by creating our truss as a 2D model in RISA 2D (See Figure 4). We first established an appropriate grid for our truss in order to account for each joint. Next, we selected the appropriate units to have our member forces solved with as well as the right global parameters. Once this was established, we went ahead and created the joint and member labels for our truss. This was followed by setting the correct boundary conditions for the truss along with the basic load combinations (BLC). After the BLC were placed, we created the load combinations acting on our roof and the individual loads acting at each joint accordingly. We then solved the truss and observed our data (See Appendix I).

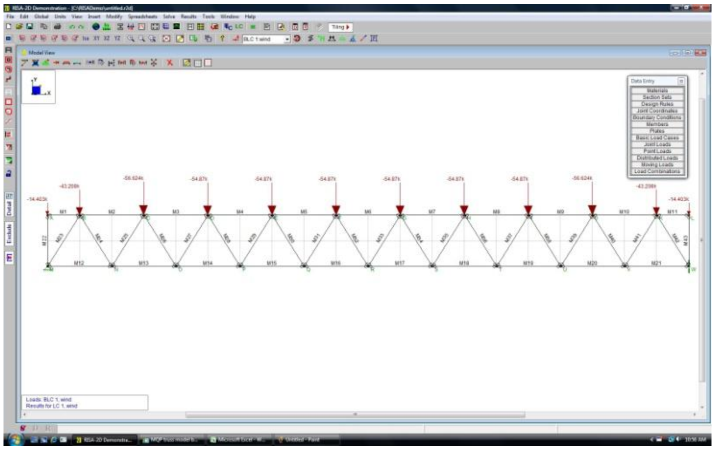

Figure 4: RISA 2D Roof Truss Design Sketch

Select Sample Truss Configuration over the Pool

Since we already determined the type of truss to use for our building's roof, we determined that using the same type of truss for our pool would be more than adequate (See Figure 5). The loads exerted on the pool truss would be smaller than the loads exerted on the roof truss, allowing us to design the pool truss to be smaller. Once our pool truss was solved, we then analyzed our data (See Appendix I).

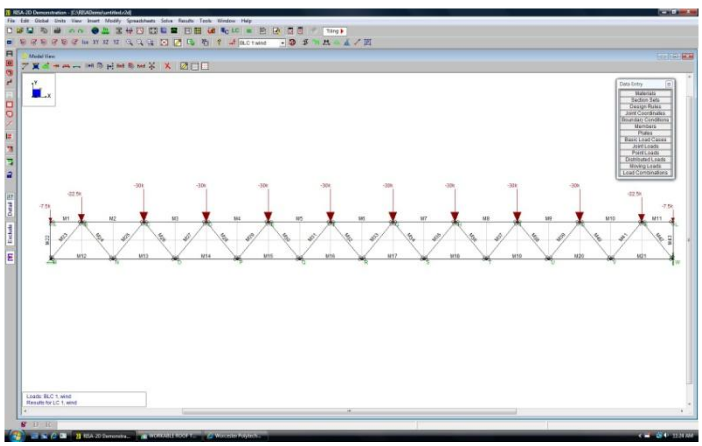

Figure 5: RISA 2D Pool Truss Design Sketch

3.2.5 Bolted Connections

Sample Truss Connection

Our trusses for both the roof and pool consisted of a bolted connection that joined the tubular shaped members with the wide flange members. By using this bolted connection, we ensured that our trusses would not resist any moment forces, as they would if a welded connection was selected. As for what typed of bolted connection to use, we conducted a sample calculation of a bolt connection using two plates, known as a butt joint, and determined that this design was suitable to use for the forces exerted on both trusses.

The procedure for the sample calculation of the two plate bolt connection consisted of the following (McCormac, 2008):

Select Type of Bolts to use

For Tubular shaped member:

Axial force (Pu) = 325.2 K

Diameter = 8.63 in.

2 plates will be ½ in. thick on each side

Try using 7/8 in. diameter A325 Bolts

Determine Bearing Strength of one Bolt

Clear Distance (Lc) = lesser of 2- (chosen bolt diameter + 1/8 bolt clearance)/2

Or

3-2*(chosen bolt diameter + 1/8 bolt clearance)/2

Lc = 2-(7/8 + 1/8)/2 = 1.5 in.

Or

3-2*((7/8 + 1/8)/2) = 2 in.

Rn = 1.2*Lc*t*Fu ≤ 2.4*d*t*Fu

Rn = 1.2*1.5*8.63*58 = **901 K** ≤ 2.4*7/8*8.63*58 = **1051 K**

Determine Shearing Strength of one Bolt

Rn = Fn*Ab (area of 7/8 in. bolts) = 60*(2*0.6) = 72

$\Phi = 0.75$

φRn = 0.75*72 = 54 K

Determine Number of Bolts required

Required = Pu/φRn = 325.2/54 = 6.02

Use 7-7/8 in. Bearing Type A325 Bolts

(Please see Appendix J *for original sample calculation document)*

3.2.6 Sample Frame Connection

The frame connection for our building consisted of a bolted connection that joined the girders with their respective columns. Using this approach helped limit the moment forces acting upon each floor of the building. The type of bolted connection we decided to use was the: single plate shear connection. Through a sample calculation, we determined just how many rows of bolts, plate sizes, fillet weld sizes, and bolt sizes we needed to account for the forces acting on our girders and columns (McCormac, 2008).

The procedure for our sample calculation was as followed:

Establish Parameters

Look at W8x28 column with a W16x26 girder located on the first floor

Try using ¾ in. A325-N high strength bolts and E70 electrodes

Fy = 36 KSI for plate

Fu = 58 KSI for plate

Fy = 50 KSI for column and girder

Fu = 65 KSI for column and girder

*Column provides rigid support; begin with 4 rows of bolts, ¼ in. plate, and 3/16 in. fillet welds (Taken from AISC Manual Table 10-9(a)

Determine Load Combination acting on the Column and Girder

RD = tributary area * [concrete slab thickness*weight of concrete*on-center length + weight of steel]

RD = 30 ft. * [(5/12)*150*25+26] = 47.7 K

RL = live load * bay area = 50*(30*30) = 45 K

Ru = 1.2*(47.7) + 1.6*(45) = 129.24 K

Check Design from AISC Manual Table 10-9(a) φRn = 52.2 K < 129.24 K **N.G.**

*Try 8 rows of bolts, 5/16 in. plate, ¼ in. fillet welds, 7/8 in. A325-N high strength bolts and E70 Electrodes

φRn = **131 K > 129.24 K** **OK**

Use 8 rows of bolts, 5/16 in. plate, ¼ in. fillet welds, 7/8 in. A325-N high strength bolts and E70 Electrodes

(Please see Appendix K for original sample calculation document)

3.2.7 Frame Analysis

Apply appropriate Wind Loads for each floor

In order to ensure that our frame analysis for our building was accurate, we first determined how much load the wind was placing on each floor. Since our floors were not the same height, the wind loads for each floor varied. Therefore, we took the wind loads of the appropriate zoning for our building, multiplied it by the tributary area of the corresponding floor bay, and then multiplied the resultant by the tributary area of each floor. Since each floor bay is 30 x 30 ft, the wind loads for each floor on the long-side of our frame were the same as the short-side of our frame (Hibbeler, 1985).

Solve the Reaction Forces of the Frame (Long-Side and Short-Side) using the Portal Method

With the wind loads for each floor in place, we began solving for the reaction forces of our frame by using the Portal Method (See Appendix K) .We first placed hinges at the centers of both the columns and the girders of our frame. Then we solved for the column shears of each floor, starting with the top floor. Once we obtained those values, we analyzed each part of the frame, starting with the top corner segment of the top floor, followed by the segments across each floor until finally reaching the base columns of our building (Hibbeler, 1985).

3.3 Foundation and Concrete Design

3.3.1 Footings

To determine the footing size that would be necessary for our foundation we added up the total load from each of the five floors that acted on each specific column to find the total factored load. We began by choosing a sample load that we had and plugged the load into a spreadsheet which can be seen in Appendix L. (Coduto, 2001) By using the spread sheet we were able to estimate the size of one of our footings needed and multiply that by the number of footings we had. We determined that the total area of the footing was less than 50% of the area of the floor of the building. Therefore it was appropriate to use spread footings as opposed to a mat foundation. Once we determined that we would use spread footings we solved for the size of the footings based on three different size loads. Footings were designed for loads from 0-372 kips, loads from 373 kips-758 kips and loads from 759 kips-934 kips. Based on these loadings we carried out the following steps to determine the size of the footing and type of reinforcement necessary (Brown, 1996); (Coduto, 2001). The steps are shown below and the calculations can be seen in Appendix M.

1. Estimate the depth of column, h, to be 1 to 2 times the column dimension
2. Solve for the allowable net soil pressure q_{net}

   q_{net}=allowable bearing pressure –(weight of footing + soil + floor + floor load)
3. Solve for the required area to find footing dimensions

   Required Area= $\frac{P}{q_{net}}$ for a square footing take the square root of the req. area
4. Solve for the factored net soil pressure

   $q_{nu}=\frac{P}{\text{Area of Footing}}$
5. Two-way Shear Check (punch)

   Solve for d= h –concrete cover –bar diameter

   Concrete cover is taken to be 3in and bar diameter as 1in

$$V_u = q_{nu} * (\text{trib. area}) = q_{nu}\left[(\text{Area of Footing})^2 - \frac{(\text{width of column } + d)^2}{12}\right]$$

V_c = to the smallest of:

$\left(\frac{2+4}{B_c}\right)\sqrt{f'_c}b_0d$ b0=perimeter of critical section- d/2 from face of column

$\left(\frac{a_s}{B_0/d}+2\right)\sqrt{f'_c}b_0d$ d=depth at which reinforcement is placed

$4\sqrt{f'_c}b_0d$ Bc=ratio of long side to short side of column

as=40 for interior columns, 30 for edge, 20 for corner

To check shear:

$V_c > \frac{V_u}{\Phi}$ with =0.85

6. One-way Shear Check

$$x = \frac{(\text{width of footing}*12) - \text{width of column}}{2} - d$$

$$V_u = q_{nu}(\text{tributary area}) = \text{factored net soil pressure} * (\text{width of footing} * \frac{x}{12})$$

$$V_c = 2\sqrt{f'_c}bd$$

$V_c > \frac{V_u}{\Phi}$ with =0.85

7. Design flexure reinforcement

$$y = \frac{(\text{Width of footing}*12) - \text{width of column}}{2}$$

$$M_u = q_{nu}\left(\text{width of footing} * \frac{y}{12} * \frac{y}{2*12}\right)$$

$M_n = A_s f_y(d - \frac{a}{2})$ Assume (d-a/2)= 0.9d

$$(A_s)_{req} = \frac{M_u/\Phi}{f_y(0.9d)}$$

(As)min=0.0018bh

Choose bars such that As>(As)min

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

Mn=Asfy(d-$\frac{a}{2}$)

Mn> $\frac{M_u}{\Phi}$ with =0.85

8. Check the development length

$$l_{db} = 0.04\frac{A_b f_y}{\sqrt{f'_c}}$$

Determine minimum development length

$$l_{d-min} = 0.03 d_b \frac{f_y}{\sqrt{f'_c}}$$

Choose ld to be the larger value.

The bar length available from the location of the max moment on each side is

y-concrete length = width of tributary area - 3in concrete cover

Bar length available must be > ld

9. Bearing Pressure Check

$$e = \frac{M}{P+W_f}$$ Wf= Weight of footing

If e ≤ B/6 then the bearing pressure is distributed trapezoidally

If e ≥ B/6 then the bearing pressure is distributed triangularly

Trapezoidal distribution:

$$q_{min} = \left(\frac{P_u+W_f}{A} - U_d\right) * (1 - \frac{6e}{B})$$

$$q_{max} = \left(\frac{P_u+W_f}{A} - U_d\right) * (1 + \frac{6e}{B})$$

$q_{max} < q_a$

10. Settlement Check

Solve for induced vertical stress:

$$\Delta\sigma_2 = \left[1 - \left(\frac{1}{1+\frac{B}{2Z_f}}\right)^{1.76}\right] * (q - \sigma'_{ZD})$$

Compute the effective stress:

$$\sigma'_{Zf} = \sigma'_{Z0} + \Delta\sigma_Z$$ Layer 1= B/2 Layer 2= B Layer 3= 2B

$$\sigma_{Z0} = \lambda H - U \qquad\qquad H = ½(\text{Layer 1} + \text{Layer 2} + \text{Layer 3})$$

Check $\delta < \delta_a$

3.3.2 Retaining Wall Design

The grade change between the upper parking lot level and the lowest level slab on grade of the building is thirty feet. This is seen now as the hill leading from the WPI Campus main level down to Alumni Field. This slope presents the need for a retaining structure at both the East and South wall. In the Geotechnical report it is suggested that a type of permanent, tied- back wall be used and the space between that wall and the building be backfilled with a compressible material (Haley & Aldrich, 2008). We chose to follow this guidance and design a reinforced concrete structure separate from the building itself.

The following are the steps and formulas used to achieve the final retaining design for the East wall (Wang, Salmon, & Pincheira, 2007):

1. Find reinforcement ratio (ρ) where $\rho = 30\% \times \rho_b$
   - From this information we can calculate Rn which will be used a number of times during the design process.
     - $R_n = \rho f_y(1\text{-}0.5\ \rho m)$ where $m = F_y / 0.85f'_c$
2. Height of wall -> Allow for 4 ft. below grade for frost penetration + original wall height
3. Estimate Thickness of Footing -> Use 7 to 10 % of wall height
4. Calculate Base Length -> Consider equilibrium of factored loads
   - Find P1 and P2 (Appendix N) and W (approximate weight of concrete structure)
   - Sum moments around point where stem meets the footing at front face of wall
   - Solve for length x (Appendix N)
   - For granular material the resultant soil pressure is to be at outer edge of middle third of footing. Therefore; Base length = 1.5x
5. Stem Thickness -> Regard bending moment and shear
   - For moment
     - Find My from top

- Find Mu @ the bottom of stem wall (See Appendix N)
- For desired Rn (as calculated previously)
  - Required d = $\sqrt{\frac{M_u}{\emptyset Rnb}}$
- Total thickness = d + 2 in. cover + 0.75 in. est. bar radius
- Use 12 in. practical thickness at top of wall (consistent)

- Check Shear at base of wall
  - Find Vy
  - Find Vu using a 1.6 overload factor
  - $\emptyset Vc = \emptyset(2\sqrt{f'c})bd$
  - $\emptyset Vc > Vu$ **If not, use above equation to find d
  - Total thickness = d + 2 in. cover + 0.75 in. est. bar radius

6. Factor of safety against overturning.
   - Calculate forces resulting from weight of materials (i.e. wt. of concrete in wall, wt.concrete in footing, wt. of soil material).
   - Using these forces, calculate the total resisting moment acting downward on footing.
   - Calculate resultant from heel = M/W
   - Calculate resisting moment = W(base length – resultant from heel)
   - Calculate overturning moment = P_1 (1/2h) + P_2 (1/3h)
   - Safety Factor must be greater than or equal to 2.0.

In this case the original overturning moment is far from achieving the required safety factor. Thus, we decided to use grouted tie-backs to help neutralize some of the pressure creating the overturning moment and sliding hazards. This procedure is further outlined in the tie-back design methodology section of this report. Upon completion of the tie-back support design we were able to return to the previous step (Step 6) with new values for P_1 and P_2. The Safety Factor was then found to satisfy the requirements.

7. Locate resultant and footing soil pressure

Location

- Find X_{bar} = (Resisting moment + Overturning Moment)/Resultant
    - Where resultant = the sum of downward forces W_1-W_4
- Find eccentricity (e) = X_{bar} – (base length/2)
- Is (6e)/base length ≤ 1.0 then resultant lies inside middle third

Use resultant to find ρ_{max}

- R = ½ (ρ_{max})(base length)
- $\rho_{max} \leq$ 5ksf

8. Factor of Safety against sliding (neglecting P_p)
    - Force causing sliding = $P_1 + P_2$
    - Frictional Force = μR
    - FS = (μR) / $P_1 + P_2 \geq 1.5$
    - If factor of safety is less than 1.5 a key may be necessary. Use Inert block concept to determine what size key.
        - $P_p = [((C_aW (h_1+a)^2)/2) – (C_aW (h_1)^2)/2)]$
        - Inert Block μ = tanα
        - $F = \mu_1R_1 + \mu_2R_2$ Using soil pressure diagram (See Appendix N)
        - Force equilibrium for FS = 1.5

          $(P_1 + P_2)1.5 = P_p + F$
9. Design of Heel Cantilever
    - Find $W_u = 1.2(W_{earth} + W_{footing}) + 1.6(W_{surcharge})$
    - Calculate M_u (downward) = ½ (W_u)(length of heel)
    - Find Factored shear at face
        - Find V_u
        - $\emptyset Vc = \emptyset(2\sqrt{f'c})bd \geq V_u$ This is not true, shear controls.
        - Required d =[(existing d) (V_u)]/ $\emptyset Vc$
        - Heel Thickness = d + cover +est. bar thickness

Increasing the heel thickness reduces stem height and can permit reducing stem thickness. It also improves stability against overturning.

- Design Reinforcement

- $As_{min} = [(3\sqrt{f'c})/F_y]b_wd \rightarrow \rho_{min} = As_{min}/ b_wd \rightarrow 200/F_y$
- Req. $As = \rho_{min}bd$
- Select reinforcement by nominal area
- Calculate development length (Category A)

$$L_d = \left(\frac{3d_b}{40}\right) * \left(\frac{F_y}{\sqrt{f'_c}}\right) * \left(\frac{\psi_e\psi_t\psi_s}{\frac{K_{tr}+c_b}{d_b}}\right)$$

In this case top bars are being used, so multiply by a factor of 1.3. The embedment length found is measured from the back face of wall.

10. Design of Toe Cantilever
    - Assume smaller thickness than cantilever heel
    - Repeat steps from design of cantilever heel
        - Take into account the resultant force caused by the soil pressure (See Appendix N)
        - Because reinforcement are bottom bars, no factor of 1.3 is included in the development length calculation.
        - Embedment of reinforcement is from front face of wall.
11. Design of Reinforcement for wall
    - Assume wall height is now smaller due to increase of heel thickness
    - Retain stem thickness
    - Find Required area for reinforcement at stem
        - Req. $R_n = (M_u(12{,}000))/(\emptyset bd)$
        - Req. $A_s = \rho_{min}bd$
        - Select reinforcement by nominal area
    - Determine development length into footing. Key can be used if length requires.

$$L_d = \left(\frac{3d_b}{40}\right) * \left(\frac{F_y}{\sqrt{f'_c}}\right) * \left(\frac{\psi_e\psi_t\psi_s}{\frac{K_{tr}+c_b}{d_b}}\right)$$

- Create Moment Capacity vs. Factored Bending Moment Diagram to help determine at what distance from the stem portions of reinforcement can be discontinued
  - Draw factored moment curve
  - For first cut use new A_s to find moment capacity

    $C = 0.85\ f'_c ba$

    $T = A_s F_y$

    $\emptyset Mn = \emptyset(T)[d - \frac{1}{2}(a)]\ (1/12)$ For both the top and bottom of wall.

  - Use Moment Capacity line , d and L_d to find the cutoff point for the first portion of reinforcement.
  - Repeat process for more reductions
  - Assume some reinforcement must remain near top of wall to serve as shear reinforcement.

12. Temperature and Shrinkage Reinforcement
- Total Amount Required ACI 14.1.2 and 14.3.3

  $A_s = 0.0025bh$ where h = average wall thickness

- Front face is more exposed to temperature changes and thus requires more reinforcement than the rear face.

  Front face = 2/3 A_s

  Rear face = 1/3 A_s

- For vertical reinforcement use nominal amount adequate enough to support horizontal temperature and shrinkage steel on face.

13. Drainage Requirements and Other Details
- Weep holes (4 in diameter) every 10 ft. of wall
- Adequate drainage @ backfill

3.3.3 Pool

Incorporating the pool into the foundation design for the building presented a unique challenge. After researching typical construction practices we decided to design a concrete foundation using retaining wall and slab on grade design. The contained area is then to be filled with a self-supporting, Gunite pool of rebar and spray concrete. Thus, the foundation designed is slightly deeper than is necessary for the competition pool to allow for ample material space to be shaped where hydrostatic pressure is the greatest. After looking at different combinations of methods for such a design, the following procedure was chosen and followed. The calculations can be seen in Appendix O.

1. Determine necessary pool dimensions and requirements.
2. Utilize values from Geotechnical Report (Haley & Aldrich, 2008)

    Glacial Till Weight

    Equivalent fluid pressure

    Angle of Internal Friction

    Friction Coefficient

    Steel reinforcement yield strength: F_y

    Concrete Strength: f'_c

3. Design of pool walls (Wang, Salmon, & Pincheira, 2007).

    This portion of the design predominantly follows the same procedure used in the design of the large retaining wall for the East elevation of the building, with a few additions/exceptions:

    - This design was completed in two parts; one for the deep end/sloped portion combined and one for the shallow end wall.
    - No tie-backs were necessary because the scale is much smaller.
    - A small surcharge was used to account for the slab/finishes/traffic on the pool deck.
    - Sliding was neglected because continuous slab at base of the pool meets the toe of the footing. This serves as increased passive pressure and neutralizes the horizontal forces not offset by the friction at the bottom of the footing.

    Other notes regarding the design of the pool walls:

- Water pressure is ignored in the design calculations. Because the difference in fluid pressures (between the external soil and the water) is negligible, the critical loading situation occurs when the pool is empty. Having a full pool of water, in effect, further stabilizes the wall structures.
- The design of the wall and footing for the sloped section of the pool will be an average if the two extremes. It will change gradually from the sizes yielded by the deep end wall and footing design to those given by the shallow end design.
- Reinforcement can be cut at ≈ 1 foot from top of wall. The slab at the pool deck stabilizes the top of wall against shear and helps to also offset some overturning moment. This fact was ignored in designing for overturning for a more conservative design.

4. Slab on grade design for base of pool (Ringo & Anderson, 1992).

Though the hydrostatic pressure at the bottom of the deep end approaches a considerable amount, the resisting force provided by the relatively stable glacial till beneath coupled with the fact that the load is uniform and lacks avenues of higher pressure, allow for a non-structurally reinforced slab on grade. The following steps were followed to determine the detailing of the concrete slab on grade for each of the three sections of pool (deep end, sloped section, shallow end).

- Slab thickness is a value that is relatively flexible in slab on grade design. The requirements for this design include:
    - At least 1/3 the toe footing height to ensure ample resistance against wall sliding (as referenced in the wall design procedure).
    - The thicker the slab, the smaller the amount of contraction/control joints necessary.
    - Must be enough thickness to support reinforcement. This can be checked once the reinforcement calculations are completed.
- Joint spacing is also an approximation for slab on grade. The higher thicknesses of slab can have larger thicknesses and the range of what the spacing can be between joints is also slightly larger, allowing more leniency in dimensioning

- Joints are placed at each change in elevation: at intersection of sloped section and both the shallow and deep end.
- The length of the toe of the footing for the pool wall must be subtracted from the total area of the slab.

▪ Steel shrinkage and tension reinforcement (non prestressed)

- Using the Subgrade Drag Method (Ringo & Anderson, 1992)

$$A_s = FLw/2F_s$$

Where

As = steel area (in^2/ft. of width)

F= Friction Factor (typical estimate = 1.5)

L=Distance between joints (ft.)

W= Dead weight of slab (psf)

Fs= Allowable working stress of reinforcement (psi)

* Use 2/3 F_y

Note: In this case, distance between joints was used as total short distance of area to be reinforced (conservative).

▪ Select style and gauge of welded wire reinforcement (WWF) according to nominal steel area. Pay attention to placement requirements for each style.

5. Create a series of design sketches depicting the following:
   - Overall pool dimensions.
   - Wall dimensions and detailing.
   - Slab on grade dimensions, joint placement and detailing.

3.4 Cost Estimating

Once the structural design of the building was complete we were able to come with a cost estimate of the materials and fabrication. The cost estimate consisted of the total cost of the steel structure; the concrete from our footings, retaining wall and pool; and the reinforcement within the footings, pool and retaining wall.

To determine the cost of the beams, girders and columns, we started by setting up a table that listed the quantity of each member needed, broken down by the member's span. We then multiplied the weight of a member by the span, and multiplied that by the quantity needed to get a total weight in pounds. After adding up the total weight of all the members we figured out the total weight of the steel members. The same approach was taken to calculate the steel needed from the trusses, the total weight of the W-shape members was found and then added to the total weight of the tubular members. Based on data from RS Means the cost of the steel was determined to be $1.00 per pound of steel with a 15% fabrication cost added on.

To determine the cost of the rebar we totaled up the amount of rebar need and multiplied it by the linear length of each bar. We then divided this by 20, because the cost of the bars are per 20 foot length.   This calculated the total amount of bars needed. The cost per bar was averaged to be about $7.00 per foot with a 15% fabrication cost resulting in $8.05 per 20 ft. bar (RS Means, 2008).

Finally the concrete cost was determined by adding up the cubic yardage of concrete for the footings, retaining wall and pool. The cost of concrete was $96.00 per cubic yard (RS Means, 2008). This cost was multiplied by the total cubic yards of concrete.

3.5 Revit Drawing

To produce a schematic for what the building is expected to look like for both the architectural and structural point of view, Revit 2009 Architectural and Revit 2009 Structural was used. For the architectural visual, Green roofs, glass curtain walls, entrances etc. were shown to create an image of ideas behind the project. The structural illustration showed the locations of columns, girders, and beam systems that acted as the skeleton of the building. These two graphic representations of our project assisted all interested parties to catch a glimpse of what the project is expected to look like once the construction is complete and according to plan. A few selected renderings can be found in

Appendix P.

4.0 RESULTS AND DISCUSSION

4.1 Structural Results

4.1.1 Beams, Girders and Columns

After designing spreadsheets to solve for all of our beam, girder and column sizes we created tables to organize the results. Several tables with the breakdowns showing the type of member, size of member, span of member and location of each member (first, second, third floor, etc.) can be found in the attached excel file. Table 3 below was created to show how many beams and girders of each different size member will be needed for our building. The red figures are the totals for each member. And the black numbers detail the span lengths and the number needed for each span length within that member.

Table 3: W-Shape Selection for Beams and GIrders

Beams & Girders	
Beam Size & Span Length	Quantity
W10x22	**19**
13.5	1
16.5	18
W12x16	**119**
5.531	18
9.172	18
12.813	18
13.5	8
16.5	11
17	31
18.193	2
25	7
30	6
W12x19	**66**
16.5	60
19.375	2
25	4
W12x22	**29**
13.5	3
16.453	18
16.5	2
17	4
20.552	2
W12x34	**1**
20	1
W14x22	**24**
16.5	1
21.75	2
22.484	2
30	19
W14x26	**43**
13.5	5
16.5	1
17	23
20.583	2
22.484	2
23.229	2
23.979	2
25	6
W14x34	**6**
25	6
W14x74	**7**
30	7

Beams & Girders	
Beam Size & Span Length	Quantity
W16x26	**14**
17	4
18.194	2
25.474	2
25	4
24.729	2
W16x31	**13**
26.25	2
26.469	2
26.714	2
26.969	2
27.308	2
27.458	2
27.708	1
W16x36	**5**
25	5
W16x40	**9**
25	9
W16x50	**2**
25	2
W18x35	**265**
25	235
19.375	2
20	2
20.552	2
21.75	2
23.073	18
25	4
W18x40	**71**
30	28
26.495	9
25	32
23.229	2
W18x55	**15**
29.912	9
26.469	2
26.25	2
20.583	2
W21x44	**19**
23.979	2
24.729	2
25.474	2
26.495	9
30	4

Beams & Girders	
Beam Size & Span Length	Quantity
W21x50	**28**
30.25	2
29.912	9
27.458	2
27.208	2
26.969	2
26.714	2
20	9
W21x55	**15**
25	12
28.708	1
30	2
W21x62	**7**
25	4
30	3
W21x68	**9**
28.729	9
W21x166	**4**
25	4
W24x62	**39**
30	30
28.729	9
W24x68	**15**
30	6
33.333	9
W24x76	**22**
20	10
30	3
33.333	9
W24x103	**3**
30	3
W24x104	**23**
30	23
W24x131	**2**
30.25	2
W24x146	**4**
30	4
W24x162	**2**
30	2

It should be noted that some members had to be increased because the shape exceeded the compact limit for flexure or because the shape did not meet the h/t_w limit for shear (American Institute of Steel Construction, 2005). All members are 50 ksi steel. Once the beams and girders were designed we were able to design the columns. Table 4 set up similar to Table 3 , but displays the column results.

Table 4: W-Shape Selection for Columns

Columns	
Column Size & Span Length	Quantity
W5x16	**6**
10	2
15	2
20	2
22	1
W8x15	**2**
10	2
W8x18	**2**
10	2
W8x21	**2**
15	2
W8x24	**15**
10	5
15	10
W8x28	**7**
10	2
20	2
22	3
W8x35	**32**
10	15
20	8
22	9
W10x26	**2**
10	2
W10x33	**13**
10	3
15	10
W10x39	**2**
15	2
W10x45	**42**
10	12
15	4
20	19
22	7
W10x49	**2**
22	2
W10x54	**4**
22	4

Columns	
Column Size & Span Length	Quantity
W10x60	**6**
22	6
W10x68	**1**
22	1
W10x77	**1**
22	1
W10x88	**2**
22	2
W12x40	**4**
15	4
W12x53	**8**
15	8
W14x53	**8**
15	8
W14x61	**12**
15	10
22	2
W14x68	**16**
15	14
22	2
W14x82	**4**
22	4
W16x77	**2**
15	2
W16x89	**5**
15	4
22	1
W18x86	**2**
15	2
W18x97	**13**
15	6
22	7
W18x106	**4**
15	4
W21x111	**5**
22	5

The column lengths were determined based on their location. The building was divided into three sections: the upper section, main floor, and lower section. Within these sections the upper section had a top and a bottom. The top of the upper section had columns of length 20 feet and the bottom of the upper section had columns of length 10 feet. The Main floor columns were 22 feet tall and the lower section has an upper and lower column, each 15 feet in length.

4.1.2 Trusses

Once all the calculations were completed by hand and by RISA 2D, the results for our Roof and Pool Truss were the following:

- The depth of the Roof Truss was 8 ft.
- The depth of the Pool Truss was 6 ft.
- The sizes of the top members for both trusses were A36 W14x82 Wide Flange Beams
- The sizes of the pipe members for both trusses were Pipe 8x (strong) pipes
- Both trusses were able to withstand the loads encountered by our building's design
    - Deflection of each member did not exceed 5 in.

Table 5 below lists the number of members needed for the W14x82 members and the 8x-strong tubular piping, broken down by the span of the member.

Table 5: Steel Selection for Trusses

	Size & Span	Quantity
Roof Truss	**W14x82**	**21**
	5	2
	10	19
	Pipe 8x-Strong	**22**
	8	2
	9.43	20
Pool Truss	**W14x82**	**21**
	5	2
	10	19
	Pipe 8x-Strong	**22**
	6	2
	7.81	20

4.1.3 Results-Frame Analysis

We determined that the major loads acting on our frame occurred at the base columns (See attached Excel spreadsheet for numerical results). This makes sense since our base columns needed to withstand the forces subjected by the wind loads in order to prevent the building from collapsing. When we compared our base column loads to the maximum loads of our column sizes, we determined that all the base column loads would be accounted for by the chosen column sizes of our building. Furthermore, the results indicated that our columns would not need any bracing to support the given loads. This coincided with our single plate shear connection design, which properly took into consideration the loads applied throughout the girders that were connected to their respective columns. Overall, the frame analysis helped ensure us that our building was able to withstand the wind loads placed upon its structure, preventing severe structural damage and possible collapse (Hibbeler, 1985).

4.2 Foundation Results

4.2.1 Footings

The footings were designed for three different ranges of loading. The first footing was designed for a load between 0 and 372 kips. The allowable bearing pressure of the soil was determined to be 8 ksf and the allowable net soil pressure was determined to be 7.255 ksf. The footing was designed with a foot of soil on top of the footing. A footing size of 8 x 8 x 3 ft. was attempted first, but it was found to fail the bearing pressure test. The bearing pressure was calculated to be 9.57 ksf, but this failed, as it is greater than 8 ksf. The next attempt at designing the footing was 10 x 10 x 3 ft. This passed the bearing pressure test with a bearing pressure of 6.14 ksf. Because this was 1.86 ksf under the allowable 8 ksf, we tried one more attempt with a 9 x 9 x 3 ft. column because this would reduce the amount of concrete needed. This final footing passed all required test (one-way shear, punch shear, development length, etc.) and was calculated to have a settlement of 0.62 inches. Therefore, all footings with loads 0 to 372 kips will be designed with a 9 x 9 x 3 ft. spread footing. The required reinforcement was determined to be 7 No. 9 bars each way.

The second footing that needed to be designed was for loading from 373 kips to 758 kips. The allowable bearing pressure and net soil pressure were the same as the first footing. One foot of soil will also be placed on top of this footing. The footing sizes that were attempted but failed were an 11 x 11 x 3 ft. and a 12 x 12 x 3 ft. footing. However, a footing size of 13 x 13 x 3 ft. was determined to have passed all necessary checks (one-way shear, punch shear, development length, etc.). The settlement for this footing was just under the allowable settlement of .75 inches. Therefore, all footings with loads between 373 kips and 758 kips will have a 13 x 13 x 3 ft. footing with reinforcement of 8 No. 10 bars each way.

Finally, one last footing was designed for loading from 759 kips to 934 kips. Because the depth of the footing was 4 feet the allowable net soil pressure was 7.105 ksf. The footing was designed using dimensions of 14 x 14 ft. with a depth of 4 feet. This footing came up 0.2 ksf over the allowable bearing pressure and therefore failed. So a 15 x 15 x 4 ft. footing was used with 10 No. 11 bars of reinforcement to successfully support column loadings from 759 kips to 934 kips. Figure 6 below shows the three footings with their appropriate reinforcement.

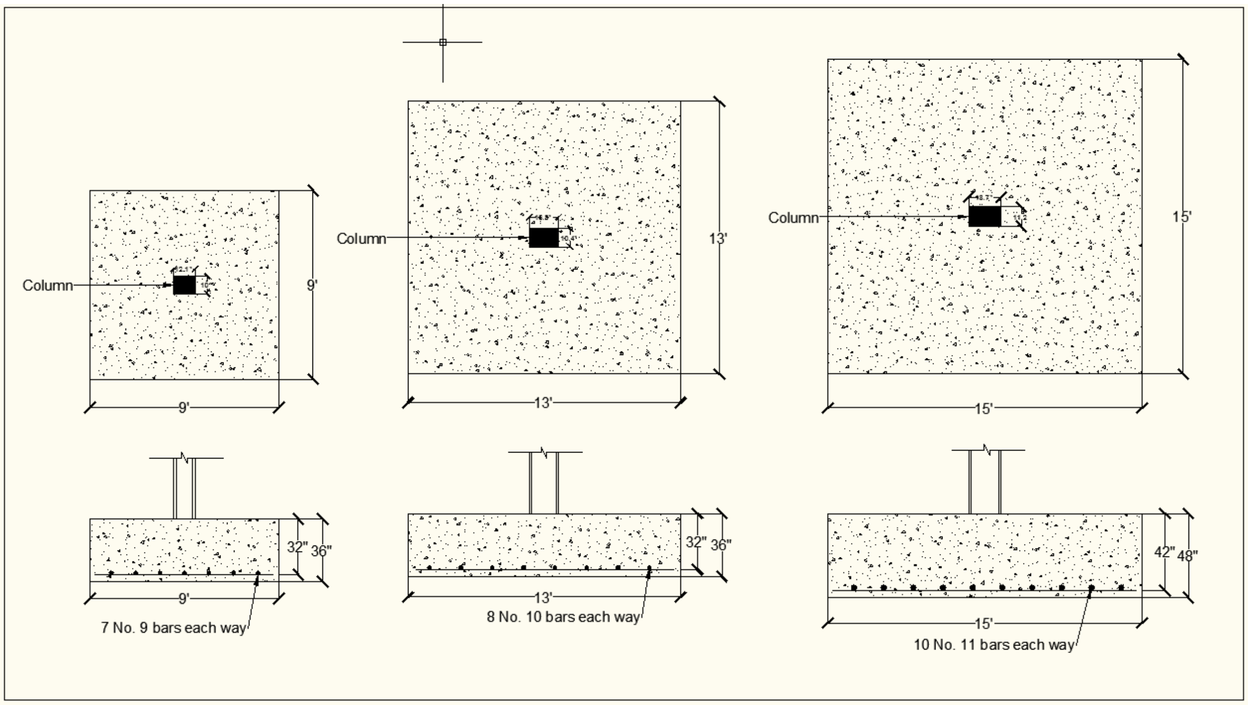

Figure 6: Footing Design with Reinforcement

4.2.2 Retaining Wall

Following the design methodology we were able to generate a design to support the earth behind the 30 ft. high by 300 ft. long wall needed at the East elevation. The South wall proposes its own challenge. The hill is 30 ft. at its greatest height and falls in height proportional to the slope of the adjacent earth until the grade reaches the project's zero elevation. Using the following information gained from the *Report on Geotechnical Investigation* the final design was achieved (2008).

Equivalent Fluid Pressure/Unit Wt.:60 pcf (conservative)

Weight of Soil: 140 pcf

Angle of internal friction: 38*

Coefficient of Friction: 0.45 (for coarse grained soil with some silt)

<u>East Wall</u>

The large retaining wall (Figure 7) was designed to be separate from the building itself. This was advice presented in the Geotechnical Report. Table 6 and Table 7 below outline the dimensions, detailing and other specifications for the design.

Table 6: Retaining Wall Dimensions

East Retaining Wall		
	Dimension	Units
Wall Height (from top of toe)	31.0	ft.
Wall height from top of heel	27.7	ft.
Total height to bottom of footing	34.0	ft.
Wall Length	300.0	ft.
Stem Width	60.0	in.
Effective width d @ stem	57.5	in.
Top Width	14.5	in.
Effective width d @ top	12.0	in.
Footing Base Length	22.0	ft.
Heel Length (includes stem width)	14.7	ft.
Toe Length(from back face)	7.3	ft.
Footing Thickness		
Heel Thickness	64.0	in.
Toe Thickness	36.0	in.
Key Depth	none	ft.
Anchors	26.0	ea.
Anchor Spacing	12.0	ft.
Anchor location (x,y)	25.62, 15.0	ft.

Table 7: Retaining Wall Construction Details

Location	Cuts	Reinforcement	Spacing (o.c.)	Development Length (L_d)
Cantilever Heel - **Top Bars**		#9 Bars	4 in.	43.5 in. —> Use embedment of 4 ft. from back face of wall
Cantilever Toe - **Bottom Bars**		#9 Bars	6 in.	26.8 in. —> Use embedment of 2.5 ft. from front face of wall
Wall - **Vertical Reinforcement**				
@ Stem		#9 Bars	4 in.	12.3 in. —> Use embedment of 2 ft. into the footing for # 9
~8 ft. from stem	1 Bar	#9 Bars	8 in.	
~18 ft. From stem	1 Bar	#9 Bar	12 in.	
~ 27.7 ft. from stem	Last Bar			
Wall - **Temperature & Shrinkage (horizontal)**				
Front Face		#7 Bars	12 in.	
Rear Face		#5 Bars	12 in.	
Wall - **Support of Temp. & Shrink. (vertical)**		#5 Bars	15 in.	
*See *Design Sketch* for more specific detailing				

Some notes regarding the large retaining wall and tie-backs include:

- Design is for an assumed surcharge of 6 feet above the restrained soil. This value is an estimate accounting for the construction of the lobby structure and any paving that will be placed on the area above.
- Tie-backs are designed as grouted anchors for constructability purposes. The placement process must incorporate ample grouting and curing time for the grout ball before backfilling can occur.
- Tie-backs will be connected through the wall and anchored on the back face using adequate connections. They are spaced at 12 ft. on center and have a nominal bar diameter of 2 ½ in.
- Construction requires enough excavation of the slope to allow the footing to be placed first with keyway. This activity should be coordinated with the placement of the rest of the spread footing foundation. After ample curing time the wall with shear key can then be poured in stages. Precautions for the protection for rebar during curing time between phases should be taken.
- Retaining wall is extended 12 ft. past the South wall. This will be explained in the following section.

South Wall

To design and construct a sloped retaining wall at the South wall of the building would be a time and cost consuming task. The grade would involve intricate formwork and more vibrating to place. To avoid the need for this structure, the large East retaining wall has been extended 12 ft. past the wall on the South side. This will allow room for the hill to slope down to zero at the base of the South wall. This slope may be very extreme so coarse backfill and slope stabilizers should be placed along the resulting hill. A safety rail should also be placed at the top of the retaining wall that is exposed.

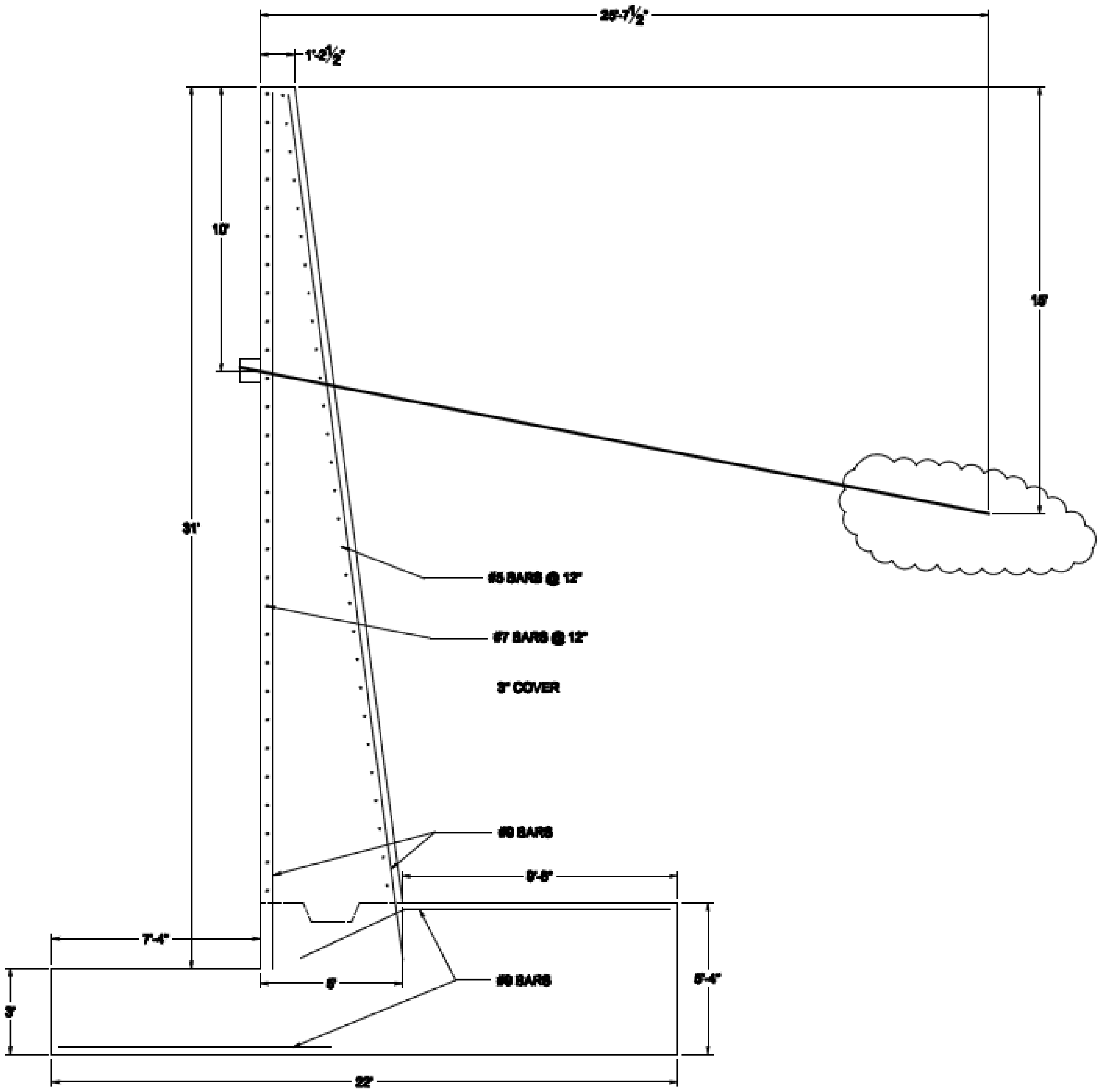

Figure 7: Final Retaining Wall Design

4.2.3 Pool Design Results

Overall the pool dimensions are illustrated in Figure 8. This design was achieved using reinforced concrete design for the sub-grade containment area that will support the Gunite competition pool. The results are presented in this section in two separate pieces: wall design and slab on grade design.

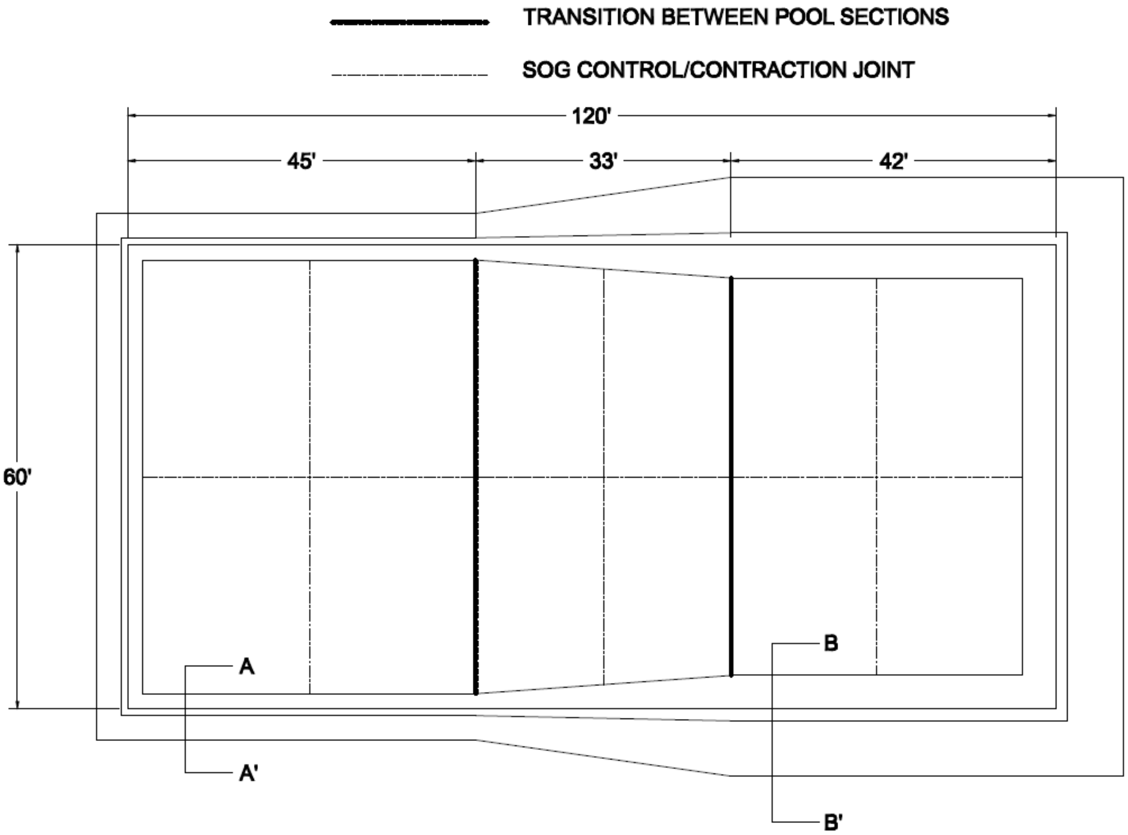

Figure 8: Pool Foundation Plan with Dimensions

Pool Walls

Deep End

This is the portion of the pool with the most critical loading situation. The diving pool requires a depth of at least 13.5 ft. This design places the top of the footing at 14.5 ft, allowing for 1 ft. of gunite, rebar and spray concrete if necessary for forming and retaining the large hydrostatic pressure. The final design is of considerable size and reinforcement (Table 8 and Table 12).

Shallow End

This 45 x 60 ft. area is only required to have a 7 ft. depth for competition swimming. Thus, the top of the footing lies at 8 ft. and the overall size of the wall and footing is considerably smaller than that of the deep end. The reinforcement is also significantly less (Table 9 and Table 11)

Sloped Section

The change in grade between the top of the slope and the bottom is 6.5 ft. Because the deep end footing and wall is of considerable size, it was decided not to continue that size all the way to the shallow end. To design for this, a gradual change from one size to the other will occur proportional to the slope. The result is a range in footing and wall size from the 10 in. footing and wall thickness of the shallow end to the 18 in. footing and wall thickness of the deep section.

The main challenge in this simplified design is determining the reinforcement. To be safe, the same reinforcement that was used for the deep end wall (the critical design loads) is considered adequate for this wall, even though it is in excess for the shorter wall stems closer to the shallow end. This excess can be eliminated by cutting it at 1 ft. from the top of the wall (Table 10 and Table 12).

Table 8: Deep End Retaining Wall Dimensions

Deep End		
	Dimension	Units
Wall Height (from top of toe)	14.5	ft.
Wall height from top of heel	14.5	ft.
Total height to bottom of footing	16.0	ft.
Wall Length	144.0	ft.
Stem Width	18.0	in.
Effective width d @ stem	15.5	in.
Top Width	18.0	in.
Effective width d @ top	15.5	in.
Footing Base Length	13.0	ft.
Heel Length (includes stem width)	8.7	ft.
Toe Length(from back face)	4.3	ft.
Footing Thickness		
Heel Thickness	18.0	in.
Toe Thickness	18.0	in.
Key Depth	none	ft.

Table 9: Shallow End Retaining Wall Dimensions

Shallow End		
	Dimension	Units
Wall Height (from top of toe)	8.0	ft.
Wall height from top of heel	8.0	ft.
Total height to bottom of footing	8.83	ft.
Wall Length	150.0	ft.
Stem Width	10.0	in.
Effective width d @ stem	7.5	in.
Top Width	10.0	in.
Effective width d @ top	7.5	in.
Footing Base Length	6.0	ft.
Heel Length (includes stem width)	4.0	ft.
Toe Length(from back face)	2.0	ft.
Footing Thickness		
Heel Thickness	10.0	in.
Toe Thickness	10.0	in.
Key Depth	none	ft.

