

Farquhar Park Aquatic Center

York, PA



Final Report

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Structural Option

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Farquhar Park Aquatic Center

York, Pennsylvania

Project Information

Occupancy - Assembly
Size - 36,987 SF
Stories - One story (53'-0") with raised seating
- Entrance, concourse, and gallery level
Construction Dates - Project Not Constructed
Cost - \$13 million
Delivery Method - Design-Bid-Build