Table 10: Sloped Section Retaining Wall Dimensions

Sloped Section		
	Dimension	Units
Wall Height (from top of toe)	8.0 to 14.5	ft.
Wall height from top of heel	8.0 to 14.5	ft.
Total height to bottom of footing	8.83 to 16.0	ft.
Wall Length	66.0	ft.
Stem Width	10.0 to 18.0	in.
Effective width d @ stem	7.5 to 15.5	in.
Top Width	10.0 to 18.0	in.
Effective width d @ top	7.5 to 15.5	in.
Footing Base Length	6.0 to 13.0	ft.
Heel Length (includes stem width)	4.0 to 8.7	ft.
Toe Length(from back face)	2.0 to 4.3	ft.
Footing Thickness		
Heel Thickness	10.0 to 18.0	in.
Toe Thickness	10.0 to 18.0	in.
Key Depth	none	ft.

Table 11: Shallow End Retaining Wall Construction Details

Location	Cuts	Reinforcement	Spacing (o.c.)	Development Length (L_d)
Cantilever Heel - Top Bars		#4 Bars	6 in.	17.13 in. —> Use embedment of 1.5 ft. from back face of wall
Cantilever Toe - Bottom Bars		#4 Bars	6 in.	13.17 in. —> Use embedment of 1.25 ft. from front face of wall
Wall - Vertical Reinforcement				
@ Stem		#4 Bars	6 in.	6.6 in. —> Use embedment of 8 in. into the footing for #4
~3 ft. from stem	1 Bar	#4 Bars	12 in.	
~ 7 ft. from stem	Last Bar			
Wall - Temperature & Shrinkage (horizontal)				
Front Face		#3 Bars	6 in.	
Rear Face		#3 Bars	12 in.	
Wall - Support of Temp. & Shrink. (vertical)		#4 Bars	24 in.	
*See *Design Sketch* for more specific detailing				

Table 12: Deep End and Sloped Section Retaining Wall Construction Details

Location	Cuts	Reinforcement	Spacing (o.c.)	Development Length (L_d)
Cantilever Heel - Top Bars		#6 Bars	6 in.	24 in. —> Use embedment of 2 ft. from back face of wall
Cantilever Toe - Bottom Bars		#6 Bars	6 in.	18 in. —> Use embedment of 1.5 ft. from front face of wall
Wall - Vertical Reinforcement				
@ Stem		#5 Bars	3 in.	10 in. —> Use embedment of 1 ft. into the footing for #4
~3.5 ft. from stem	2 Bars	#5 Bars	6 in.	
~8.5 ft. from stem	1 Bar	#5 Bars	12 in.	
~ 13.5 ft. from stem	Last Bar			
Wall - Temperature & Shrinkage (horizontal)				
Front Face		#5 Bars	10 in.	
Rear Face		#4 Bars	12 in.	
Wall - Support of Temp. & Shrink. (vertical)		#5 Bars	1.5 ft.	
*See *Design Sketch* for more specific detailing				

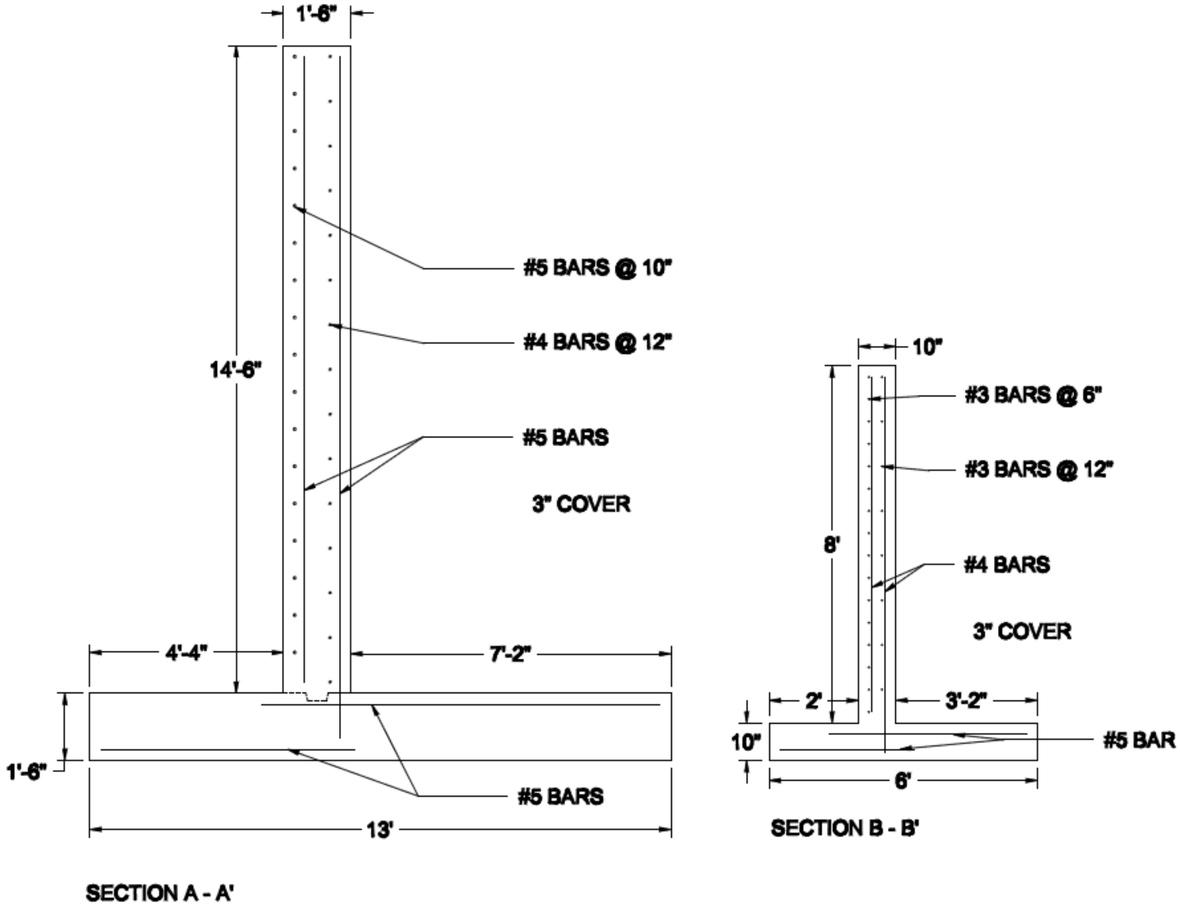

Figure 9: Pool Retaining Wall Design

Pool Slab on Grade

The slab on grade plan for all three sections of the pool is relatively straight forward. Each set of design criteria include slab thickness, joint spacing and a welded wire fabric (WWF) gauge. Overall, there are a few design factors that affect the whole pool:

- Control/ Contraction joints are to be placed where each end (deep and shallow) meet the sloped section (Figure 8)
- The perimeter of the slab lies where the footings for the retaining walls end. This means that there is a built in construction joint where the slab meets the footing of the surrounding wall structures.
- WWF is placed at 2 in. below top of slab.
- WWF is kept continuous @ control joints.

- The slab has been designed in three sections, assuming a uniform loading situation per section. It has also been assumed that the addition of a formed, Gunite pool body on top of the slab will be of uniform weight and depth.

Deep End

- Slab Thickness = 8 in.
- Control/Contraction joints @ 18'-10" from end footing and @ 25 ft. --8 in. from side footings.
- WWF - 4 x 4 – W2.9 x W2.9 (Wt. = 62 pounds per 100 sf)

Shallow End

- Slab Thickness = 6 in.
- Control/Contraction joints @ 21 ft. --6 in. from end footing and @ 28 ft. from side footings.
- WWF - 4x4 – W2.9 x W2.9 (Wt. = 62 pounds per 100 sf)

Sloped Section

- Slab Thickness = 8 in.
- Control/Contraction joints @ 16 ft. –6 in. from ends of slope and @ 26 ft. –10 in. from side footings.
- WWF - 4x4 – W2.9 x W2.9 (Wt. = 62 pounds per 100 sf)

4.3 Cost Estimate

By following the cost estimate methodology we were able to come up with a total cost for the concrete, steel and rebar. The total tons of steel needed for the construction of the building can be found in Table 13 broken down by the type of member. The total amount of structural steel need for the building was 537 tons.

Table 13: Total Tonnage of Steel

	Tons of Steel
Beams/Girders	438.324574
Columns	98.354
Trusses	24.48976
Total	536.678574

The total amount of concrete needed for the retaining wall and pool was calculated to be 2,660 cubic yards. A table showing how we solved for the total cubic yards for the footing is shown below. Adding the concrete from the pool, retaining wall and footing we discovered that we needed 3,381 cubic yards of concrete as shown in Table 14.

Table 14: Total Volume of Concrete

Footing Size	Cubic Feet	Quantity	Total Cubic Feet	Cubic Yards
9'x9'x3'	243	22	5346	198
13'x13'x3'	507	19	9633	357
15'x15'x4'	900	5	4500	167
			Total Cubic Yards	721

Finally, the total amount of rebar was calculated. A spreadsheet can be found in Appendix Q that was used to solve for the total rebar needed for the footings, retaining wall and pool design. From this spreadsheet we determined that we need 6,183 bars of length 20 feet, a rundown of the cost can be seen below in Table 16.

The last step involved adding the cost of the steel, rebar, and concrete together to get a total cost estimate. One last table (Table 16) was created to summarize all of these results.

Table 15: Total Cost for Reinforcement Bars

Steel (lbs)	Cost (per pound) with Fab.	Total Cost
1073358	1.15	1234361.7

Concrete (Cu. Yd.)	Cost (per cubic yard)	Total Cost
3381	96	324576

Rebar (Each)	Cost Per Bar with Fab.	Total Cost
6183	8.05	49773.15

Table 16: Total Cost

Total Cost	$1,608,710.85

It shows the amount of steel, concrete and rebar needed, along with the unit price to come up with a final total cost of $1,608,710.85 for the bare and fabricated materials of the building.

5.0 CONCLUSION

The purpose of the Capstone Design experience is to create a final product that reflects the thought and substance invested in the process. The simple act of combining design data and numbers does not do justice to the procedural and worldly constraints that face projects in a professional setting.

This report is together a cohesive look at a process. This process represents, as a whole, a comprehensive design of the structural and foundation elements of the given building, the new Recreation Center. Throughout the course of the undertaking the group has researched, designed, modeled, and analyzed a very real set of data; a set of data that is given true meaning and substance when looking back on the topics discussed in the Capstone Design section of this report.

From this project we enhanced our overall design skills by creating a structural frame for the building made up of beams, girders, trusses and columns. We also created a foundation for the building by designing spread footing, a retaining wall and a pool foundation. Finally we used computer software to create drawings to show what the WPI Athletic Facility will look like. With the completion of the building, WPI will realize the potential benefits for itself and the surrounding community. The facility will become an added attraction for visitors, promote an active lifestyle and will allow the school to become more involved in the community by hosting community events. The new Recreation Center will, in the end, have positive effects on WPI.

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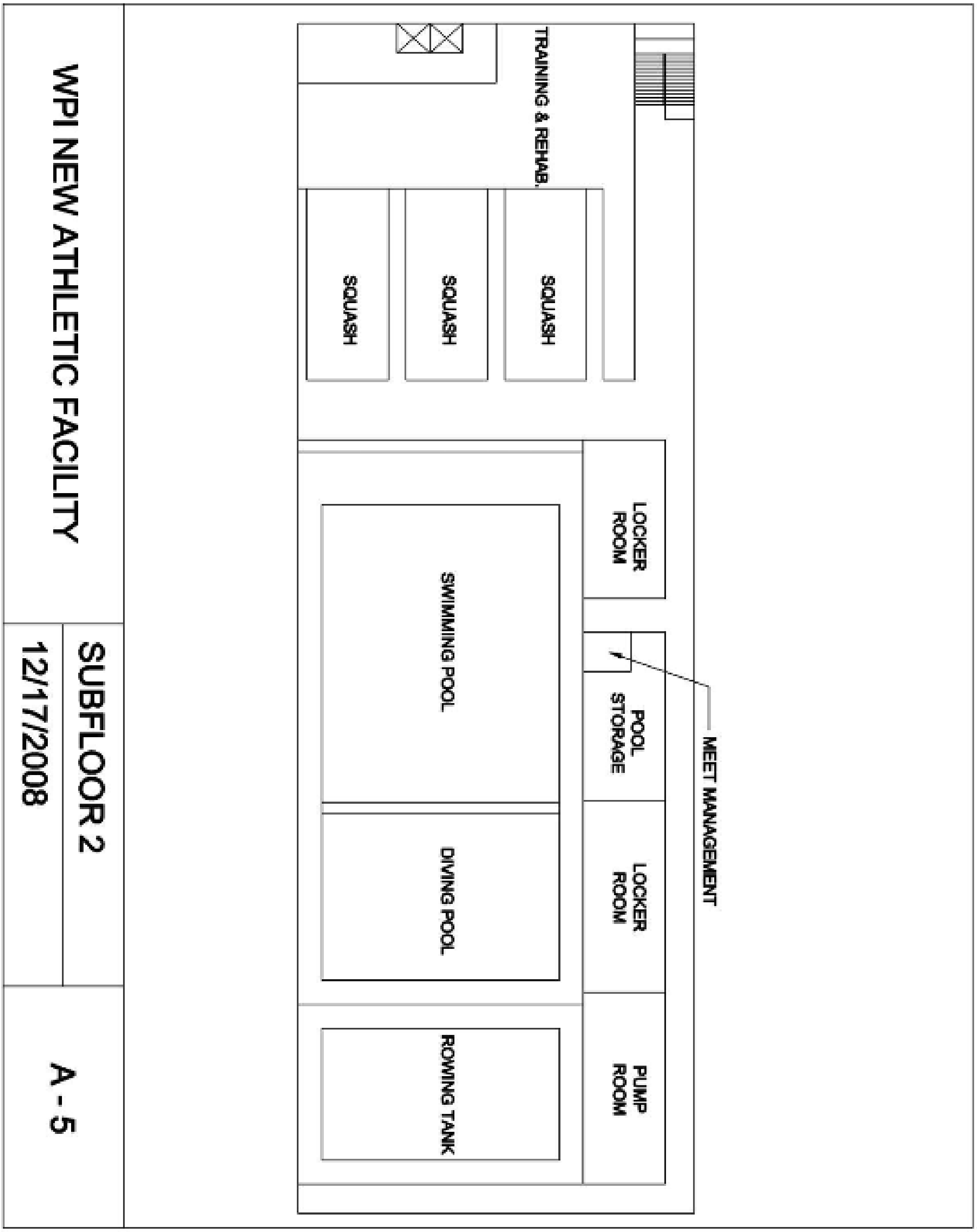
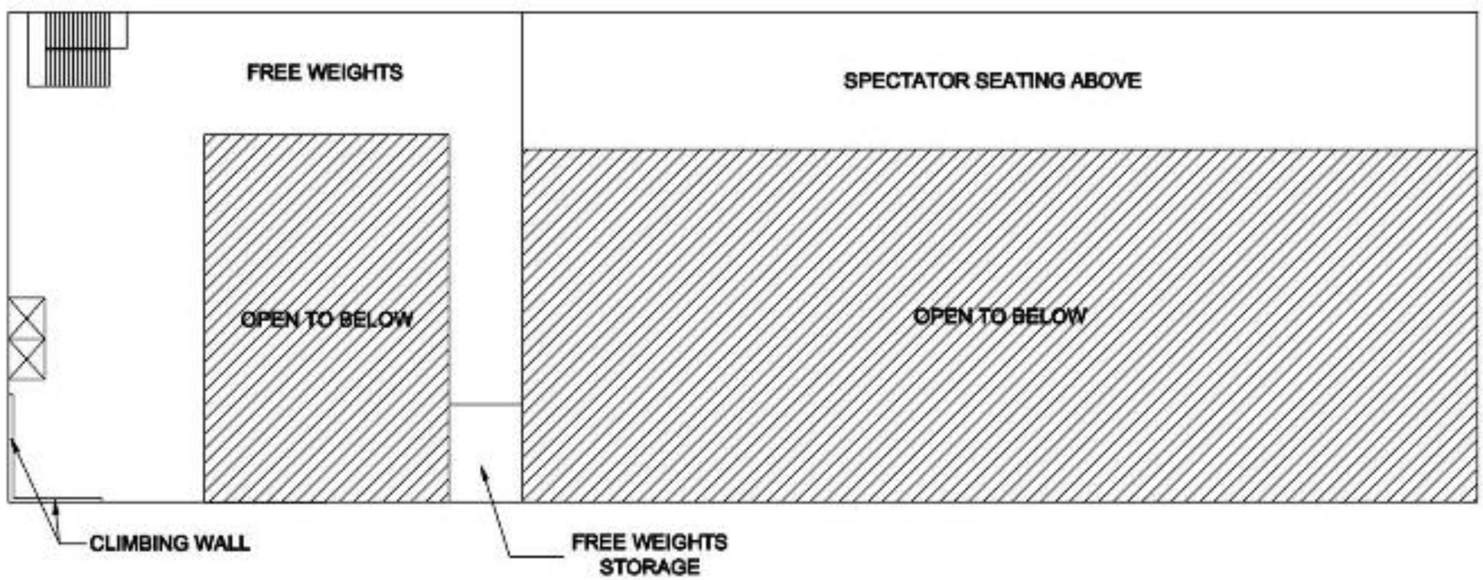
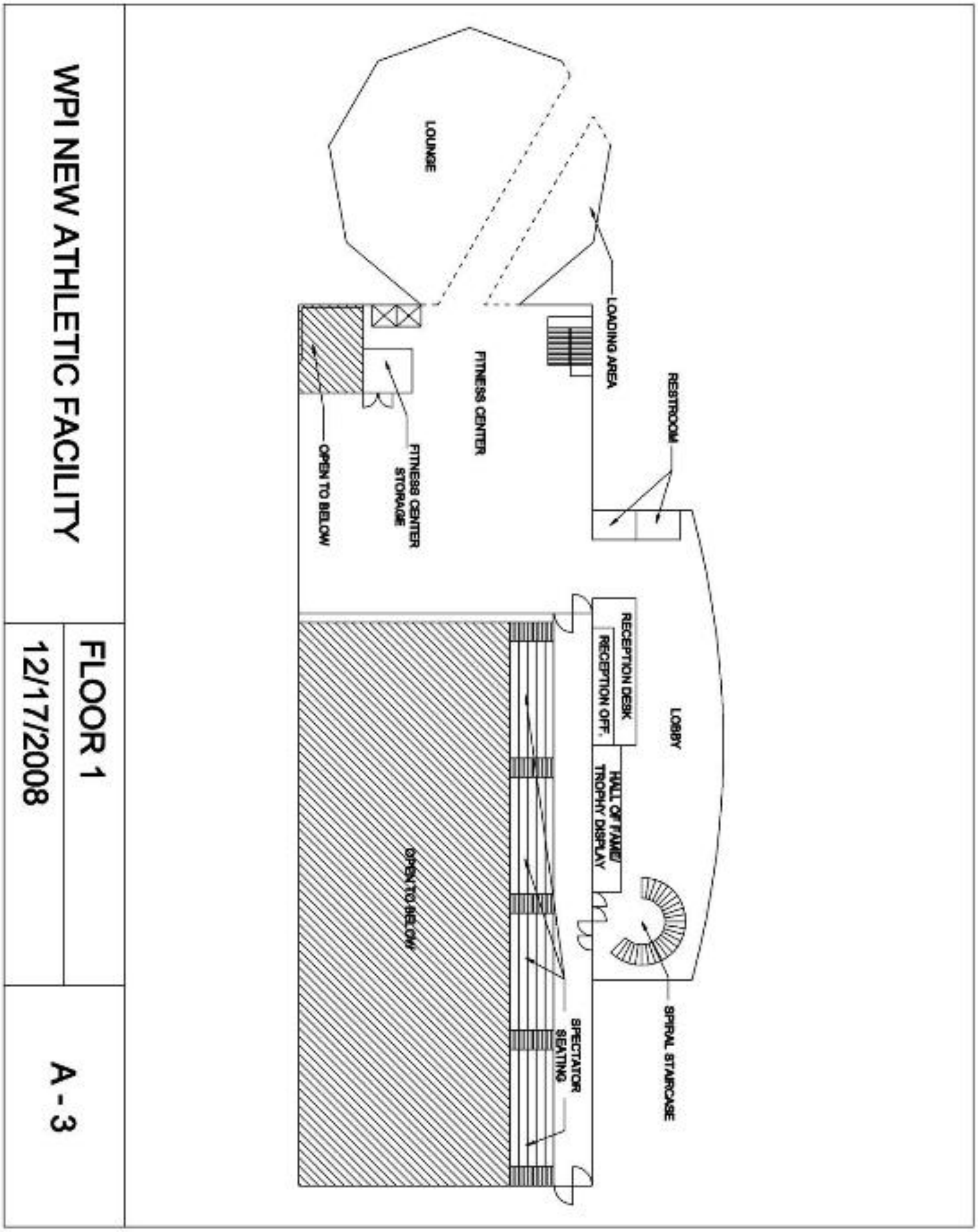
Appendix A

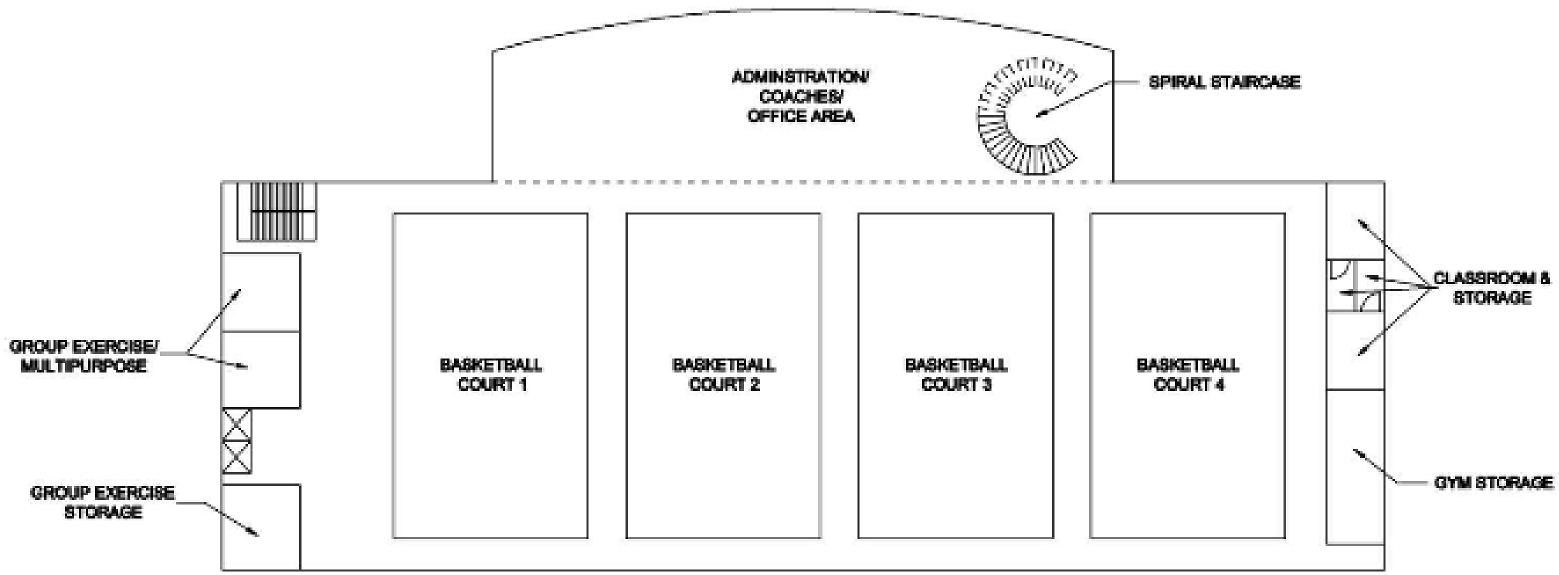
Floor Plans





WPI NEW ATHLETIC FACILITY	SUBFLOOR 1 12/17/2008	A - 4


WPI NEW ATHLETIC FACILITY
FLOOR 1
12/17/2008
A - 3
LOUNGE
LOADING AREA
RESTROOM
FITNESS CENTER
FITNESS CENTER STORAGE
OPEN TO BELOW
LOBBY
RECEPTION DESK
RECEPTION OFF.
HALL OF FAME/ TROPHY DISPLAY
SPIRAL STAIRCASE
SPECTATOR SEATING
OPEN TO BELOW



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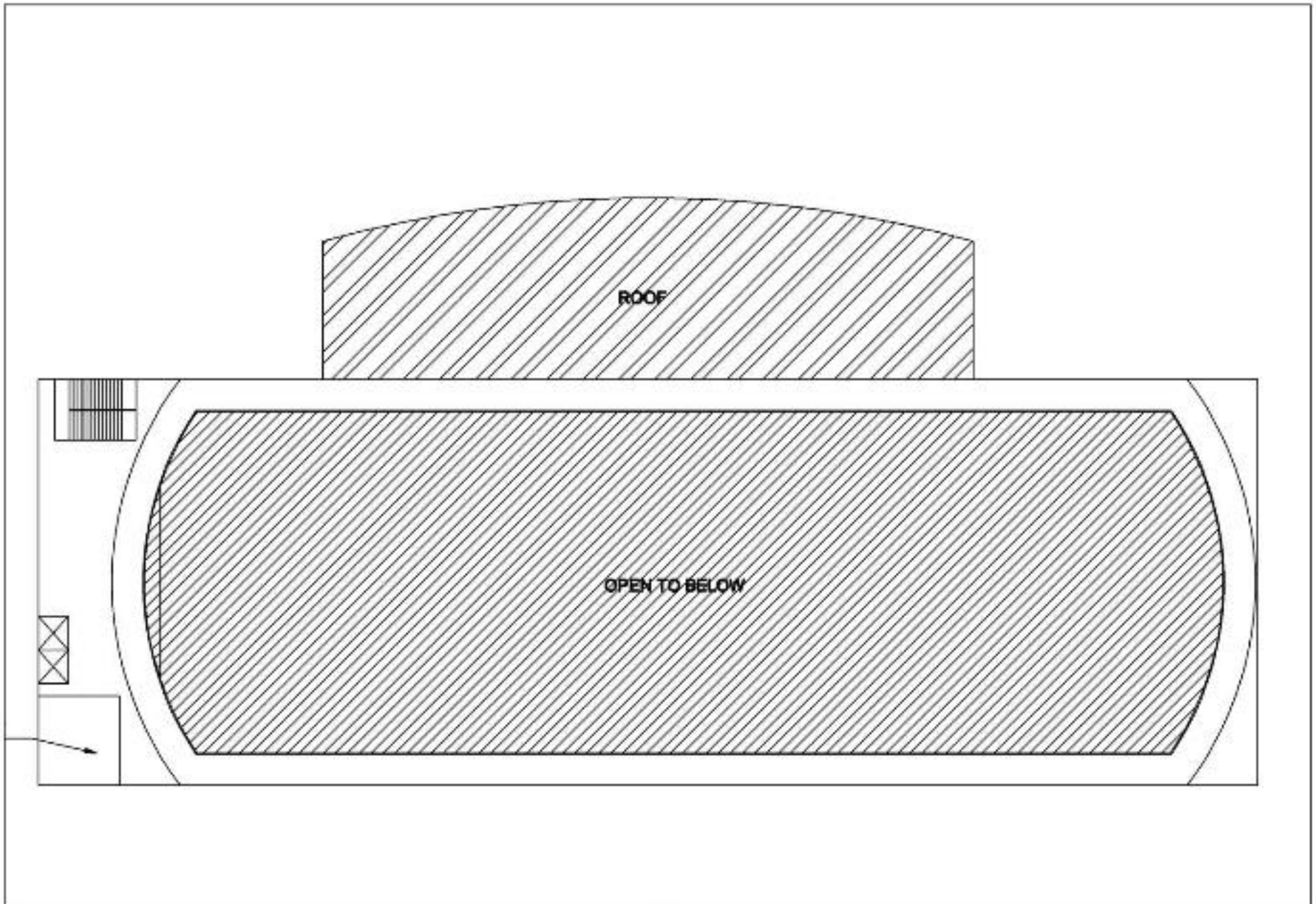

WPI NEW ATHLETIC FACILITY	FLOOR 3 12/17/2008	A - 1

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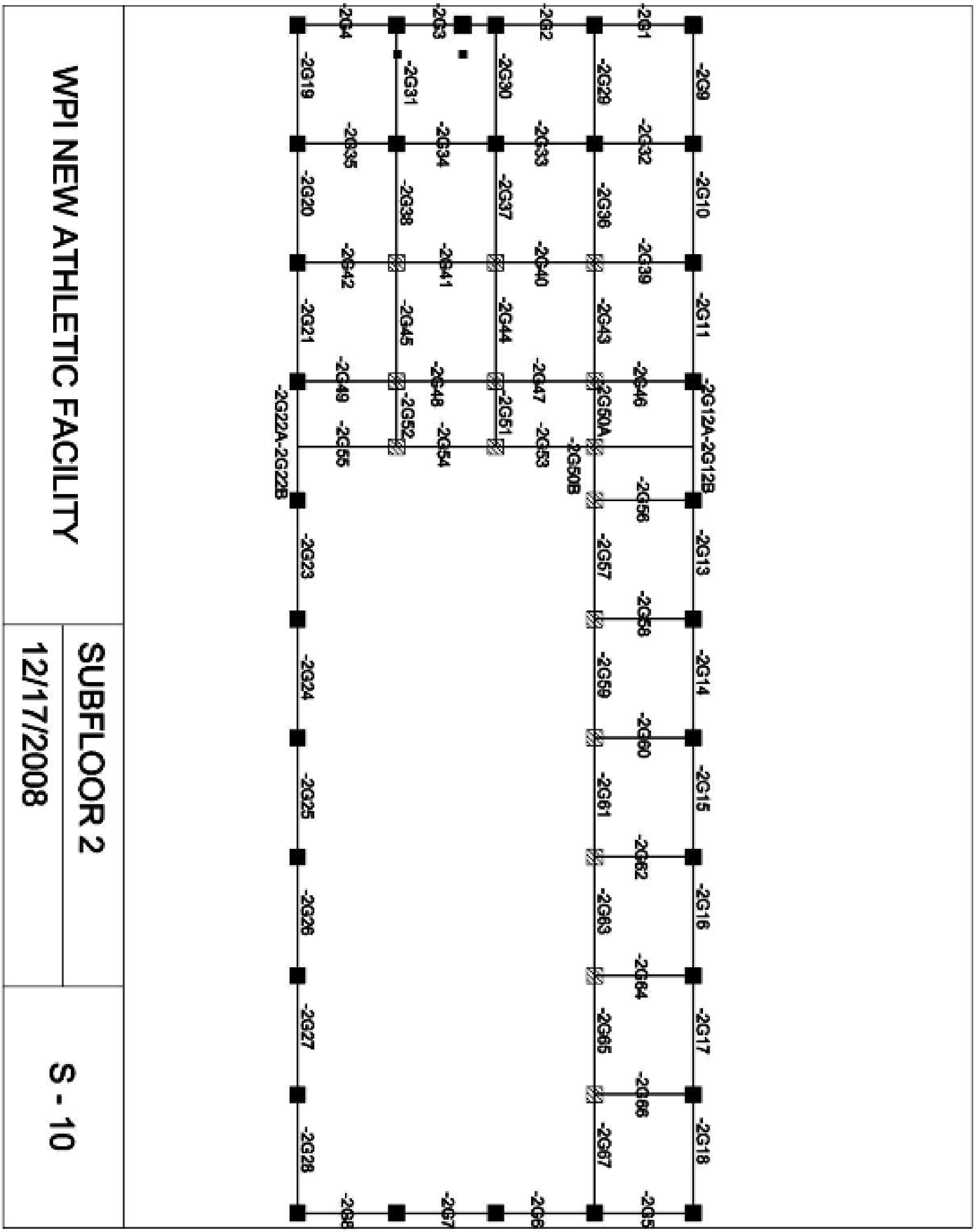
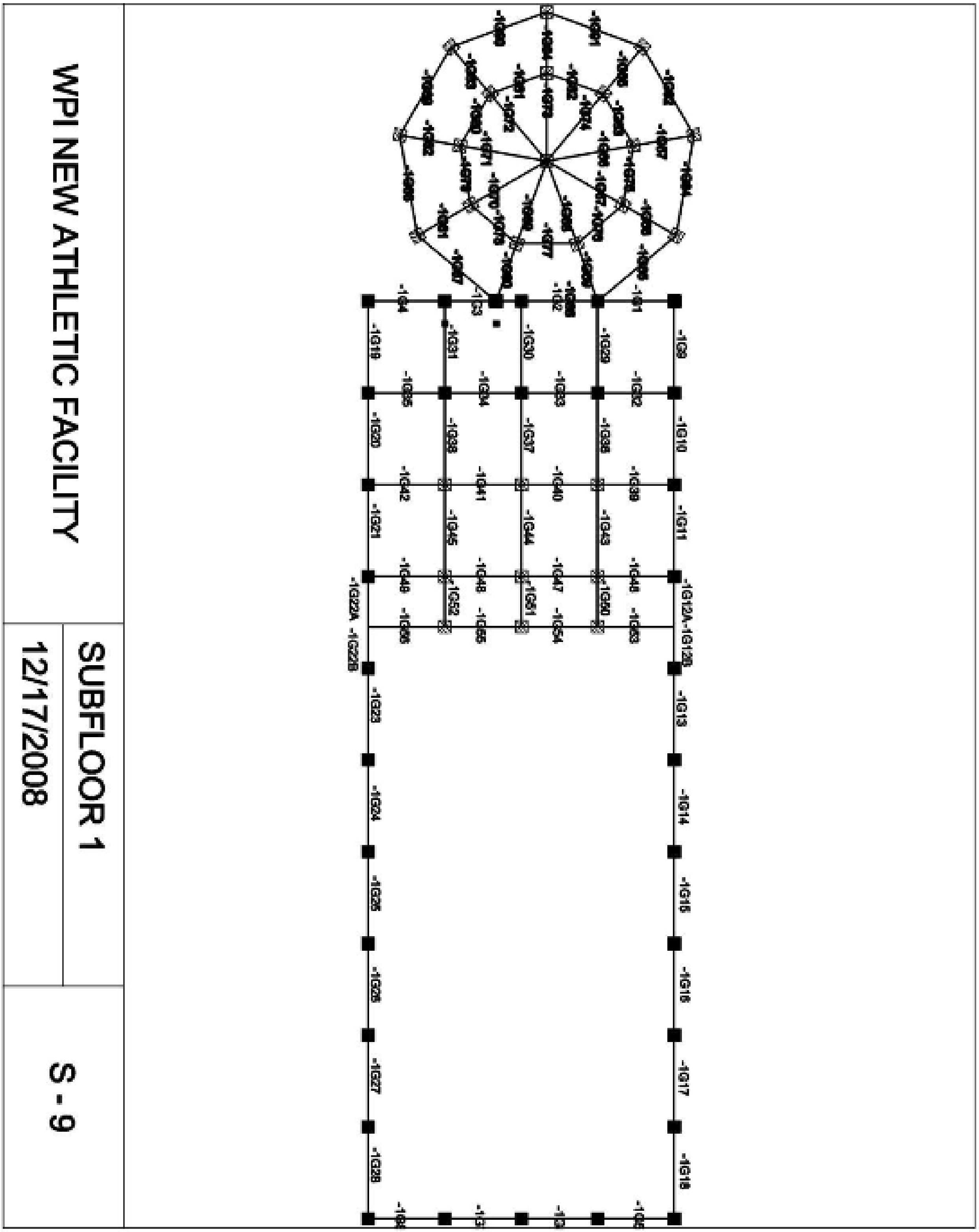
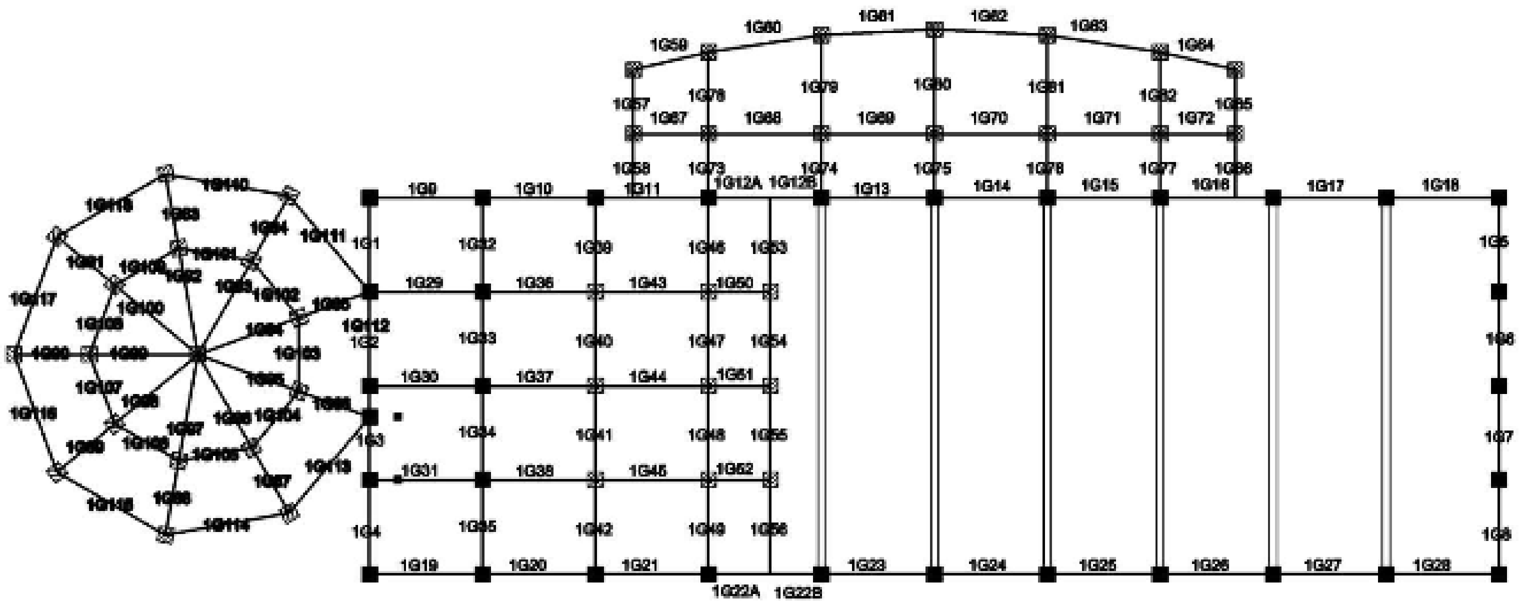



WPI NEW ATHLETIC FACILITY
SUBFLOOR 1
12/17/2008
S - 9



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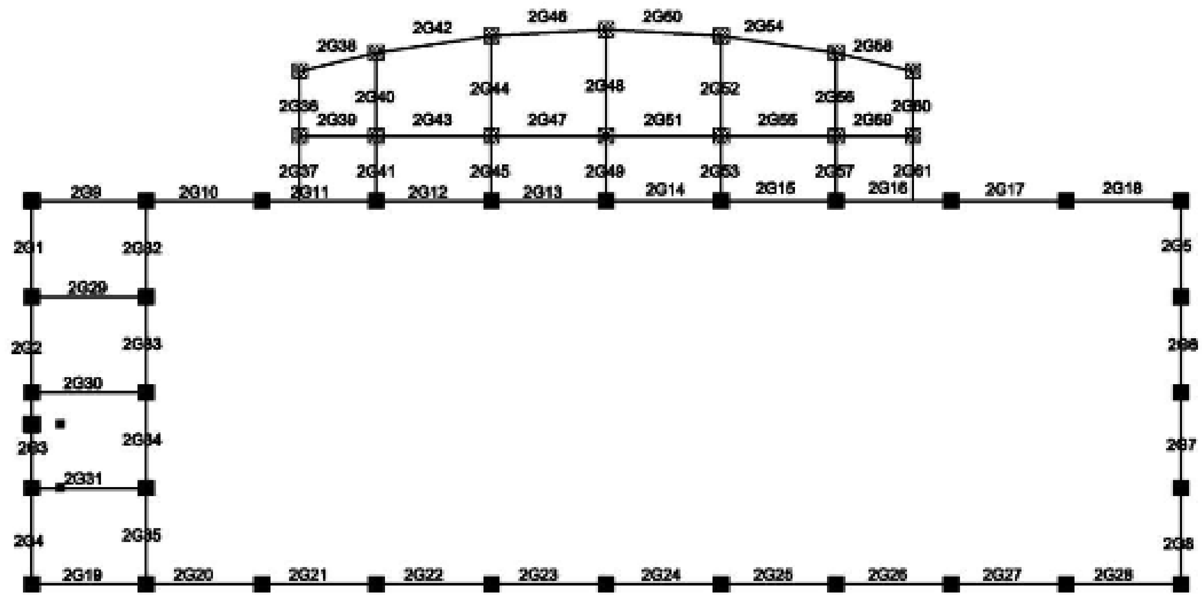
	12/17/2008	



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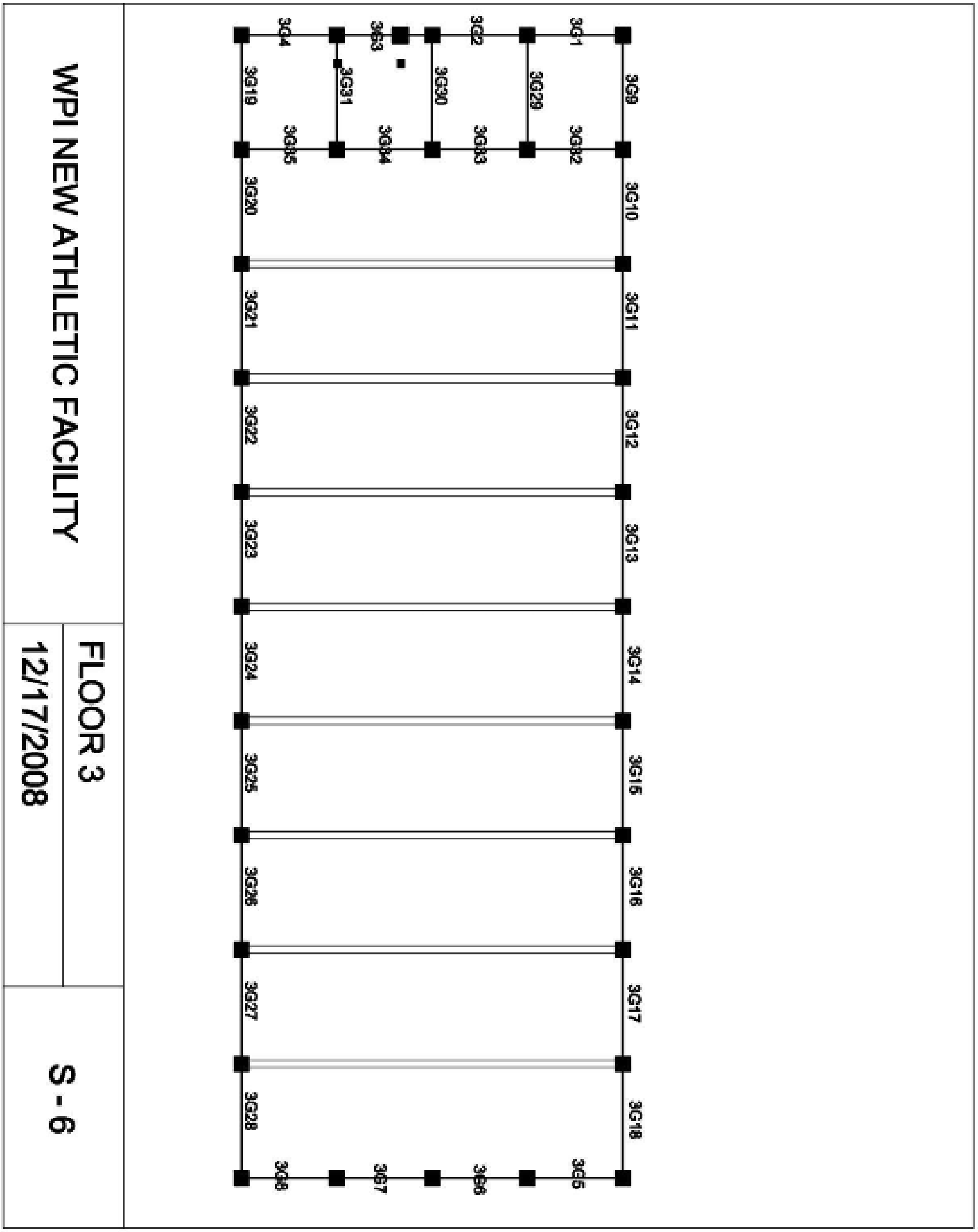
WPI NEW ATHLETIC FACILITY
FLOOR 3
12/17/2008
S - 6
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3G3
3G4
3G5
3G6
3G7
3G8
3G9
3G10
3G11
3G12
3G13
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3G21
3G22
3G23
3G24
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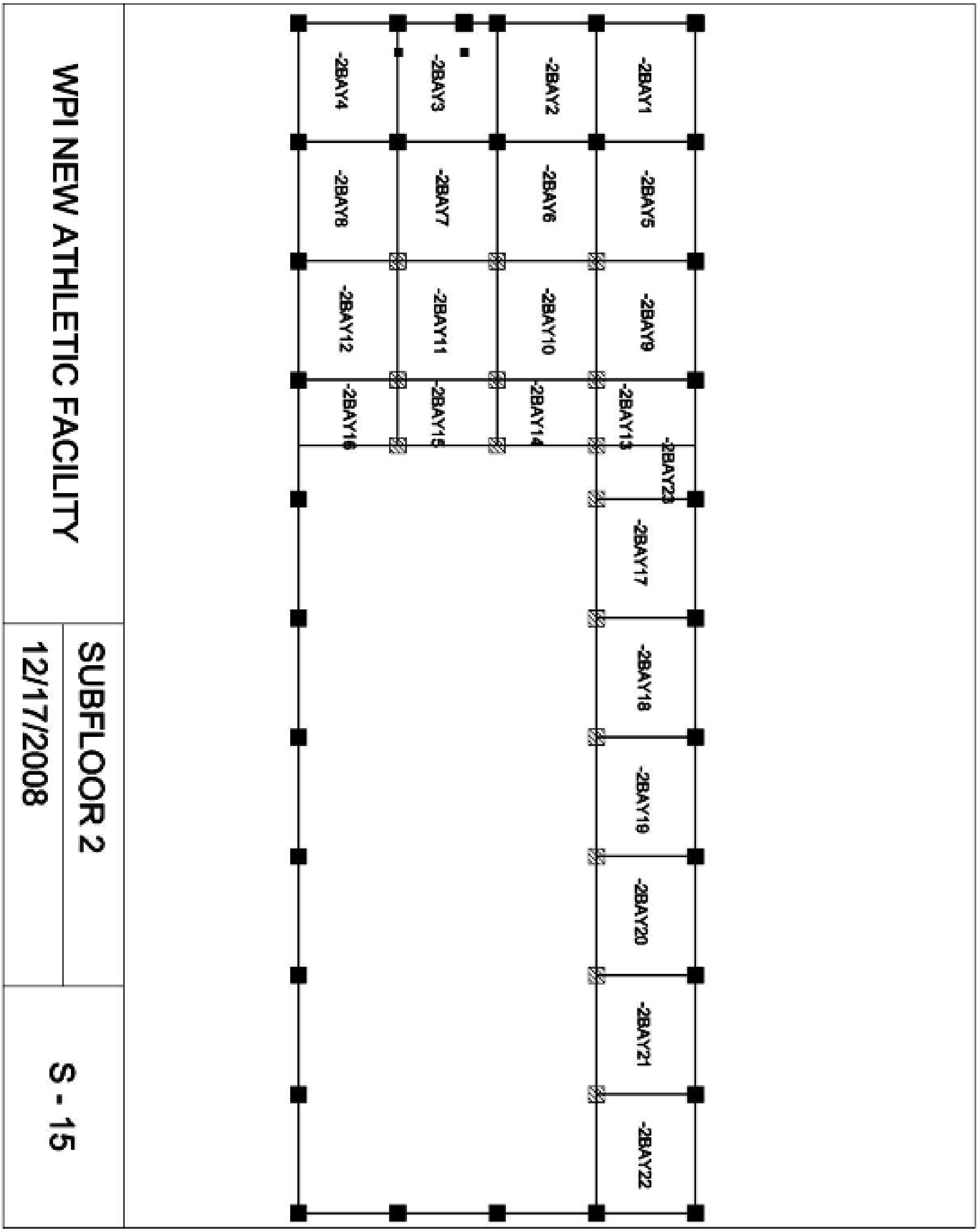
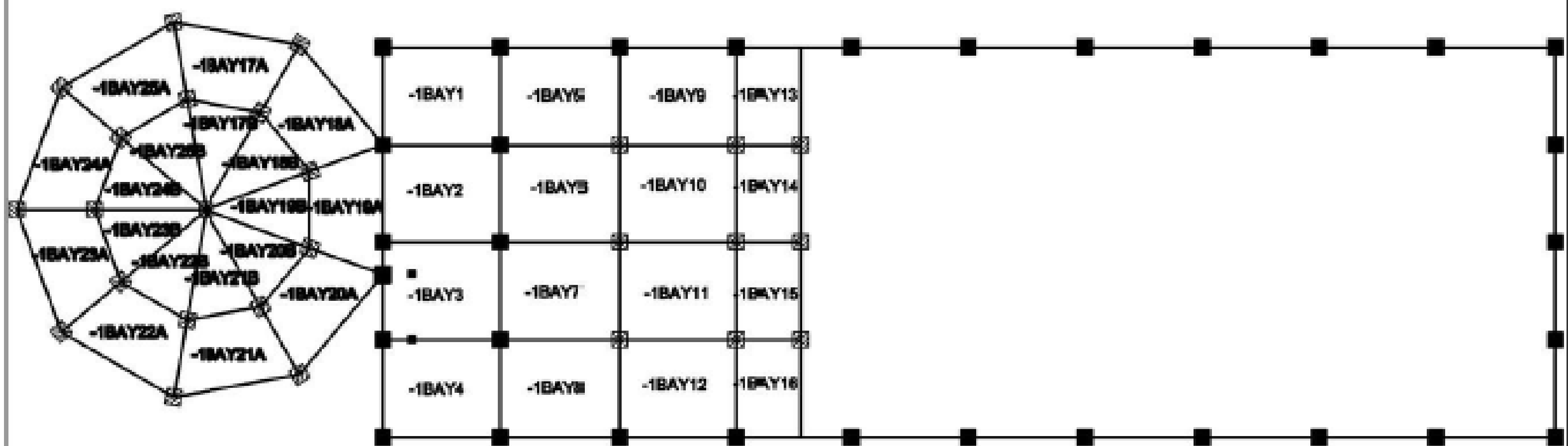
3G31
3G32
3G33
3G34
3G35

Labeled Beam Layout





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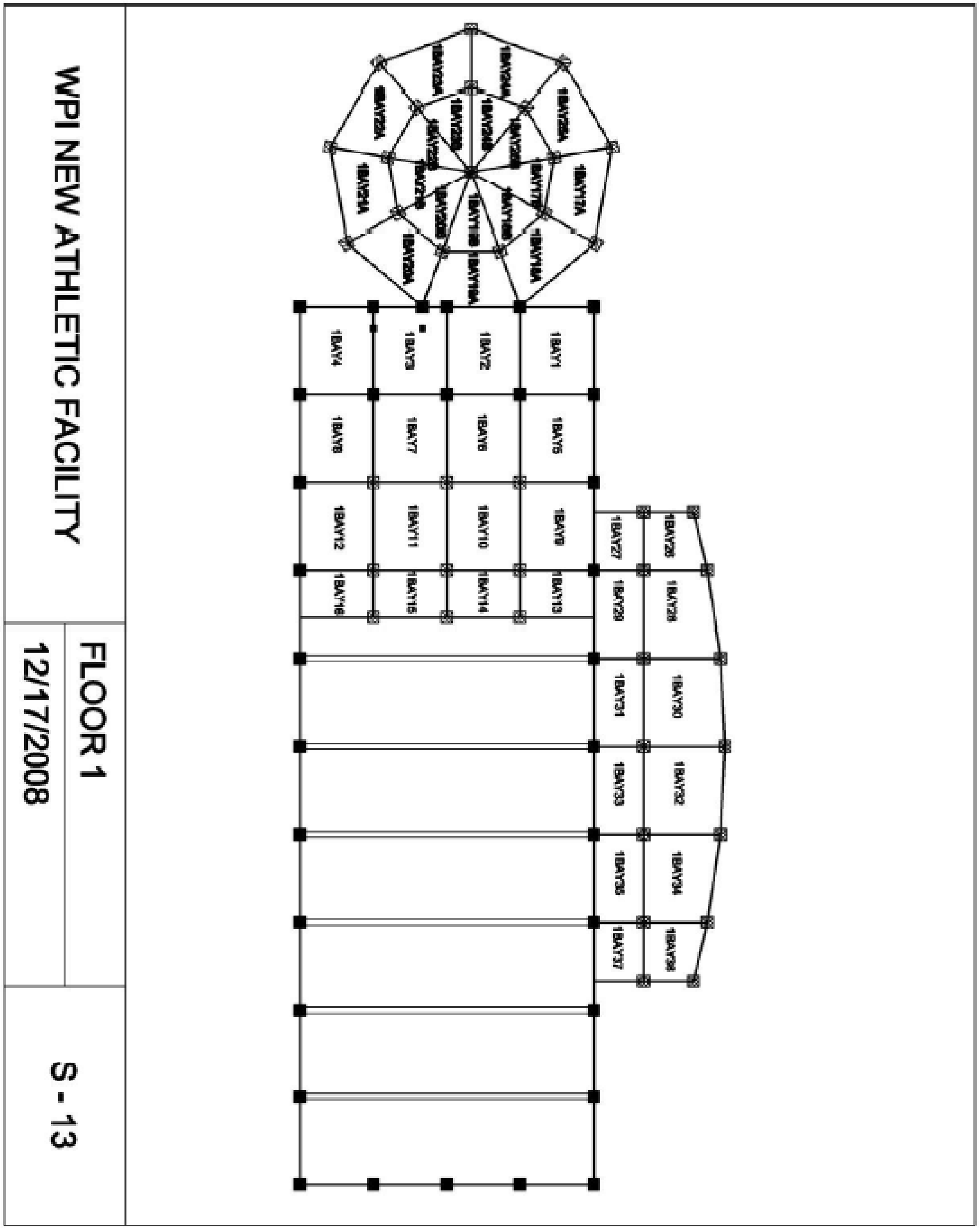
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FLOOR 1
12/17/2008
S - 13
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1BAY7
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1BAY15
1BAY16
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1BAY27
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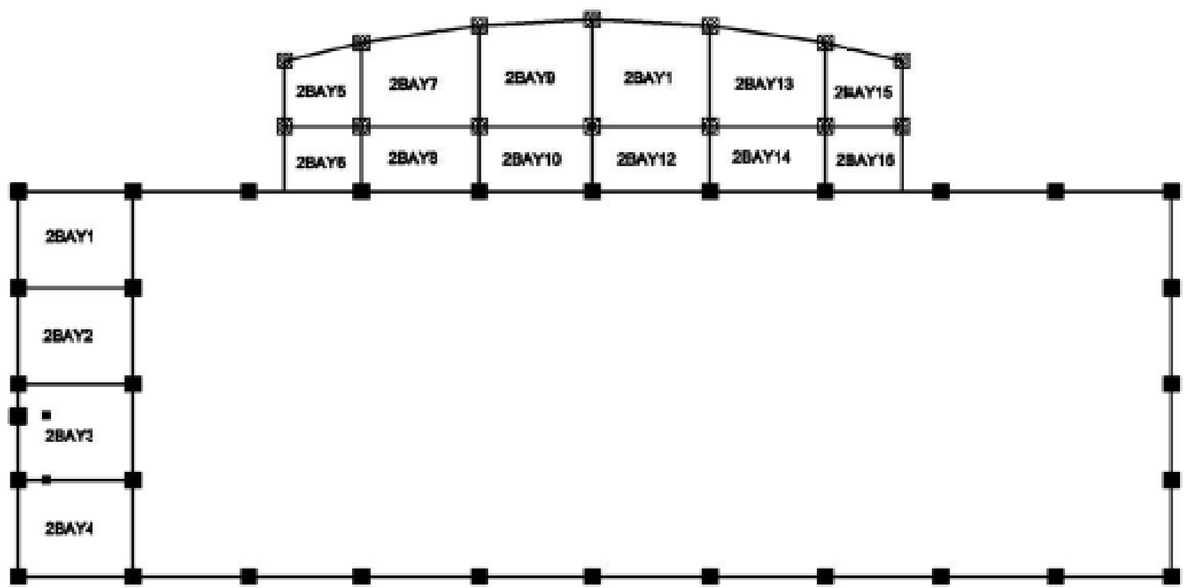
1BAY36
1BAY37



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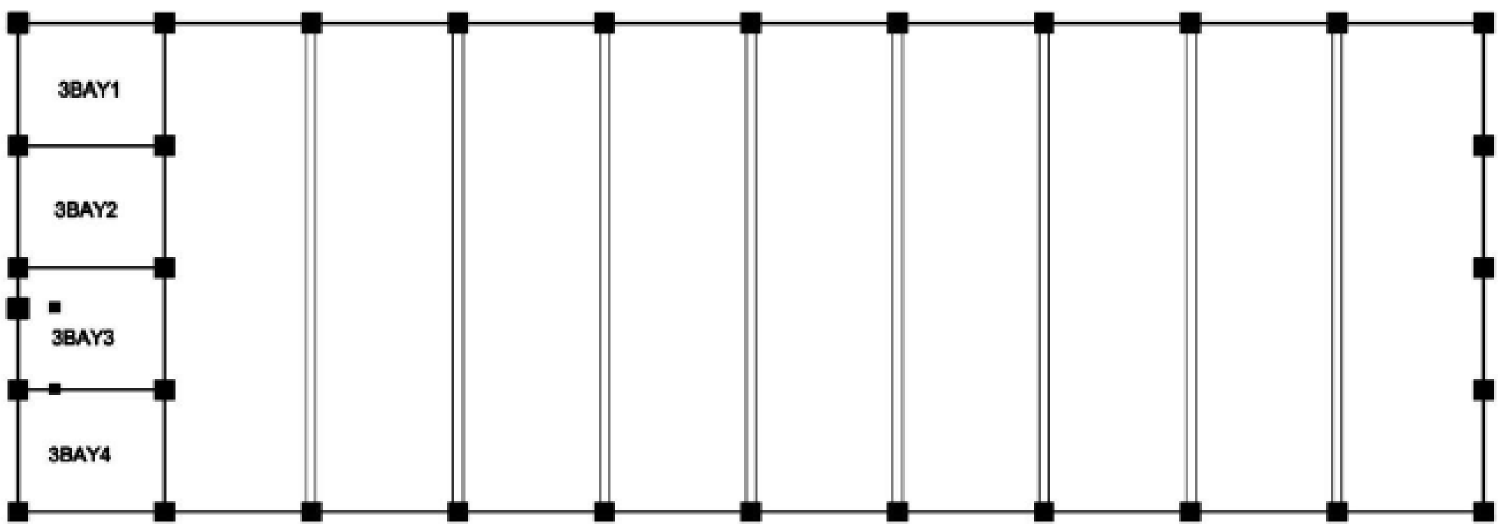

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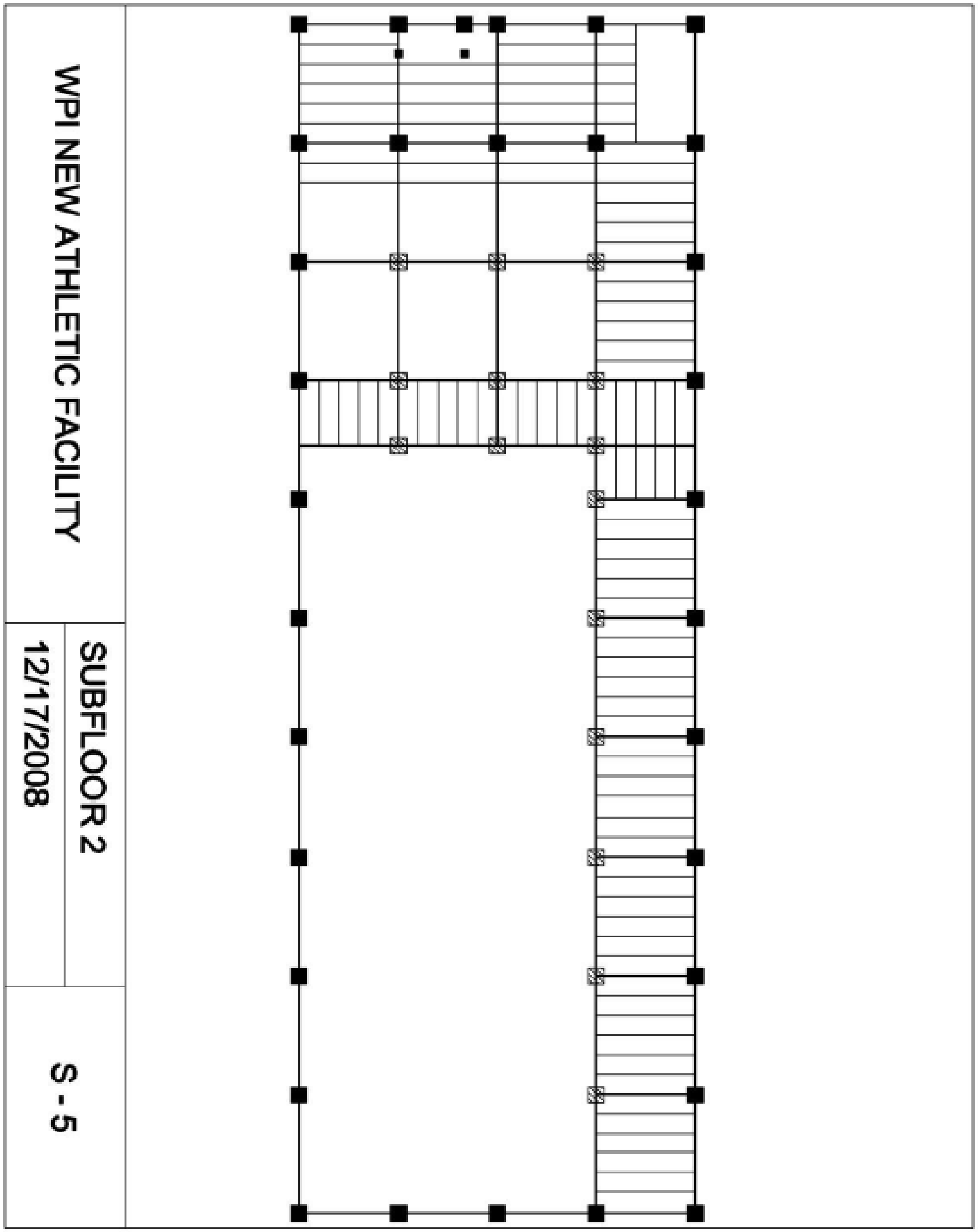
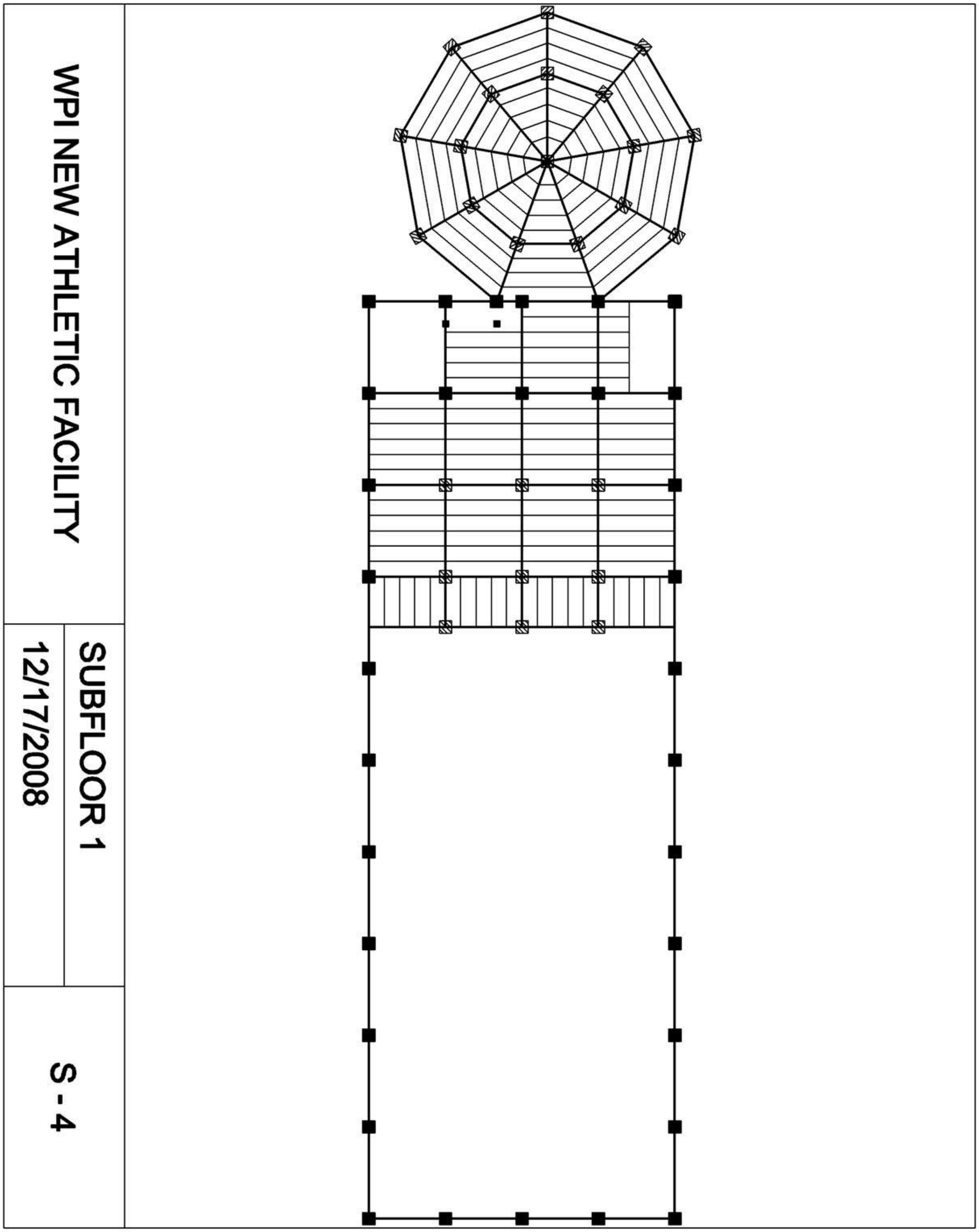
	12/17/2008	

Structural Layout

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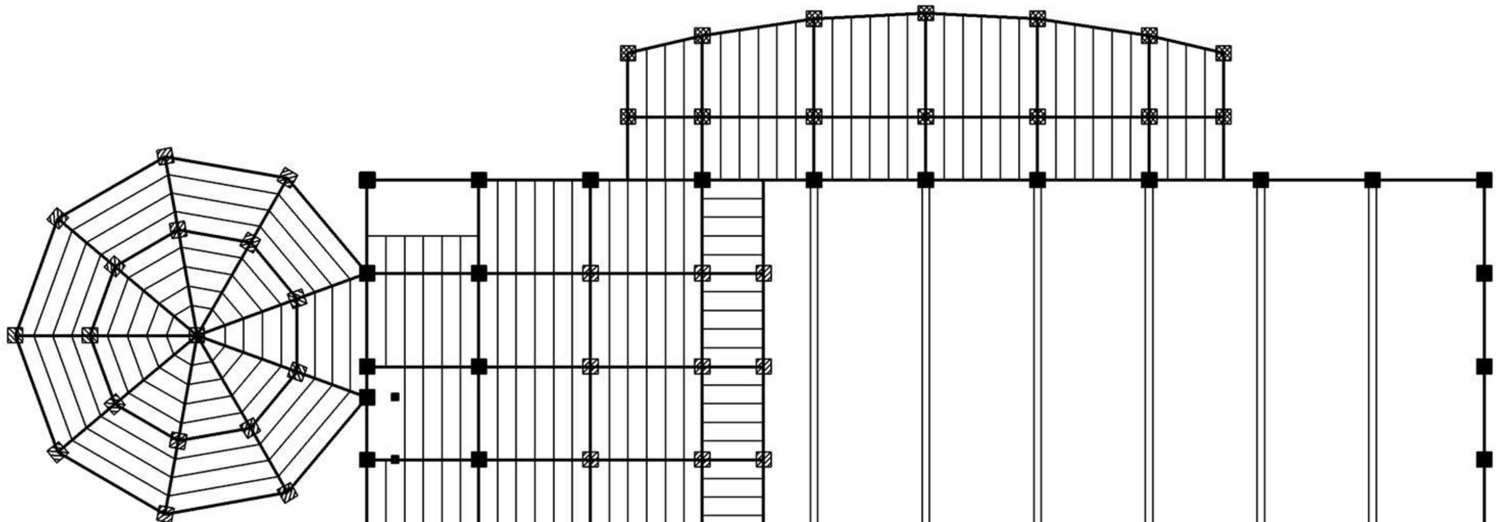
SUBFLOOR 1
12/17/2008
S - 4



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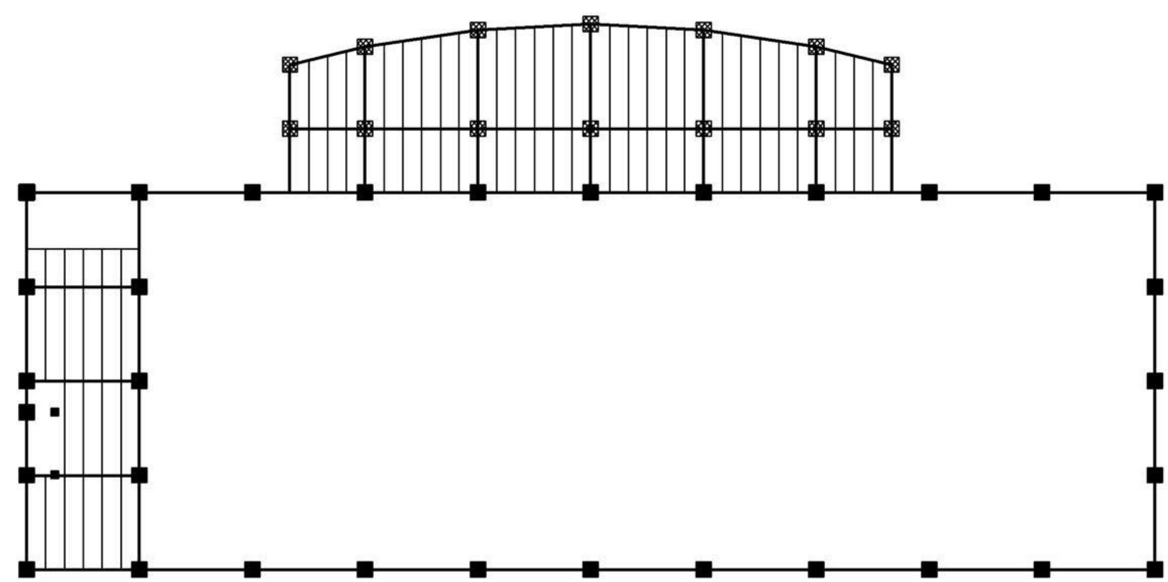

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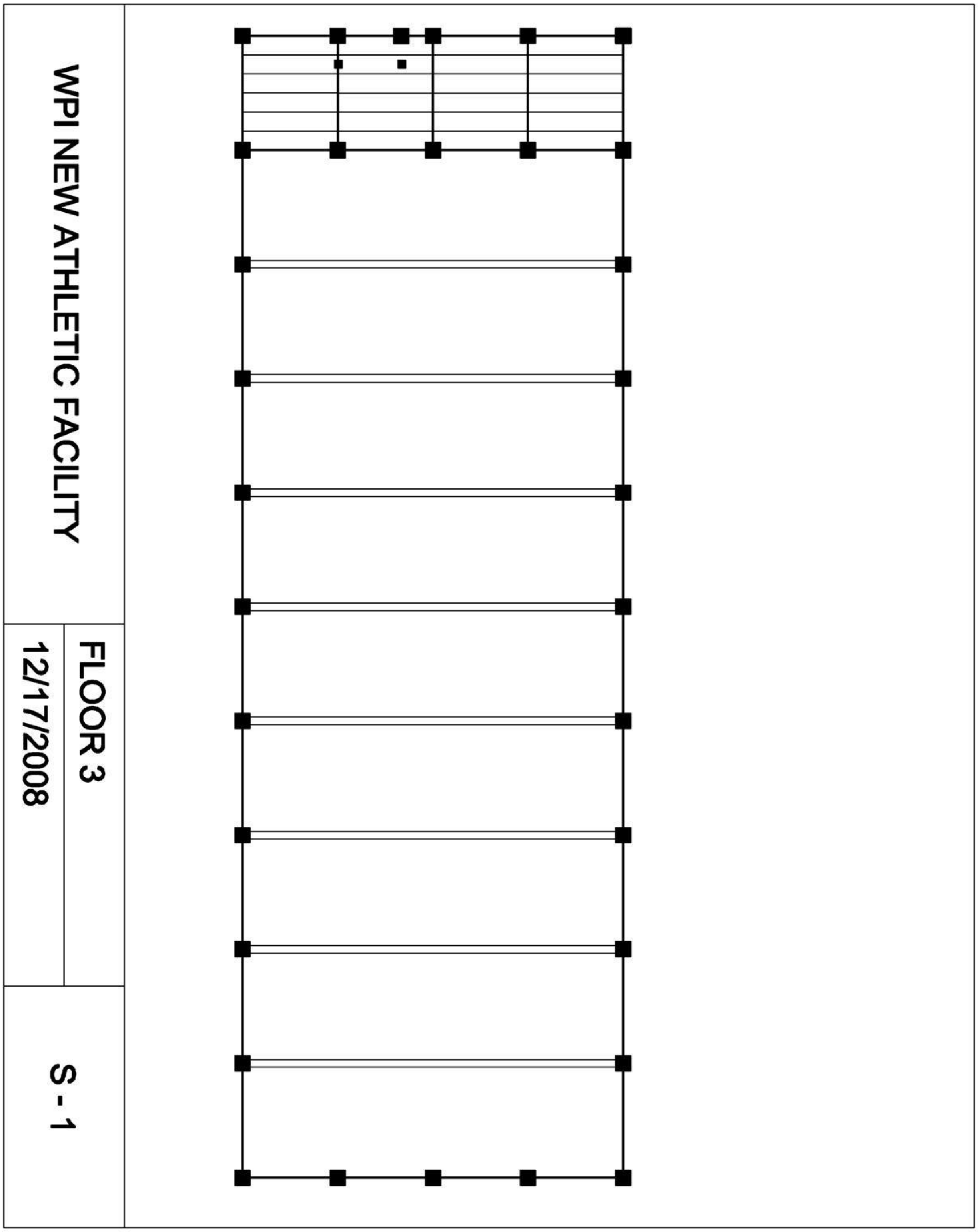
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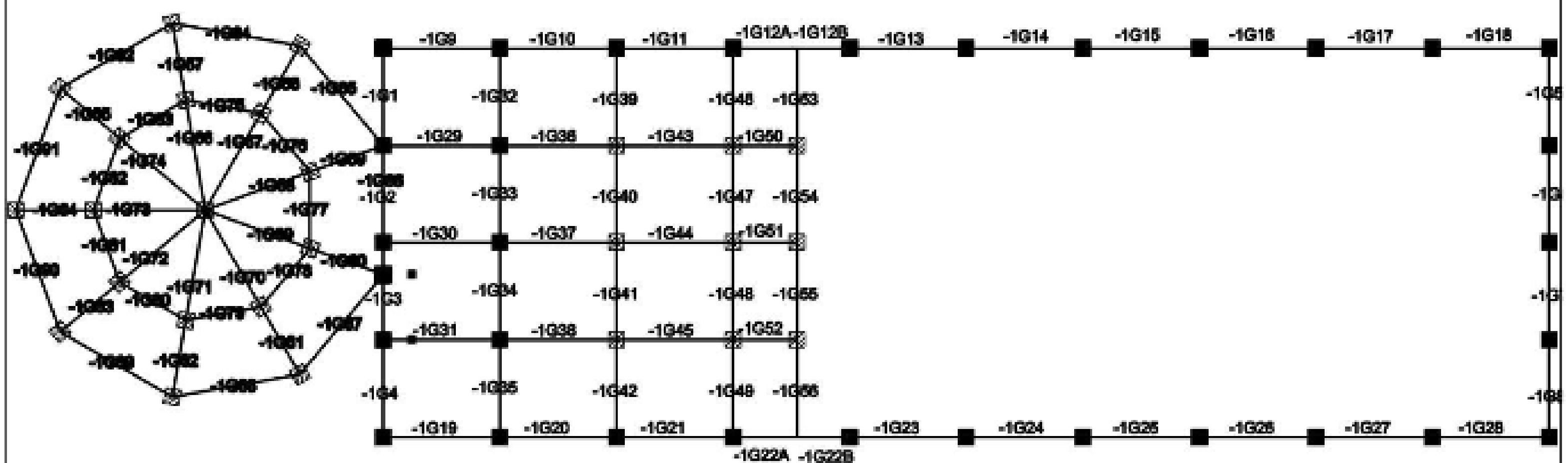
FLOOR 3
12/17/2008
S - 1

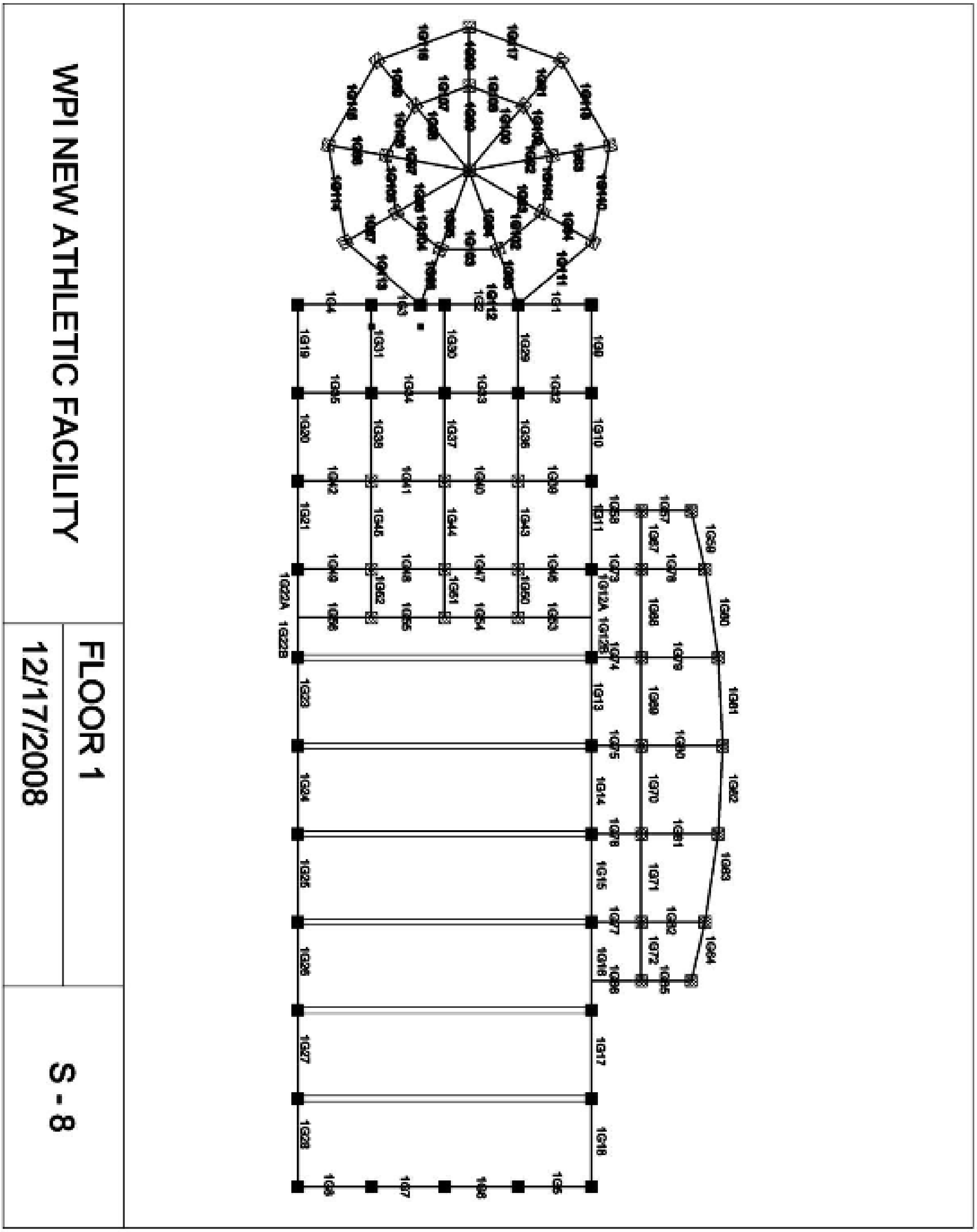


WPI NEW ATHLETIC FACILITY

SUBFLOOR 1

12/17/2008

S - 9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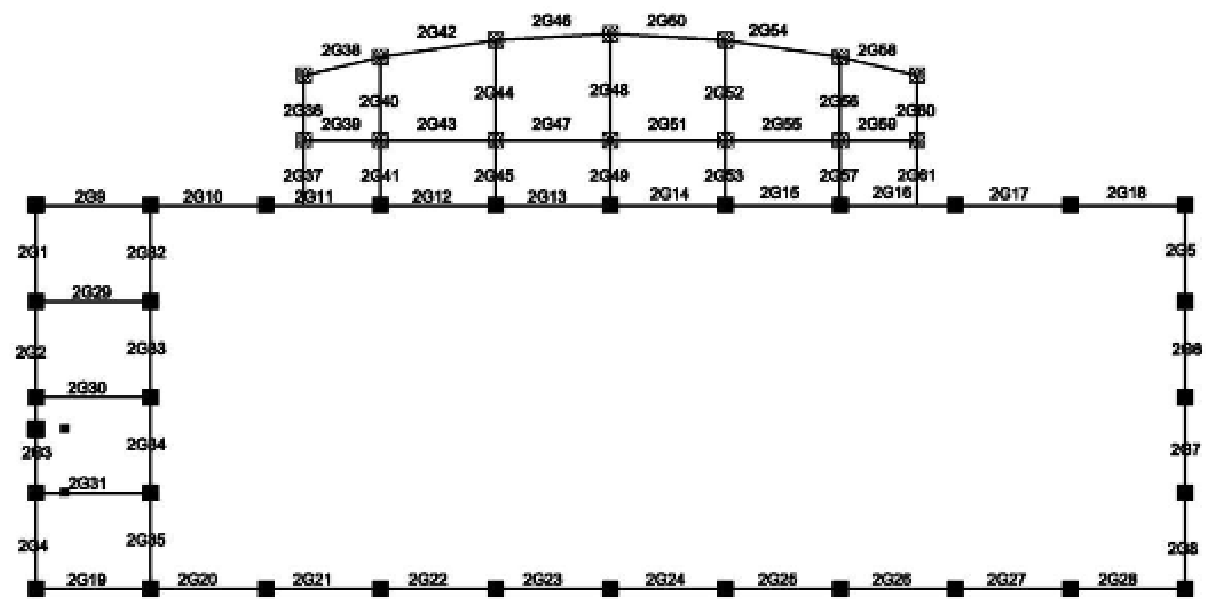
FLOOR 1
12/17/2008
S - 8



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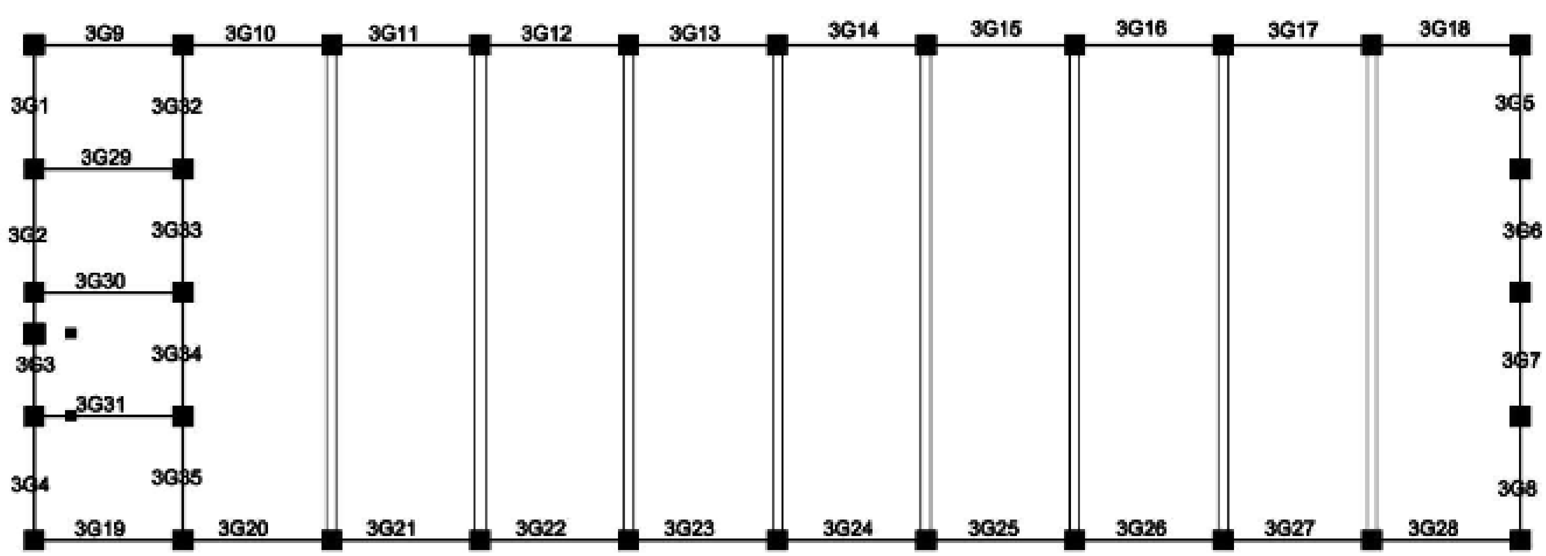
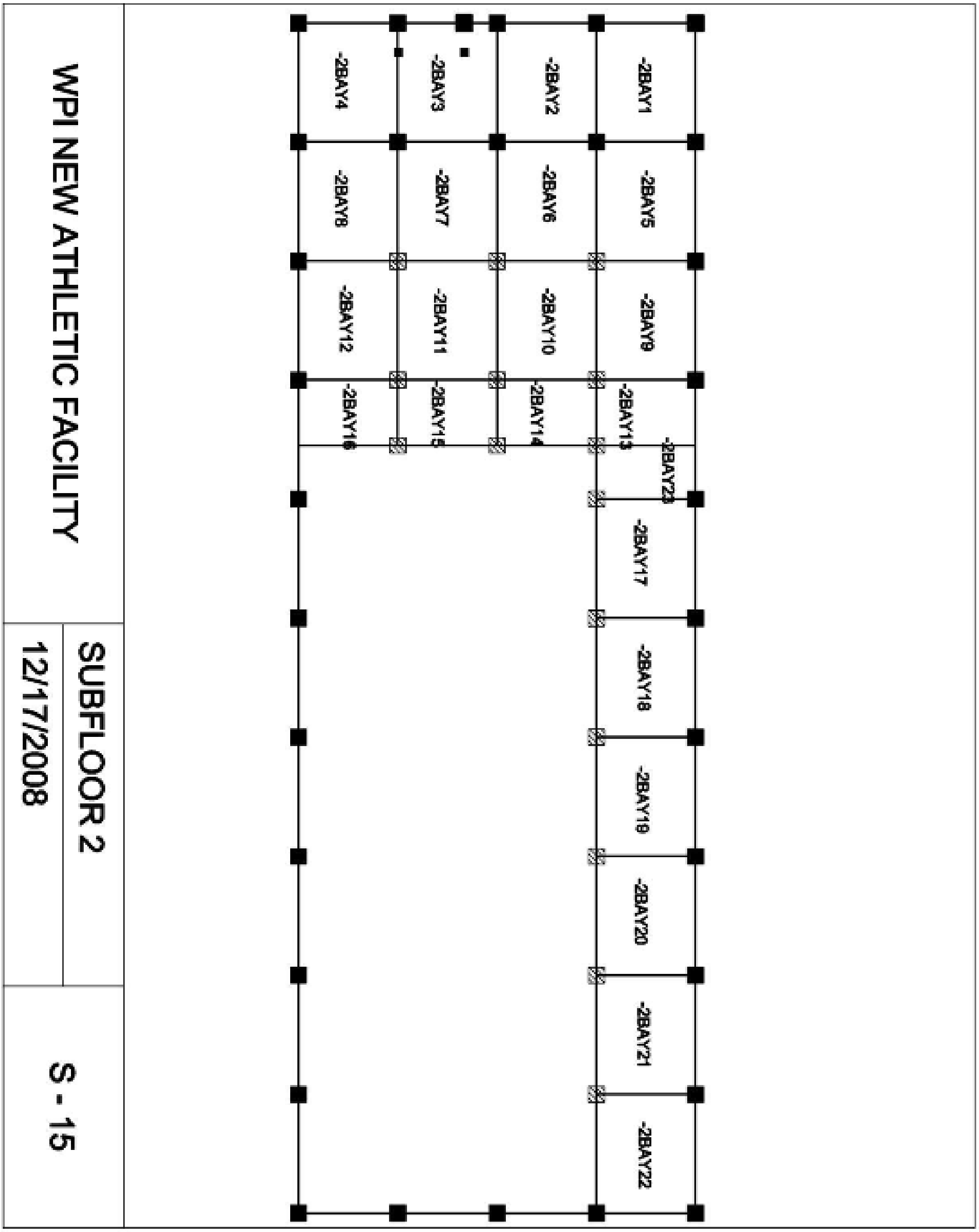
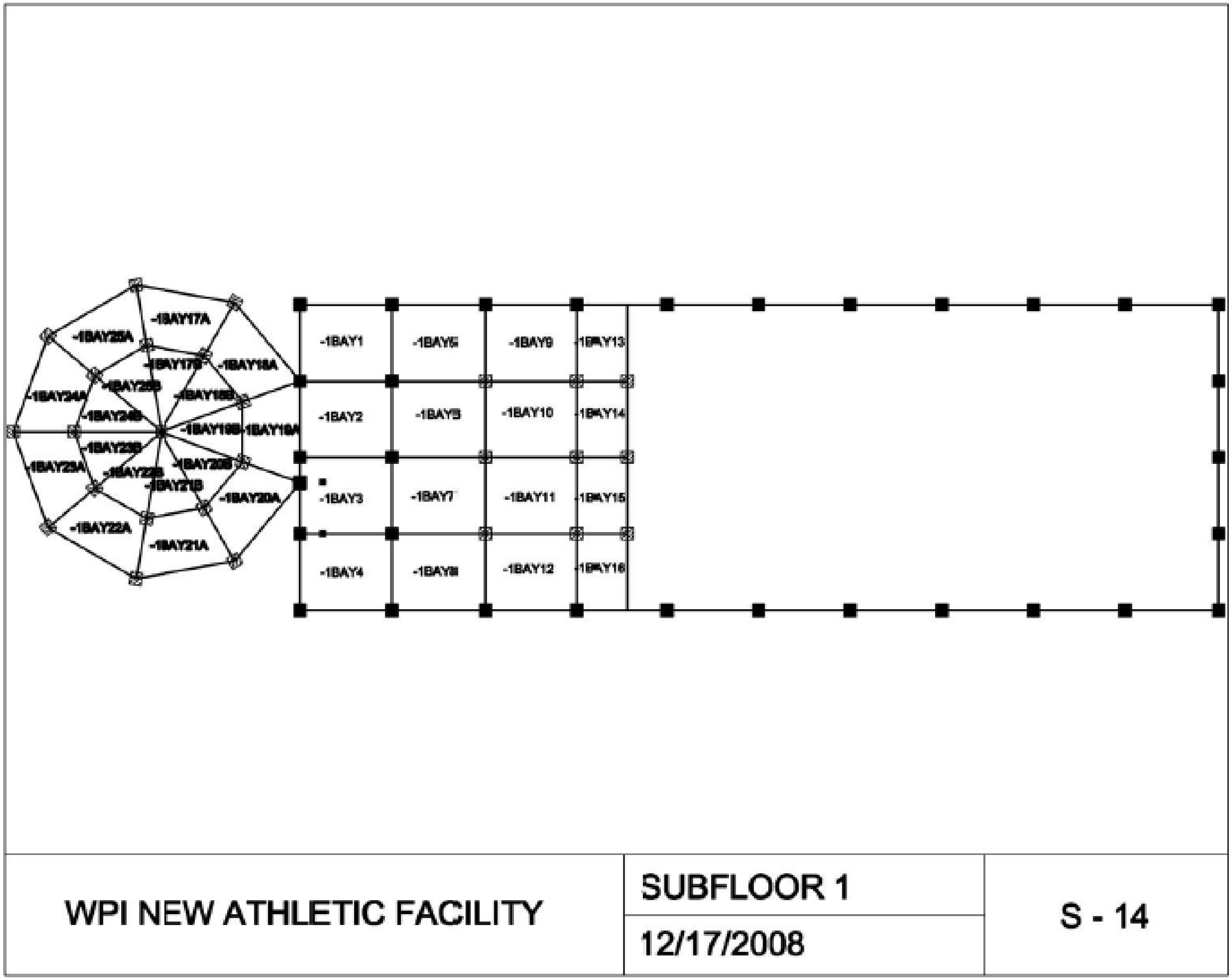


WPI NEW ATHLETIC FACILITY	FLOOR 3 12/17/2008	S - 6

Labeled Beam Layout





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-1BAY2
-1BAY3
-1BAY4
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-1BAY10
-1BAY11
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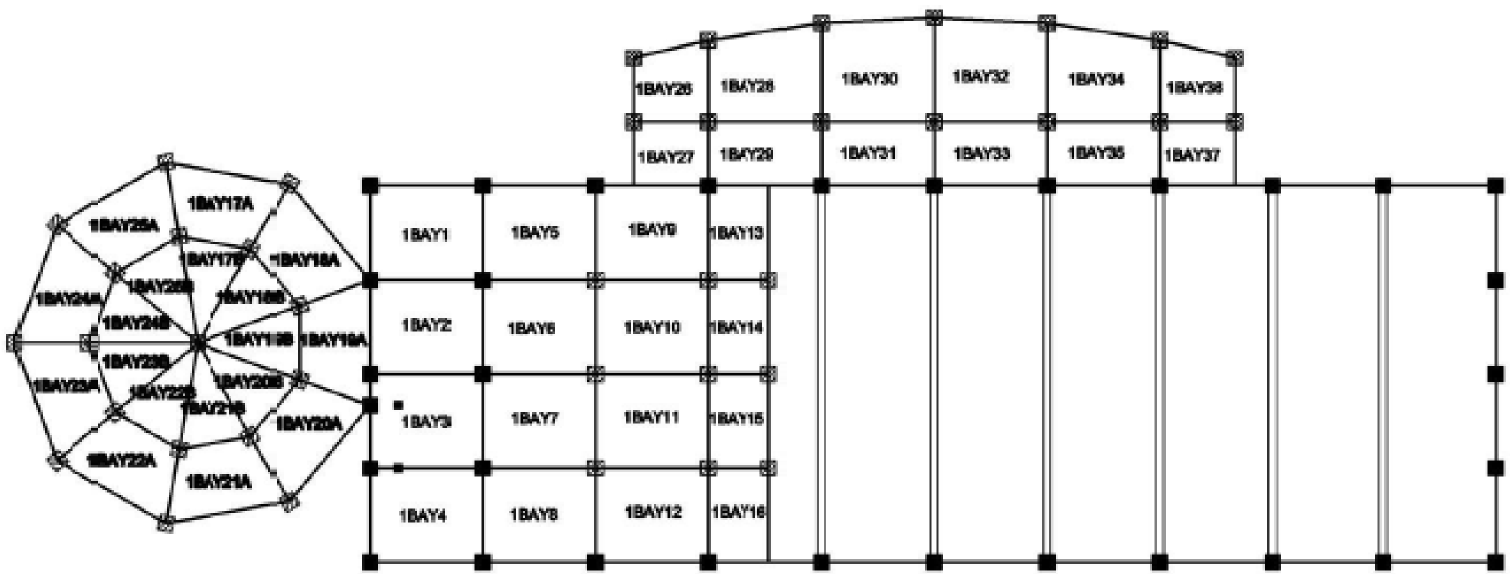
WPI NEW ATHLETIC FACILITY
SUBFLOOR 1
12/17/2008
S - 14



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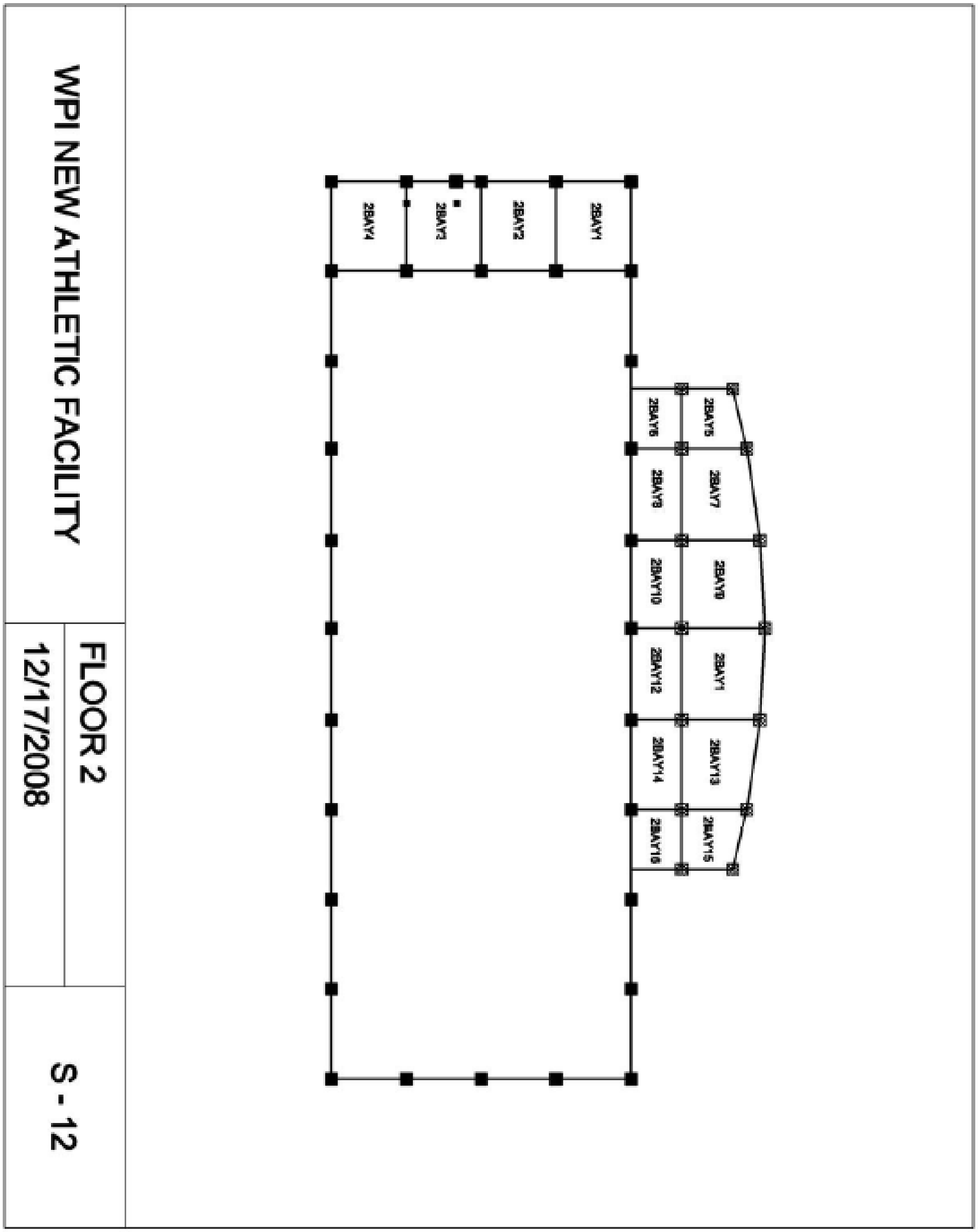
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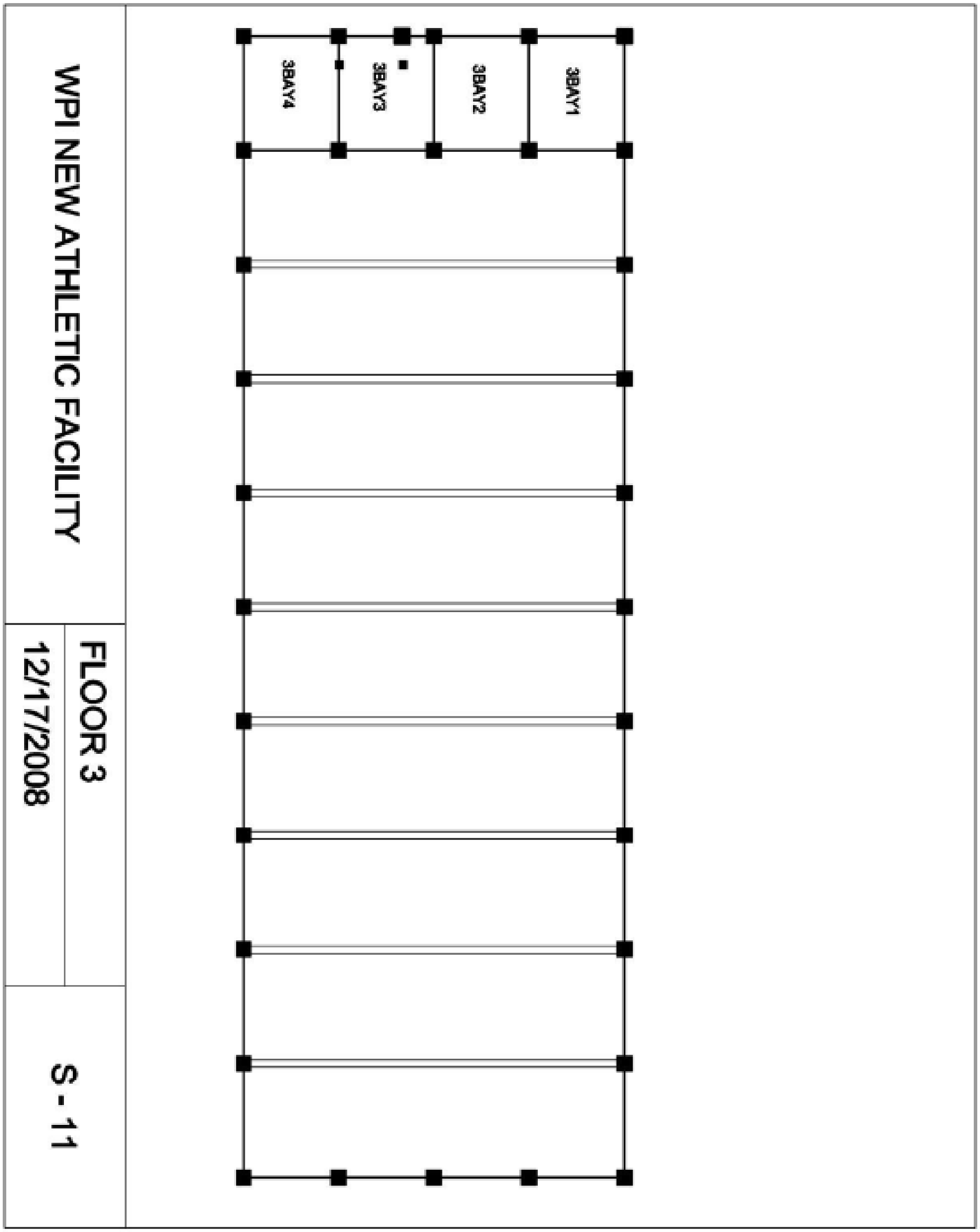
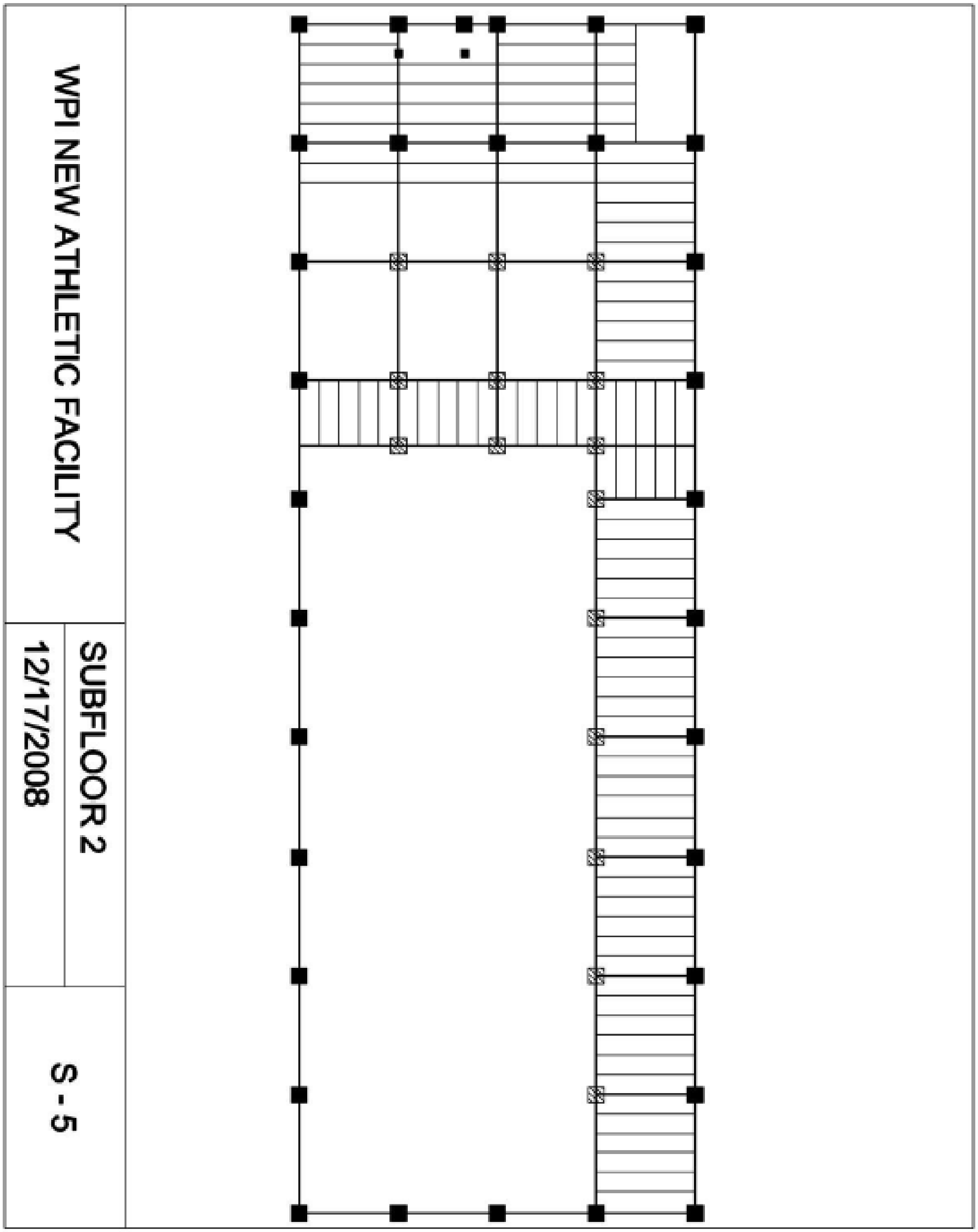
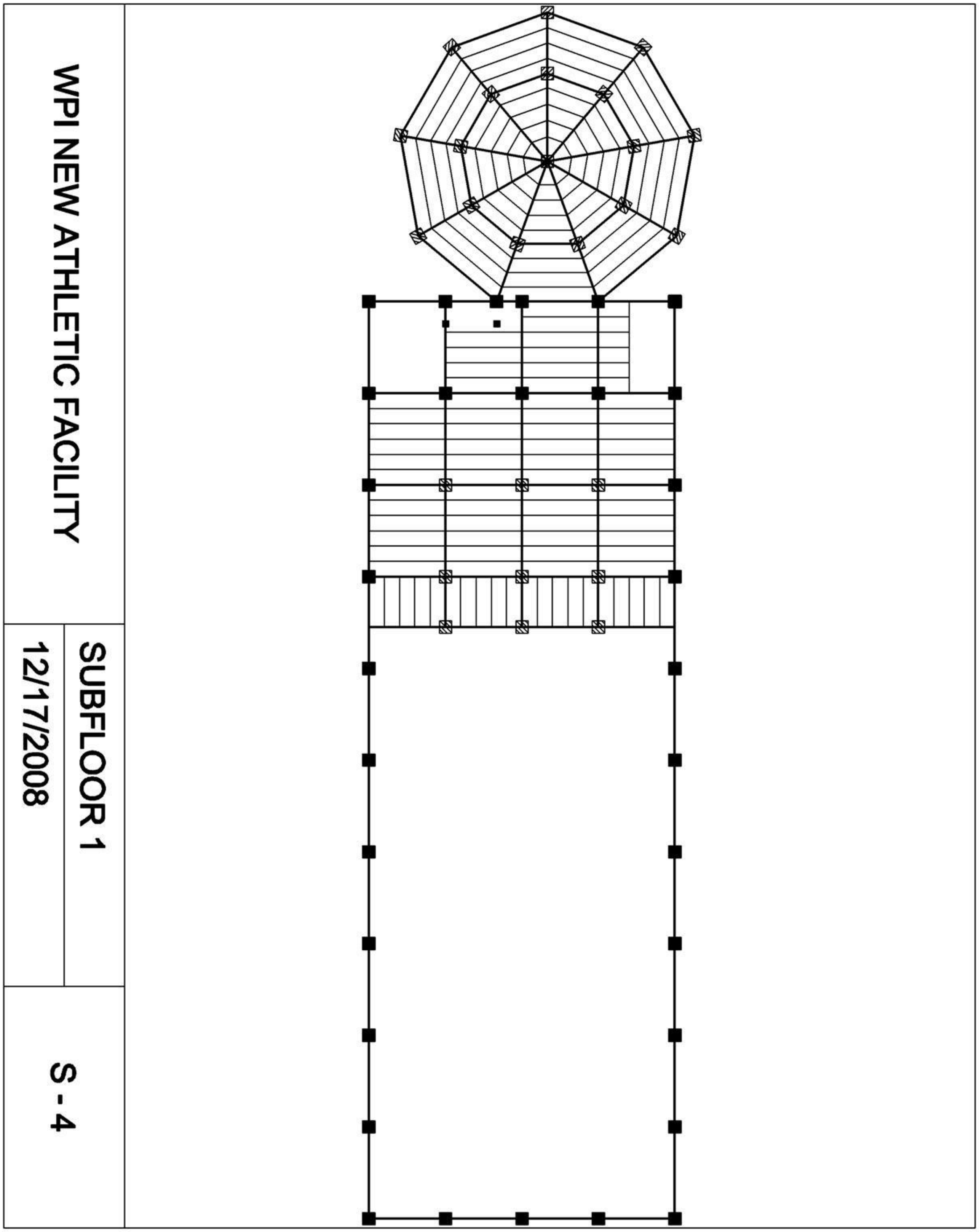
FLOOR 2
12/17/2008
S - 12


WPI NEW ATHLETIC FACILITY
FLOOR 3
12/17/2008
S - 11
3BAY1
3BAY2
3BAY3
3BAY4

Structural Layout

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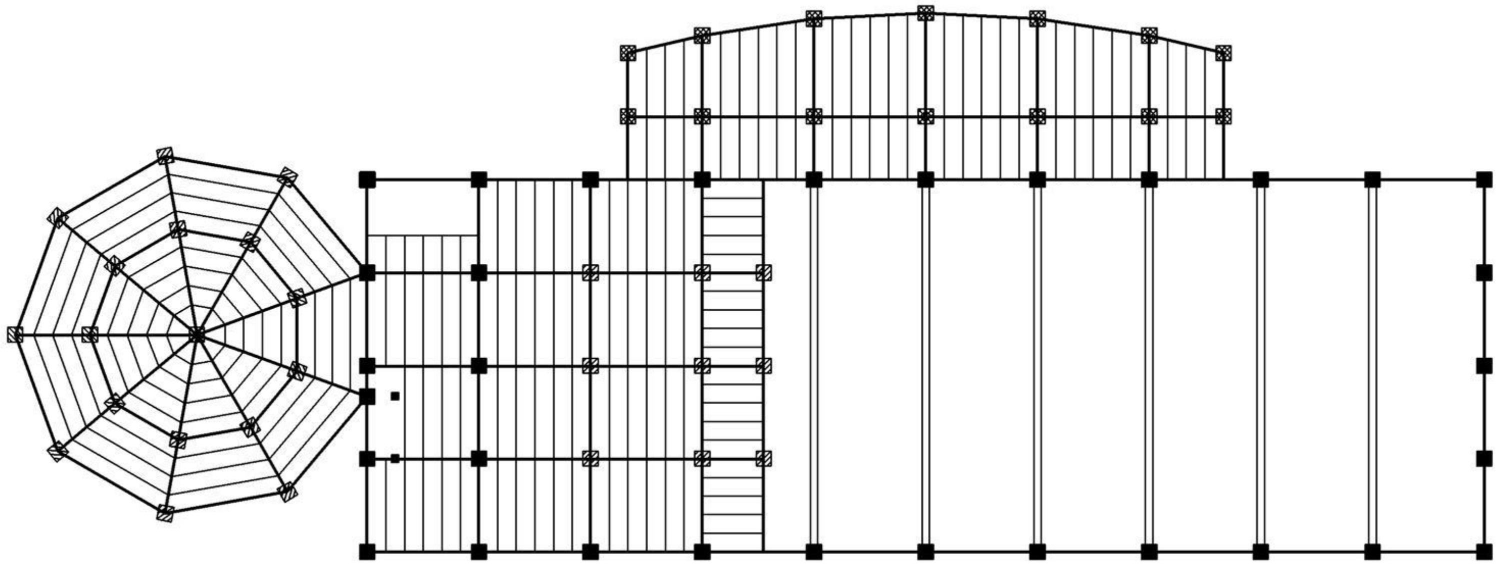
SUBFLOOR 1
12/17/2008
S - 4



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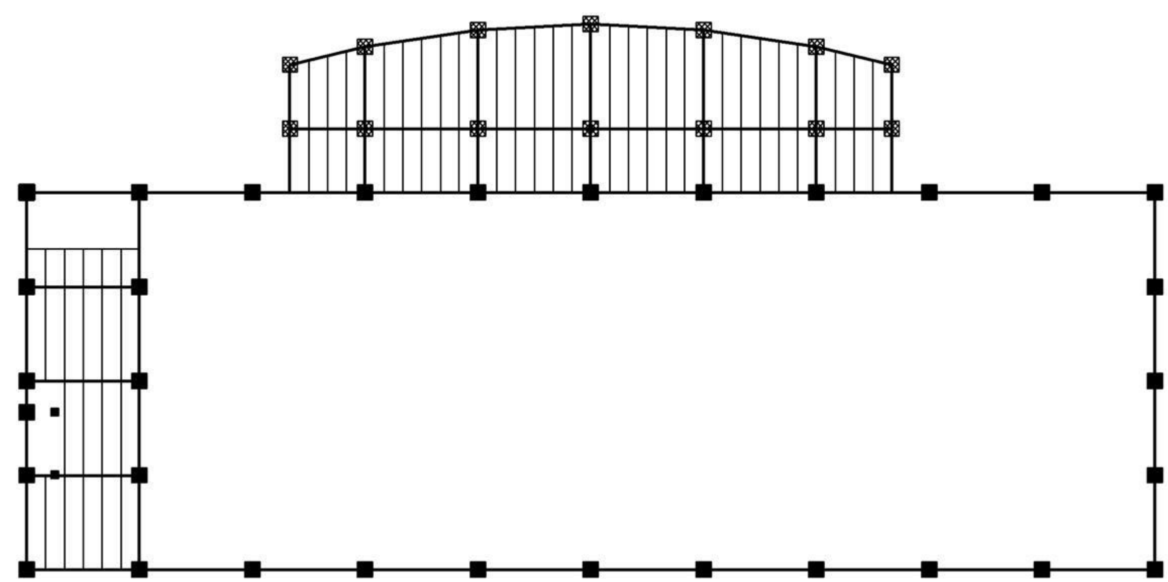
	12/17/2008	



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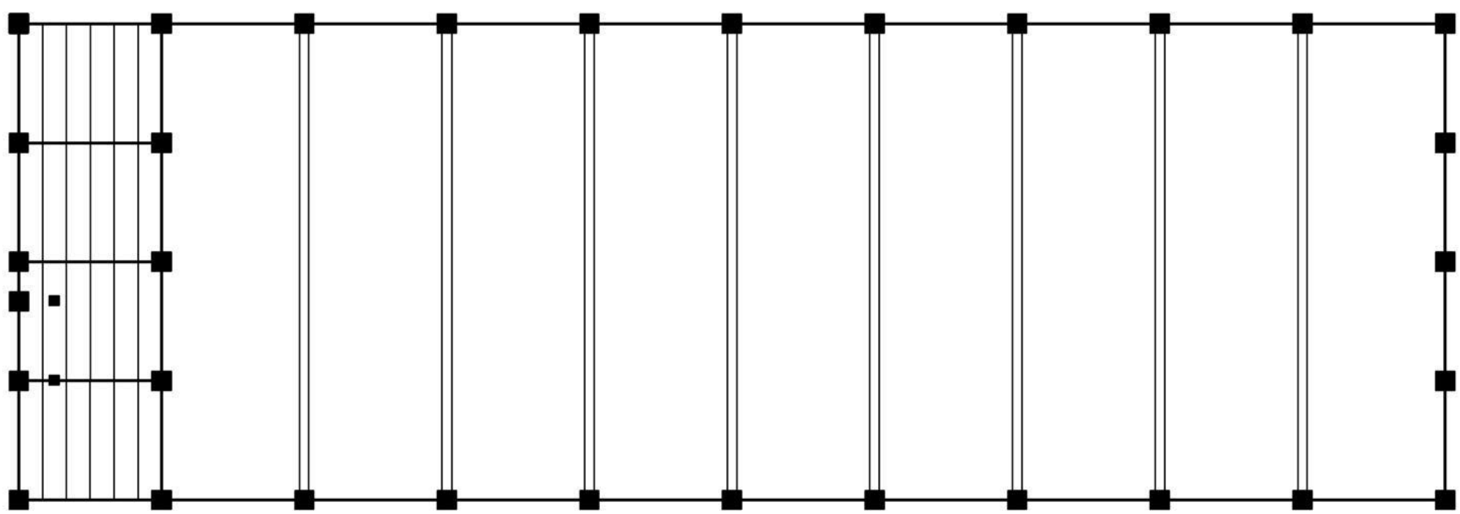
	12/17/2008	



WPI NEW ATHLETIC FACILITY	FLOOR 3 12/17/2008	S - 1

Appendix B

Permitting

I. DEPT. OR PUBLIC WORKS AND PARKS

- A. Completely filled out Application
  1. Current Dig Safe numbers
  2. Notification to Worcester Water and Sewer Department
  3. Explanation of Application
  4. Sketch
     - a. Location of work
     - b. Risk of injury to road users
     - c. Probability of damage to
       - i. trees
       - ii. highways structures
       - iii. private property
     - d. Property lines
     - e. Roads/Intersections
     - f. Pavement lines
     - g. Sidewalks
     - h. Trees
     - i. Drainage Structures
     - j. Utility Poles (with pole numbers)
     - k. Character and extent of work
     - l. Excavation
       - i. Cut lines
       - ii. Dimensions
       - iii. Relations to existing buildings
       - iv. Sieve Analysis
       - v. Proposed back fill material
  5. Proposed Methods and Materials
- B. Bond Requirement
  - --In the amount and form of Commissioner of Public Works request
  - --Amount will be separate for each permit or Annual Blanket Bond
- C. Insurance Requirement
  - --Automobile
  - --Property damage liability
  - --Bodily injury liability
  - --Workmen's Compensation
- D. Payment of Fees

II. FIRE DEPARTMENT

- A. Blasting
  1. Blaster submits application
  2. Blaster submits explosives user's certificate.
  3. Blaster submits copy of bond from State Treasurer - $20,000.
  4. Blaster submits copies of photo I.D. of blasters.

5. Blaster submits Pre blast survey address list.
6. Blaster submits Dig Safe Number.
7. Blaster renders $50.00 check or money order.
8. Blaster must give minimum 24 hour notice for the start of blast.

B. Cutting and Welding

1. Applicant applies for permit
2. Permit Fee - $50.00 check or money order.
3. Inspection of area by the Worcester Fire Department prior to start of work. This will determine if a fire watch will be conducted by the fire department or the permit holder. A fire watch shall be mandated.
4. The fire watch shall remain on the job at least 30 minutes after welding or cutting operations have been completed.

C. Dumpster

1. Dumpster contractor or customer applies for permit
2. Permit Fee - $25.00 per container and $10.00 for each additional container on a particular site. Check or money order.

D. Fire Alarm Installation

1. Licensed contractor applies for permit.
2. Permit Fee - $50.00 check or money order.
3. Inspection within one week.
4. All newly wired and updated systems must first be signed-off by Wiring Inspector.

E. Fire Lane

1. Fire Lane Form shall be completed by the owner of concerned property.
2. The Worcester Fire Department reviews and inspects area for designated Fire Lane.
3. Agreement signed by property owner and Fire Prevention

F. Flammable Storage

1. Applicant must contact Fire Prevention at (508) 799-1822 before applying.
2. Applicant applies for permit.
3. Applicant supplies a list of product names and quantities.
4. Permit Fee - $50.00 check or money order

G. Oil Burner Permit

1. The oil burner technician applies for permit at:
Worcester Fire Department OR,
Department of Inspectional Services, Building & Zoning Division.
2. Permit Fee - $50.00 check or money order
3. Installation to be inspected and approved by Code Enforcement Wiring Inspector.
4. Fire inspection will be set up after installation is complete.

H. Ceremonial Fire

1. Applicant applies for permit
2. Permit Fee - $50.00 check or money order
3. Area of burn is inspected to determine if a fire detail is required.
4. Permit must be applied for 7-10 days in advance of event.

I. Propane Storage

1. Applicant applies for permit at least 3 business days prior to event.
2. Permit Fee - $50.00 check or money order
3. Inspection by The Worcester Fire Department within 7-10 days

J. Smoke and Carbon Monoxide Detector

1. Applicant applies for permit
2. Permit Fee - $50.00 check or money order
3. Inspection by The Worcester Fire Department within 10-14days
4. All newly wired and updated systems must first be signed off by Wiring Inspector.

K. Sprinkler Installation

1. Licensed sprinkler contractor submits plans to the Building & Zoning Division and Fire Prevention.
2. Contractor applies for permit.
3. Permit Fee - $50.00 check or money order

L. Tank Installation

1. Contractor applies for permit.
2. Permit Fee is according to gallonage and size ($ 50.00 - $250.00)
3. Make check or money order
4. Must give two hour notice to The Worcester Fire Department for inspection

III. INSPECTIONAL SERVICES

A. Building Permit

1. Zoning Approval Form
2. Land Use Form
3. Posting Permit Affidavit
4. Disposal Affidavit
5. Construction Control Affidavit (only for buildings over 35,000cu.ft.)
6. Consumer Information Form (for sunrooms)
7. Fire Extinguishing Equipment

B. Electrical Permit and Inspection

C. Plumbing/gas Permit and Inspection

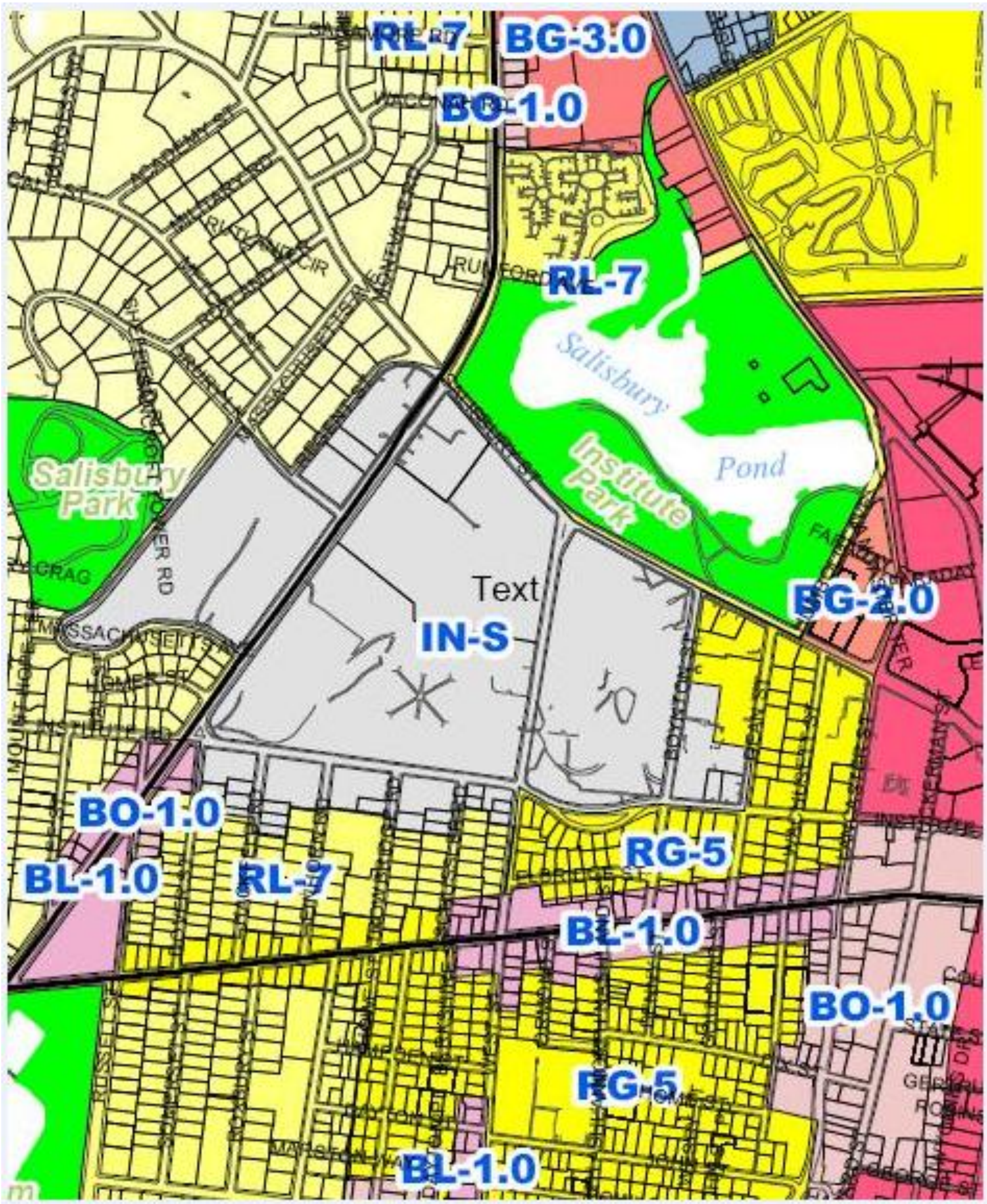
IV. PLANNING AND REGULATORY SERVICES

A. Historical Commission

B. Zoning Board of Appeals

C. Planning Board

Appendix C
Zoning District



Appendix D

Wind Loads

- Three methods of Wind Loads design
  - Our building is not a "low-rise" by definitions in 6.2 because height exceeds 60ft at 82ft on the football field side, therefore cannot use Method 1 (simplified procedure).
  - Method 2 (Analytical Procedure) -
  - Method 3 (Wind Tunnel Procedure) – does not apply to this project as a wind tunnel is not available/the most practical approach.
- The basic wind speed V and wind directionality factor K_d shall be determined in accordance with Section 6.5.4
  - V = 100 mph
  - K_d = 0.85
- An importance factor I shall be determined in accordance with section 6.5.5
  - Cat III – I = 1.15
- An exposure category or exposure categories and velocity pressure exposure coefficient K_z or K_h, as applicable, shall be determined for each wind direction in accordance with Section 6.5.6
  - Surface Roughness B – α= 7.0; z_g (ft) = 1200; a^ = 1/7; b^ = 0.84; α-bar = 1/4.0; b-bar = 0.45; c = 0.30; l (ft) = 320; Є-bar = 1/3.0; z_{min} (ft) = 30
  - Exposure B – K_z 0.936
- A topographic factor K_{zt} shall be determined in accordance with Section 6.5.7
  - N/A
- A gust effect factor G or G_f, as applicable, shall be determined in accordance with Section 6.5.8
  - 0.85 or consider equation
- An enclosure classification shall be determined in accordance with Section 6.5.9
  - Building, Enclosed, No coefficient here
- ~~Internal pressure coefficient GC_{pi} shall be determined in accordance with Section 6.5.11.1~~
  - ~~+0.18, -0.18 check figure 6-5 for details~~ NOT APPLICABLE
- External pressure coefficients C_p or GC_{pf}, or force coefficients C_f, as applicable, shall be determined in accordance with Section 6.5.11.2 or 6.5.11.3, respectively
  - C_p = N/A?; GC_{pf} = (see table 1[4] below); C_f = N/A?
- Velocity pressure q_z or q_h, as applicable, shall be determined in accordance with Section 6.5.10
  - $q_z = q_h = 0.00256K_zK_{zt}K_dV^2I$ (lb/ft^2) = 23.422 lb/ft^2

[4] Tentative table numbering

- Design wind load p or F shall be determined in accordance with Section 6.5.12, 6.5.13, 6.5.14, and 6.5.15, as applicable
    - $p = q(GC_p) - q_i(GC_{pi})$ (lb/ft^2) (N/m^2)[5]
    - $p = q(GC_{pf} - GC_{pi}) = 23.422(\cancel{1.18GC_{pf}}) =$ (see table 2[6] below)

[5] Use absolute value to produce most critical load cases

[6] Tentative table numbering

Table 1[7]

Level	Zone 1	Zone 2	Zone 3	Zone 4	Zone 5
1	N/A	N/A	N/A	0.7	1.0
2	N/A	N/A	N/A	0.7	1.0
3	N/A	N/A	N/A	0.7	1.0
4	N/A	N/A	N/A	0.7	1.0
5	N/A	N/A	N/A	0.7	1.0
6	N/A	N/A	N/A	0.7	1.0
Roof	0.9	1.6	2.3	N/A	N/A

Table 2[8]

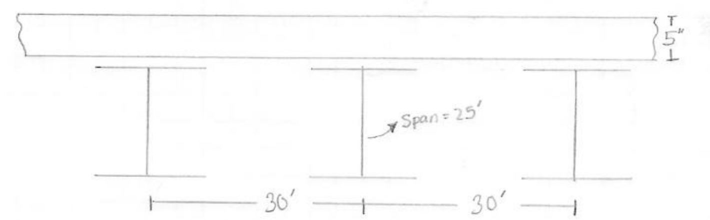
Level	Zone 1	Zone 2	Zone 3	Zone 4	Zone 5
1	N/A	N/A	N/A	16.4	23.422
2	N/A	N/A	N/A	16.4	23.422
3	N/A	N/A	N/A	16.4	23.422
4	N/A	N/A	N/A	16.4	23.422
5	N/A	N/A	N/A	16.4	23.422
6	N/A	N/A	N/A	16.4	23.422
Roof	21.08	37.48	53.87	N/A	N/A

[7] See table 6-17 from ASIC specs book

[8] All units in lb/ft^2

Appendix E
Sample Beam Calculation

SAMPLE BEAM CALCULATION



CONCRETE WEIGHT = 150 lb/ft³ HARDWOOD FLOOR = 4 lb/sq-ft

ASSUME BEAM WT = 50 lb/ft

SLAB WT $= \left(\frac{5}{12}\right)(150)(30) = 1925$ lb/ft

$W_D = 1975$ lb/ft

$W_L = 30(100) = 3{,}000$ lb/ft

$W_U = 1.2(1975) + 1.6(3000)$ → Use governing load equation from spreadsheet Created

$W_U = 7170$ lb/ft

$= 7.170$ k/ft

$M_U = \frac{(7.17\ \text{k/ft})(25')^2}{8} = 560.2$ ft-k

TRY W24×62 $\Rightarrow \phi_b M_{px} = 574 > 560.2$

BEAM WT = 62 lb/ft

$W_D = 1925 + 62 = 1987$ lb/ft

$W_U = 1.2(1987) + 1.6(3000)$

$W_U = 7.184$ k/ft

$\frac{(7.184)\times(25^2)}{8} = 561.3$ ft-k

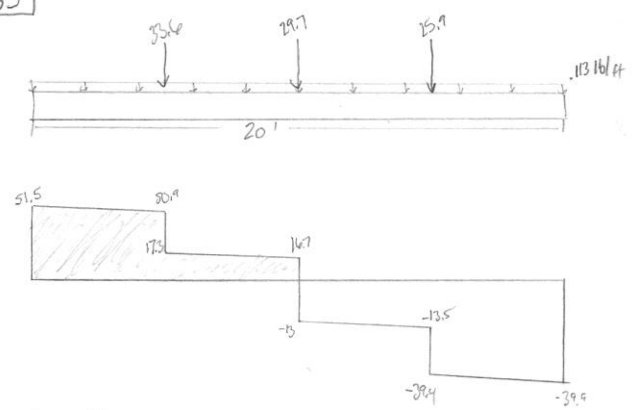
☑ $574_{\text{ft-k}} > 561.3$ ft-k

$\boxed{\text{USE W24x62}}$

Appendix F

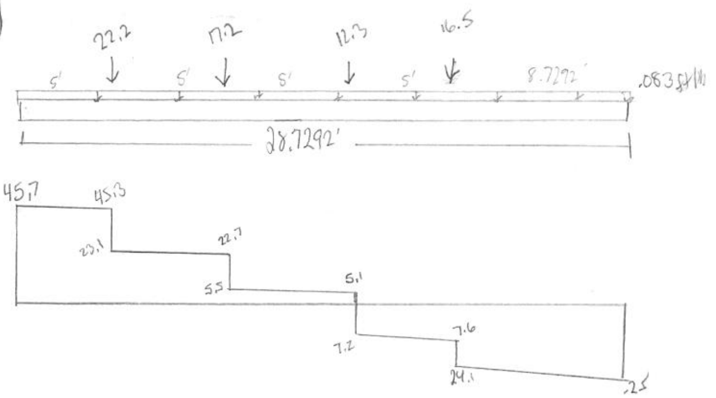
Sample Girder Calculation

-1G57 - -1G65



Max Moment = (.56×5)½×2 = 2.8
(50.9×5) = 254.5
(16.7×5) = 83.5
= 340.8 ft·kips

-1G66-74



Max Moment
(.42×5×½×3) = 3.15
(45.3×5) = 226.5
(22.7×5) = 113.5
(5.1×5) = 25.5
= 368.7 ft·kips

Appendix G

Sample Column Calculation

Sample Column - (Girder 3G30, 3G34, 3G33)

Top Section

$P_u = 0.657^k + 0.657^k + 2(13.14)^k(5)$

$P_u = 132.714$

Assume $\frac{KL}{r} = 50$ $\quad \phi_c F_{cr} = 37.5$ ksi

$A_{required} = \frac{132.714}{37.5} = 3.53 \text{ in}^2$

Try W 8 x 35 $\quad (A = 10.3 \text{ in}^2 \quad r_x = 3.51 \text{ in} \quad r_y = 2.03 \text{ in})$

$\left(\frac{KL}{r}\right)_y = \frac{12(20)}{2.03} = 118.22 \quad \phi_c F_{cr} = 16.1$ ksi

$\phi_c P_n = \phi_c F_{cr}(\text{Area}) = 16.1(10.3) = 165.83 > 132.714$ ok

Bottom Section (Above ℄ 2G30, 2G33, 2G34)

$P_u = 132.714^k + 0.727^k + 0.727^k + 2(14.275^k)5 = 491.043^k$

Assume $\phi_c F_{cr} = 37.5$ ksi

$A \text{ req'd} = \frac{491.043}{37.5} = 13.09 \text{ in}^2$

Try W 8 X 58 $\quad (A = 17.1 \text{ in}^2 \quad r_x = 3.65 \text{ in} \quad r_y = 2.10 \text{ in})$

$\left(\frac{KL}{r}\right)_y = \frac{12(10)}{2.1} = 57.14 \quad \phi_c F_{cr} = 35.4$ ksi

$\phi_c P_n = 35.4(17.1) = 605.34 > 491.043$

Sample Column (Con't) girder 1G30, 1G33, 1G34

$P_u = 491.043^k + 2(14.275^k)5 + 2(2.855)$

$P_u = 639.5^k$

Assume $\phi_c F_{cr} = 37.5$ ksi

A req'd $= \frac{639.5}{37.5} = 17.05 \text{ in}^2$

Try W 18 × 130 ($A = 38.2 \text{ in}^2$ $r_x = 7.9$ in $r_y = 2.69$ in)

$\left(\frac{KL}{r}\right)_y = \frac{12(22)}{2.69} = 98.14$

$\phi_c F_{cr} = 19.3$

$\phi_c P_n = 737^k > 639.5^k$ O.K

Sample Column (con't) girder

$P_u = 639.5 + 2(14.275)5 + 2(14.275)5 + 2(2.855) + 2(2.855) = 936.42$

Assume $\phi_c F_{cr} = 37.5$ ksi A req'd $= \frac{936.42}{37.5} = 24.97 \text{ in}^2$

Try W24 × 176 ($A = 51.7 \text{ in}^2$ $r_x = 10.5$ in $r_y = 3.04$ in)

$\left(\frac{KL}{r}\right)_y = \frac{12(15)}{3.04} = 59.21$

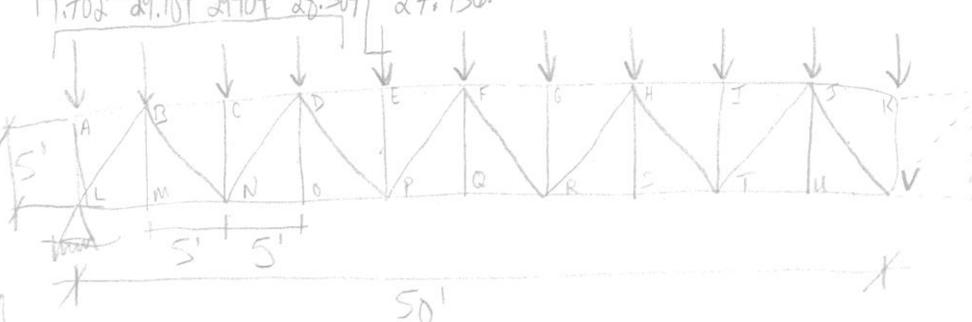
$\phi_c F_{cr} = 35$

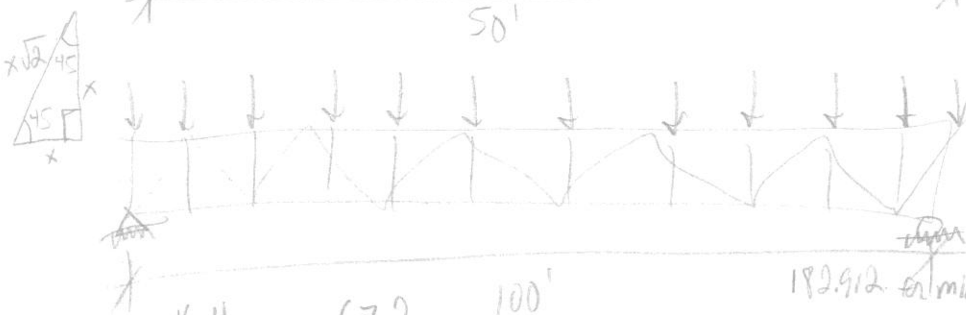
$\phi_c P_n = 35(51.7) = 1809.5 > 936.42$

should look for a lighter beam.

Appendix H

Roof Truss Design using Method of Joints

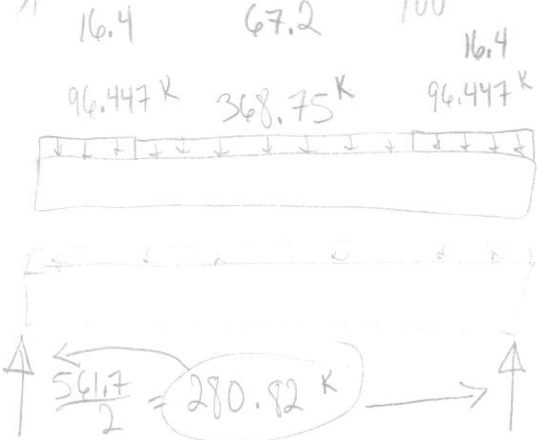




182.912 @ middle.

$196.032 \times 30 = p/f$

5881 p/f



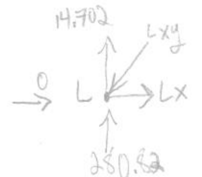
members = 41
joints = 22

$\frac{561.7}{2} = 280.82^K$

Joint Method

$\Sigma F_x = 0 = A_x$

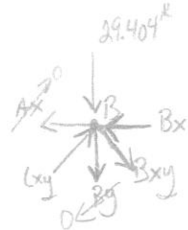
$\Sigma F_y = 0 = A_y - 14.702$ $\boxed{A_y = 14.702^K}$



$\Sigma F_y = 0 = 14.702 + 280.82 - L_{xy}\left(\frac{5}{5\sqrt{2}}\right)$

$\boxed{L_{xy} = 417.93^K}$

$\Sigma F_x = 0 = 0 - 417.93\left(\frac{5}{5\sqrt{2}}\right) + L_x$ $\boxed{L_x = 295.52^K}$



$\Sigma F_y = 0 = 295.52 - 29.404 - B_{xy}(5/5\sqrt{2})$

$\boxed{B_{xy} = 376.34^K}$

$\Sigma F_x = 0 = 295.52 + 376.34(5/5\sqrt{2}) - 0 - B_x$

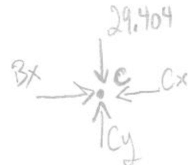
$\boxed{B_x = 561.63^K}$

266.11

$\Sigma F_x = 0 = M_x - 295.52$

$\boxed{M_x = 295.52^K}$

$\Sigma F_y = \boxed{0 = B_y}$



$\Sigma F_x = 0 = 561.63 - C_x$ $\boxed{C_x = 561.63^K}$

$\Sigma F_y = 0 = C_y - 29.404$ $\boxed{C_y = 29.404^K}$

$\Sigma F_y = 0 = 266.11 - 29.404 - N_{xy}(5/5\sqrt{2})$

$\boxed{N_{xy} = 334.75^K}$

$\Sigma F_x = 0 = -295.52 - 266.11 - 236.7 + N_x$

$\boxed{N_x = 798.33^K}$

$\Sigma F_y = \boxed{0 = O_y}$

$\Sigma F_x = 0 = O_x - 798.33$ $\boxed{O_x = 798.33^K}$

$\Sigma F_y = 0 = 236.7 - 28.504 - D_{xy}(5/5\sqrt{2})$

$\boxed{D_{xy} = 294.43^K}$

$\Sigma F_x = 0 = 561.63 + 236.7 + 208.19 - D_x$

$\boxed{D_x = 1006.52^K}$

$\Sigma F_x = 0 = 1006.52^K - E_x$ $\boxed{E_x = 1006.52^K}$

$\Sigma F_y = 0 = E_y - 27.436$ $\boxed{E_y = 27.436^K}$

$\Sigma F_y = 0 = \underline{208.19} - 27.436 - P_{xy}(5/5\sqrt{2})$

$\boxed{P_{xy} = 255.62^K}$

$\Sigma F_x = 0 = -\underline{798.33} - \underline{208.19} - 180.75 + P_x$

$\boxed{P_x = 1187.27^K}$

$\Sigma F_y = \boxed{0 = Q_y}$

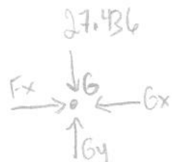
$\Sigma F_x = 0 = Q_x - 1187.27$ $\boxed{Q_x = 1187.27^K}$

$\Sigma F_y = 0 = \underline{180.75} - 27.436 - F_{xy}(5/5\sqrt{2})$

$\boxed{F_{xy} = 216.82^K}$

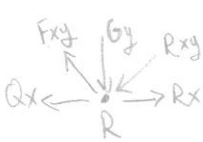
$\Sigma F_x = 0 = \underline{1006.52} + \underline{180.75} + 153.31 - F_x$

$\boxed{F_x = 1340.58^K}$



$\Sigma F_x = 0 = 1340.58 - G_x$ $\boxed{G_x = 1340.58^K}$

$\Sigma F_y = 0 = G_y = 27.436$ $\boxed{G_y = 27.436^K}$



$\Sigma F_y = 0 = \underline{153.31} - \underline{27.436} - R_{xy}(5/5\sqrt{2})$

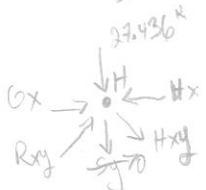
$\boxed{R_{xy} = 178.01^K}$

$\Sigma F_x = 0 = -\underline{1187.27} - \underline{153.31} - 125.87 + R_x$

$\boxed{R_x = 1466.45^K}$

$\Sigma F_y = \boxed{0 = S_y}$

$\Sigma F_x = 0 = S_x - 1466.45 \quad \boxed{S_x = 1466.45^K}$

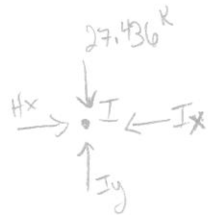


$\Sigma F_y = 0 = 125.87 - 27.436 - H_{xy}\left(5/5\sqrt{2}\right)$

$\boxed{H_{xy} = 139.2^K}$

$\Sigma F_x = 0 = 1340.58 + 125.87 + 98.43 - H_x$

$\boxed{H_x = 1564.88^K}$



$\Sigma F_x = 0 = 1564.88 - I_x \quad \boxed{I_x = 1564.88^K}$

$\Sigma F_y = 0 = I_y - 27.436 \quad \boxed{I_y = 27.436^K}$

$\Sigma F_y = 0 = 98.43 - 27.436 - T_{xy}\left(5/5\sqrt{2}\right)$

$\boxed{T_{xy} = 100.4^K}$

$\Sigma F_x = 0 = -1466.45 - 98.43 - 70.99 + T_x$

$\boxed{T_x = 1635.87^K}$

$\Sigma F_y = \boxed{0 = U_y}$

$\Sigma F_x = 0 = U_x - 1635.87 \quad \boxed{U_x = 1635.87^K}$

$\Sigma F_y = 0 = 70.99 - 27.436 - J_{xy}\left(5/5\sqrt{2}\right)$

$\boxed{J_{xy} = 61.59^K}$

$\Sigma F_x = 0 = 1564.88 + 70.99 + 43.55 - J_x$

$\boxed{J_x = 1679.42^K}$

$\Sigma F_x = 0 = 1679.42 - K_x$ $\boxed{K_x = 1679.42^K}$

$\Sigma F_y = 0 = K_y - 27.436$ $\boxed{K_y = 27.436^K}$

$\Sigma F_y = 0 = 43.55 - 27.436 - V_{xy}\left(5/5\sqrt{2}\right)$

$\boxed{V_{xy} = 22.79^K}$

$\Sigma F_x = 0 = -1635.87 - 43.55 - 16.11 + V_x$

$\boxed{V_x = 1695.33^K}$

Appendix I

Roof & Pool Truss Design Data

ROOF TRUSS

JOINT REACTIONS			
LABEL	X(K)	Y(K)	MZ(FT-K)
M	0	291.188	0
W	0	291.188	0
Totals:	0	582.377	
COG (ft):	X: 50	Y: 7.83	

JOINT DEFLECTIONS		
LABEL	X(IN)	Y(IN)
A	1.022	-0.004
B	1.022	-0.764
C	0.967	-2.181
D	0.87	-3.351
E	0.741	-4.181
F	0.595	-4.612
G	0.442	-4.612
H	0.295	-4.181
I	0.167	-3.351
J	0.069	-2.181
K	0.015	-0.764
L	0.015	-0.004
M	0	0
N	0.03	-1.49
O	0.109	-2.797
P	0.225	-3.806
Q	0.365	-4.443
R	0.518	-4.66
S	0.671	-4.443
T	0.812	-3.806
U	0.928	-2.797
V	1.007	-1.49
W	1.036	0

MEMBER FORCES			
LABEL	SEC	AXIAL(K)	SHEAR(K)
M1	1	0	0.205
	2	0	-0.205
M2	1	316.942	0.41
	2	316.942	-0.41
M3	1	568.612	0.41
	2	568.612	-0.41
M4	1	747.594	0.41
	2	747.594	-0.41
M5	1	854.983	0.41
	2	854.983	-0.41
M6	1	890.78	0.41
	2	890.78	-0.41
M7	1	854.983	0.41
	2	854.983	-0.41
M8	1	747.594	0.41
	2	747.594	-0.41
M9	1	568.612	0.41
	2	568.612	-0.41
M10	1	316.942	0.41
	2	316.942	-0.41
M11	1	0	0.205
	2	0	-0.205
M12	1	-172.285	0.41
	2	-172.285	-0.41
M13	1	-460.848	0.41
	2	-460.848	-0.41
M14	1	-675.626	0.41
	2	-675.626	-0.41
M15	1	-818.811	0.41
	2	-818.811	-0.41
M16	1	-890.404	0.41
	2	-890.404	-0.41
M17	1	-890.404	0.41
	2	-890.404	-0.41
M18	1	-818.811	0.41
	2	-818.811	-0.41
M19	1	-675.626	0.41
	2	-675.626	-0.41
M20	1	-460.848	0.41
	2	-460.848	-0.41
M21	1	-172.285	0.41
	2	-172.285	-0.41
M22	1	14.608	0
	2	14.931	0

MEMBER DEFLECTIONS			
LABEL	SEC	X(IN)	Y(IN)
M1	1	1.022	-0.004
	2	1.022	-0.764
M2	1	1.022	-0.764
	2	0.967	-2.181
M3	1	0.967	-2.181
	2	0.87	-3.351
M4	1	0.87	-3.351
	2	0.741	-4.181
M5	1	0.741	-4.181
	2	0.595	-4.612
M6	1	0.595	-4.612
	2	0.442	-4.612
M7	1	0.442	-4.612
	2	0.295	-4.181
M8	1	0.295	-4.181
	2	0.167	-3.351
M9	1	0.167	-3.351
	2	0.069	-2.181
M10	1	0.069	-2.181
	2	0.015	-0.764
M11	1	0.015	-0.764
	2	0.015	-0.004
M12	1	0	0
	2	0.03	-1.49
M13	1	0.03	-1.49
	2	0.109	-2.797
M14	1	0.109	-2.797
	2	0.225	-3.806
M15	1	0.225	-3.806
	2	0.365	-4.443
M16	1	0.365	-4.443
	2	0.518	-4.66
M17	1	0.518	-4.66
	2	0.671	-4.443
M18	1	0.671	-4.443
	2	0.812	-3.806
M19	1	0.812	-3.806
	2	0.928	-2.797
M20	1	0.928	-2.797
	2	1.007	-1.49
M21	1	1.007	-1.49
	2	1.036	0
M22	1	0.004	1.022
	2	0	0

POOL TRUSS

JOINT REACTIONS			
LABEL	X(K)	Y(K)	MZ(FT-K)
M	0	161.606	0
W	0	161.606	0
Totals:	0	323.213	
COG (ft):	X: 50	Y: 5.785	

JOINT DEFLECTIONS		
LABEL	X(IN)	Y(IN)
A	0.763	-0.002
B	0.763	-0.706
C	0.722	-2.025
D	0.649	-3.123
E	0.553	-3.906
F	0.444	-4.313
G	0.329	-4.313
H	0.22	-3.906
I	0.124	-3.123
J	0.051	-2.025
K	0.011	-0.706
L	0.011	-0.002
M	0	0
N	0.022	-1.383
O	0.081	-2.605
P	0.167	-3.555
Q	0.272	-4.155
R	0.387	-4.361
S	0.501	-4.155
T	0.606	-3.555
U	0.692	-2.605
V	0.751	-1.383
W	0.773	0

M23	1	325.228	0.101
	2	324.905	-0.101
M24	1	-273.1	0.101
	2	-272.776	-0.101
M25	1	271.683	0.101
	2	271.359	-0.101
M26	1	-203.492	0.101
	2	-203.168	-0.101
M27	1	202.074	0.101
	2	201.751	-0.101
M28	1	-135.952	0.101
	2	-135.628	-0.101
M29	1	134.534	0.101
	2	134.21	-0.101
M30	1	-68.411	0.101
	2	-68.087	-0.101
M31	1	66.994	0.101
	2	66.67	-0.101
M32	1	-0.871	0.101
	2	-0.547	-0.101
M33	1	-0.547	0.101
	2	-0.871	-0.101
M34	1	66.67	0.101
	2	66.994	-0.101
M35	1	-68.087	0.101
	2	-68.411	-0.101
M36	1	134.21	0.101
	2	134.534	-0.101
M37	1	-135.628	0.101
	2	-135.952	-0.101
M38	1	201.751	0.101
	2	202.074	-0.101
M39	1	-203.168	0.101
	2	-203.492	-0.101
M40	1	271.359	0.101
	2	271.683	-0.101
M41	1	-272.776	0.101
	2	-273.1	-0.101
M42	1	324.905	0.101
	2	325.228	-0.101
M43	1	14.608	0
	2	14.931	0

M23	1	0	0
	2	-0.107	-1.272
M24	1	1.19	0.461
	2	1.279	-0.765
M25	1	-1.248	-0.815
	2	-1.337	-1.977
M26	1	2.363	-0.336
	2	2.429	-1.39
M27	1	-2.314	-1.574
	2	-2.38	-2.513
M28	1	3.302	-1.038
	2	3.347	-1.827
M29	1	-3.109	-2.208
	2	-3.153	-2.845
M30	1	3.939	-1.587
	2	3.961	-2.045
M31	1	-3.574	-2.664
	2	-3.596	-2.949
M32	1	4.226	-1.94
	2	4.226	-2.03
M33	1	-3.677	-2.909
	2	-3.677	-2.819
M34	1	4.145	-2.07
	2	4.123	-1.786
M35	1	-3.412	-2.924
	2	-3.389	-2.466
M36	1	3.702	-1.966
	2	3.658	-1.329
M37	1	-2.798	-2.706
	2	-2.753	-1.917
M38	1	2.93	-1.635
	2	2.863	-0.696
M39	1	-1.88	-2.269
	2	-1.813	-1.215
M40	1	1.886	-1.098
	2	1.797	0.064
M41	1	-0.73	-1.644
	2	-0.641	-0.417
M42	1	0.656	-0.393
	2	0.549	0.879
M43	1	0.004	0.015
	2	0	1.036

MEMBER FORCES			
LABEL	SEC	AXIAL(K)	SHEAR(K)
M1	1	0	0.205
	2	0	-0.205
M2	1	235.624	0.41
	2	235.624	-0.41
M3	1	423.881	0.41
	2	423.881	-0.41
M4	1	558.351	0.41
	2	558.351	-0.41
M5	1	639.032	0.41
	2	639.032	-0.41
M6	1	665.926	0.41
	2	665.926	-0.41
M7	1	639.032	0.41
	2	639.032	-0.41
M8	1	558.351	0.41
	2	558.351	-0.41
M9	1	423.881	0.41
	2	423.881	-0.41
M10	1	235.624	0.41
	2	235.624	-0.41
M11	1	0	0.205
	2	0	-0.205
M12	1	-127.575	0.41
	2	-127.575	-0.41
M13	1	-342.726	0.41
	2	-342.726	-0.41
M14	1	-504.089	0.41
	2	-504.089	-0.41
M15	1	-611.665	0.41
	2	-611.665	-0.41
M16	1	-665.453	0.41
	2	-665.453	-0.41
M17	1	-665.453	0.41
	2	-665.453	-0.41
M18	1	-611.665	0.41
	2	-611.665	-0.41
M19	1	-504.089	0.41
	2	-504.089	-0.41
M20	1	-342.726	0.41
	2	-342.726	-0.41
M21	1	-127.575	0.41
	2	-127.575	-0.41
M22	1	7.705	0
	2	7.948	0

MEMBER DEFLECTIONS			
LABEL	SEC	X(IN)	Y(IN)
M1	1	0.763	-0.002
	2	0.763	-0.706
M2	1	0.763	-0.706
	2	0.722	-2.025
M3	1	0.722	-2.025
	2	0.649	-3.123
M4	1	0.649	-3.123
	2	0.553	-3.906
M5	1	0.553	-3.906
	2	0.444	-4.313
M6	1	0.444	-4.313
	2	0.329	-4.313
M7	1	0.329	-4.313
	2	0.22	-3.906
M8	1	0.22	-3.906
	2	0.124	-3.123
M9	1	0.124	-3.123
	2	0.051	-2.025
M10	1	0.051	-2.025
	2	0.011	-0.706
M11	1	0.011	-0.706
	2	0.011	-0.002
M12	1	0	0
	2	0.022	-1.383
M13	1	0.022	-1.383
	2	0.081	-2.605
M14	1	0.081	-2.605
	2	0.167	-3.555
M15	1	0.167	-3.555
	2	0.272	-4.155
M16	1	0.272	-4.155
	2	0.387	-4.361
M17	1	0.387	-4.361
	2	0.501	-4.155
M18	1	0.501	-4.155
	2	0.606	-3.555
M19	1	0.606	-3.555
	2	0.692	-2.605
M20	1	0.692	-2.605
	2	0.751	-1.383
M21	1	0.751	-1.383
	2	0.773	0
M22	1	0.002	0.763
	2	0	0

M23	1	199.4	0.101
	2	199.157	-0.101
M24	1	-168.9	0.101
	2	-168.66	-0.101
M25	1	167.42	0.101
	2	167.177	-0.101
M26	1	-126.89	0.101
	2	-126.65	-0.101
M27	1	125.411	0.101
	2	125.168	-0.101
M28	1	-84.88	0.101
	2	-84.637	-0.101
M29	1	83.401	0.101
	2	83.158	-0.101
M30	1	-42.871	0.101
	2	-42.628	-0.101
M31	1	41.391	0.101
	2	41.149	-0.101
M32	1	-0.861	0.101
	2	-0.618	-0.101
M33	1	-0.618	0.101
	2	-0.861	-0.101
M34	1	41.149	0.101
	2	41.391	-0.101
M35	1	-42.628	0.101
	2	-42.871	-0.101
M36	1	83.158	0.101
	2	83.401	-0.101
M37	1	-84.637	0.101
	2	-84.88	-0.101
M38	1	125.168	0.101
	2	125.411	-0.101
M39	1	-126.65	0.101
	2	-126.89	-0.101
M40	1	167.177	0.101
	2	167.42	-0.101
M41	1	-168.66	0.101
	2	-168.9	-0.101
M42	1	199.157	0.101
	2	199.4	-0.101
M43	1	7.705	0
	2	7.948	0

M23	1	0	0
	2	-0.054	-1.038
M24	1	1.03	0.134
	2	1.076	-0.868
M25	1	-1.048	-0.902
	2	-1.094	-1.851
M26	1	2.018	-0.742
	2	2.053	-1.605
M27	1	-1.949	-1.73
	2	-1.983	-2.498
M28	1	2.815	-1.5
	2	2.838	-2.147
M29	1	-2.624	-2.404
	2	-2.646	-2.926
M30	1	3.355	-2.075
	2	3.366	-2.451
M31	1	-3.018	-2.869
	2	-3.029	-3.102
M32	1	3.597	-2.42
	2	3.597	-2.495
M33	1	-3.102	-3.089
	2	-3.102	-3.014
M34	1	3.524	-2.508
	2	3.513	-2.275
M35	1	-2.871	-3.045
	2	-2.86	-2.669
M36	1	3.141	-2.332
	2	3.119	-1.81
M37	1	-2.343	-2.741
	2	-2.32	-2.094
M38	1	2.478	-1.904
	2	2.444	-1.136
M39	1	-1.558	-2.199
	2	-1.523	-1.336
M40	1	1.589	-1.257
	2	1.543	-0.308
M41	1	-0.581	-1.462
	2	-0.536	-0.46
M42	1	0.549	-0.444
	2	0.495	0.594
M43	1	0.002	0.011
	2	0	0.773

Appendix J

Sample Butt Joint Bolt Connection Design

Bolt Connection Sample Calc. A36 ($F_y = 36$ksi $F_u = 58$ksi)

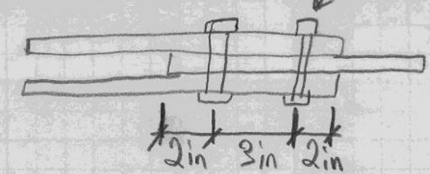
$P_u = 325.2^K$

2 plates 1/2 in thick on each side. ($A = 0.6\,in^2$)

pipe 8.63 in. thick in diam. (t)

use 7/8 in diam A325 bolts (d)

7/8 in bolts



How many are required?

Bearing strength of 1 bolt

$L_c =$ lesser of $2 - \frac{7/8 + 1/8}{2} = 1.5$ in

or $3 - 2\left(\frac{7/8 + 1/8}{2}\right) = 2$ in

$R_n = 1.2 L_c t F_u \leq 2.4 d t F_u$

$= 1.2(1.5)(8.63)(58) = 901^K < (2.4)(7/8)(8.63)(58) = 1051^K$

Shearing strength of 1 bolt

$R_n = F_n A_b = (60)(2 \times 0.6) = 72^K$

$\phi = 0.75$

$\phi R_n = 0.75(72) = 54\ K$

$\#$ of bolts req'd $= \frac{P_u}{\phi R_n} = \frac{325.2}{54} = 6.02$

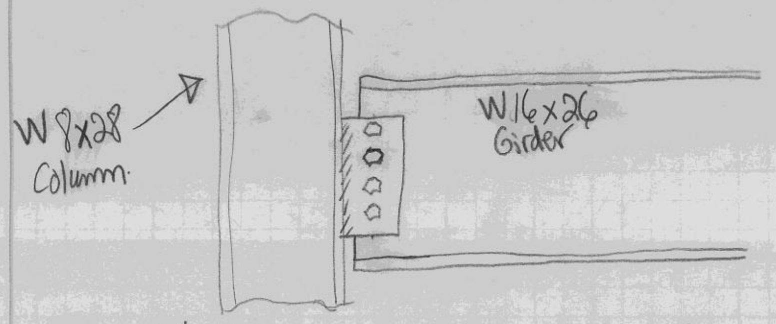
$\boxed{\text{use 7 7/8 in bearing type A325 bolts}}$

Sample Single Plate Shear Connection Design

MQP | Sample calc

W16x26
25' o.c.
30' Span

Single Plate Shear Connection



Use 3/4" A325-N high strength bolts and E70 electrodes

$F_y = 36$ ksi for plate
$F_u = 58$ ksi for plate

$F_y = 50$ ksi
$F_u = 65$ ksi
column + girder

* Assume column provides rigid support, 4 rows of bolts, 1/4" plate, and 3/16" fillet welds

$$R_D = 30'\left[\tfrac{5}{12}(150)(25) + 26\right] = \underline{47.7^K}$$

$$R_L = 50\,(30 \times 30) = \underline{45^K}$$

$$R_u = 1.2(47.7) + 1.6(45) = 129.24^K$$

$$\phi R_n = 52.2\,K < 129.24^K \quad \text{N.G.}$$

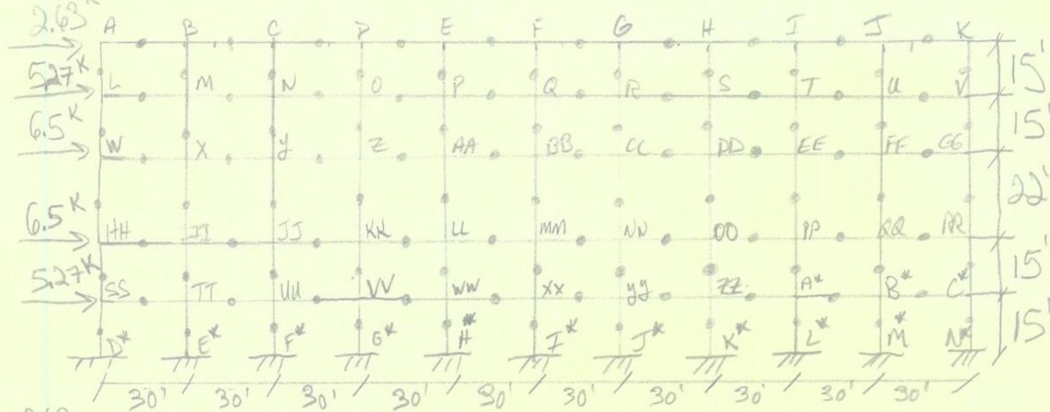
* Try 8 rows of bolts, 5/16" plate, 1/4" fillet welds, 7/8" A325-N high strength bolts and E70 Electrodes

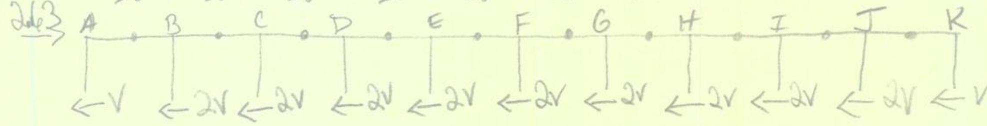
$$\boxed{\phi R_n = 131^K > 129.24^K \text{ OK}✓}$$

Appendix K

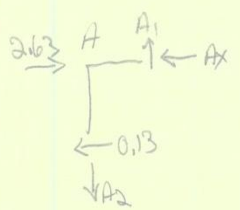
Frame Analysis using Portal Method (Long-Side)

MQP Frame Analysis (LS)





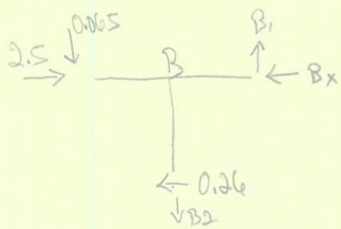
$\Sigma F_x = 0 = 2.63 - 20V \qquad V = 0.13^K$



$\Sigma F_x = 0 = 2.63 - 0.13 - A_x \qquad \underline{A_x = 2.5^K}$

$\Sigma M_A = 0 = 0.13(7.5) - A_1(15) \qquad \underline{A_1 = 0.065^K}$

$\Sigma F_y = 0 = 0.065 - A_2 \qquad \underline{A_2 = 0.065^K}$

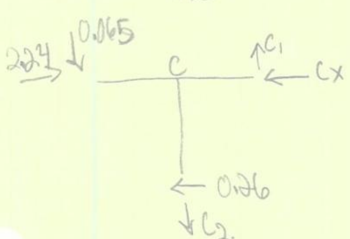


$\Sigma F_x = 0 = 2.5 - 0.26 - B_x \qquad \underline{B_x = 2.24^K}$

$\Sigma M_B = 0 = 0.26(7.5) - 0.065(15) - B_1(15)$

$\Sigma F_y = 0 = 0.065 - 0.065 - B_2 \qquad \underline{B_1 = 0.065^K}$

$\underline{B_2 = 0^K}$

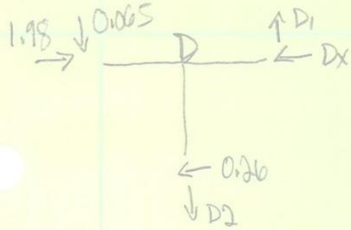


$\Sigma F_x = 0 = 2.24 - 0.26 - C_x \qquad \underline{C_x = 1.98^K}$

$\Sigma M_C = 0 = 0.26(7.5) - 0.065(15) - C_1(15)$

$\Sigma F_y = 0.065 - 0.065 - C_2 \qquad \underline{C_1 = 0.065^K}$

$\underline{C_2 = 0^K}$

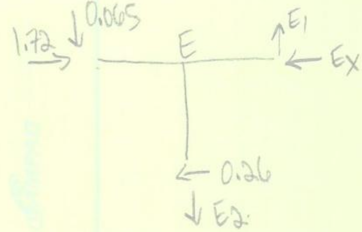


$\Sigma F_x = 0 = 1.98 - 0.26 - D_x \qquad \underline{D_x = 1.72^K}$

$\Sigma M_D = 0 = 0.26(7.5) - 0.065(15) - D_1(15)$

$\Sigma F_y = 0 = 0.065 - 0.065 - D_2 \qquad \underline{D_1 = 0.065^K}$

$\underline{D_2 = 0^K}$

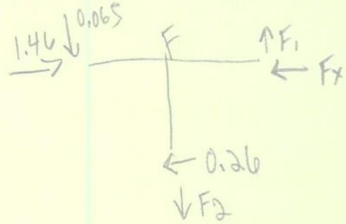


$\Sigma F_x = 0 = 1.72 - 0.26 - E_x \qquad \underline{E_x = 1.46^K}$

$\Sigma M_E = 0 = 0.26(7.5) - 0.065(15) - E_1(15)$

$\Sigma F_y = 0 = 0.065 - 0.065 - E_2 \qquad \underline{E_1 = 0.065^K}$

$\underline{E_2 = 0^K}$

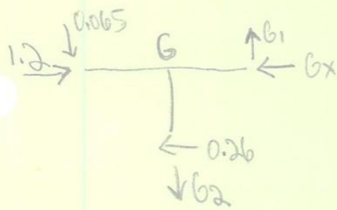


$\Sigma F_x = 0 = 1.46 - 0.26 - F_x \qquad \underline{F_x = 1.2^K}$

$\Sigma M_F = 0 = 0.26(7.5) - 0.065(15) - F_1(15)$

$\Sigma F_y = 0 = 0.065 - 0.065 - F_2 \qquad \underline{F_1 = 0.065^K}$

$\underline{F_2 = 0^K}$

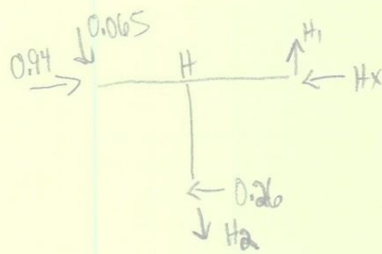


$\Sigma F_x = 0 = 1.2 - 0.26 - G_x \qquad \underline{G_x = 0.94^K}$

$\Sigma M_G = 0 = 0.26(7.5) - 0.065(15) - G_1(15)$

$\Sigma F_y = 0 = 0.065 - 0.065 - G_2 \qquad \underline{G_1 = 0.065^K}$

$\underline{G_2 = 0^K}$

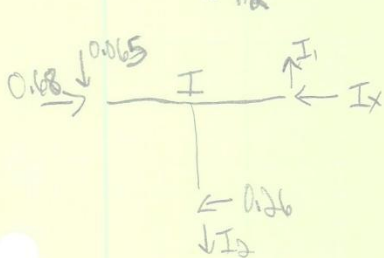


$\Sigma F_x = 0 = 0.94 - 0.26 - H_x \qquad \underline{H_x = 0.68^K}$

$\Sigma M_H = 0 = 0.26(7.5) - 0.065(15) - H_1(15)$

$\Sigma F_y = 0 = 0.065 - 0.065 - H_2 \qquad \underline{H_1 = 0.065^K}$

$\underline{H_2 = 0^K}$

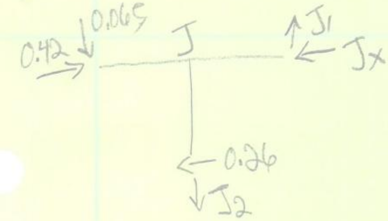


$\Sigma F_x = 0 = 0.68 - 0.26 - I_x \qquad \underline{I_x = 0.42^K}$

$\Sigma M_I = 0 = 0.26(7.5) - 0.065(15) - I_1(15)$

$\Sigma F_y = 0 = 0.065 - 0.065 - I_2 \qquad \underline{I_1 = 0.065^K}$

$\underline{I_2 = 0^K}$

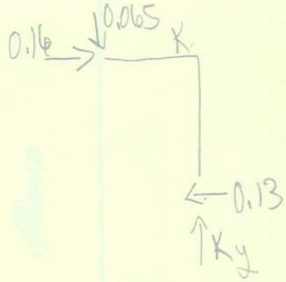


$\Sigma F_x = 0 = 0.42 - 0.26 - J_x$ $\quad \underline{J_x = 0.16^K}$

$\Sigma M_J = 0 = 0.26(7.5) - 0.065(15) - J_1(15)$

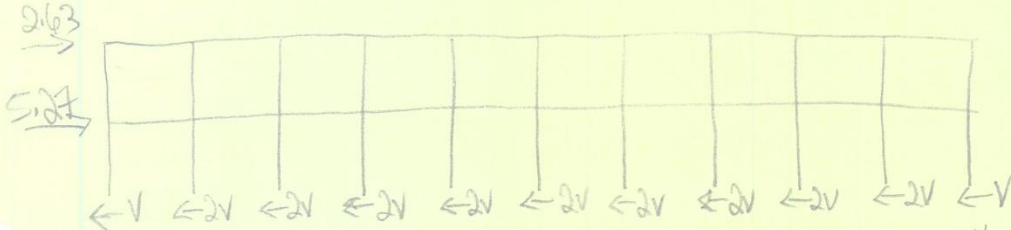
$\Sigma F_y = 0 = 0.065 - 0.065 - J_2$ $\quad \underline{J_1 = 0.065^K}$

$\underline{J_2 = 0^K}$

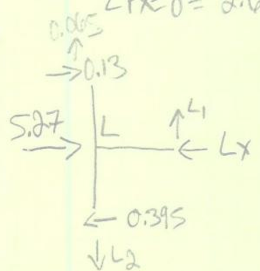


$\Sigma F_y = 0 = K_y - 0.065$

$\underline{K_y = 0.065^K}$



$\Sigma F_x = 0 = 2.63 + 5.27 - 20V$ $\quad V = 0.395^K$ $\quad 2V = 0.79^K$

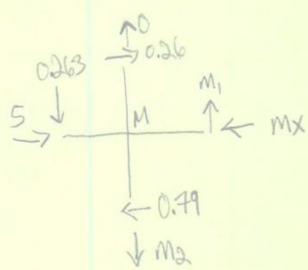


$\Sigma F_x = 0 = 0.13 - 0.395 + 5.27 - L_x$ $\quad \underline{L_x = 5^K}$

$\Sigma M_L = 0 = 0.395(7.5) + 0.13(7.5) - L_1(15)$

$\Sigma F_y = 0 = 0.065 + 0.263 - L_2$ $\quad \underline{L_1 = 0.263^K}$

$\underline{L_2 = 0.328^K}$

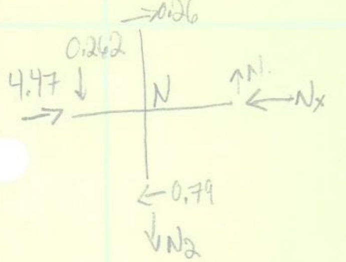


$\Sigma F_x = 0 = -M_x + 0.26 - 0.79 + 5$

$\underline{M_x = 4.47^K}$

$\Sigma M_M = 0 = 0.79(7.5) - 0.263(15) + 0.26(7.5) - M_1(15)$

$\Sigma F_y = 0 = 0.262 - 0.263 - M_2$ $\quad \underline{M_2 = 0^K}$ $\quad \underline{M_1 = 0.262^K}$



$\sum F_x = 0 = -N_x + 0.26 - 0.79 + 4.47$

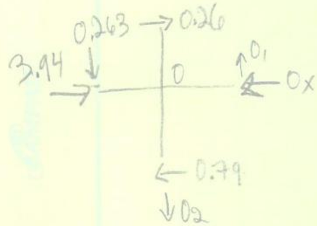
$\underline{N_x = 3.94^K}$

$\sum M_N = 0 = 0.79(7.5) - 0.262(15) + 0.26(7.5) - N_1(15)$

$\sum F_y = 0 = 0.263 - 0.262 - N_2$

$\underline{N_1 = 0.263^K}$

$\underline{N_2 = 0^K}$

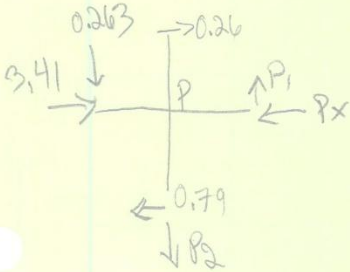


$\sum F_x = 0 = -O_x + 0.26 - 0.79 + 3.94$ $\underline{O_x = 3.41^K}$

$\sum M_O = 0 = 0.79(7.5) - 0.263(15) + 0.26(7.5) - O_1(15)$

$\sum F_y = 0 = 0.263 - 0.263 - O_2$ $\underline{O_1 = 0.263^K}$

$\underline{O_2 = 0^K}$

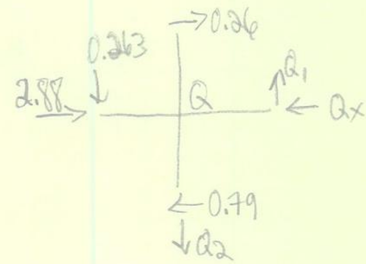


$\sum F_x = 0 = 3.41 + 0.26 - 0.79 - P_x$ $\underline{P_x = 2.88^K}$

$\sum M_P = 0 = 0.79(7.5) - 0.263(15) + 0.26(7.5) - P_1(15)$

$\sum F_y = 0 = 0.263 - 0.263 - P_2$ $\underline{P_1 = 0.263^K}$

$\underline{P_2 = 0^K}$

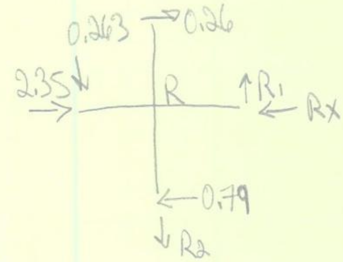


$\sum F_x = 0 = 2.88 + 0.26 - 0.79 - Q_x$ $\underline{Q_x = 2.35^K}$

$\sum M_Q = 0 = 0.79(7.5) - 0.263(15) + 0.26(7.5) - Q_1(15)$

$\sum F_y = 0 = 0.263 - 0.263 - Q_2$ $\underline{Q_1 = 0.263^K}$

$\underline{Q_2 = 0^K}$



$\sum F_x = 0 = 2.35 + 0.26 - 0.79 - R_x$

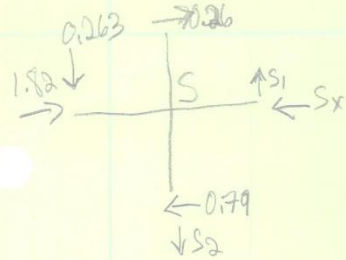
$\underline{R_x = 1.82^K}$

$\sum M_R = 0 = 0.79(7.5) - 0.263(15) + 0.26(7.5) - R_1(15)$

$\underline{R_1 = 0.263^K}$

$\sum F_y = 0 = 0.263 - 0.263 - R_2$

$\underline{R_2 = 0^K}$

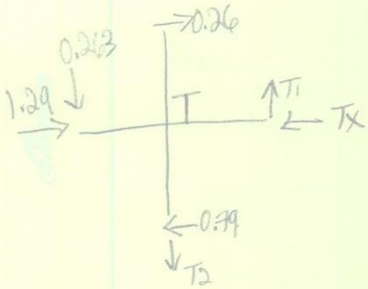


$\Sigma F_x = 0 = 1.82 + 0.26 - 0.79 - S_x$ $\quad \underline{S_x = 1.29^K}$

$\Sigma M_S = 0 = 0.79(7.5) - 0.263(15) + 0.26(7.5) - S_1(15)$

$\Sigma F_y = 0 = 0.263 - 0.263 - S_2$ $\quad \underline{S_1 = 0.263^K}$

$\underline{S_2 = 0^K}$



$\Sigma F_x = 0 = 1.29 + 0.26 - 0.79 - T_x$ $\quad \underline{T_x = 0.76^K}$

$\Sigma M_T = 0 = 0.79(7.5) - 0.263(15) + 0.26(7.5) - T_1(15)$

$\Sigma F_y = 0 = 0.263 - 0.263 - T_2$ $\quad \underline{T_1 = 0.263^K}$

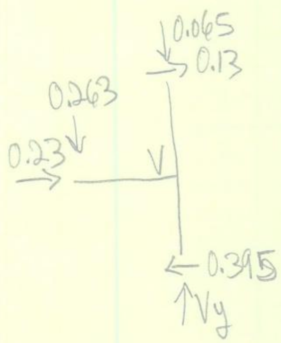
$\underline{T_2 = 0^K}$

$\Sigma F_x = 0 = 0.76 + 0.26 - 0.79 - U_x$ $\quad \underline{U_x = 0.23^K}$

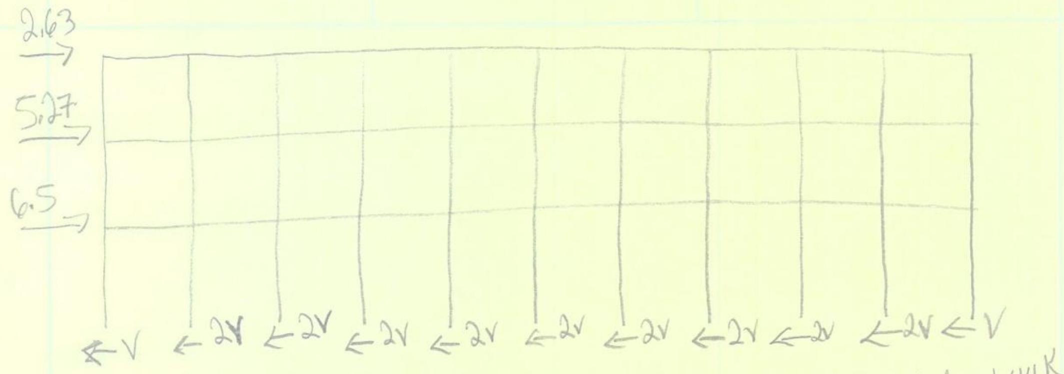
$\Sigma M_U = 0 = 0.79(7.5) - 0.263(15) + 0.26(7.5) - U_1(15)$

$\Sigma F_y = 0 = 0.263 - 0.263 - U_2$ $\quad \underline{U_1 = 0.263^K}$

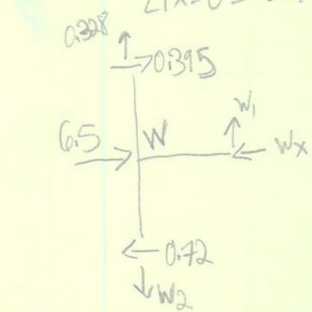
$\underline{U_2 = 0^K}$



$\Sigma F_y = 0 = V_y - 0.065 - 0.263$ $\quad \underline{V_y = 0.328^K}$



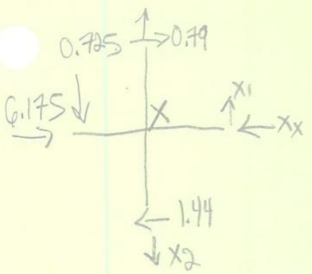
$\Sigma F_x = 0 = 2.63 + 5.27 + 6.5 - 20V$ $\quad V = 0.72^K \quad 2V = 1.44^K$



$\Sigma F_x = 0 = 6.5 + 0.395 - 0.72 - W_x$ $\quad \underline{W_x = 6.175^K}$

$\Sigma M_W = 0 = 0.72(11) + 0.395(7.5) - W_1(15)$ $\quad \underline{W_1 = 0.725^K}$

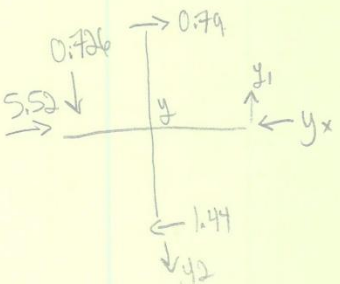
$\Sigma F_y = 0 = 0.328 + 0.725 - W_2$ $\quad \underline{W_2 = 1.05^K}$



$\Sigma F_x = 0 = 6.175 + 0.79 - 1.44 - X_x$ $\quad \underline{X_x = 5.52^K}$

$\Sigma M_X = 0 = 1.44(11) - 0.725(15) + 0.79(7.5) - X_1(15)$

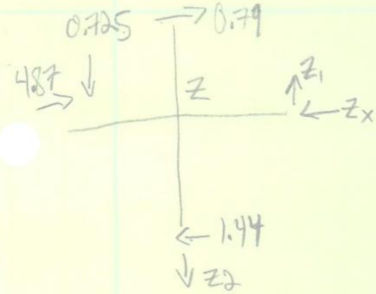
$\Sigma F_y = 0 = 0.726 - 0.725 - X_2$ $\quad \underline{X_2 = 0^K} \quad \underline{X_1 = 0.726^K}$



$\Sigma F_x = 0 = 5.52 + 0.79 - 1.44 - y_x$ $\quad \underline{y_x = 4.87^K}$

$\Sigma M_y = 0 = 1.44(11) - 0.726(15) + 0.79(7.5) - y_1(15)$

$\Sigma F_y = 0 = 0.725 - 0.726 - y_2$ $\quad \underline{y_1 = 0.725^K} \quad \underline{y_2 = 0^K}$

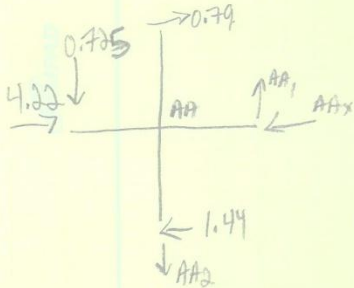


$\Sigma F_x = 0 = 4.87 + 0.79 - 1.44 - Z_x$ $\quad \underline{Z_x = 4.22^K}$

$\Sigma M_Z = 0 = 1.44(11) - 0.725(15) + 0.79(7.5) - Z_1(15)$

$\Sigma F_y = 0 = 0.725 - 0.725 - Z2$ $\quad \underline{Z_1 = 0.725^K}$

$\underline{Z2 = 0^K}$

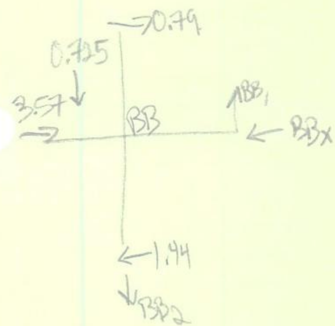


$\Sigma F_x = 0 = 4.22 + 0.79 - 1.44 - AA_x$ $\quad \underline{AA_x = 3.57^K}$

$\Sigma M_{AA} = 0 = 1.44(11) - 0.725(15) + 0.79(7.5) - AA_1(15)$

$\Sigma F_y = 0 = 0.725 - 0.725 - AA2$ $\quad \underline{AA_1 = 0.725^K}$

$\underline{AA2 = 0^K}$

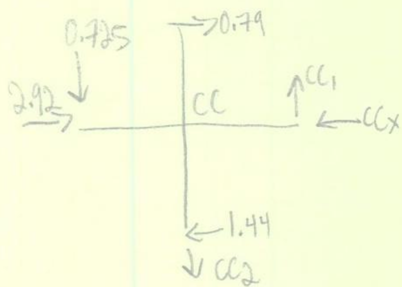


$\Sigma F_x = 0 = 3.57 + 0.79 - 1.44 - BB_x$ $\quad \underline{BB_x = 2.92^K}$

$\Sigma M_{BB} = 0 = 1.44(11) - 0.725(15) + 0.79(7.5) - BB_1(15)$

$\Sigma F_y = 0 = 0.725 - 0.725 - BB2$ $\quad \underline{BB_1 = 0.725^K}$

$\underline{BB2 = 0^K}$



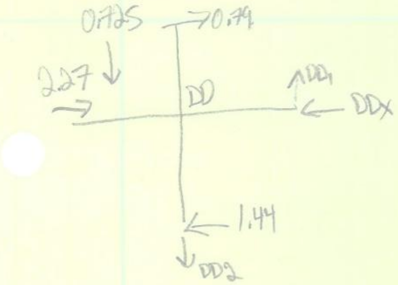
$\Sigma F_x = 0 = 2.92 + 0.79 - 1.44 - CC_x$

$\underline{CC_x = 2.27^K}$

$\Sigma M_{CC} = 0 = 1.44(11) - 0.725(15) + 0.79(7.5) - CC_1(15)$

$\Sigma F_y = 0 = 0.725 - 0.725 - CC_2$ $\quad \underline{CC_1 = 0.725^K}$

$\underline{CC2 = 0^K}$



$\Sigma F_x = 0 = 2.27 + 0.79 - 1.44 - DD_x$

$\underline{DD_x = 1.62^K}$

$\Sigma M_{DD} = 0 = 1.44(11) - 0.725(15) + 0.79(7.5) - DD_1(15)$

$\underline{DD_1 = 0.725^K}$

$\Sigma F_y = 0 = 0.725 - 0.725 - DD_2$

$\underline{DD_2 = 0^K}$

$\Sigma F_x = 0 = 1.62 + 0.79 - 1.44 - EE_x$

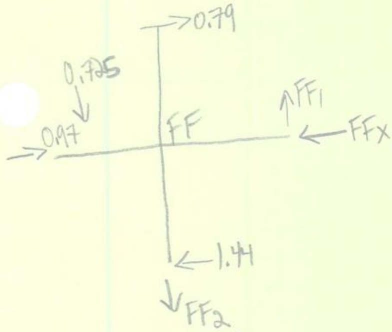
$\underline{EE_x = 0.97^K}$

$\Sigma M_{EE} = 0 = 1.44(11) - 0.725(15) + 0.79(7.5) - EE_1(15)$

$\underline{EE_1 = 0.725^K}$

$\Sigma F_y = 0 = 0.725 - 0.725 - EE_2$

$\underline{EE_2 = 0^K}$



$\Sigma F_x = 0 = 0.97 + 0.79 - 1.44 - FF_x$

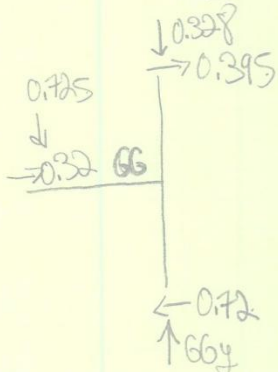
$\underline{FF_x = 0.32^K}$

$\Sigma M_{FF} = 0 = 1.44(11) - 0.725(15) + 0.79(7.5) - FF_1(15)$

$\underline{FF_1 = 0.725^K}$

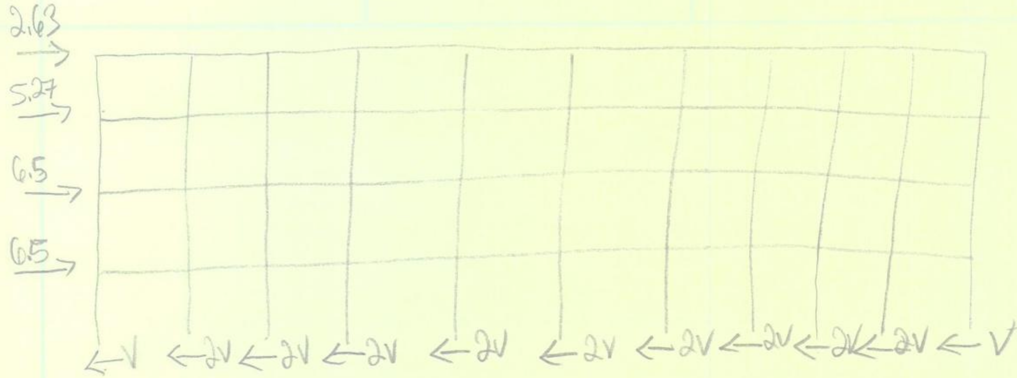
$\Sigma F_y = 0 = 0.725 - 0.725 - FF_2$

$\underline{FF_2 = 0^K}$

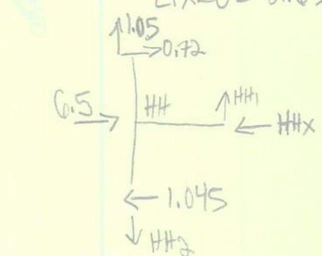


$\Sigma F_y = 0 = GG_y - 0.725 - 0.328$

$\underline{GG_y = 1.05^K}$



$\Sigma F_x = 0 = 2.63 + 5.27 + 6.5 + 6.5 - 20V$ $\quad V = 1.045^K \quad 2V = 2.09^K$



$\Sigma F_x = 0 = 6.5 + 0.72 - 1.045 - HH_x$

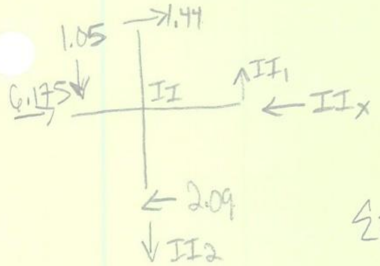
$\underline{HH_x = 6.175^K}$

$\Sigma M_{HH} = 0 = 1.045(7.5) + 0.72(11) - HH_1(15)$

$\underline{HH_1 = 1.05^K}$

$\Sigma F_y = 0 = 1.05 + 1.05 - HH_2$

$\underline{HH_2 = 2.1^K}$



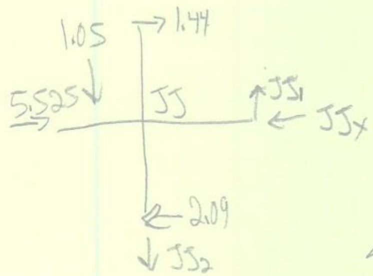
$\Sigma F_x = 0 = 6.175 + 1.44 - 2.09 - II_x$

$\underline{II_x = 5.525^K}$

$\Sigma M_{II} = 0 = 2.09(7.5) - 1.05(15) + 1.44(11) - II_1(15)$

$\Sigma F_y = 0 = 1.05 - 1.05 - II_2$

$\underline{II_2 = 0^K}$ $\quad \underline{II_1 = 1.05^K}$



$\Sigma F_x = 0 = 5.525 + 1.44 - 2.09 - JJ_x$

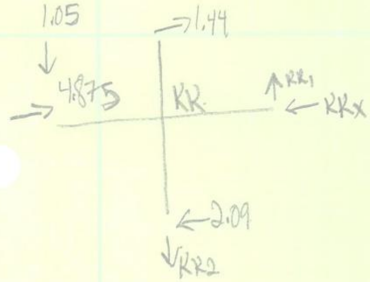
$\underline{JJ_x = 4.875^K}$

$\Sigma M_{JJ} = 0 = 2.09(7.5) - 1.05(15) + 1.44(11) - JJ_1(15)$

$\underline{JJ_1 = 1.05^K}$

$\Sigma F_y = 0 = 1.05 - 1.05 - JJ_2$

$\underline{JJ_2 = 0^K}$



$\Sigma F_x = 0 = 4.875 + 1.44 - 2.09 - KK_x$

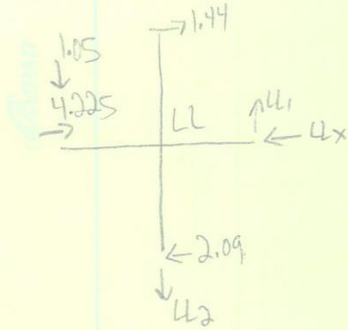
$\underline{KK_x = 4.225^K}$

$\Sigma M_{KK} = 0 = 2.09(7.5) - 1.05(15) + 1.44(11) - KK_1(15)$

$\underline{KK_1 = 1.05^K}$

$\Sigma F_y = 0 = 1.05 - 1.05 - KK_2$

$\underline{KK_2 = 0^K}$



$\Sigma F_x = 0 = 4.225 + 1.44 - 2.09 - LL_x$

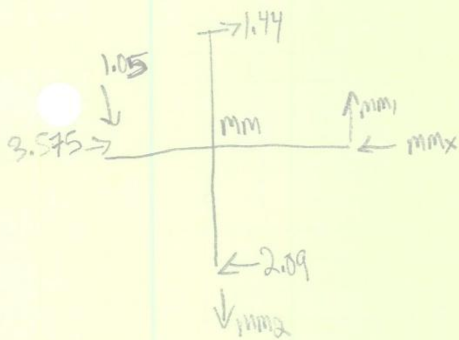
$\underline{LL_x = 3.575^K}$

$\Sigma M_{LL} = 0 = 2.09(7.5) - 1.05(15) + 1.44(11) - LL_1(15)$

$\underline{LL_1 = 1.05^K}$

$\Sigma F_y = 0 = 1.05 - 1.05 - LL_2$

$\underline{LL_2 = 0^K}$



$\Sigma F_x = 0 = 3.575 + 1.44 - 2.09 - mm_x$

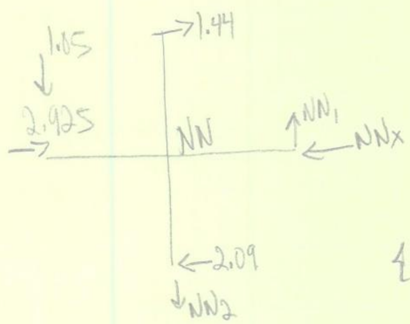
$\underline{mm_x = 2.925^K}$

$\Sigma M_{mm} = 0 = 2.09(7.5) - 1.05(15) + 1.44(11) - mm_1(15)$

$\underline{mm_1 = 1.05^K}$

$\Sigma F_y = 0 = 1.05 - 1.05 - mm_2$

$\underline{mm_2 = 0^K}$



$\Sigma F_x = 0 = 2.925 + 1.44 - 2.09 - NN_x$

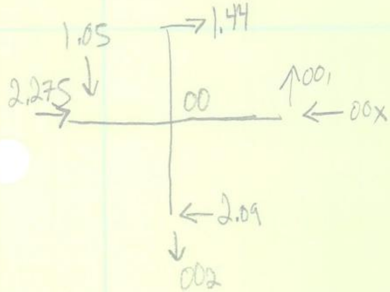
$\underline{NN_x = 2.275^K}$

$\Sigma M_{NN} = 0 = 2.09(7.5) - 1.05(15) + 1.44(11) - NN_1(15)$

$\underline{NN_1 = 1.05^K}$

$\Sigma F_y = 0 = 1.05 - 1.05 - NN_2$

$\underline{NN_2 = 0^K}$



$\Sigma F_x = 0 = 2.275 + 1.44 - 2.09 - OO_x$

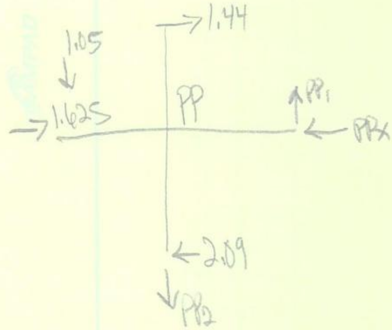
$\underline{OO_x = 1.625^K}$

$\Sigma M_{OO} = 0 = 2.09(7.5) - 1.05(15) + 1.44(11) - OO_1(15)$

$\underline{OO_1 = 1.05^K}$

$\Sigma F_y = 0 = 1.05 - 1.05 - OO_2$

$\underline{OO_2 = 0^K}$



$\Sigma F_x = 0 = 1.625 + 1.44 - 2.09 - PP_x$

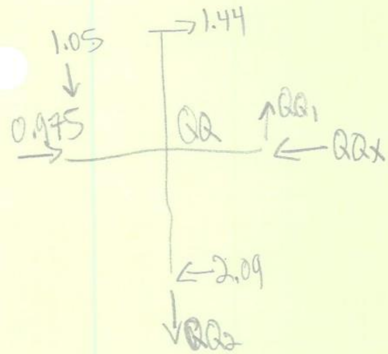
$\underline{PP_x = 0.975^K}$

$\Sigma M_{PP} = 0 = 2.09(7.5) - 1.05(15) + 1.44(11) - PP_1(15)$

$\Sigma F_y = 0 = 1.05^K - 1.05^K - PP_2$

$\underline{PP_1 = 1.05^K}$

$\underline{PP_2 = 0^K}$



$\Sigma F_x = 0 = 0.975 + 1.44 - 2.09 - QQ_x$

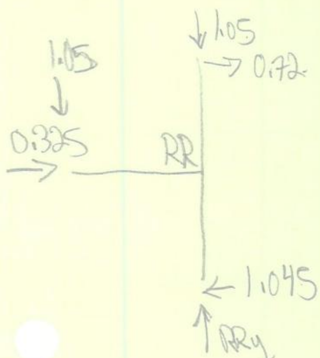
$\underline{QQ_x = 0.325^K}$

$\Sigma M_{QQ} = 0 = 2.09(7.5) - 1.05(15) + 1.44(11) - QQ_1(15)$

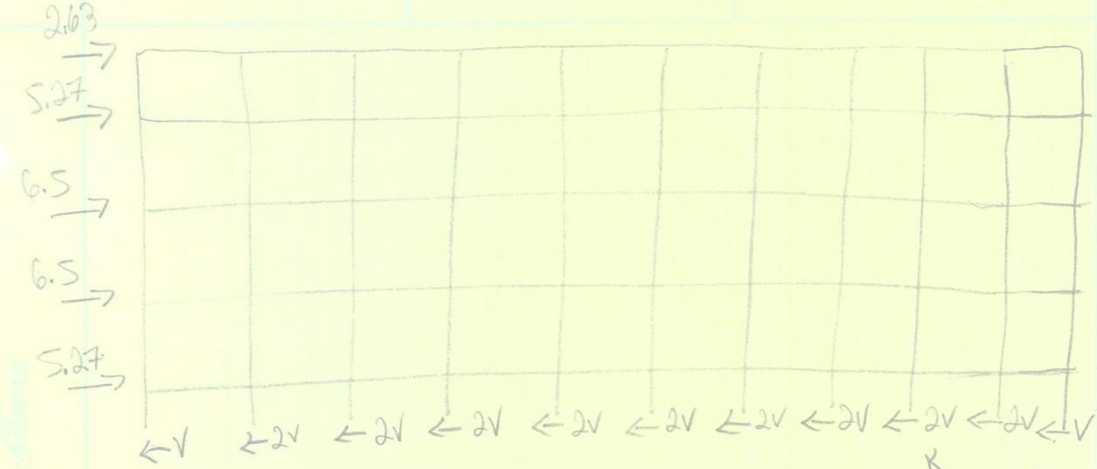
$\underline{QQ_1 = 1.05^K}$

$\Sigma F_y = 0 = 1.05 - 1.05 - QQ_2$

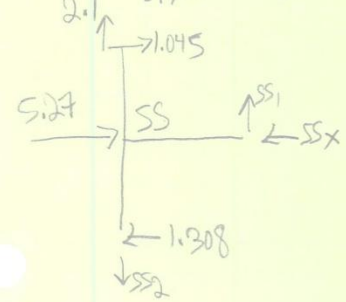
$\underline{QQ_2 = 0^K}$



$\Sigma F_y = 0 = RR_y - 1.05 - 1.05$ $\quad \underline{RR_y = 2.1^K}$



$\Sigma F_x = 0 = 2.63 + 5.27 + 6.5 + 6.5 + 5.27 - 20V$ $\quad V = 1.308^K \quad 2V = 2.617^K$



$\Sigma F_x = 0 = 5.27 + 1.045 - 1.308 - SS_x$

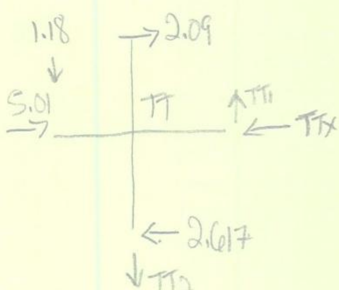
$\underline{SS_x = 5.01^K}$

$\Sigma M_{SS} = 0 = 1.308(7.5) + 1.045(7.5) - SS_1(15)$

$\underline{SS_1 = 1.18^K}$

$\Sigma F_y = 0 = 2.1 + 1.18 - SS_2$

$\underline{SS_2 = 3.28^K}$



$\Sigma F_x = 0 = 5.01 + 2.09 - 2.617 - TT_x$

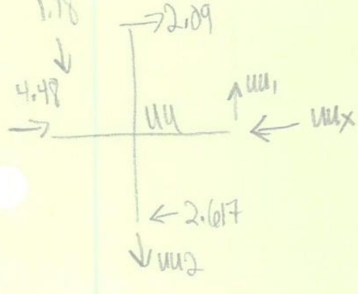
$\underline{TT_x = 4.48^K}$

$\Sigma M_{TT} = 0 = 2.617(7.5) - 1.18(15) + 2.09(7.5) - TT_1(15)$

$\Sigma F_y = 0 = 1.18 - 1.18 - TT_2$

$\underline{TT_1 = 1.18^K}$

$\underline{TT_2 = 0^K}$



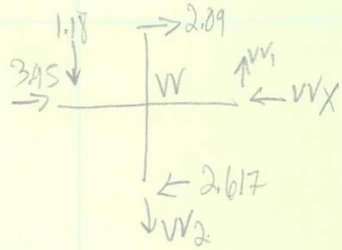
$\Sigma F_x = 0 = 4.48 + 2.09 - 2.617 - UU_x$

$\underline{UU_x = 3.95^K}$

$\Sigma M_{UU} = 0 = 2.617(7.5) - 1.18(15) + 2.09(7.5) - UU_1(15)$

$\underline{UU_1 = 1.18^K}$

$\Sigma F_y = 0 = 1.18 - 1.18 - UU_2$ $\quad \underline{UU_2 = 0^K}$



$\Sigma F_x = 0 = 3.95 + 2.09 - 2.617 - VV_x$

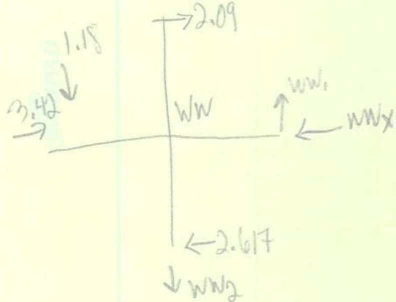
$\underline{VV_x = 3.42^K}$

$\Sigma M_{VV} = 0 = 2.617(7.5) - 1.18(15) + 2.09(7.5) - VV_1(15)$

$\underline{VV_1 = 1.18^K}$

$\Sigma F_y = 0 = 1.18 - 1.18 - VV_2$

$\underline{VV_2 = 0^K}$



$\Sigma F_x = 0 = 3.42 + 2.09 - 2.617 - WW_x$

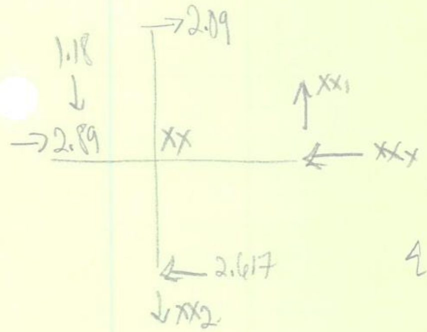
$\underline{WW_x = 2.89^K}$

$\Sigma F_{WW} = 0 = 2.617(7.5) - 1.18(15) + 2.09(7.5) - WW_1(15)$

$\underline{WW_1 = 1.18^K}$

$\Sigma F_y = 0 = 1.18 - 1.18 - WW_2$

$\underline{WW_2 = 0^K}$



$\Sigma F_x = 0 = 2.89 + 2.09 - 2.617 - XX_x$

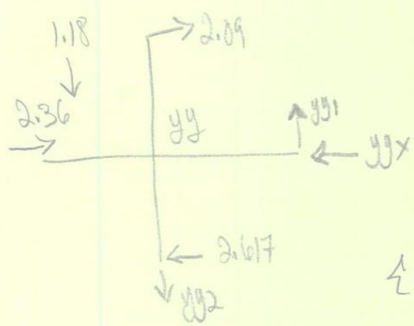
$\underline{XX_x = 2.36^K}$

$\Sigma M_{XX} = 0 = 2.617(7.5) - 1.18(15) + 2.09(7.5) - XX_1(15)$

$\underline{XX_1 = 1.18^K}$

$\Sigma F_y = 0 = 1.18 - 1.18 - XX_2$

$\underline{XX_2 = 0^K}$



$\Sigma F_x = 0 = 2.36 + 2.09 - 2.617 - YY_x$

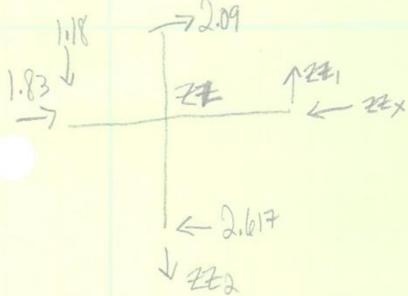
$\underline{YY_x = 1.83^K}$

$\Sigma M_{YY} = 0 = 2.617(7.5) - 1.18(15) + 2.09(7.5) - YY_1(15)$

$\underline{YY_1 = 1.18^K}$

$\Sigma F_y = 0 = 1.18 - 1.18 - YY_2$

$\underline{YY_2 = 0^K}$



$$\Sigma F_x = 0 = 1.83 + 2.09 - 2.617 - ZZ_x$$

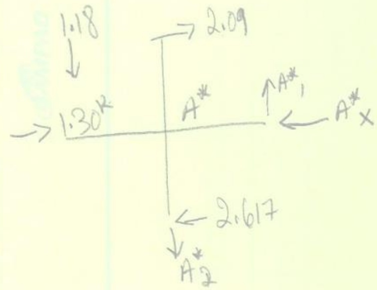
$$\underline{ZZ_x = 1.30^K}$$

$$\Sigma M_{ZZ} = 0 = 2.617(7.5) - 1.18(15) + 2.09(7.5) - ZZ_1(15)$$

$$\underline{ZZ_1 = 1.18^K}$$

$$\Sigma F_y = 0 = 1.18 - 1.18 - ZZ_2$$

$$\underline{ZZ_2 = 0^K}$$



$$\Sigma F_x = 1.30 + 2.09 - 2.617 - A^*_x$$

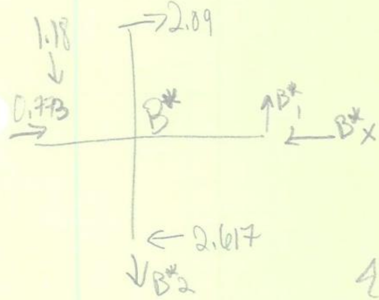
$$\underline{A^*_x = 0.773^K}$$

$$\Sigma M_{A^*} = 0 = 2.617(7.5) - 1.18(15) + 2.09(7.5) - A^*_1(15)$$

$$\underline{A^*_1 = 1.18^K}$$

$$\Sigma F_y = 0 = 1.18 - 1.18 - A^*_2$$

$$\underline{A^*_2 = 0^K}$$



$$\Sigma F_x = 0 = 0.773 + 2.09 - 2.617 - B^*_x$$

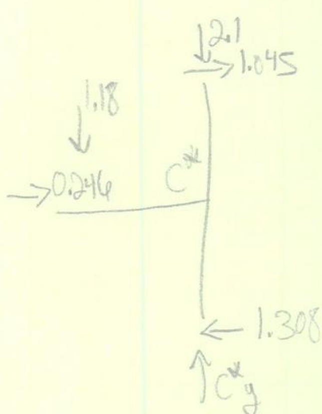
$$\underline{B^*_x = 0.246^K}$$

$$\Sigma M_{B^*} = 0 = 2.617(7.5) - 1.18(15) + 2.09(7.5) - B^*_1(15)$$

$$\underline{B^*_1 = 1.18^K}$$

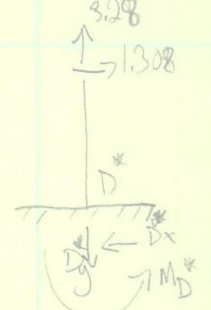
$$\Sigma F_y = 0 = 1.18 - 1.18 - B^*_2$$

$$\underline{B^*_2 = 0^K}$$



$$\Sigma F_y = 0 = C^*_y - 2.1 - 1.18$$

$$\underline{C^*_y = 3.28^K}$$

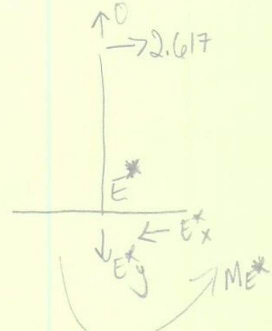


$\Sigma F_x = 0 = 1.308 - D^*_x$ $\qquad$ $\underline{D^*_x = 1.308^K}$

$\Sigma F_y = 0 = 3.28 - D^*_y$ $\qquad$ $\underline{D^*_y = 3.28}$

$\Sigma M_D = 0 = 1.308(7.5) - M^*_D$

$\underline{M^*_D = 9.81^{ft\text{-}K}}$

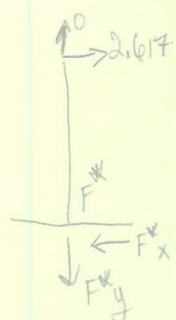


$\Sigma F_x = 0 = 2.617 - E^*_x$ $\qquad$ $\underline{E^*_x = 2.617^K}$

$\Sigma F_y = 0 = 0 - E^*_y$ $\qquad$ $\underline{E^*_y = 0^K}$

$\Sigma M_{E^*} = 0 = 2.617(7.5) - M_{E^*}$

$\underline{M_{E^*} = 19.6^{ft\text{-}K}}$



$\underline{F^*_x = 2.617^K}$

$\underline{F^*_y = 0^K}$

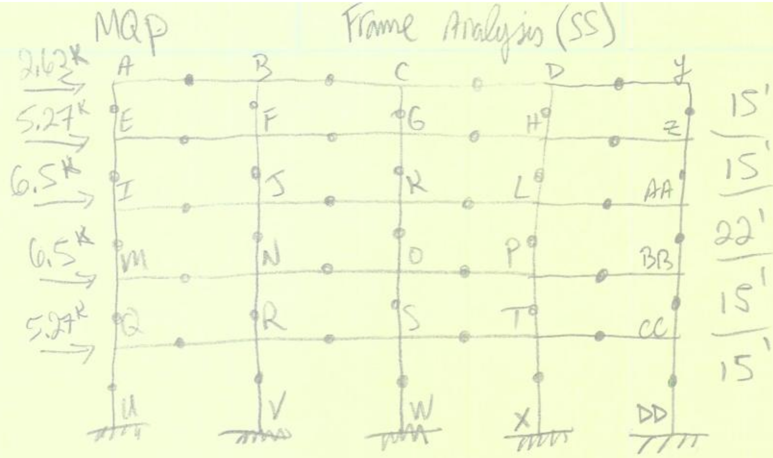
$\underline{M_{F^*} = 19.6^{ft\text{-}K}}$

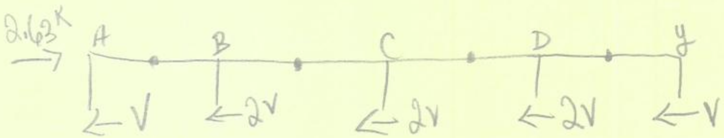
Through repetition

$G^*_x = 2.617^K$	$H^*_x = 2.617^K$	$I^*_x = 2.617^K$	$J^*_x = 2.617^K$
$G^*_y = 0^K$	$H^*_y = 0^K$	$I^*_y = 0^K$	$J^*_y = 0^K$
$M_{G^*} = 19.6^{ft\text{-}K}$	$M_{H^*} = 19.6^{ft\text{-}K}$	$M_{I^*} = 19.6^{ft\text{-}K}$	$M_{J^*} = 19.6^{ft\text{-}K}$
$K^*_x = 2.617^K$	$L^*_x = 2.617^K$	$M^*_x = 2.617^K$	$N^*_x = 1.308^K$
$K^*_y = 0^K$	$L^*_y = 0^K$	$M^*_y = 0^K$	$N^*_y = 3.28^K$
$M_{K^*} = 19.6^{ft\text{-}K}$	$M_{L^*} = 19.6^{ft\text{-}K}$	$M_{M^*} = 19.6^{ft\text{-}K}$	$M_{N^*} = 9.81^{ft\text{-}K}$

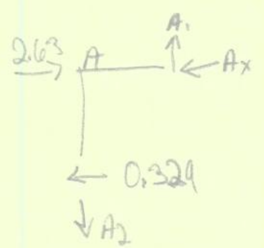
Short-Side

MQP Frame Analysis (SS)





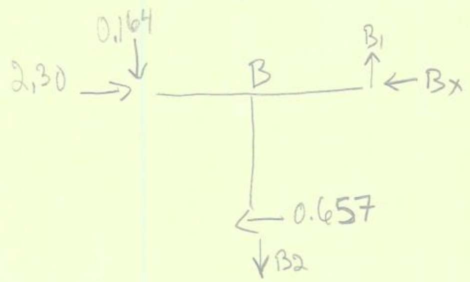
$\Sigma F_x = 0 = 2.63 - 8V \qquad V = 0.329^K \qquad 2V = 0.657^K$



$\Sigma F_x = 0 = 2.63 - 0.329 - A_x \qquad \underline{A_x = 2.30^K}$

$\Sigma M_A = 0 = 0.329(7.5) - A_1(15) \qquad \underline{A_1 = 0.164^K}$

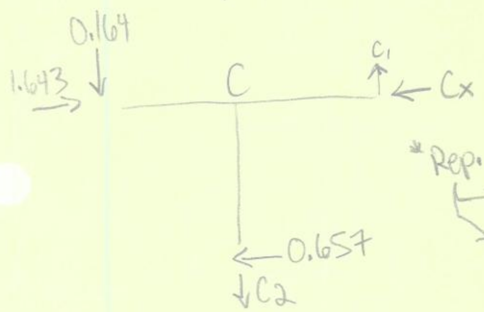
$\Sigma F_y = 0 = 0.164 - A_2 \qquad \underline{A_2 = 0.164^K}$



$\Sigma F_x = 0 = 2.30 - 0.657 - B_x \qquad \underline{B_x = 1.643^K}$

$\Sigma M_B = 0 = 0.657(7.5) - 0.164(15) - B_1(15) \qquad \underline{B_1 = 0.164^K}$

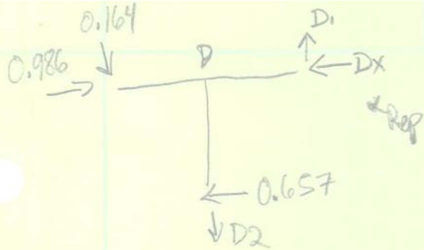
$\Sigma F_y = 0.164 - 0.164 - B_2 \qquad \underline{B_2 = 0^K}$



$\Sigma F_x = 0 = 1.643 - 0.657 - C_x \qquad \underline{C_x = 0.986}$

*Rep. $\underline{C_1 = 0.164^K}$

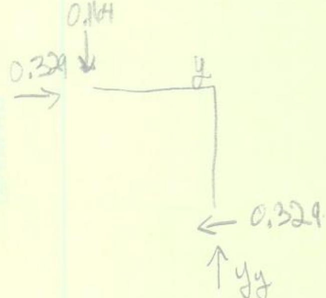
$\underline{C_2 = 0^K}$



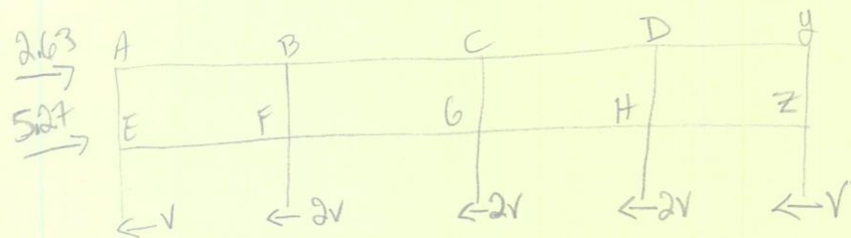
$\sum F_x = 0 = 0.986 - 0.657 - D_x$ $\quad D_x = 0.329^K$

$D_1 = 0.164^K$

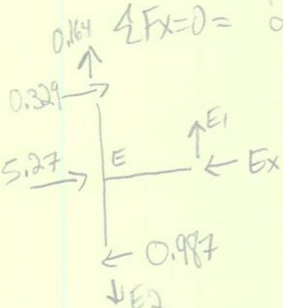
$D_2 = 0^K$



$\sum F_y = 0 = y_y - 0.164$ $\quad y_y = 0.164^K$



$\sum F_x = 0 = 2.63 + 5.27 - 8V$ $\quad V = 0.987^K \quad 2V = 1.975^K$



$\sum F_x = 0 = 5.27 + 0.329 - 0.987 - E_x$ $\quad E_x = 4.61^K$

$\sum M_E = 0 = 0.987(7.5) + 0.329(7.5) - E_1(15)$

$\sum F_y = 0 = 0.658 + 0.164 - E_2$

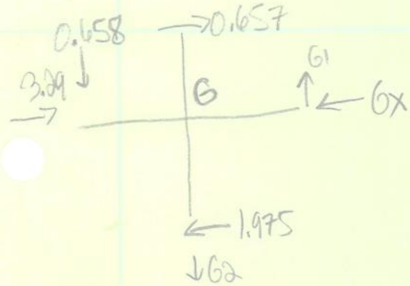
$E_2 = 0.822^K \quad E_1 = 0.658^K$

$\sum F_x = 0 = 4.61 + 0.657 - 1.975 - F_x$ $\quad F_x = 3.29^K$

$\sum M_F = 0 = 1.975(7.5) - 0.658(15) + 0.657(7.5) - F_1(15)$

$\sum F_y = 0 = 0.658 - 0.658 - F_2$

$F_2 = 0^K \quad F_1 = 0.658^K$

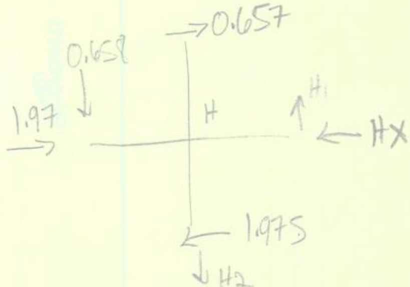


$\Sigma F_x = 0 = 3.29 + 0.657 - 1.975 - G_x$

$G_x = 1.97^K$

$G_1 = 0.658^K$

$G_2 = 0^K$

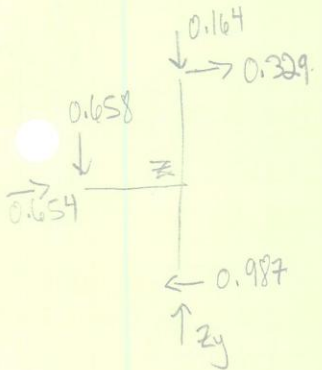


$\Sigma F_x = 0 = 1.97 + 0.657 - 1.975 - H_x$

$H_x = 0.654^K$

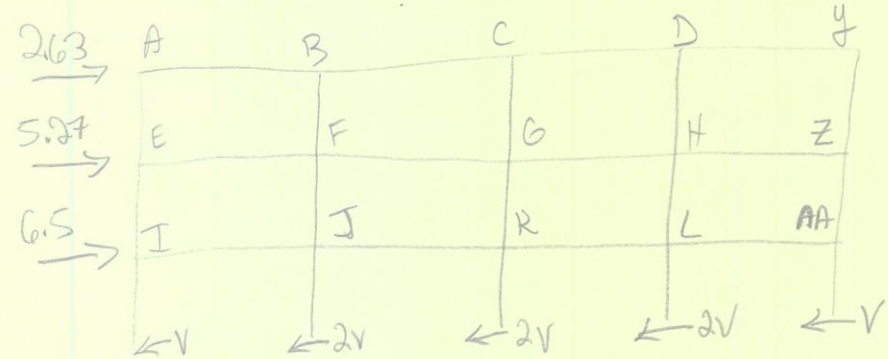
$H_1 = 0.658^K$

$H_2 = 0^K$



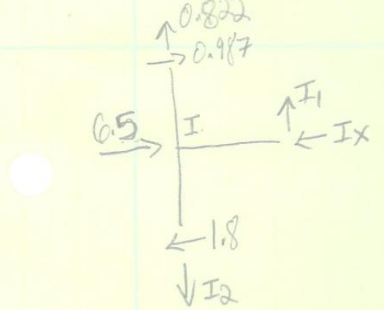
$\Sigma F_y = 0 = Z_y - 0.658 - 0.164$

$Z_y = 0.822^K$



$\Sigma F_x = 0 = 2.63 + 5.27 + 6.5 - 8V$

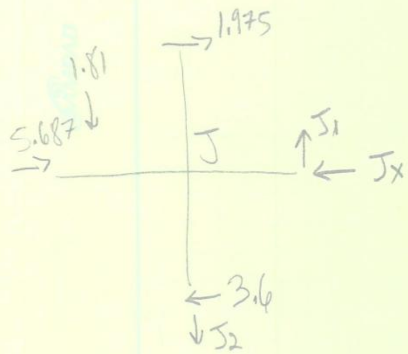
$V = 1.8^K$ $2V = 3.6^K$



$$\Sigma F_x = 0 = 6.5 + 0.987 - 1.8 - I_x \qquad \underline{I_x = 5.687^K}$$

$$\Sigma M_I = 0 = 1.8(11) + 0.987(7.5) - I_1(15) \qquad \underline{I_1 = 1.81^K}$$

$$\Sigma F_y = 0 = 1.81 + 0.822 - I_2 \qquad \underline{I_2 = 2.63^K}$$



$$\Sigma F_x = 0 = 5.687 + 1.975 - 3.6 - J_x \qquad \underline{J_x = 4.06^K}$$

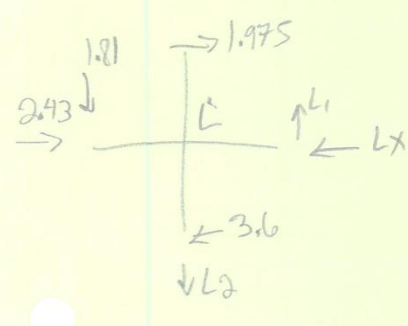
$$\Sigma M_J = 0 = 3.6(11) - 1.81(15) + 1.975(7.5) - J_1(15) \qquad \underline{J_1 = 1.81^K}$$

$$\Sigma F_y = 0 = 1.81 - 1.81 - J_2 \qquad \underline{J_2 = 0^K}$$

$$\Sigma F_x = 0 = 4.06 + 1.975 - 3.6 - K_x \qquad \underline{K_x = 2.43^K}$$

$$\underline{K_1 = 1.81^K}$$

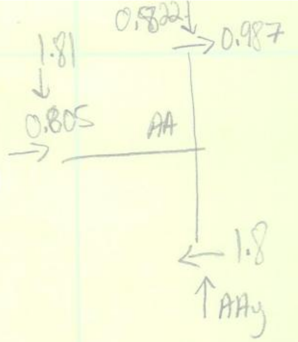
$$\underline{K_2 = 0^K}$$



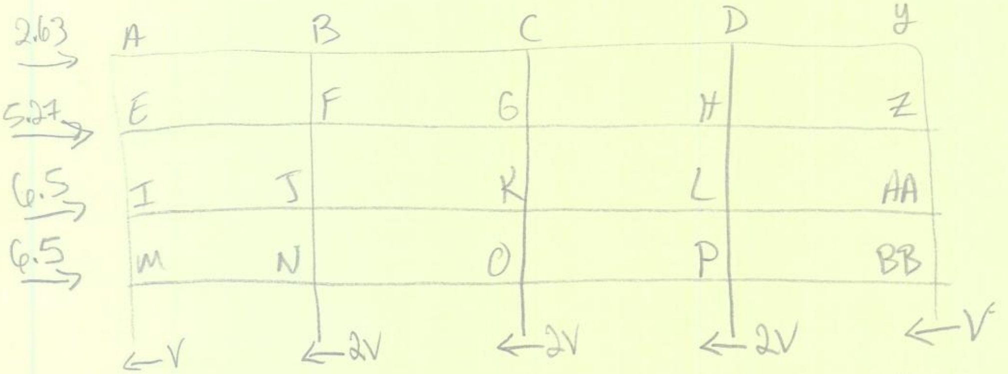
$$\Sigma F_x = 0 = 2.43 + 1.975 - 3.6 - L_x \qquad \underline{L_x = 0.805^K}$$

$$\underline{L_1 = 1.81^K}$$

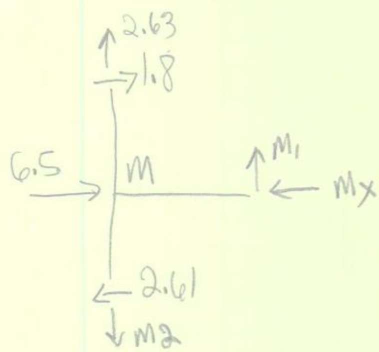
$$\underline{L_2 = 0^K}$$



$\Sigma F_y = 0 = AA_y - 1.81 - 0.822$ $\qquad$ $AA_y = 2.63^K$



$\Sigma F_x = 0 = 2.63 + 5.27 + 6.5 + 6.5 - 8V$ $\qquad$ $V = 2.61^K$ $\quad$ $2V = 5.22^K$



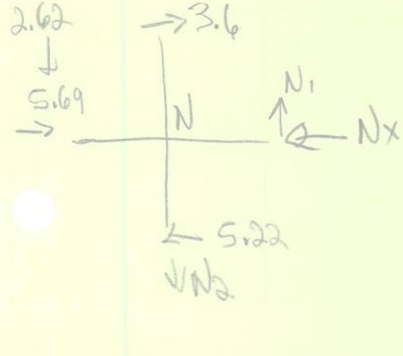
$\Sigma F_x = 0 = 6.5 + 1.8 - 2.61 - M_x$ $\qquad$ $M_x = 5.69^K$

$\Sigma M_m = 0 = 2.61(7.5) + 1.8(11) - M_1(15)$

$M_1 = 2.62^K$

$\Sigma F_y = 0 = 2.63 + 2.62 - M_2$

$M_2 = 5.25^K$

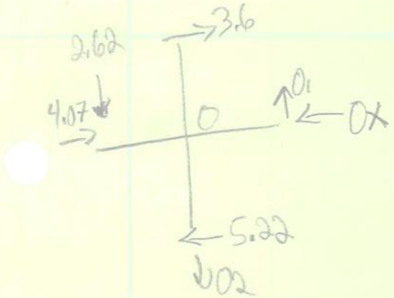


$\Sigma F_x = 0 = 5.69 + 3.6 - 5.22 - N_x$ $\qquad$ $N_x = 4.07^K$

$\Sigma M_N = 0 = 5.22(7.5) - 2.62(15) + 3.6(11) - N_1(15)$

$\Sigma F_y = 0 = 2.62 - 2.62 - N_2$ $\qquad$ $N_1 = 2.62^K$

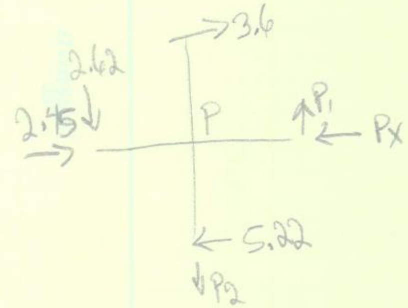
$N_2 = 0^K$



$$\Sigma F_x = 0 = 4.07 + 3.6 - 5.22 - O_x \qquad O_x = 2.45^k$$

$$O_1 = 2.62^k$$

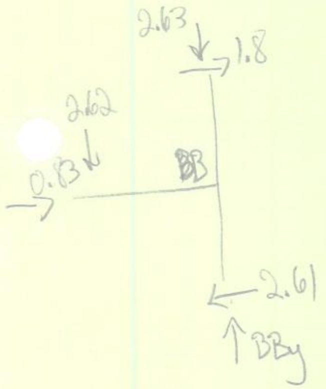
$$O_2 = 0^k$$



$$\Sigma F_x = 0 = 2.45 + 3.6 - 5.22 - P_x \qquad P_x = 0.83^k$$

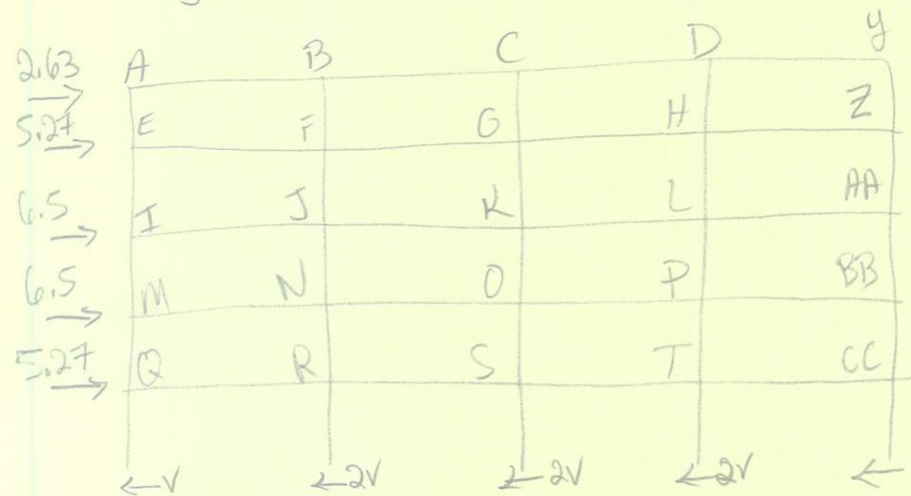
$$P_1 = 2.62^k$$

$$P_2 = 0^k$$

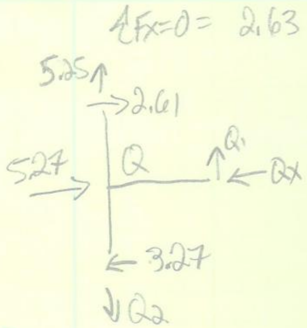


$$\Sigma F_y = 0 = BB_y - 2.62 - 2.63$$

$$BB_y = 5.25^k$$



$\Sigma F_x = 0 = 2.63 + 5.27 + 6.5 + 6.5 + 5.27 - 8V \qquad V = 3.27^K \quad 2V = 6.54^K$



$\Sigma F_x = 0 = 5.27 + 2.61 - 3.27 - Q_x$

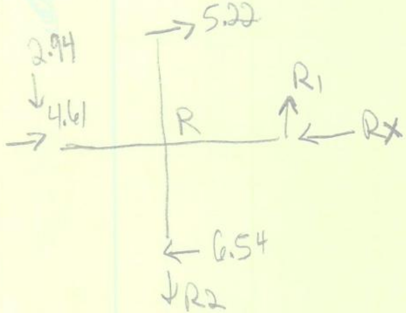
$\underline{Q_x = 4.61^K}$

$\Sigma M_Q = 0 = 3.27(7.5) + 2.61(7.5) - Q_1(15)$

$\underline{Q_1 = 2.94^K}$

$\Sigma F_y = 0 = 5.25 + 2.94 - Q_2$

$\underline{Q_2 = 8.19^K}$



$\Sigma F_x = 0 = 4.61 + 5.22 - 6.54 - R_x$

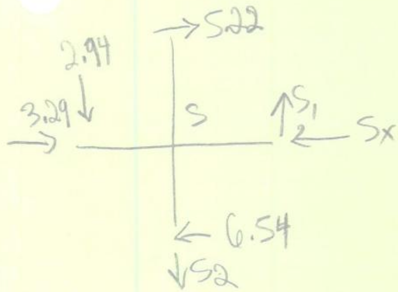
$\underline{R_x = 3.29^K}$

$\Sigma M_R = 0 = 6.54(7.5) - 2.94(15) + 5.22(7.5) - R_1(15)$

$\underline{R_1 = 2.94^K}$

$\Sigma F_y = 0 = 2.94 - 2.94 - R_2$

$\underline{R_2 = 0^K}$

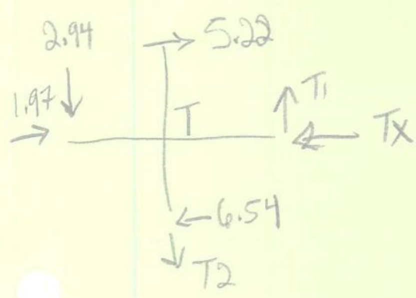


$\Sigma F_x = 0 = 3.29 + 5.22 - 6.54 - S_x$

$\underline{S_x = 1.97^K}$

$\underline{S_1 = 2.94^K}$

$\underline{S_2 = 0^K}$

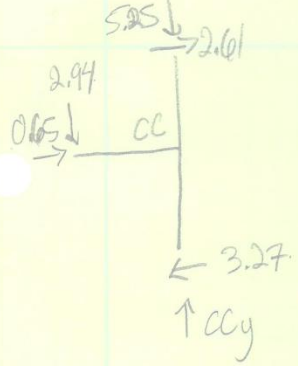


$\Sigma F_x = 0 = 1.97 + 5.22 - 6.54 - T_x$

$\underline{T_x = 0.65^K}$

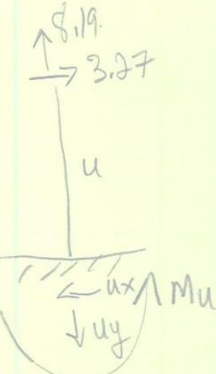
$\underline{T_1 = 2.94^K}$

$\underline{T_2 = 0^K}$



$\Sigma F_y = CC_y - 2.94 - 5.25$

$\underline{CC_y = 8.19^K}$



$\Sigma F_x = 0 = 3.27 - U_x$ $\quad \underline{U_x = 3.27^K}$

$\Sigma F_y = 0 = 8.19 - U_y$ $\quad \underline{U_y = 8.19^K}$

$\Sigma M_u = 3.27(7.5) - M_u$ $\quad \underline{M_u = 24.5^{ft\text{-}K}}$

$\underline{V_y = 0^K}$

$\underline{V_x = 6.54^K}$

$\underline{M_V = 49.05^{ft\text{-}K}}$

$\underline{W_y = 0^K}$

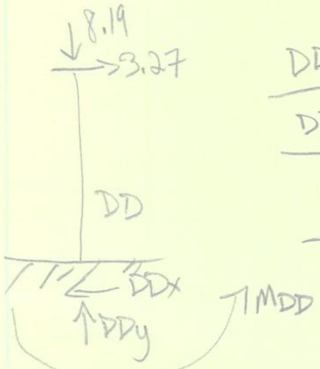
$\underline{W_x = 6.54^K}$

$\underline{M_W = 49.05^{ft\text{-}K}}$

$\underline{X_y = 0^K}$

$\underline{X_x = 6.54^K}$

$\underline{M_X = 49.05^{ft\text{-}K}}$



$\underline{DD_y = 8.19^K}$

$\underline{DD_x = 3.27^K}$

$\underline{M_{DD} = 24.5^{ft\text{-}K}}$

Appendix L
Settlement Spreadsheet

SETTLEMENT ANALYSIS OF SHALLOW FOUNDATIONS
Classical Method

Date March 1, 2009
Identification Problem 7.5

Input

Units	e	E or SI
Shape	SQ	SQ, CI, CO, or RE
B =	10	ft
L =		ft
D =	3	ft
P =	708	k
Dw =		ft
r =	0.85	

Results

q =	7343	lb/ft^2
delta =	4.42	in

Depth to Soil Layer Top (ft)	Bottom (ft)	Cc/(1+e)	Cr/(1+e)	sigma m' (lb/ft^2)	gamma (lb/ft^3)	zf (ft)	sigma c' (lb/ft^2)	sigma zo' (lb/ft^2)	delta sigma (lb/ft^2)	sigma zf' (lb/ft^2)	strain (%)	delta (in)
0.0	3.0				113							
3.0	3.5	0.1	0.022	5000	113	0.25	5164	164	7190	7355	4.10	0.246
3.5	4.0	0.1	0.022	5000	113	0.75	5190	190	7173	7363	3.98	0.239
4.0	4.5	0.1	0.022	5000	113	1.25	5215	215	7113	7328	3.85	0.231
4.5	5.0	0.1	0.022	5000	113	1.75	5240	240	6992	7232	3.69	0.222
5.0	5.5	0.1	0.022	5000	113	2.25	5266	266	6804	7070	3.51	0.211
5.5	6.0	0.1	0.022	5000	113	2.75	5291	291	6555	6846	3.31	0.198
6.0	6.5	0.1	0.022	5000	113	3.25	5316	316	6257	6573	3.08	0.185
6.5	7.0	0.1	0.022	5000	113	3.75	5342	342	5925	6266	2.82	0.169
7.0	7.5	0.1	0.022	5000	113	4.25	5367	367	5574	5941	2.55	0.153
7.5	8.0	0.1	0.022	5000	113	4.75	5392	392	5217	5609	2.27	0.136
8.0	8.5	0.1	0.022	5000	113	5.25	5417	417	4865	5283	2.06	0.124
8.5	9.0	0.1	0.022	5000	113	5.75	5443	443	4525	4968	1.96	0.118
9.0	9.5	0.1	0.022	5000	113	6.25	5468	468	4202	4670	1.87	0.112
9.5	10.0	0.1	0.022	5000	113	6.75	5493	493	3898	4391	1.78	0.107
10.0	10.5	0.1	0.022	5000	113	7.25	5519	519	3615	4134	1.69	0.101

Appendix M

Footing #1 Design

FOOTINGS FOR COLUMNS WITH LOAD BETWEEN 0-372 KIPS

MAX LOAD ON COLUMN = 372 KIPS

① ESTIMATE THE DEPTH OF THE COLUMN TO BE 1 TO 2 TIMES THE COLUMN DIMENSION

Take $h = 36\text{ in}$

ALLOWABLE NET SOIL PRESSURE = ALLOWABLE BEARING PRESSURE − (WT. OF FOOTING + SOIL + FLOOR + FLOOR LOAD)

$$= 8\text{ ksf} - \left(\left(\frac{36\text{ in}}{12\text{ in/ft}} \times .15\frac{\text{kip}}{\text{ft}^3}\right) + \left(1\text{ ft} \times .12\frac{\text{kip}}{\text{ft}^3}\right) + \left(.5\text{ ft} \cdot .15\frac{\text{kip}}{\text{ft}^3}\right) + .1\text{ ksf}\right)$$

$$= 7.255\text{ ksf}$$

② REQUIRED AREA $= \dfrac{P_u}{q_{net}} = \dfrac{372\text{ kips}}{7.255\text{ ksf}} = 51.3\text{ ft}^2$

TRY $8_{ft} \times 8_{ft}$ by 36" THICK FOOTING

③ SOLVE FOR THE FACTORED NET SOIL PRESSURE

$$q_{nu} = \frac{372\text{ kips}}{8' \times 8'} = 5.81\text{ ksf}$$

④ 2-WAY SHEAR CHECK (PUNCH)

d = h − concrete cover − bar diameter

$d = 36\text{in} - 3\text{in} - 1\text{in} = 32\text{ in}$

$$V_u = q_{nu} \times \text{TRIB. AREA} = 5.8\left[8^2 - \left(\frac{12+32}{12}\right)^2\right] = 292.9\text{ kips}$$

$$\frac{V_u}{\phi} = \frac{292.9\text{ kips}}{0.85} = 344.5\text{ kips}$$

V_c = SMALLEST OF:

$$\left(\frac{2+4}{\beta_c}\right)\sqrt{f'_c}\,b_o d = \left(\frac{2+4}{12.1/10}\right)\cdot\sqrt{4000}\cdot(32+12)\cdot 4 \times 32 = 1766\text{ kips}$$

$$\left(\frac{\alpha_s}{b_o/d}+2\right)\sqrt{f'_c}\,b_o d = \left(2+\frac{20\cdot 30\cdot 40}{(44\cdot 4)/32}\right)\cdot\sqrt{4000}\cdot(32+12)\cdot 4 \times 32 = 2007\text{ kips}$$

$$4\sqrt{f'_c}\,b_o d = 4\cdot\sqrt{4000}\cdot(32+12)\cdot 4 \times 32 = \underline{1425\text{ kips}}$$

$$V_c = 1425\text{ kips} > \frac{V_u}{\phi} = 344.5\text{ kips}\ ☑$$

⑤ ONE-WAY SHEAR CHECK

$$X = \frac{(8' \times 12) - 12.1}{2} - 32 = 9.95 \text{ inches}$$

$$V_U = 5.8\left(8 \times \frac{9.95}{12}\right) = 38.5 \text{ kips}$$

$$\frac{V_U}{\phi} = \frac{38.5}{0.85} = 45.3 \text{ kips}$$

$$V_c = 2\sqrt{f'_c}\, b\, d = \frac{2\sqrt{4000} \cdot 8 \cdot 12 \cdot 32}{1000} = 388.6 \text{ kips}$$

$$V_c = 388.6 \text{ kips} > \frac{V_U}{\phi} = 45.3 \text{ kips} \quad ☑$$

⑥ DESIGN FLEXURAL REINFORCEMENT

$$y = \frac{(8 \times 12) - 12}{2} = 42 \text{ in}$$

$$M_U = 5.8\left(8\text{ft} \times \frac{42}{12} \times \frac{42}{12 \times 2}\right) = 284.2$$

$$\frac{M_U}{\phi} = \frac{284.2}{0.9} = 315.8 \text{ ft-kips}$$

$$(A_s)_{req} = \frac{M_U/\phi}{f_y(.9d)} = \frac{315.8 \times 12000}{50000\,(.9 \times 32)} = 2.63 \text{ in}^2$$

$$(A_s)_{min} = 0.0018bh = .0018 \cdot (8 \times 12) \cdot 36 = 6.22 \text{ in}^2$$

CHOOSE 8#8 BARS (BOTH WAYS) (EACH WAY) $A_s = 6.32 \text{ in}^2$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{6.32 \times 50{,}000}{0.85 \times 4000 \times 96} = 0.97 \text{ in}$$

$$M_n = A_s f_y\left(d - \frac{a}{2}\right) = 6.32 \text{ in}^2 \cdot 50{,}000\left(32 - \frac{.97}{2}\right)$$

$$M_n = 9959034 \text{ in-lb} = 829.9 \text{ ft-kips}$$

$$M_n = 829 \text{ ft kips} > \frac{M_U}{\phi} = 315.8 \text{ ft-kips} \quad ☑$$

⑦ CHECK THE DEVELOPMENT LENGTH

$$l_{db} = 0.04\frac{A_b f_y}{\sqrt{f'_c}} = 0.04 \times \frac{.89 \times 50{,}000}{\sqrt{4000}} = 28.1 \text{ in}$$

$$(l_d)_{min} = 0.03 d_b \frac{f_y}{\sqrt{f'_c}} = .03 \times 1.0 \times \frac{50{,}000}{\sqrt{4000}} = 23.7 \text{ in}$$

BAR LENGTH AVAILABLE = y - CONCRETE COVER = 42-3 = 39 inches

39 inches > 28.1 inches ☑

(8) CHECK BEARING PRESSURE

$$e = \frac{M}{P + W_f} = \frac{284.2 \text{ ft-kips}}{372_{\text{kips}} + \left(\frac{8 \times 8 \times 3 \times 150}{1000}\right)} = 0.709$$

$$D/6 = 8/6 = 1.33$$

$e = 0.709 \le D/6 = 1.33$ → BEARING DISTRIBUTED TRAPEZOIDALLY

$$q_{min} = \left(\frac{P_u + W_f}{A} - U_d\right)\left(1 - \frac{6e}{B}\right)$$

$$= \left(\frac{372 + 28.8}{8 \times 8} - 0\right)\left(1 - \frac{6 \times .709}{8}\right)$$

$$= 6.26 \times .468$$

$$= 2.93_{\text{ksf}}$$

$q_{max} = 6.26 \times 1.53 = 9.57_{\text{ksf}} > 8_{\text{ksf}}$ → $\underline{\underline{\text{FAIL}}}$

$\boxed{\text{8x8 FAILED BEARING PRESSURE TEST...}}$

SO TRY 10×10

$q_{av} = 3.72$ ksf

2-WAY $V_u = 3.72\left[10^2 - \left(\frac{12.1 + 32}{12}\right)^2\right] = 501.7$ kip

$V_c = 4\sqrt{4000} \cdot (32+12) \cdot 4 \times 32 = 1425$

$1425 > \frac{501.7}{0.85} = 590.2$ ☑

1-WAY

$x = \frac{(10' \times 12) - 12.1}{2} - 32 = 21.95$

$V_u = 3.72\left[10 \times \frac{21.95}{12}\right] = 68.05$ kips

$V_c = \frac{2\sqrt{4000} \times 10 \times 12 \times 32}{1000} = 485.7$ kips

$485.7 > \frac{68.05}{.85} = 80.1$ ☑

development length

$l_{db} = \frac{.04 \times 1.22 \times 50{,}000}{\sqrt{4000}} = 38.6$

$l_{d\,min} = \frac{.03 \times 1.375 \times 50{,}000}{\sqrt{4000}} = 32.6$

bar length available $= y - \text{Con. cov} = 54 - 3 = 49$ in

49 in > 38.6 in ☑

Bearing pressure

$e = \frac{376.7}{372 + (45)} = .903$ $B/6 = 10/6 = 1.66$

$q_{min} = \left(\frac{372 + 45}{10 \times 10}\right)\left(1 - \frac{6 \cdot .903}{10}\right) =$

$4.217(1 - .458) =$

$q_{min} = 2.29$

$\boxed{q_{max} = 4.217(1 + .458) = 6.14_{\text{ksf}} \le 8_{\text{ksf}}}$ ☑

10x10 passed bearing pressure test

Design of Flex. Rein.

$Y = \frac{(10 \times 12) - 12}{2} = 54$ in

$M_u = 3.72\left[10 \times \frac{54}{12} \times \frac{54}{24}\right] = 376.7$

$\frac{M_u}{\phi} = \frac{376.7}{0.9} = 418.5$

$(A_s)_{req} = \frac{M_u/\phi}{f_y(.9d)} = \frac{418.5 \times (12000)}{50000(.9 \times 32)} = 3.49 \text{ in}^2$

$(A_s)_{min} = 0.0018bh = .0018(10 \times 12) \times 36 = 7.78 \text{ in}^2$

Choose 5#11 bars $A_s = 7.80 \text{ in}^2$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{7.80 \cdot 50000}{0.85 \cdot 4000 \times 120} = .955$

$M_n = 7.80 \times 50000 \left(32 - \frac{.955}{2}\right)$

$M_n = 1024$

$M_n\, 1024 > 418.5$ ☑

10X10 WAS 1.86ksf UNDER ALLOWABLE BEARING PRESSURE

SO TRY 9x9

$q_{nu} = \frac{372}{9 \times 9} = 4.59$ ksf

2-way

$V_u = 4.59\left[9^2 - \left(\frac{12.1+32}{12}\right)^2\right] = 309.8$ kips

$V_c = 4\sqrt{4000} \cdot (32+12) \times 4 \times 32 = 1425$

$1425 > 309.8$ ☑

1-way

$x = \frac{(9' \times 12) - 12.1}{2} - 32 = 15.95$

$V_u = 4.59\left[9 \times \frac{15.95}{12}\right] = 54.9$ kips

$V_c = \frac{2\sqrt{4000} \times 9 \times 12 \times 32}{1000} = 437.2$ kips

$437.2 > \frac{54.9}{.85}$ ☑

design of flex. reinf.

$x = \frac{(9 \times 12) - 12}{2} = 48$ $\quad M_u = 4.59\left[9 \times \frac{48}{12} \times \frac{48}{24}\right] = 330.5$

$\frac{M_u}{\phi} = \frac{330.5}{.9} = 367.2$ $\quad (A_s)_{req} = \frac{367.2 \times 12000}{50000\ (.9 \times 32)} = 3.06$ in²

$(A_s)_{min} = .0018bh = .0018 \cdot 9 \times 12 \times 36 = 7.00$ in²

Choose 7#9 bars — BOTH WAYS $\quad A_s = 7.00$ in²

$a = \frac{7.00 \times 50{,}000}{.85 \times 4000 \times 108} = .953$

$M_n = 7.00 \cdot 50{,}000 \cdot \left(32 - \frac{.957}{2}\right) = 919.4 > 367.2$ ☑

development length

$l_{db} = \frac{.04 \times 1.0 \times 50000}{\sqrt{4000}} = 31.6$ in

$l_{d\,min} = \frac{.03 \times 1.125 \times 50000}{\sqrt{4000}} = 26.7$ in

bar length available $= 48$in $- 3$in $= 45$in > 31.6in ☑

Bearing Pressure

$e = \frac{330.5}{372 + 36.5} = .81$ $\quad B/6 = 1.5$

$q_{min} = \left(\frac{372+36.5}{9 \times 9}\right)\left(1 - \frac{6 \cdot .81}{9}\right) = 5.04(1 - .54) = 2.318$

$q_{max} = 5.04(1 + .54) = 7.76$ ksf < 8.0 ksf ☑

(9) CHECK SETTLEMENT FOR 9x9 FOOTING

$\delta < \delta_a$

INDUCED VERTICAL STRESS

$\Delta\sigma_z = \left[1 - \left(\frac{1}{1+(B/2z)^2}\right)^{1.5}\right](7.76 - .480)$

$\Delta\sigma_z = 6.36$

Effective stress

$\sigma'_{zf} = \sigma'_{z0} + \Delta\sigma_z$ $\quad$ Layer 1 $= 9/2 = 4.5$ $\quad$ Layer 2 $= B = 9$ $\quad$ Layer 3 $= 2B = 18$ $\quad H = \frac{1}{2}(4.5 + 9 + 18) = 16'$

$\sigma'_{z0} = 120$ lb/ft³ $\times 16$ ft

$\sigma'_{z0} = 1920$

$\sigma'_{zf} = 1.920 + 6.36 = 8.28$ ksf

NORMALLY CONSOLIDATED SOILS

$\delta_c = r\sum \frac{C_c}{1+e_0} H \log\left(\frac{\sigma'_{zf}}{\sigma'_{z0}}\right)$

$= 0.85(.006)(16)\log\left(\frac{8.28}{1.920}\right)$

$= .0518$ ft $= .62$ inches $< .75$ inches ☑

$\boxed{\text{FOR } P_u \text{ BETWEEN } 0\text{-}372 \text{ kips USE } 9' \times 9' \times 3' \text{ SPREAD FOOTING W/ 7\#9 BARS BOTH WAYS}}$

Footing #2 Design

FOOTINGS FOR COLUMNS WITH LOAD BETWEEN 373 kips - 758 kips

① ESTIMATE THE DEPTH OF THE COLUMN TO BE 1 TO 2 TIMES THE COLUMN DIMENSION.

TAKE $h = 36 in$

ALLOWABLE NET SOIL PRESSURE = ALLOWABLE BEARING PRESSURE - (WT OF FOOTING + SOIL + FLOOR + FLOOR LOAD)

$= 8_{ksf} - \{(36/12 \times .150 kip/ft^3) + (1 ft \times .120 k/ft^3) + (.5 \times .15 kip/ft^3 + .1 ksf)\}$

$= 7.255 ksf$

② REQUIRED AREA $\frac{P_u}{q_{net}} = \frac{758}{7.255} = 104.5 ft^2$

TRY 11'x11' by 36" THICK FOOTING

③ SOLVE FOR FACTORED NET SOIL PRESSURE

$q_{nu} = \frac{758 kips}{11 \times 11} = 6.26 ksf$

④ 2-WAY SHEAR CHECK (PUNCH)

d = h - concrete cover - bar diameter

$d = 36_{in} - 3_{in} - 1_{in} = 32 in$

$V_u = q_{nu} \times Trib.Area = 6.26\left[11^2 - \left(\frac{16.8+32}{12}\right)^2\right] = 653.9 kips$

V_c = smallest of: $\left(\frac{2+4}{\beta_c}\right) = \frac{6}{\ldots} = 3.71$ ← smallest $3.71 \times \sqrt{f'_c}\, b_o d = 3.71 \times \sqrt{4000} \times (32+16.8) \times 4 \times 32$

$\left(\frac{\alpha_s}{b_o/d} + 2\right) = \frac{30.30 \ldots}{(32+16.8)4/32} = 13.1$ $V_c = 1466 kips$

$(4) = 4$

$V_c = 1466 kips > \frac{653.9}{0.85} = 769.3 kips$ ☑

⑤ CHECK 1-WAY SHEAR

$x = \frac{(11 \times 12) - 16.8}{2} - 32 = 23.6 inches$ $V_u = 6.26\left(11 \times \frac{23.6 in}{12}\right) = 135.4 kips$

$V_c = 2\sqrt{f'_c}\, bd = 2\sqrt{4000} \times 11 \times 12 \times 32 = 534.3 kips$

$V_c = 534.3 > \frac{135.4}{0.85} = \frac{V_u}{\phi} = 159.3$ ☑

⑥ DESIGN FLEXURAL REINFORCEMENT

$Y = \frac{(11\times12) - 16.8}{2} = 57.6$ in

$M_u = 6.26\left(11 \times \frac{57.6}{12} \times \frac{57.6}{2\times12}\right)$

$M_u = 793.3$ ft-kips $\quad \frac{M_u}{\phi} = \frac{793.3}{0.9} = 881.4$ ft-kip

$(A_s)_{req} = \frac{M_u/\phi}{f_y(jd)} = \frac{881.4}{50000(.9\times32)} = 7.35$ in²

$(A_s)_{min} = .0018bh = .0018 \times (11\times12)\times36 = 8.55$ in²

CHOOSE 7 #10 BARS BOTH WAYS

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{8.89 in^2 \times 50{,}000}{0.85\times4000\times132} = .99$

$M_n = A_s f_y (d - \frac{a}{2}) = 8.89 \times 50{,}000 \times (32 - \frac{.99}{2})$

$M_n = 1167$ ft-kips $> \frac{M_u}{\phi} = 881.4$ ☑

⑦ CHECK THE DEVELOPMENT LENGTH

$l_{db} = .04\frac{A_b f_y}{\sqrt{f'_c}} = 0.04 \times \frac{1.11\times50{,}000}{\sqrt{4000}} = 35.1$ in

$(l_d)_{min} = .03 d_b \frac{f_y}{\sqrt{f'_c}} = \frac{.03\times1.25\times50000}{\sqrt{4000}} = 29.65$ in

BAR LENGTH AVAILABLE = Y - Concrete cover = 57.6 - 3 = 54.6 in > 35.1 in ☑

⑧ CHECK BEARING PRESSURE

$e = \frac{M}{P + W_f} = \frac{793.3}{758 + \left(\frac{11\times11\times3\times150}{1000}\right)} = 0.976$

$B/6 = 11/6 = 1.83$

$0.976 < 1.83$

→ BEARING PRESSURE DISTRIBUTED TRAPEZOIDALLY

$q_{min} = \left(\frac{P_u + W_f}{A} - U_s\right)\left(1 - \frac{6e}{B}\right)$

$= \left(\frac{758 + 54.45}{11\times11} - 0\right)\left(1 - \frac{6(.976)}{11}\right)$

$= 6.714 \cdot (1 - .532)$

$= 3.142$ ksf

$q_{max} = 6.714 \cdot (1 + .532)$

$= 10.29$ ksf > 8 ksf → FAIL

$\boxed{\text{TRY } \boxed{12\times12}}$

$h = 36$ in

allowable net soil pressure = 7.255 ksf

$q_{nu} = \frac{758 \text{ kip}}{12\times12} = 5.26$ ksf

2 WAY SHEAR

$V_u = 5.26\left[12^2 - \left(\frac{16.8+32}{12}\right)^2\right] = 670.5$ kip

$V_c = 3.71 \times \sqrt{4000} \times (32+16.8)\times4\times32 = 1466$ kip

1466 kip $> \frac{670.5}{.85} = 788.8$ ☑

one-way shear

$x = \frac{(12\times 12)-16.8}{2} - 32 = 31.6\,in$ $V_u = 5.26\left(12 \times \frac{31.6}{12}\right) = 166.2\,kips$

$V_c = 2\sqrt{f'_c}\,bd = 2\sqrt{4000} \times 12 \times 12 \times 32 = 582.9\,kips$ $582.9\,kips > \frac{166.2}{.85} = \frac{V_u}{\phi}$ ☑

DESIGN FLEXURAL REINFORCEMENT

$Y = \frac{(12\times 12)-16.8}{2} = 63.6\,in$ $M_u = 5.26\left[12 \times \frac{63.6}{12} \times \frac{63.6}{24}\right] = 886.5\,ft\text{-}kips$ $\frac{M_u}{\phi} = \frac{886.5}{0.9} = 985\,ft\text{-}kips$

$(A_s)_{req} = \frac{M_u/\phi}{f_y(.9d)} = \frac{985}{50000(.9\times 32)} = 8.21\,in^2$ $(A_s)_{min} = .0018 \times (12\times 12) \times 36 = 9.33\,in^2$

CHOOSE 6#11 BARS BOTH WAYS $a = \frac{A_s f_y}{.85 f'_c b} = \frac{9.36 \times 5000}{.85 \times 4000 \times 144} = 0.96$

$M_n = 9.36\,in^2 \times 50000 \times \left(32 - \frac{.96}{2}\right) = 1229.3 > \frac{M_u}{\phi} = 985$ ☑

CHECK DEVELOPMENT LENGTH

$l_{db} = \frac{.04 \times 1.22 \times 50{,}000}{\sqrt{4000}} = 38.6\,in$ $(l_d)_{min} = \frac{.03 \times 1.375 \times 50000}{\sqrt{4000}} = 32.6\,in$

BAR LENGTH AVAILABLE $= 63.6 - 3 = 60.6\,in > 38.6\,in$ ☑

CHECK BEARING PRESSURE

$e = \frac{886.5\,ft\text{-}kips}{758 + (12\times 12\times 3\times .150)} = 1.077$ $B/6 = 2.0$ $1.077 < 2.0$ → BEARING PRESSURE DISTRIBUTED TRAPEZOIDALLY

$q_{min} = \left(\frac{758 + 64.8}{12\times 12} - 6\right)\left(1 - \frac{6(1.077)}{12}\right)$
$= (5.71) \times (1.0 - .5385)$
$= 2.64\,ksf$

$q_{max} = 5.71 \times 1.5385$
$= 8.78\,ksf > 8_{ksf}$ FAIL

$\boxed{\text{TRY } 13\times 13}$

$h = 36\,in$

allowable net soil pressure 7.255 ksf

$q_{nu} = \frac{758}{13\times 13} = 4.49\,ksf$

2-way shear

$V_u = 4.49\left[13^2 - \left(\frac{16.8+32}{12}\right)^2\right] = 684.6\,kips$

$V_c = 1466\,kips > \frac{684.6}{0.85} = 805.4\,kips$ ☑

One-way Shear

$X = \frac{(13\times12)-16.8}{2} - 32 = 37.6$ in $\qquad V_u = 4.49\left(13\times\frac{37.6}{12}\right) = 182.9$ kip

$V_c = 2\sqrt{4000}\times 13\times12\times32 = 631.4$ kips $> \frac{182.9}{.85} = 215.2$ kips ☑

DESIGN FOR FLEXURAL REINFORCEMENT

$Y = \frac{(13\times12)-16.8}{2} = 69.6$ in $\qquad M_u = 4.49\left[13\times\frac{69.6}{12}\times\frac{69.6}{24}\right] = 981.8 \qquad M_u/\phi = \frac{981.8}{.9} = 1090.9$ ft-kips

$(A_s)_{req} = \frac{1090.9 \text{ ft-kips}}{50{,}000\times(.9\times32)} = 9.09 \text{ in}^2 \qquad (A_s)_{min} = .0018\times(13\times12)\times36 = 10.11 \text{ in}^2$

CHOOSE 13 #10 BARS BOTH WAYS $A_s = 10.16$ in² $\qquad a = \frac{10.16\times50{,}000}{.85\times4000\times(13\times12)} = 0.96$

$M_n = 10.11\times50{,}000\times\left(32-\frac{.96}{2}\right) = 1593 > 1090.9$ ☑

CHECK DEVELOPMENT LENGTH

$l_{db} = \frac{.04\times1.11\times50{,}000}{\sqrt{4000}} = 35.1$ in $\qquad l_d|_{min} = \frac{.03\times1.25\times50{,}000}{\sqrt{4000}} = 29.64$ in

BAR LENGTH AVAILABLE $= 69.6 - 3 = 66.6$ in > 35.1 in ☑

CHECK BEARING PRESSURE

$e = \frac{981.8}{758+(13\times13\times3\times.150)} = 1.18 \qquad B/6 = 2.17 \qquad 2.17 > 1.18 \rightarrow$ BEARING PRESSURE DISTRIBUTED TRAPEZOIDALLY

$q_{min} = \left(\frac{758+76.05}{13\times13} - 0\right)\left(1-\frac{6(1.18)}{13}\right)$

$= 4.935\times(1-.545)$

$= 2.25$

$q_{max} = 4.935\times(1+.545)$

$= 7.62_{ksf} < 8.0_{ksf}$ ☑

CHECK SETTLEMENT

INDUCED VERTICAL STRESS:

$\Delta\sigma_z = \left[1-\left(\frac{1}{1+(.76)^2}\right)^{3/2}\right](7.62-.480)$

$\Delta\sigma_z = 6.81$

EFFECTIVE STRESS

$\sigma'_{zf} = \sigma'_{z0} + \Delta\sigma_z$

$\sigma'_{z0} = 120\times23' = 2.73$

$\sigma'_{zf} = 2.73+6.81 = 9.54_{ksf}$

layer 1 = 9/2

layer 2 = 13

layer 3 = 26

$H = \frac{1}{2}(6.5+13+26)$

$H = 22.75'$

NORMALLY CONSOLIDATED SETTLEMENT

$\delta_c = r\sum\frac{C_c}{1+e_0}H\log\left(\frac{\sigma'_{zf}}{\sigma'_{z0}}\right) = .85\times.006\times22.75\times\log\left(\frac{9.54}{2.73}\right) = .06' = .72'' < .75''$ ☑

$\boxed{\text{FOR } P_u \text{ BETWEEN 373 kips - 758 kips USE } 13'\times13'\times3' \text{ SPREAD FOOTING WITH 13 \#10 BARS BOTH WAYS}}$

Footing #3 Design

FOOTINGS FOR COLUMNS WITH LOAD 759 kips – 934 kips

① ESTIMATE DEPTH OF COLUMN TO BE 1 to 2 TIMES COLUMN DIMENSION

Take $h = 48$ inches

Allowable net soil pressure $= 8\,ksf - \left(\frac{48}{12}\times .150\right) + (1\times .120) + (.5\times .15)\,(1)$

$= 7.105\,ksf$

② Required Area $\frac{P_u}{q_{net}} = \frac{934}{7.105} = 131.5$

Try 14'×14'×4' footing

③ SOLVE FOR FACTORED NET SOIL PRESSURE

$q_{nu} = \frac{934}{14\times 14} = 4.77\,ksf$

④ TWO-WAY SHEAR CHECK

$d = 48 - 3\text{in} - 1\text{in} = 44\text{in}$

$V_u = q_{nu} \times \text{Trib Area} = 4.77\left[14^2 - \left(\frac{18.7+44}{12}\right)^2\right] = 805.9\,\text{kips}$

$V_c =$ smallest: $\frac{2+4}{(18.7/18.7)} = 3.59$

$\frac{20{,}30\text{or}40}{(44+18.7)/44} = 14$ Govern $\quad 3.59\times\sqrt{4000}\times(44+18.7)\times 4\times 44$

$V_c = 2505.6$

4

$2505.6 > \frac{805.9}{.85}$ ☑

⑤ CHECK 1-WAY SHEAR

$x = \frac{(14\times 12) - 18.7}{2} - 44 = 30.65 \qquad V_u = 4.77\left(14\times\frac{30.65}{12}\right) = 170.6\,\text{kips}$

$V_c = 2\sqrt{4000}\times 14\times 12\times 44 = 935\,\text{kips}$

$935 \geq \frac{170.6}{.85}$ ☑

⑥ DESIGN FLEXURAL REINFORCEMENT

$X = \frac{(14\times12)-18.7}{2} = 74.65\,in$

$(A_s)_{req} = \frac{1435.7(\times12000)}{50000\times.9\times44} = 8.70$

$M_u = 4.77\left(14\times\frac{74.65}{12}\times\frac{74.65}{24}\right) = 1292.2$

$(A_s)_{min} = .0018bh = .0018\times(14\times12)\times44 = 13.3\,in^2$

$M_n = \frac{1292.2}{.9} = 1435.7\,ft\text{-}kips$

CHOOSE 9#11 bars

$a = \frac{14.04\times50{,}000}{.85\times4000\times168} = 1.24$

$M_n = 14.04\times50{,}000\times(44-\frac{1.24}{2}) = 2537.7\,ft\text{-}kip$

$2537.7_{ft\text{-}k} > 1435.7_{ft\text{-}kip}$ ☑

⑦ CHECK DEVELOPMENT LENGTH

$l_{db} = \frac{.04\times1.22\times50000}{\sqrt{4000}} = 38.6\,in$

$(l_d)_{min} = \frac{.03\times1.375\times50{,}000}{\sqrt{4000}} = 32.6\,in$

BAR LENGTH AVAILABLE $= 74.65 - 3 = 71.65_{in} > 38.6\,in$ ☑

⑧ CHECK BEARING PRESSURE

$e = \frac{1292}{934+\left(\frac{14\times14\times4\times150}{1000}\right)} = 1.23$

$B/6 = 2.33$

$2.33 > 1.23 \rightarrow$ Bearing pressure distributed trapezoidally

$q_{min} = \left(\frac{934+117.6}{(14\times14)}\right)\left(1-\frac{6(1.23)}{14}\right)$

$= 5.36\times(1-.53)$

$= 5.36\times.417 = 2.24$

$q_{max} = 5.36\times1.53 = 8.20 > 8.0$ $\boxed{FAIL}$

TRY 15'x15'x4'

$q_{nu} = \frac{934}{15\times15} = 4.15$

$M_u = 4.15\left(15\times\frac{80.65}{12}\times\frac{80.65}{24}\right) = 1405$

$q_{max} = \left(\frac{934+135}{15\times15}\right)\times\left(1+\frac{6(1.31)}{15}\right)$

$Y = \frac{(15\times12)-18.7}{2} = 80.65$

$e = \frac{1405}{934+(135)} = 1.31$

$= 4.75\times1.524 = 7.24 < 8.00$ ☑

FLEXURAL REINFORCEMENT

$Y = 80.65$

$M_u = 1405$

$M_v = \frac{1405}{\phi} = \frac{1405}{0.9} = 1561$

$(A_s)_{req} = \frac{1561 \times 12000}{60{,}000 \times 9 \times 44} = 9.46$

$(A_s)_{min} = .0018 \times (15 \times 12)\, 44 = 14.26$

CHOOSE 10 #11 bars both ways

FOR LOAD 759 kips - 934 kips USE 15'x15'x4' FOOTING WITH 10 #11 BARS BOTH WAYS

Appendix N

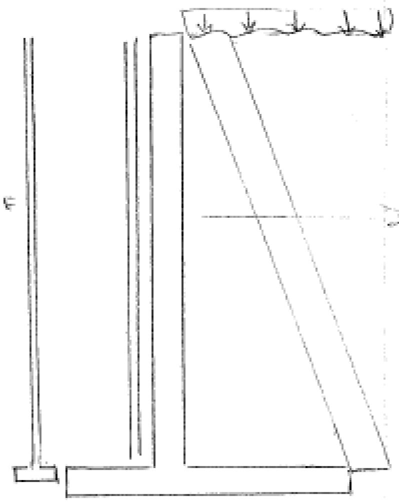
Retaining wall

CANTILEVER RETAINING WALL @ EAST ELEVATION

$P_a = C_a \omega h^2/2$ WHERE $C_a \omega$ = 60 pcf (EQUIV. FLUID PRESSURE)

$P_p = C_p \omega h^2/2$ SAME ⟶ = 90 pcf

ASSUME UNIFORM SURCHARGE LOAD
(LOBBY & PAVED SURFACE)



$\sigma = Kq$ — σ = ADDITIONAL LAT. LOAD
K_a = FOR ACTIVE COND.
q = SURCHARGE PRESSURE

$K_a = \tan^2(45° - \phi/2) = 0.2379$

$\sigma = K_a q = 0.2379\, q$

* ASSUME INSTEAD A 6 ft. SURCHARGE WILL APPROXIMATE THE LATERAL EARTH PRESSURE EFFECT.

WEIGHT OF SOIL = 140 pcf
EQUIV. FLUID PRESSURE - 60 pcf
∢ INTERNAL FRICTION - 38°
COEFF. FRICTION - ≈ 0.45 (COARSE GRAINED w/ SILT)

f'_c = 4000 psi
f_y = 50,000 psi

FOR PASSIVE USE
EQUIV. FLUID PRESSURE = 90 pcf

MAX. SOIL PRESSURE
5 KSF

① FIND $\rho \approx 30\% \rho_b$ $\rho \approx (0.3)\cdot(0.0367)$ T. 3. 6. 1

$\rho = 0.0110$ REINFORCEMENT RATIO

$R_N = \rho f_y (1 - \frac{1}{2}\rho m)$ WHERE $m = f_y / 0.85 f'_c = 14.71$

$R_N = (0.0110)\cdot(50{,}000)[1 - (\frac{1}{2}\cdot(0.0110)(14.71))] = 505.92$

$R_N \approx 505$ psi

② HEIGHT OF WALL DIFF IN GRADE = 30 ft.

ALLOW 4 FT FOR FROST PENETRATION

34 ft.

③ THICKNESS OF FOOTING   MAY BE ESTIMATED

USE 7 TO 10 % OF WALL HEIGHT

USE 3 ft. THICKNESS ≈ 9%

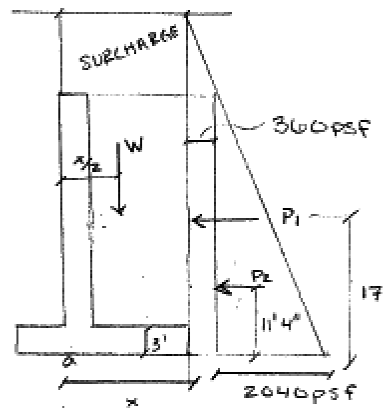
④ BASE LENGTH → CONSIDER EQUILIBRIUM OF FACTORED LOADS

USE 1-ft LENGTH OF WALL WHERE MAT. $w = 140$ pcf.

$P_1 = (0.360)(34)(1) = 12.24$ K

$P_2 = \frac{1}{2}(2040)(34)(1) = 34.68$ K

$W = 0.140(34+6)x = 5.60x$



SUM OF MOMENTS @ POINT a

$W(x/2) = P_1(17.0) + P_2(11.33)$

$\frac{5.60x^2}{2} = (12.24)(17) + (34.68)(11.33)$

$2.8x^2 = 601.0$ $x = 14.65$ ft.

FOR GRANULAR MATERIAL THE RESULTANT SOIL PRESSURE IS TO BE @ OUTER EDGE OF MIDDLE 1/3 OF FOOTING.

BASE LENGTH $= 1.5x = 1.5(14.65) = 21.976$ ft

ASSUME BASE = 22 ft. LONG

⑤ STEM THICKNESS → REGARD BENDING M & SHEAR.

BENDING MOMENT USUALLY GOVERNS.

$M_y = \frac{0.360y^2}{2} + \frac{0.060y^3}{6} = 0.18y^2 + 0.01y^3$ (FROM TOP)

@ THE BOTTOM OF 31' STEM WALL →

$M_u = 1.6[0.180(31)^2 + 0.01(31)^3] = 753.424$ ft-kips

FOR DESIRED $R_n \approx 505$ psi:

REQUIRED $d = \sqrt{\frac{M_u}{\phi R_n b}} = \sqrt{\frac{(753)(12{,}000)}{0.9(505)12}} = 40$ IN.

TOTAL THICKNESS = 40" + 2" COVER + 0.75 (EST. BAR RAD)

= 42.75"   USE 43"   12" PRACTICAL MIN @ TOP.

CHECK SHEAR @ BASE OF WALL

$$V_y = 0.360y + 1/2(0.060)y^2 = 0.36y + 0.03y^2$$

@ BASE (31') USING 1.6 OVERLOAD FACTOR

$$V_u = 1.6[0.36(31) + 0.03(31)^2] = 63.984 \text{ KIPS}$$

$$\phi V_c = \phi(2\sqrt{f'c})bd$$

$$= 0.75(2\sqrt{4000})(12)(43)\tfrac{1}{1000} = 48.95 < 63.984 \quad \times \quad \underline{\text{NOT OK}}$$

$$0.75(2\sqrt{4000})(12)(d)\tfrac{1}{1000} > 63.984$$

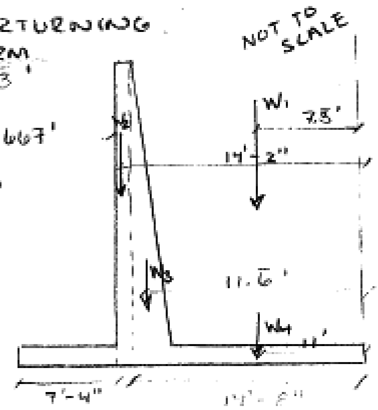
$d = 56.2$ IN. USE 57 INCHES

NEW BASE STEM THICKNESS $d = 57'' + 2'' + 0.75'' = \underline{60 \text{ INCHES}}$

MIN TOP THICKNESS → $\underline{12 \text{ INCHES}}$

⑥ FACTOR OF SAFETY AGAINST OVERTURNING

	ARM
$W_1 = (0.140)(34+6)(5.54) = 31.024$	$7.\overline{33}'$
$W_2 = (0.150)(1.00)(31) = 4.65$	$14.1667'$
$W_3 = 1/2(4.0)(31)(0.150) = 9.3$	$11.\overline{6}'$
$W_4 = (0.150)(22)(3) = 9.9$	$11'$
54.874	



$M_1 = 31.024 \times 7.333 = 227.51$ ft-K

$M_2 = 4.65 \times 14.1\overline{6} = 65.844$ ft-K

$M_3 = 9.3 \times 11.\overline{6} = 108.5$ ft-K

$M_4 = 9.9 \times 11 = \underline{108.9 \text{ ft-K}}$

510.754 ft-K

RESULTANT FROM HEEL $= M/W = 510.754/54.874 = 9.308$ ft.

RESISTING MOMENT $= 54.874(22 - 9.308) = \underline{696.461}$ ft-K

OT MOMENT $= P_1(17) + P_2(11.\overline{3}) = 12.24(17) + 34.68(11.33) = 599.964$ ft-K

FS AGAINST OT $= \frac{696.461}{599.964} = \underline{1.161} < 2$ $\boxed{X}$

OR $0.9(696.461) \geq 1.6(600) \rightarrow 626.8 \leq 960$

BECAUSE RESISTING MOMENT WHEN COMPARED W/ OT MOMENT DOES NOT MEET FS = 1.5 TO 2.0 —>

ACCOUNT FOR EXTRA MOMENT USING TIE-BACK ANCHORS FOR A MOMENT RESISTANCE OF ATLEAST —>

2 TIMES THE OT MOMENT (600 FT-K)

MINUS THE RESISTING MOMENT PROVIDED BY THE FOOTING/WALL ITSELF. (696.461 FT-K)

TIE BACK ANCHORS FOR $2(600) - 696.5 = 503.5$ ft-K

USE 500 ft-K

TIE BACK DESIGN

USE FACTOR OF SAFETY = 1.3

OVERTURNING MOMENT

$$P_1(17) + P_2(11.3) < 350 \text{ ft}\cdot\text{K}$$

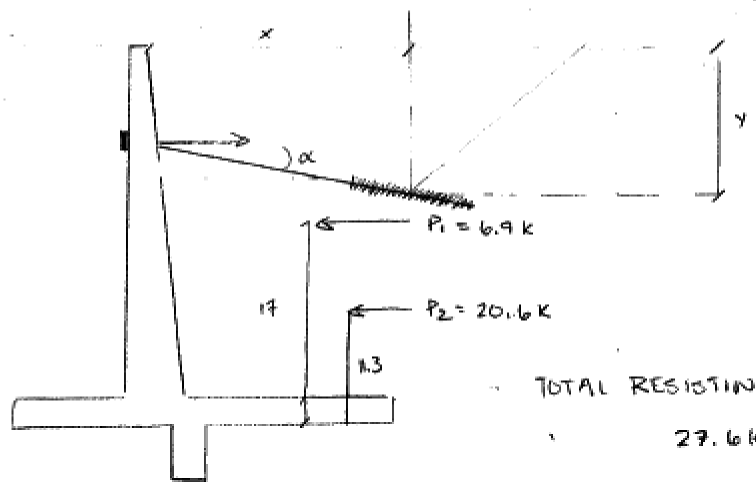
EXISTING PROPORTION P_1/P_2 =

$$\tfrac{1}{3}P_2(17) + P_2(11.3) < 350 \text{ ft}\cdot\text{K} \qquad 12.24/34.68 \approx 1/3$$

$$17P_2 < 350 \text{ ft}\cdot\text{K}$$

$$P_2 = 20.6 \text{ K} \quad \therefore \quad P_1 = 6.9 \text{ K} \qquad = 27.6 \text{ K} \;(1\text{f})$$

APPLY 1.3 FS TO SHEAR STRENGTH OF SOIL IN NEXT STEP (FINDING x, y LOCATION)



TOTAL RESISTING FORCE

27.6K

FIND ∡ OF FAILURE PLANE

27.6K PER LINEAR FOOT.

x	y	
48	19.5	$\alpha = 13^\circ$ 15ft
16.2	13	$\alpha = 48.01^\circ$
25.5	15	$\alpha = 32.11^\circ$

$\boxed{25.62' \quad 15' \quad \alpha = 32^\circ}$

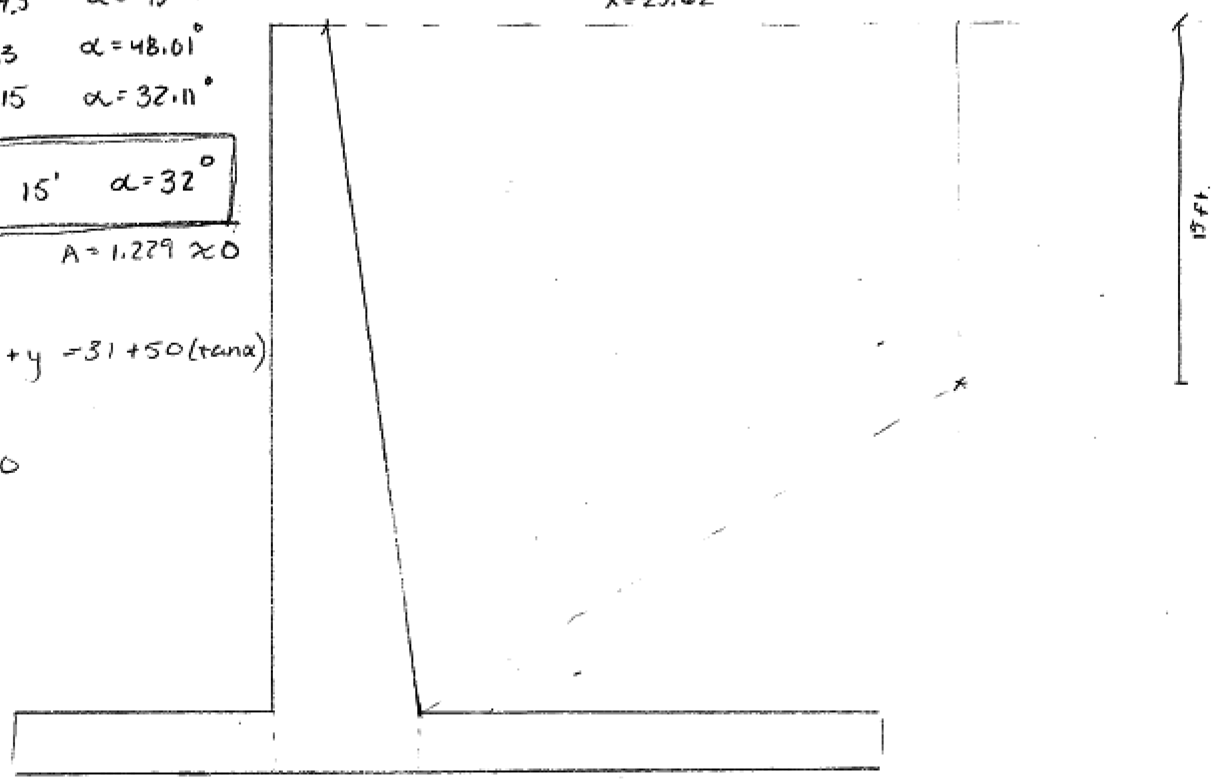
$A = 1.229 \approx 0$

FOR Y

$5y(\tan\alpha) + y = 31 + 50(\tan\alpha)$

FOR X

$x = 5y - 50$



TENSILE CAPACITY OF ANCHORS

USE FS = 1.3

REQUIRED TO RESIST 276 K IF PLACE / PER 10 FT

0.60 – 0.80 SMTS IS ACCEPTABLE

SMTS = SPECIFIED MIN. TENSILE STRENGTH OF PRESTRESSED STEEL.

"FOR LOSS OF SINGLE ANCHOR" → MAX DESIGN LOAD = 0.60 SMTS

FOR 10 FOOT SPACING →

$$SMTS = \frac{276\,K}{0.60} = 460\,K$$

STEEL → 150 KSI ALL-THREAD-BAR ASTM A722

NOMINAL BAR D	MIN. ULT. STRENGTH	MIN. YIELD STRENGTH
2-1/2"	778 KIPS	622 KIPS

ADJUST SPACING →

$$SMTS = 620\,K = \frac{27.6\,K \cdot x}{0.60}$$

$x = 13.47'$ ROUND DOWN

SPACING @ 12' O.C. TOTAL # = 25 TIE-BACKS.

$$\text{FINAL SMTS} = \frac{331\,K}{0.6} = 552 < 622$$ ☑

7 LOCATION OF RESULTANT & FOOTING SOIL PRESSURES

$R = 54.9$ KIPS

$$\bar{X} = \frac{\text{RESIST MOMENT} + \text{OT MOMENT}}{R} = \frac{(510.8) + (249.9)}{54.9} = 13.86 \text{ Ft.}$$

$$\text{OT MOMENT (NEW)} = \underbrace{(12.24 - 6.9)}_{5.34}^{P_1}(17) + \underbrace{(34.68 - 20.6)}_{14.08}^{P_2}(11.3)$$

$$90.8 + 159.1 = 249.9$$

$$e = 13.86 - \frac{22'}{2} = 2.86 \text{ Ft.}$$

$$\frac{6e}{\text{BASE } L} = \frac{6(2.86)}{22'} = 0.78 < 1.0$$

RESULTANT LIES JUST INSIDE MIDDLE THIRD.

$$R = \tfrac{1}{2}(p_{max})(\text{BASE } L)$$

$$54.9 = \tfrac{1}{2}(p_{max})(22) \longrightarrow p_{max} = 4.99 < 5 \text{ KSF} \ \boxed{\checkmark}$$

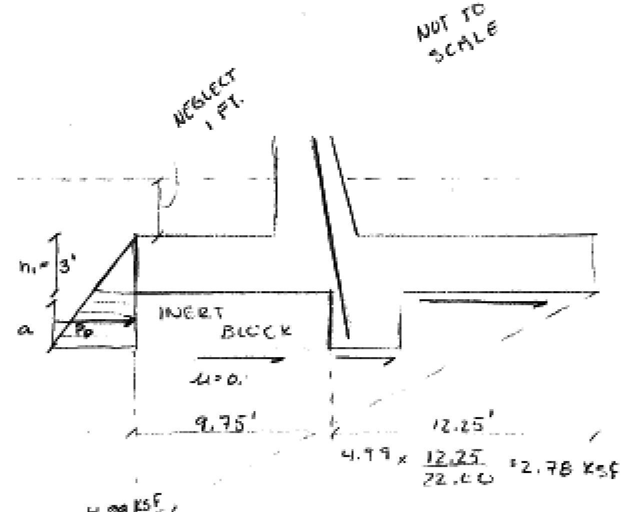
8 FS AGAINST SLIDING (NEGLECTING P_P)

FORCE CAUSING SLIDING $\rightarrow \overset{(NEW)}{P_1 + P_2} = 19.42$ K

FRICTIONAL FORCE $\rightarrow \mu R = 0.45(54.9) = 24.71$ K

$$FS = \frac{24.71^k}{19.42^k} = 1.27 < 2.0 \qquad \text{USE KEY}$$

*USING INERT BLOCK CONCEPT



4.99 KSF / SOIL PRESSURE DIAGRAM

EQUIV. FLUID PRESSURE = 60 pcf

$$P_p = \frac{0.06\,(h_1 + a)^2}{2} - \frac{0.06\,(h_1)^2}{2} \qquad \text{FOR } h_1 = 3\,ft$$

$$P_p = 0.03\,(3+a)^2 - 0.27$$

INERT BLOCK $\mu \rightarrow \tan\alpha = \tan 38 = 0.78$
REGULAR $\mu \rightarrow \mu = 0.45$

$$F = \mu_1 R_1 + \mu_2 R_2 \rightarrow 0.78(1/2)(4.99 + 2.78)(9.75) + 0.45(1/2)(2.78)(12.25)$$
$$29.545 + 7.662$$

$F = 37.21$ KIPS

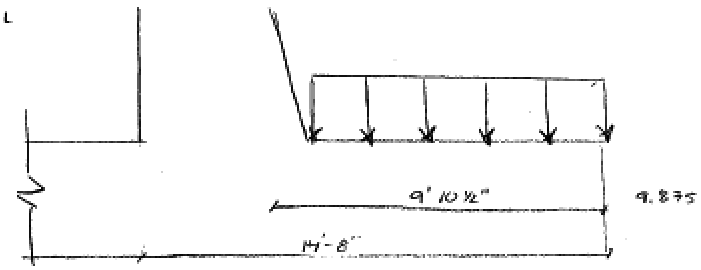
FORCE EQUILIBRIUM, w/ FS = 1.5

$$(P_1 + P_2)1.5 = P_p + F \rightarrow (49.42)(1.5) = P_p + 37.21$$

$$P_p = 37$$

NO KEY NECESSARY FOR FACTOR OF SAFETY OF 1.5

⑨ DESIGN OF HEEL
CATILEVER



$W_u = 1.2\,(W_{EARTH} + W_{FOOTING}) + 1.6\,(W_{SURCHARGE})$

$W(EARTH) = 0.140\,(30) = 4.2\ K/Ft$
$W(FOOT) = 0.140\,(3) = 0.42\ K/Ft$
$W(SURC) = 0.140\,(6) = 0.84\ K/Ft.$

$W_u = 1.2\,(4.2 + .42) + 1.6\,(0.84) = 6.89\ K/Ft.$

$M_{u\,(DOWNWARD)} = \frac{1}{2}(6.89)(9.875)^2 = 335.94\ Ft \cdot K.$

* NEGLECT UPWARD SOIL PRESSURE

FACTORED SHEAR @ FACE $\qquad 0.9\,(W_D) + 1.6\,(W_L)$

$V_u = (7.14)(9.66) = 69.02$ Kips

$\phi V_c = \phi\,(2\sqrt{f'c}\,)\,bd$

$= 0.75(2\sqrt{4000}\,)(12)(33.5)\frac{1}{1000} = 38.137\ K < 69.02\ K$ NG

SHEAR CONTROLS

REQUIRED $d = \frac{33.5\,(69.02)}{38} = 60.84$ IN.

USE HEEL THICKNESS → 64 IN.

FOR FLEX. STRENGTH. $\qquad M_u = 328.6\ ft \cdot k$ (CORRECTED FOR 64IN)

REQUIRED $R_N = \frac{M_u}{\phi b d^2} = \frac{328.6\,(12{,}000)}{0.9(12)(61.5)^2} = 96.5$ psi < 500 psi ☑

* INCREASING HEEL THICKNES TO 64 IN. CAN REDUCE STEM H AND CAN PERMIT REDUCING STEM THICKNESS. IT ALSO IMPROVES STABILITY AGAINST OVERTURNING,

FOR $R_N = 96.5$ psi $\longrightarrow \rho \approx 0.002$ FIG 3.8.1

$A_{s\,min} = \frac{3\sqrt{f'c}}{f_y}\,b_w d \rightarrow \rho_{min} = \frac{A_{s\,min}}{b_w d} = \frac{3(\sqrt{f'c})}{f_y}$ OR $\frac{200}{f_y}$

0.00379 OR $0.004 = \underline{0.004}$

REQ. $A_s = 0.004\,(12)(61.5) = 2.952\ in^2/Ft.$ $\boxed{\text{USE \#9 @ 4" o.c.}}$
$(A_s = 3.0\ in^2/Ft)$

DEVELOPMENT LENGTH. #9 BARS. @ 4 in LATERALLY

CLR SPCING $> 2d_b = 2.256$ & 2 in CLR CVR > 1.128 ☑ CAT A

FOR #9 BARS $f_y = 50$ ksi $f'_c = 4$ ksi

$L_d = 44.6 \rightarrow$ TOP BARS ($\lambda = 1.3$) = 58 in. SIMPLIFIED

$$l_d = \frac{3 d_b}{40} \cdot \frac{f_y}{\sqrt{f'_c}} \cdot \frac{\psi_t \psi_e \psi_s}{\left(\frac{c_b + K_{tr}}{d_b}\right)}$$

C_b = SMALLER OF
① CENTER OF BAR TO CONC. SURFACE
② C to C DISTANCE *

$$L_d = \frac{3(1.128)}{40} \cdot \frac{50{,}000}{\sqrt{4000}} \cdot \frac{1.3\,(1.0)(1.0)(1.0)}{2.0}$$

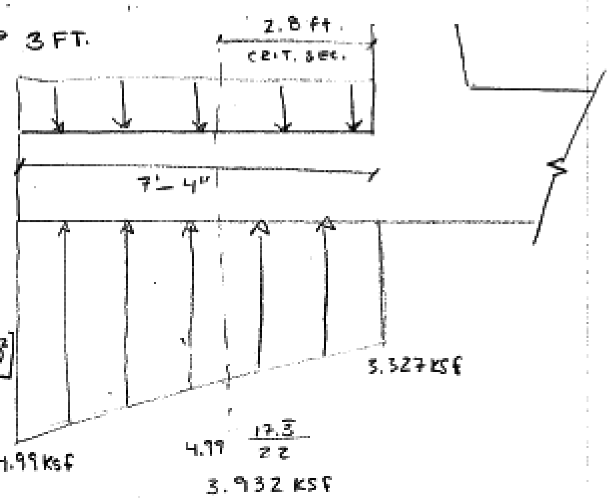
$L_d = 43.5$ in. $= 3.625$ FT. $\rightarrow$ 3' 7½" + 5"

$\boxed{\text{USE EMBEDMENT OF 4 FT. FROM BACK FACE OF WALL}}$

⑩ DESIGN OF TOE CANTILEVER

TOE MAY HAVE SMALLER THICKNESS THAN HEEL

USE 36 IN. $\rightarrow$ 3 FT.



$W = 0.140(3) = 0.42$ K/ft

$$V_u = 1.6\left(\frac{4.99 + 3.932}{2} - 0.42\right)(4.6)$$

$V_u = 18.9$ K

$$M_u = 1.6\left[\tfrac{1}{2}(4.99)(7.3)^2(\tfrac{2}{3}) + \tfrac{1}{2}(3.327)(7.3)^2(\tfrac{1}{3}) - \tfrac{1}{2}(.42)(7.5)^2\right]$$

$= 172.76$ ft·K

$$\text{REQ } R_N = \frac{172.76\,(12{,}000)}{0.90\,(12)\,(33.5)^2}$$

171.05 psi $\rightarrow \rho \approx 0.0035$

MIN $\rho = 1.33\,(0.0035) = 0.0047$

REQ $A_s = 0.0047(12)(33.5) = 1.89$ in²/ft.

$\boxed{\text{USE \#9 @ 6" } (A_s = 2.00\ \text{in}^2/\text{ft})}$

THE SHEAR STRENGTH FOR TOE

$\phi V_s = \phi (2\sqrt{f'_c}) bd = 0.75(2\sqrt{4000})(12)(33.5)\frac{1}{1000} = 38.14^k > 18.9^k$ OK ☑

NOT TOP BARS → EMBEDMENT

L_d (FOR #9) $= \frac{3 d_b}{40} \frac{f_y}{\sqrt{f'_c}} \frac{\psi_t \psi_e \psi_s \lambda}{2.5} = \frac{3(1.128)}{40} \frac{50{,}000}{\sqrt{4000}} \frac{1}{2.5}$

$L_d = 26.8$ IN. → USE EMBEDMENT OF 2.5 FT FROM FRONT FACE OF WALL

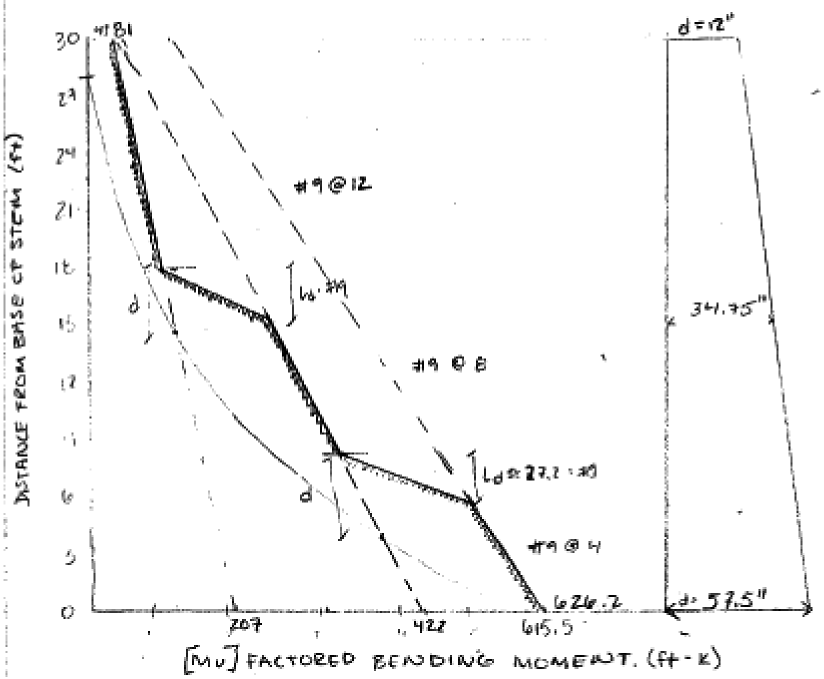
(11) REINFORCEMENT FOR WALL

- WALL IS NOW 2.5 ft. LESS THAN ORIGINAL CALCULATIONS.
- RETAIN 60 IN THICKNESS @ BASE STEM OF WALL

$M_U = 1.6\,[.180(28.7)^2 + 0.01(28.7)^3] = 615.5$ ft·k.

REQ $R_N = \frac{(615.5)(12{,}000)}{(0.90)(12)(57.5)^2} = 206.8$ psi

REQ $A_s = 0.004(12)(57.5) = 2.76$ in^2/foot.



USE #9 BARS @ 4" → $A_s = 3.00\ in^2/ft.$

$$\text{CLR SPACING} = \frac{4 - 1.128/2 - 1.0/2}{1.128} d_b = 3.04\ d_b$$

$$c_b = 2 + .0.564 = \frac{2.564\,in + K_{tr}}{d_b} = 2.27\ in$$

$$L_d\ (\text{for } \#11) = \frac{3 d_b}{40} \frac{f_y}{\sqrt{f'c}} \frac{1.6}{2.27} = 27.2\ \text{IN.}$$

$$L_d \text{ INTO FOOTING} = 27.2 \left(\frac{1.128}{2.5}\right) = 12.3\ \text{IN.}$$

USE AN EMBEDMENT OF 2 FT INTO THE FOOTING FOR #9

~~USE BASE KEY OF 1 FT (NECESSARY)~~

FOR #9 BARS @ 4 IN. MOMENT CAPACITY →

$C = 0.85 f'c\, ba = 0.85(4)(12)a = 40.8a$

$T = A_s f_y = 3.00(50) = 150$ $\qquad a = 3.68$ IN

@ TOP OF WALL

$$\phi M_n = 0.90(150)[(12) - 0.5(3.68)]\ 1/12 = 114.32\ ft\text{-}k$$

@ BOTTOM OF WALL

$$\phi M_n = 0.90(150)[57.5 - 0.5(3.68)]\ 1/12 = 626.18\ ft\text{-}k$$

TO FIND WHERE #9 BARS CAN BE CUT (EVERY OTHER)

$C = 40.8a$ $\quad T = 2.00(50) = 100$ $\qquad a = 2.45\ in$

ϕM_n @ TOP $= 0.9(100)[12 - (0.5 \times 2.45)]\ 1/12 = 80.8\ ft\text{-}k$

ϕM_n @ BOTTOM $= 0.9(100)[57.5 - (0.5 \times 2.45)]\ 1/12 = 422.06\ ft\text{-}k$

$C = 40.8\,a \quad T = 1.00(50) \quad a = 1.225$

ϕM_n @TOP $= 0.9(50)[12 - (0.5 \times 1.225)]\,1/12 = 42.7$ ft-K

ϕM_n @BOTTOM $= 0.9(50)[57.5 - (0.5 \times 1.225)]\,1/12 = 206.6$ ft·k

FOR AMPLE SHEAR PROTECTION CONTINUE W/ #9@12" UNTIL
CUT @ 1 ft FROM TOP

(12) TEMPERATURE & SHRINKAGE REINFORCEMENT.

TOTAL AMOUNT REQUIRED ACI-14.1.2 & 14.3.3

$A_s = 0.0025\,bh = 0.0025(12)(22.75) = 0.683\ in^2/ft.$

WHERE h = AVERAGE WALL THICKNESS

FRONT FACE → $2/3\,A_s = .453\ in^2/ft.$ #7 @ 12" $A_s = 0.60\ in^2$

REAR FACE → $1/3\,A_s = 0.23\ in^2/ft.$ #5 @ 12" $A_s = 0.31\ in^2$

FOR VERT. REINFORCEMENT US NOMINAL AMOUNT ADEQUATE ENOUGH TO SUPPORT HORIZ. TEMP. & SHRINKAGE STEEL ON FACE.

USE #5 @ 1 ft. 3 in. SPACING.

(13) DRAINAGE & OTHER DETAILS.

- WEEP HOLES (4 in DIAMETER) EVERY 10 ft. OF WALL.
- ADEQUATE DRAINAGE OF BACKFILL
- FOOTING PLACE FIRST THEN WALL W/ SHEAR KEY

(14) DESIGN SKETCH

CANTILEVER WALL - POOL DESIGN (DEEP END)

$f_y = 50$ KSI
$f'_c = 5$ KSI

ASSUME FILL = 140 pcf EQUIV. FLUID PRESSURE = 60 pcf

∢ INTERNAL FRICTION = 38° COEFF. FRICTION = 0.40

MAXIMUM SOIL PRESSURE - 5 KSF

DEEP END

① REINFORCEMENT RATIO $\rho = 25-35\%$ OF ρ_b

$\rho = 0.35 \cdot \rho_b = 0.35(0.0367) = 0.0129$

$R_N = \rho f_y (1 - \tfrac{1}{2}\rho m)$ $m = f_y / 0.85 f'_c = 11.76$

$R_N = (0.0129)(50{,}000)(1 - \tfrac{1}{2}(0.0129)(11.76))$

$R_N = 598.6$ psi

② HEIGHT OF WALL → DO NOT HAVE TO ACCOUNT FOR FOOTING TO BE BELOW FROSTLINE AS POOL IS @ INTERIOR OF BUILDING.

DEPTH OF POOL → 13.5 ft. (TO TOP OF FOOTING = 14.5')

③ FOOTING THICKNESS (7 TO 10 % h) = USE 1.45 ft FOOTING

* MAKE 1.5 ft THICK = 18"

④ BASE L
FACTOR 1.6 FOR EARTH PRESSURE & 0.9 FOR EARTH & CONC.

USING 1 ft. OF WALL LENGTH

$P_1 = \left(\frac{(2/3)(60\text{pcf})}{1000}\right)(16)(1) = 0.64$ Kips

$P_2 = \tfrac{1}{2}\left(\frac{(60\text{pcf})(16)}{1000}\right)(16)(1) = 15.3$ Kips

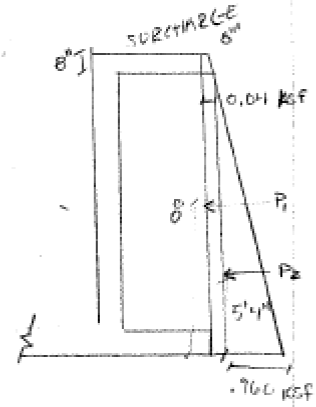
$W = .140(16 + \tfrac{2}{3})x = 2.\bar{3}x$ Kips

SUM MOMENTS @ STEM

$W(x/2) = (0.64)(8') + (15.3)(5.\bar{3}')$

$\frac{2.\bar{3}x^2}{2} = 86.72$ $x = 8.62$

BASE L = $1.5x$ = $\underline{\underline{13'}}$



Appendix O

Pool

CANTILEVER WALL - POOL DESIGN (DEEP END)

$f_y = 50$ KSI
$f'_c = 5$ KSI

ASSUME TILL = 140 pcf EQUIV. FLUID PRESSURE = 60 pcf

∢ INTERNAL FRICTION = 38° COEFF. FRICTION = 0.40

MAXIMUM SOIL PRESSURE - 5 KSF

$\boxed{\text{DEEP END}}$

① REINFORCEMENT RATIO ρ = 25-35% OF ρ_b

$\rho = 0.35 \rho_b = 0.35(0.0367) = 0.0129$

$R_N = \rho f_y (1 - \tfrac{1}{2}\rho m)$ $m = f_y / 0.85 f'_c = 11.76$

$R_N = (0.0129)(50{,}000)\left(1 - \tfrac{1}{2}(0.0129)(11.76)\right)$

$R_N = 598.6$ psi

② HEIGHT OF WALL → DO NOT HAVE TO ACCOUNT FOR FOOTING TO BE BELOW FROSTLINE AS POOL IS @ INTERIOR OF BUILDING.

DEPTH OF POOL → 13.5 ft. (TO TOP OF FOOTING = 14.5'

③ FOOTING THICKNESS (7 TO 10% h) = USE 1.45 ft FOOTING

* MAKE 1.5 ft THICK = 18"

④ BASE L
FACTOR 1.6 FOR EARTH PRESSURE & 0.9 FOR EARTH & CONC.

USING 1 ft. OF WALL LENGTH

$P_1 = \left(\frac{(2/3)(60\,pcf)}{1000}\right)(16)(1) = 0.64$ Kips

$P_2 = \tfrac{1}{2}\left(\frac{(60\,pcf)(16)}{1000}\right)(16)(1) = 15.3$ Kips

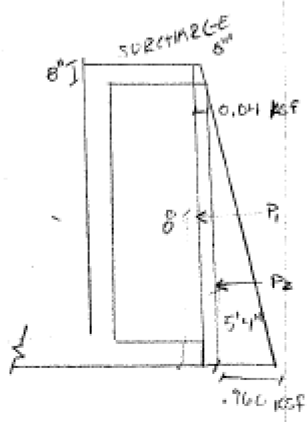
$W = .140(16 + 2/3)x = 2.3x$ Kips

SUM MOMENTS @ STEM

$W(x/2) = (0.64)(8') + (15.3)(5.3')$

$\frac{2.3x^2}{2} = 86.72$ $x = 8.62$

BASE L = $1.5x$ = $\underline{13'}$



⑤ STEM THICKNESS BY SHEAR

$V_y = 0.04y + 1/2(0.06)y^2 = 0.04y + 0.03y^2$ USE OL = 1.6

$V_u = 1.6[0.04(14.5) + 0.03(14.5)^2] = 11.02\ K$

$\phi V_c = \phi 2\sqrt{f'_c}\,bd = 0.75(2\sqrt{4000})(12)\times 18(\frac{1}{1000}) = 20.5\ K > 11.02\ K$ OK

USE THICKNESS = **18"**

⑥ OT

		ARM
$W_1 = (0.140)(16.67)(7.16) = 16.7\ K$		3.58
$W_2 = (0.140)(1.5)(14.5) = 3.05\ K$		7.92
$W_3 = (0.140)(13')(1.5) = 2.73\ K$		6.5
	22.48	

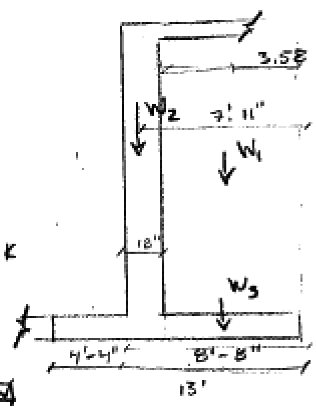
TOTAL M = 101.7 ft · K

RESULT FROM HEEL $= \frac{101.7}{22.48} = 4.52$ ft.

RESISTING M = 22.5 (13 − 4.52) = 190.8 ft − K

OT MOMENT $= P_1(8) + P_2(5.\bar{3})$
$= 0.64(8) + 15.3(5.\bar{3})$
$= 86.72$ ft − K

FS AGAINST OT $= \frac{190.8}{86.72} = 2.2 > 2.0$ ☑



⑦ LOCATION OF RESULTANT & FOOTING PRESSURES

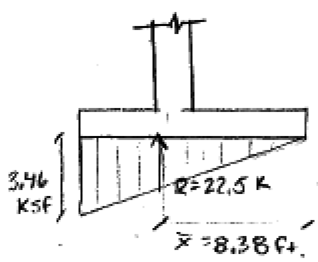
R = 22.48 k $\bar{x} = \frac{101.7 + 86.72}{22.48} = 8.38$ ft

$e = 8.38 - \frac{13}{2} = 1.88$ ft. $\frac{6e}{BASE} = \frac{6(1.88)}{13} = 0.8677 < 1.0$ ☑

RESULTANT LIES INSIDE MID THIRD.

$R = 1/2\, p_{max}$ (BASE L) $22.48 = 1/2\, p_{max}\,(13)$

$p_{max} = 3.46$ ksf < 5 ksf ☑



⑧ SLIDING

$P_1 + P_2 = 15.94\ K \qquad \mu R = 0.45(22.48) = 10.116\ K$

$FS = \frac{10.116}{15.94} = 0.635 < 1.5$

BECAUSE SLAB @ BASE OF POOL WILL SERVE AS A CONTINUATION OF FOOTING, KEY IS NOT NECESSARY TO ACHIEVE A SAFETY FACTOR OF 1.5.

⑨ DESIGN OF HEEL CANTILEVER

$w_u = 1.2(2.03 + .21) + 1.6(.093) = 2.84\ K/ft.$

$M_u = \frac{1}{2}(2.84)(7.1\bar{6})^2 = 72.8\ ft \cdot K.$

$V_u = (2.165)(7.16) = 15.5\ K.$

$\phi V_c = \phi 2\sqrt{f'_c}\, bd \rightarrow 0.75(2\sqrt{4000})(12)(18)\frac{1}{1000} = 20.5\ K > 15.5$ ☑

$A_{s\,MIN} = \frac{3\sqrt{f'_c}}{f_y} b_w d \quad \rightarrow \rho_{min} = \frac{200}{f_y} = 0.004$

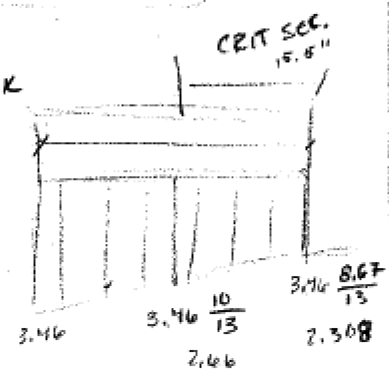
$REQ\ A_s = 0.004(12)(15.5) = 0.744\ in^2/ft.$

USE # 6 @ 6 $\quad A_s = 0.88\ in^2/ft \qquad c_b = 2 + 0.375 = 2.375$

$L_d\ (FOR\ \#6) = \frac{3(.75)}{40}\ \frac{56{,}000}{\sqrt{4000}}\ \frac{1.3}{2.375} = 24.34\ IN. \approx 2\ ft$

USE EMBEDMENT OF 2 Ft. FROM BACK OF WALL

⑩ DESIGN OF TOE CANTILEVER.



$V_u = 1.6\left(\frac{3.46 + 2.66}{2} - 0.21\right)(1.29) = 5.67\ K$

$M_u = 1.6\left[\frac{1}{2}(3.46)(4.\bar{3})^2(\frac{2}{3}) + \frac{1}{2}(2.308)(4.3)^2(\frac{1}{3}) - \frac{1}{2}(0.21)(4.\bar{3})^2\right] = 26.9\ ft\text{-}K.$

$Req.\ R_N = \frac{26.9(12000)}{0.9(12)(15.5)^2} = 124.41\ psi$

$\rho_{min} = 0.004$

$Req.\ A_s = 0.004(12)(15.5) = 0.744\ in^2/ft$

USE #6 @ 6

$L_d = \frac{24.34}{1.3} = 18.7\ IN$

USE EMBEDMENT OF 19 IN FROM FRONT FACE OF WALL

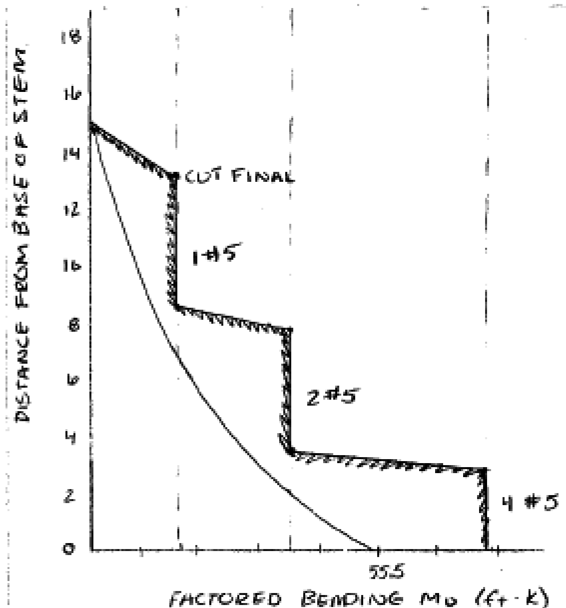
(11) REINFORCEMENT FOR WALL

$$M_y = \frac{0.04y^2}{2} + \frac{0.66y^3}{6} = 0.02y^2 + 0.01y^3$$

$$M_U = 1.6\left[0.02(14.5)^2 + 0.01(14.5)^3\right] = 55.5 \text{ ft}\cdot\text{K}$$

$$\text{Req. } R_N = \frac{55.5(12{,}000)}{0.9(12)(15.5)^2} = 256.7 \text{ psi}$$

$$\text{Req } A_S = 0.005(12)(15.5) = .93 \text{ in}^2/\text{ft}$$



$C = 46.8a$ $a = 1.52$ in

$T = (1.24)(50) = 62$

$$\phi M_N = 0.9(62)\left[15.5 - \tfrac{1}{2}(1.52)\right]\tfrac{1}{12} = 68 \text{ ft}\cdot\text{K}$$

USE #5 @ 3 ($A_S = 1.24$)

$$C_b = \frac{2.31}{.625} = 3.7$$

$$L_d = \frac{3(0.625)}{40}\,\frac{50000}{\sqrt{4000}}\,\frac{1}{3.7} = 10''$$

USE EMBEDMENT OF 1 FT. INTO FOOTING

TO CUT TWO #5 → $A_S = 0.63$ in²/ft

$C = 0.85 f'_c ba = 40.8a$ $T = A_S f_y = 0.62(50) = 31.$

$a = .76$ in

$$\phi M_N = 0.9(31)\left[15.5 - 0.5(.76)\right]\frac{1}{12} = 35.2 \text{ ft}\cdot\text{K}$$

TO CUT ONE #5 → $A_S' = 0.31$ in²/ft #5 @ 12"

$C = 40.8a$

$T = 0.31(50) = 15.5$

$a = 0.3799$ in

$$\phi M_N = 0.9(15.5)\left[15.5 - 0.5(.38)\right]\tfrac{1}{12} = 17.8 \text{ ft}\cdot\text{k}$$

(12) TEMP & SHRINKAGE

$A_s = 0.0025\,bh = 0.0025(12)(18) = 0.54\ in^2/ft.$

$2/3\,A_s = 0.36\ in^2/ft.$ [FRONT FACE #5 @ 10] $A_s = 0.37\ in^2/ft$

$1/3\,A_s = 0.78\ in^2/ft.$ [REAR FACE #4 @ 12] $A_s = 0.20\ in^2/ft$

* FOR VERT. REINF @ FRONT FACE <u>#5 @ 1.5'</u>
TO SUPPORT TEMP & SHRINK.

CANTILEVER POOL WALL $\boxed{\text{SHALLOW END}}$

① REINF. RATIO $\rho = 0.35(0.0367) = 0.0129$

$R_N = 598.6\ psi$

② HEIGHT OF WALL → TOP OF FOOTING @ 8' (7' POOL)

③ FOOTING THICKNESS → 10% h = 10 IN FOOTING

④ BASE L 1 FT WALL

5" SLAB $P_1 = \left(\frac{2/3 \cdot 60}{1000}\right)(8.83)(1) = .353$ kips

$P_2 = 1/2\left(\frac{(60pcf)(8.83)}{1000}\right)(8.83)(1) = 2.34$ kips

$w = 0.140(8.83 + 2/3)x = 1.33x$

$w(x/2) = 0.353(4.42) + (2.34)(2.94) = \frac{1.33x^2}{2}$ $x = 3.6$ in

BASE L = 1.5 x = 5.34 → BASE L = $\underline{\underline{6\ ft.}}$

⑤ STEM THICKNESS BY SHEAR

$v_y = 0.04y + 1/2(0.06)y^2$

$v_u = 1.6[0.04(8) + 0.03(8)^2] = 3.58\ k$

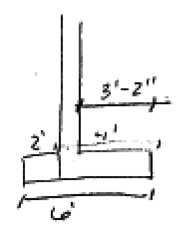
$\phi v_c = 0.75(2\sqrt{4000})(12) \times 12 \frac{1}{1000} = 13.66\ k > 3.58\ k$ TOO MUCH

$\phi v_c = 0.75(2\sqrt{4000})(12)(10)\frac{1}{1000} = 11.38\ k > 3.58\ k$ ☑

USE MIN. $\underline{\underline{10''}}$

⑥ OT

		ARM
$W_1 = (0.14)(8.67)(3.167) =$	$3.844\ k$	$1.58\bar{3}$
$W_2 = (0.14)(0.83)(8) =$	$0.933\ k$	$3.58\bar{3}$
$W_3 = (0.14)(6)(0.83) =$	$0.697\ k$	3.00
	$5.474\ k$	



$M = 11.519\ ft \cdot k$

RES. FROM HEEL = $\frac{11.519}{5.474} = 2.10$

RESIST. M $= 5.474(6 - 2.10) = 21.35\ ft \cdot k$

OT M $= (0.353)(4.42) + (2.34)(2.94) = 8.439$

SF AGAINST OT = $\frac{21.35}{8.44} = 2.53 > 2.0$ ☑

⑦ LOCATION OF RESULTANT & FOOTING PRESSURES

$R = 5.474\ k$ $\bar{x} = \frac{11.59 + 8.44}{5.474} = 3.7$ ft.

$e = 3.7 - \frac{6}{2} = 0.66\ ft$ $\frac{6(0.66)}{6} = 0.66 < 1.0$ ☑

$R = 1/2\ p_{max}$ (BASE L) $5.475 = 1/2\ p_{max}(6)$ $p_{max} = 1.825\ KSF$

⑧ SLIDING → CAN BE IGNORED BECAUSE OF SLAB

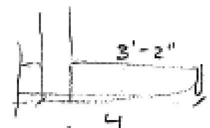
⑨ DESIGN OF HEEL CANTILEVER

$W_U = 1.2(1.12 + 0.12) + 1.6(1.21) = 3.42$ k/ft

$M_U = \frac{1}{2}(3.42)(3.167)^2 = 17.15$ ft-k

$V_U = (3.052)(3.167) = 9.67$ k

$\phi V_C = 0.75(2\sqrt{4000})(12)(10)\frac{1}{1000} = 11.38\text{ K} > 9.67$ ☑



A_S REQ $= 0.004(12)(7.5) = 0.36\ in^2/ft/ft$

USE #4 @ 6 $A_S = 0.40\ in^2$ $C_D = 2 + .25 = 2.25$

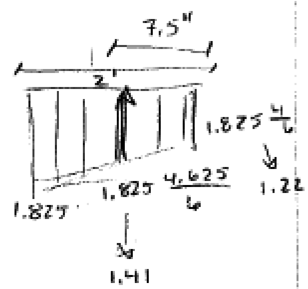
L_d FOR #4 $= \frac{(3)(.500)}{40}\frac{50{,}000}{\sqrt{4000}}\frac{1.3}{2.25} = 17.13$ in

$\boxed{\text{USE EMBEDMENT OF } 1\frac{1}{2} \text{ ft FROM BACK OF WALL}}$

⑩ DESIGN OF TOE CANTILEVER

$V_U = 1.6\left(\frac{1.83 + 1.41}{2} - 0.12\right)(1.38) = 3.31$ K

$M_U = 1.6\left[\frac{1}{2}(1.83)(1.38)^2(\frac{2}{3}) + \frac{1}{2}(1.22)(1.38)^2(\frac{1}{3}) - \frac{1}{2}(.12)(1.38)^2\right] = 2.3$ ft-k



Req $R_N = \frac{(2.3)(12{,}000)}{0.9(12)(7.5)^2} = 45.43$

$\rho_{min} = 0.004$

Req $A_S = 0.004(12)(7.5) = 0.36 in^2$

USE #4 @ 6 $A_S = .40\ in^2$

L_d FOR #4 $= \frac{17.13\text{ in}}{1.3} = 13.17$ in

$\boxed{\text{USE EMBEDMENT OF } 1\frac{1}{4} \text{ ft FROM FRONT OF WALL}}$

⑪ REINFORCEMENT FOR WALL

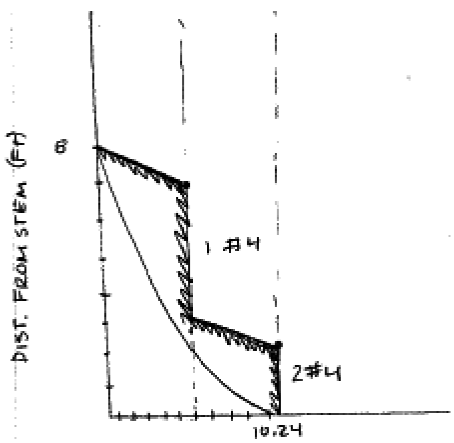
$M_y = \frac{0.04y^2}{2} + \frac{0.06y^3}{6} = 0.02y^2 + 0.01y^3$

$M_u = 1.6[0.02(8)^2 + 0.01(8)^3] = 10.24$ ft-k

Req. $R_N = \frac{(10.24)(12000)}{0.9(12)(7.5)^2} = 202.3$

Req. $A_s = 0.0041(12)(7.5) = 0.36$ in²/ft WIDTH

USE #4 @ 6 in. $A_s = 0.40$ in²/ft



$C = 40.8a$ $a = .49$ in

$T = (0.40)(50) = 20$

$\phi M_N = 0.9(20)[7.5 - 1/2(.49)]\ 1/12$

$\phi M_N = 10.88$ ft-k ☑

CUT ONE #4 $A_s = 0.20$ in²

$C = 40.8a$ $a = 0.27$ in

$T = (0.20)(50) = 10$

$\phi M_N = 0.9(10)[7.5 - 1/2(.27)]\ 1/12$

$\phi M_N = 6.08$ ft-k

CUT FINAL #4 @ 1 ft. FROM TOP

$C_b = \frac{2.25}{0.5} = 4.5$ $L_d = \frac{3(.5)}{40} \frac{50000}{\sqrt{4000}} \frac{1}{4.5} = 6.6$ in

$\boxed{\text{USE EMBEDMENT OF 8 in INTO FOOTING}}$

⑫ TEMP & SHRINKAGE

$A_s = 0.0025\,bh = 0.0025(12)(10) = 0.30$ in²/ft

2/3 A_s = 0.2 in²/ft FRONT FACE #3 @ 6 $A_s = 0.22$ in²

1/3 A_s = 0.1 in²/ft REAR FACE #3 @ 12 $A_s = 0.11$ in²

FOR VERT REINF @ FRONT FACE #4 @ 2'
TO SUPPORT TEMP & SHRINK.

Slab on Grade for Pool

SLAB ON GRADE DESIGN FOR POOL

IN THIS CASE LOADS ARE ASSUMED TO BE UNIFORM FOR EACH OF THE 3 SECTIONS OF SLAB

- SLOPED SECTION WILL BE DESIGNED THE SAME AS THE DEEP END TO BE CONSERVATIVE

$\boxed{\text{UNIFORM LOAD @ DEEP END}}$

GUNITE POOL = 150 pcf × 1 ft = 150 psf

+ H_2O PRESSURE = 62.4 pcf × 13.5 ft = 842.4 psf

992.4 psf

f_y = 50 ksi

f'_c = 4 ksi

MOR = $7.5\sqrt{f'_c}$ = 474.3 psi

USE A SAFETY FACTOR OF 1.7

ALLOWABLE BENDING STRESS = 279 psi

- THIS ALLOWABLE STRESS IS NOT NEEDED BECAUSE UNIFORM LOADING CAUSES NO BENDING STRESSES.
- SLOPED SECTION IS GRADUAL AND HAS A CHANGE IN LOAD THAT CAN BE NEGLECTED FOR BENDING STRESSES.

SLAB THICKNESS – TRY <u>8 IN</u> FOR DEEP END

JOINT SPACING – TOTAL SLAB AREA = 37'-8" × 51'-4"

CONTRACTION/CONTROL

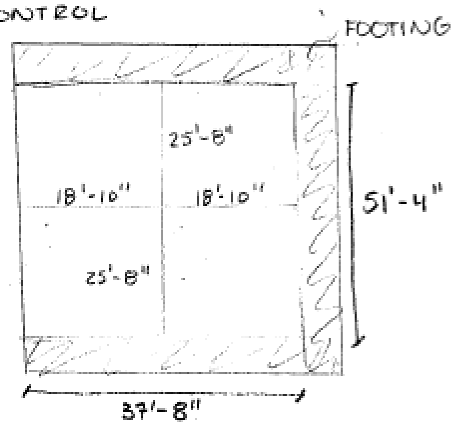
@ 18'-10" FROM END FOOTING

AND

@ 25'8" FROM SIDE FOOTINGS

AND @ FOOTING

AND @ BEGINNING OF SLOPE



SUBGRADE DRAG

~ CONSERVATIVE

$$A_s = \frac{FLW}{2(f_s)} = \frac{(1.5)(37.6)(100psf)}{2\ (2/3 \cdot 50{,}000)} = \underline{\underline{0.084}}\ in^2/ft/ft.$$

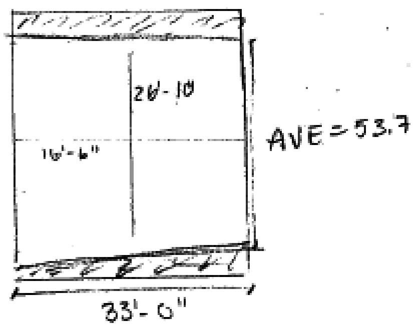
- USE WWF 2 in. BELOW TOP OF SLAB. to CENTER OF SLAB.
- KEEP CONTINUOUS @ CONTROL / CONTRACTION JOINTS

WWF →

ROLL		A_s (in^2/ft)	Wt. (lb/100sf)
4x4 - W2.9 x W2.9	(4x4 - 6x6)	0.087	62.

$\boxed{\text{SLOPE}}$

CONTROL JOINTS
@ 16'-6" FROM START & END OF INCLINE
AND @ TOP OF INCLINE
AND @ 25'-8" FROM SIDE FOOTINGS.

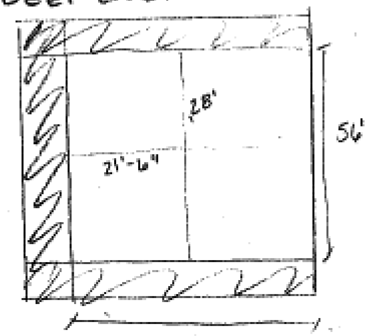


$$A_s = \frac{FLW}{2(f_s)} = \frac{(1.5)(33)(100)}{2\ (33{,}333.3)} = 0.743\ in^2/ft\ /\ ft\ \text{OF WIDTH}$$

WWF USE SAME ROLL AS DEEP END. CONTINE @ JOINTS

$\boxed{\text{SHALLOW}}$

CONTROL JOINTS
@ 28' FROM SIDE FOOTINGS
AND
@ 21'-6" FROM END FOOTING



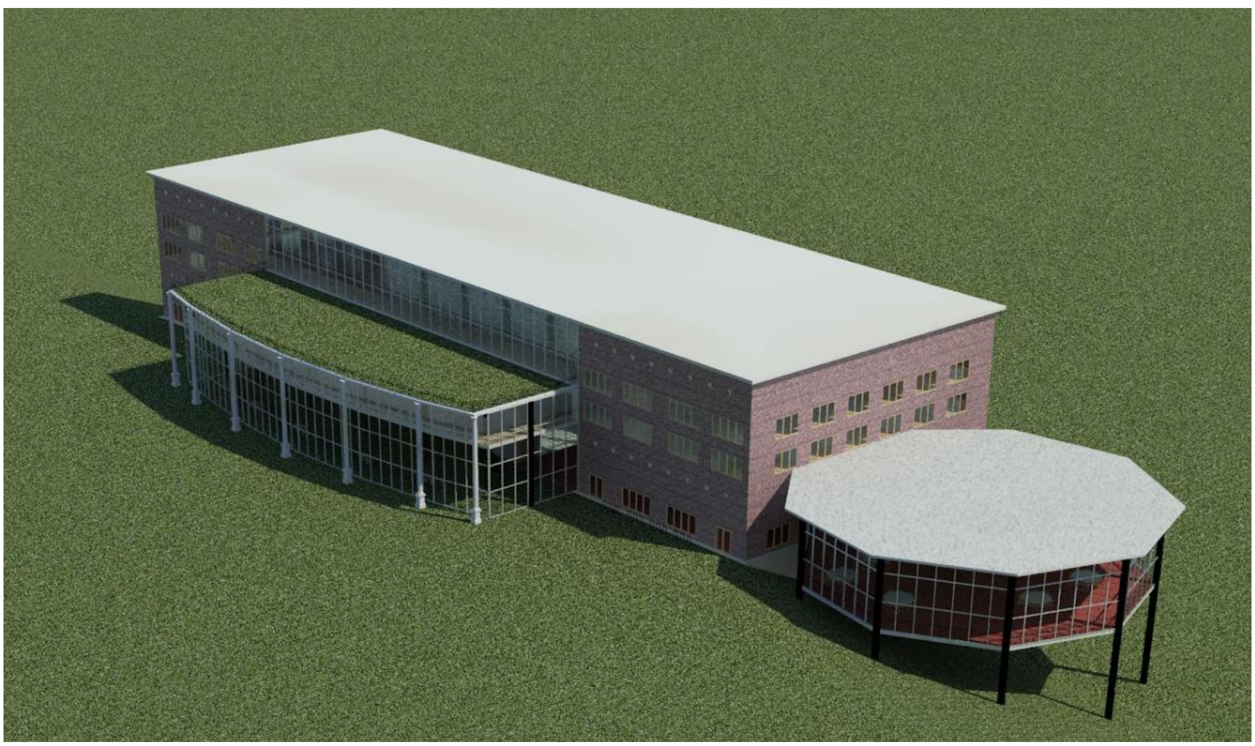
$$A_s = \frac{FLW}{2(f_s)} = \frac{(1.5)(43)(100)}{2(2/3 \cdot 50000)} = 0.073\ in^2/ft$$

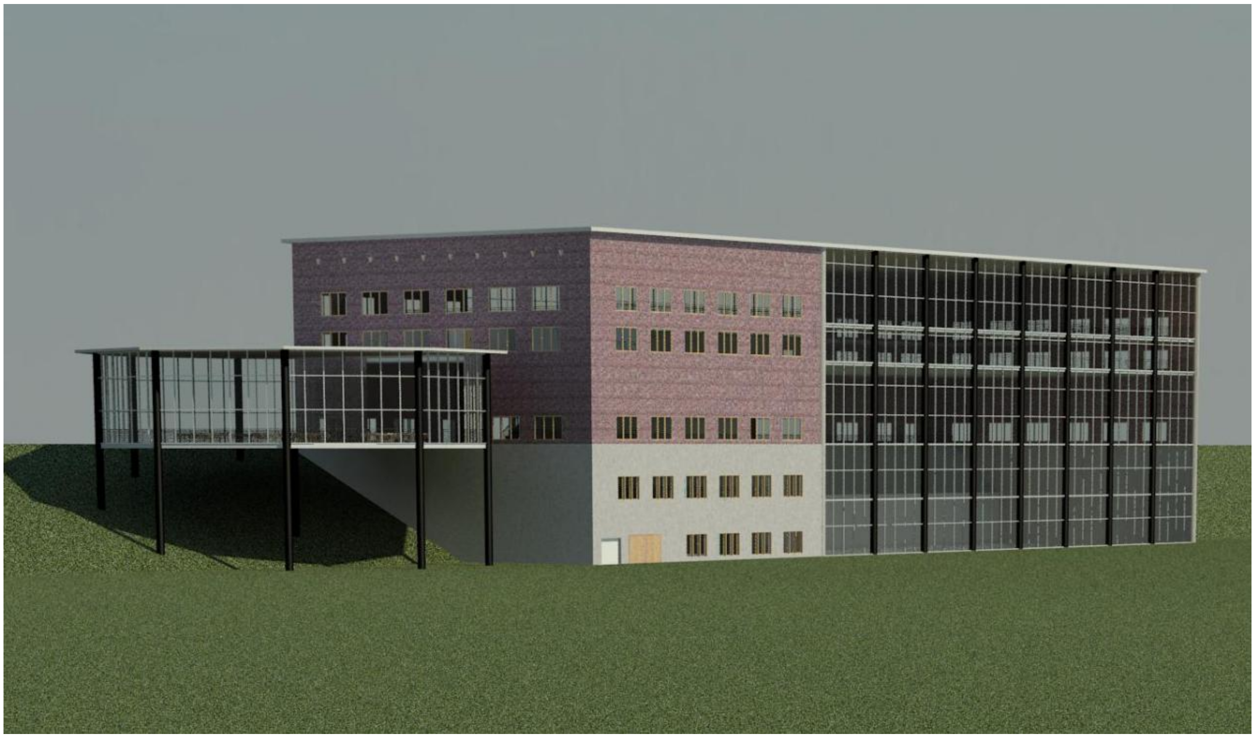
43'

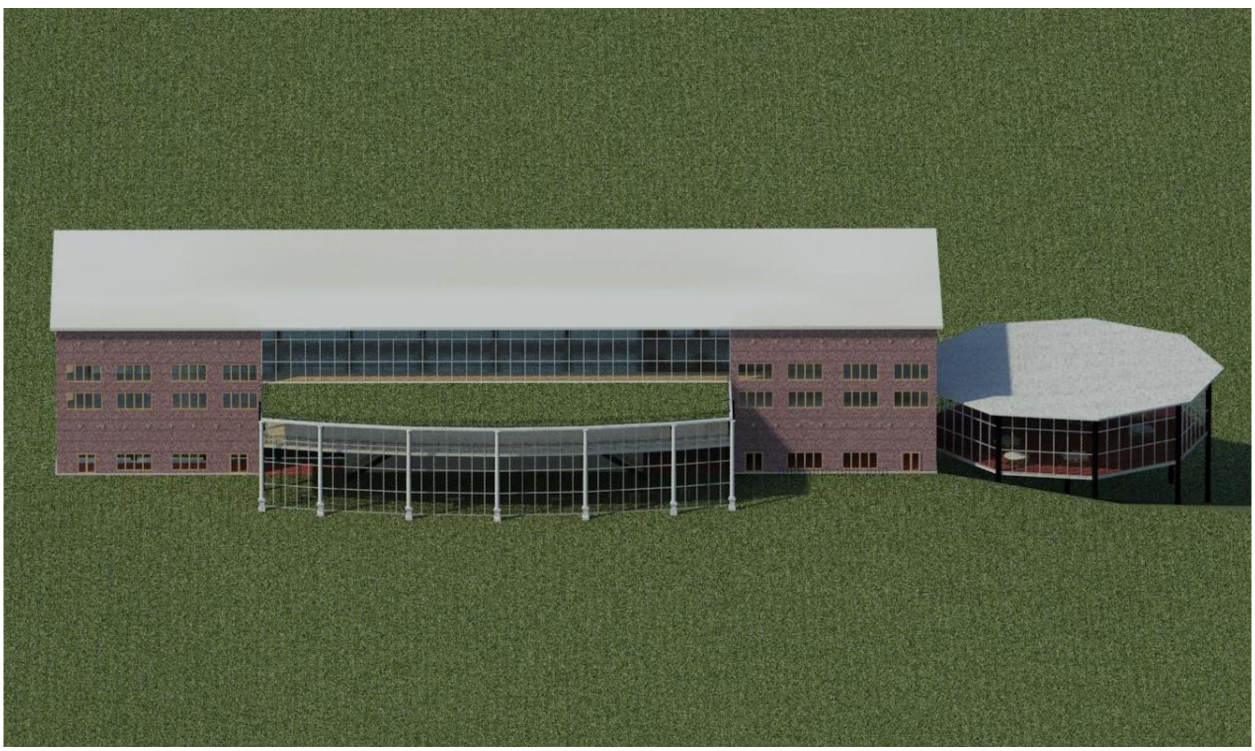
ROLL		As/ft	wt/100 sf
4x4 - W2.9 x W2.9	(4x4 - 6x6)	0.087	62 lbs

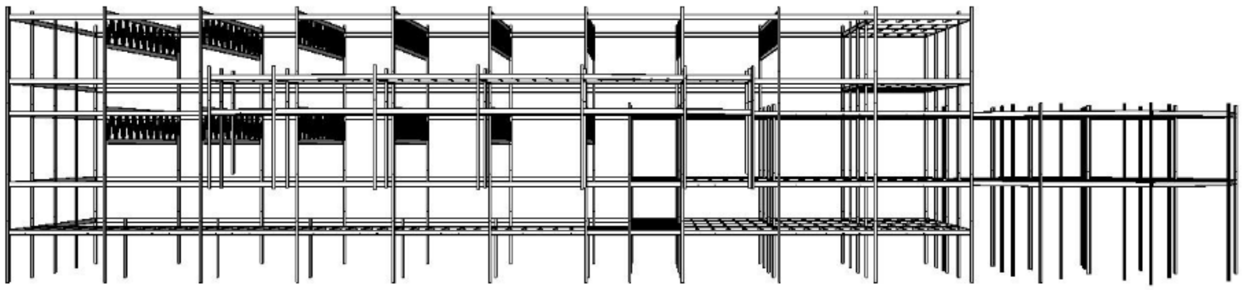
Appendix P

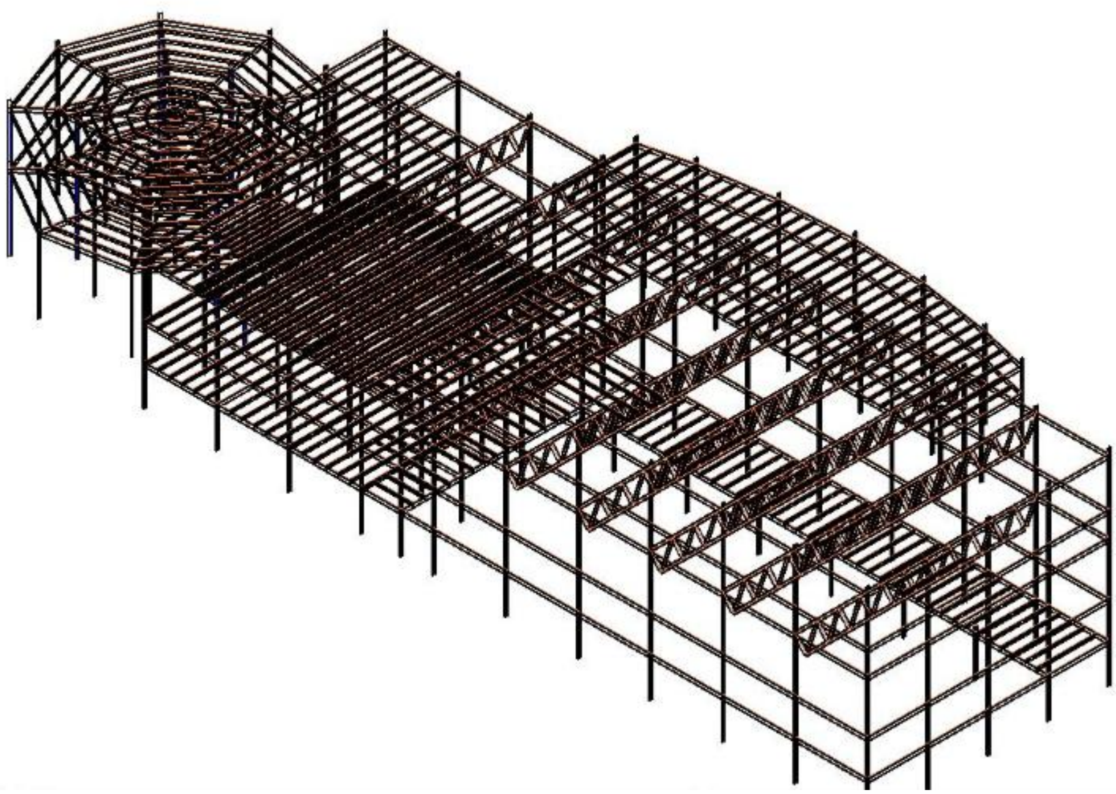
Revit Drawings

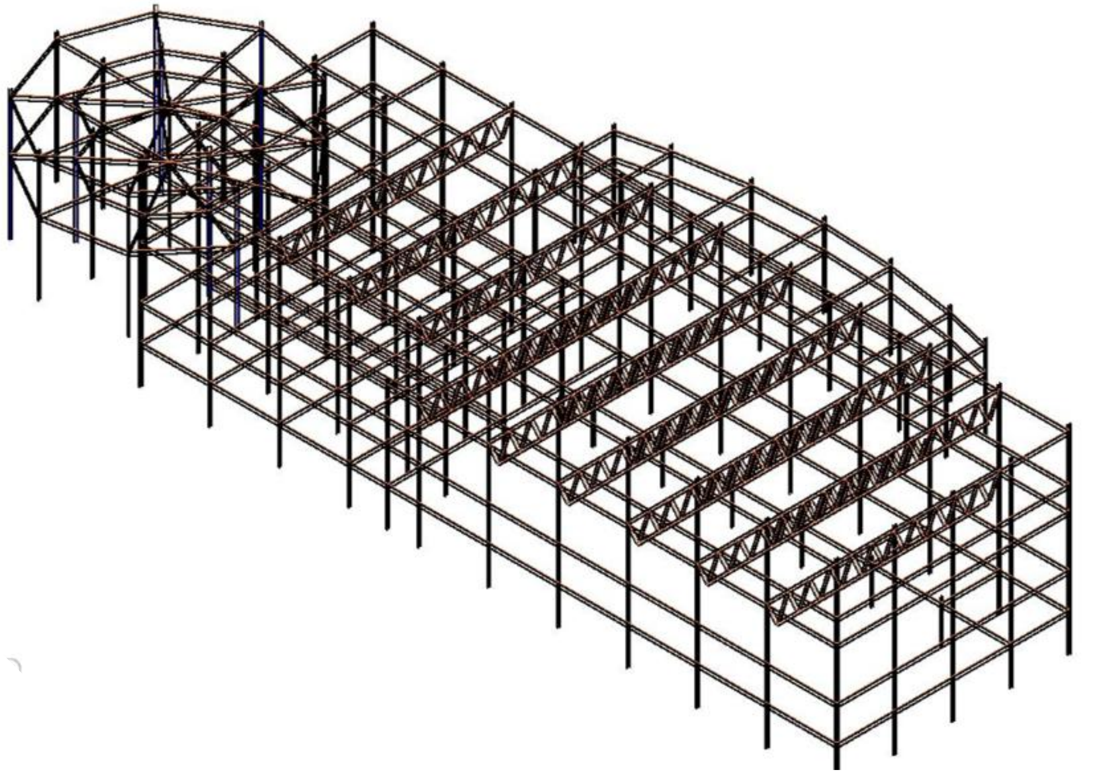


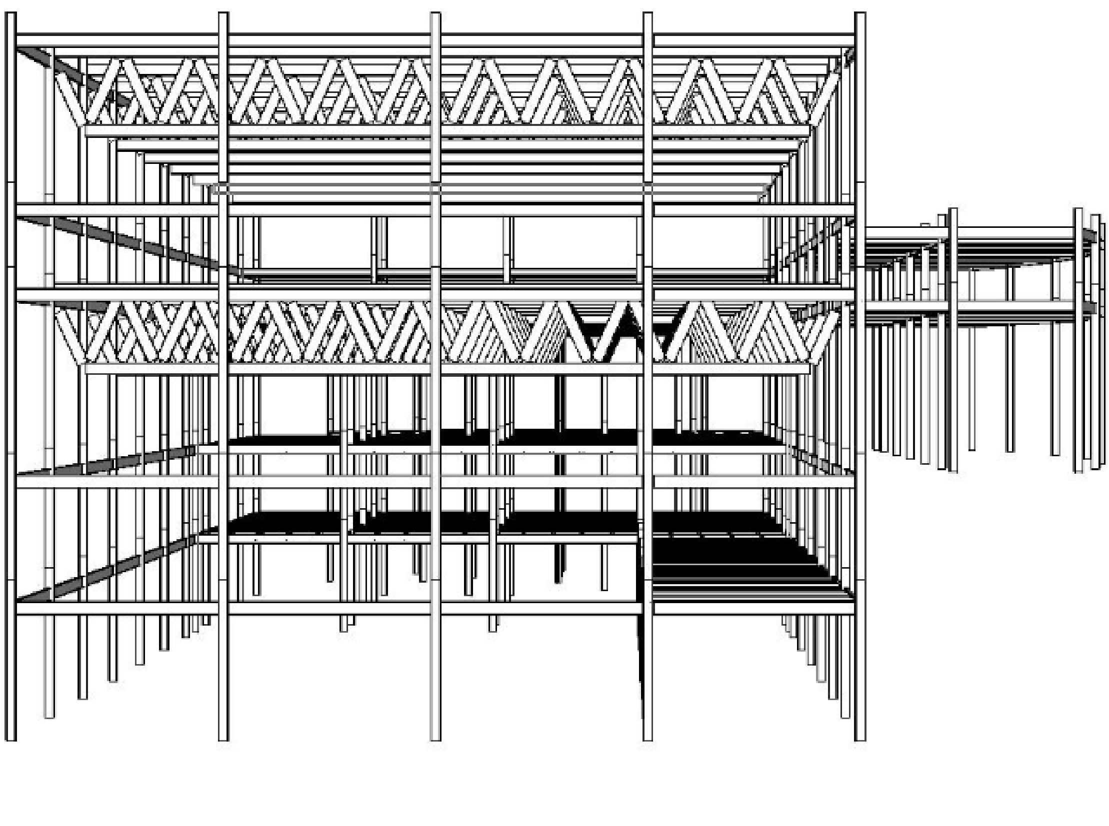


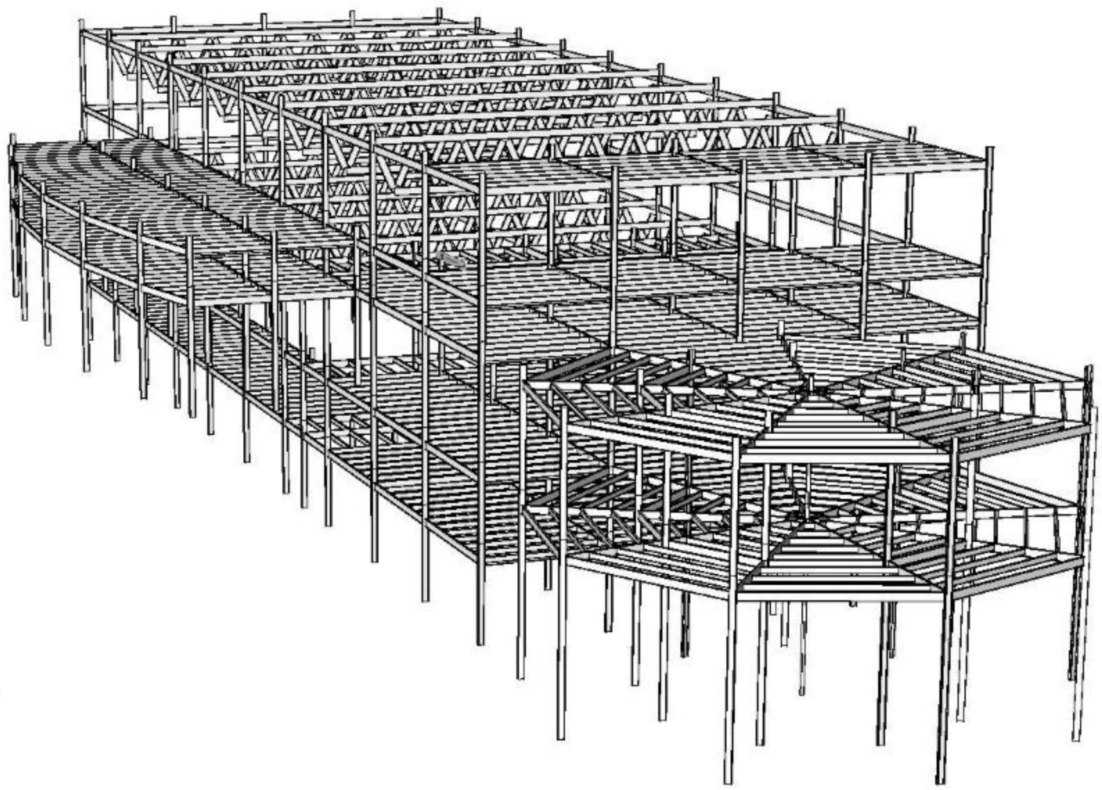












Appendix Q

Rebar Spreadsheet

East Wall						
		#/foot	total feet	Ld	Length	LF
Rebar for wall						
vertical reinf	#9	3	312	2	8	9360
	#9	2.3333	312	0	10	7279.896
	#9	1	312	0	9.7	3026.4
horiz	#7	1	31	0	312	9672
	#5	1	31	0	312	9672
vert.	#5	0.8	31	0	312	7737.6
Rebar for footing		#/foot	total feet	Ld	Length	LF
heel	#9 bars	4	312	4	14.7	23337.6
toe	#9 bars	2	312	2.5	7.3	6115.2

Deep End Wall							w/ sloped
		#/foot	total feet	Ld	Length	LF	
Rebar for deep end wall							
Vertical	#5	4.00	144	1	3.5	2592	5184
	#5	2.00	144	0	5	1440	2880
	#5	1.00	144	0	5	720	1440
horiz	#5	1.20	14.5	0	144	2505.6	5011.2
	#4	1.00	14.5	0	144	2088	4176
vert						0	0
	#5	0.67	14.5	0	144	1392.007	2784.01392
For footing						0	0
	#6	2.00	144	2	8.7	3081.6	6163.2
	#6	2.00	144	1.5	4.3	1670.4	3340.8

Shallow						
		#/foot	total feet	Ld	Length	LF
Rebar for deep end wall						
Vertical	#4	2.00	150	0.66667	3	1100.001
	#4	1.00	150	0	4	600
horiz	#3	2.00	8	0	150	2400
	#3	1.00	8	0	150	1200
vert						0
	#4	0.50	8	0	150	600
For footing						0
	#4	2.00	150	1.5	4	1650
	#4	2.00	150	1.25	2	975

Rebar for footings	quantity	# of bars	Lin. Ft.	Total LF of Bars	Total # of Bars
14 # 9 bars for 9'x9'x3' footing	22	308	8.5	2618	130.9
16 # 10 bars for 13'x13'x3' footing	19	304	12.5	3800	190
21 # 11 bars for 15'x15'x4' footing	5	105	14.5	1522.5	76.125
				Total	397.025

Bar	LF	lb/ft	lbs
#3	3600	0.376	1353.6
#4	9101.001	0.668	6079.469
#5	34708.81	1.043	36201.29
#6	9504	1.502	14275.01
#7	9672	2.044	19769.57
#9	49119.1	3.4	167004.9
Total Lf	115704.9		

Total # of bars (pool & retaining wall)	5785.246
Total # of bars (footings)	397.025

Total # of Bars 6182.271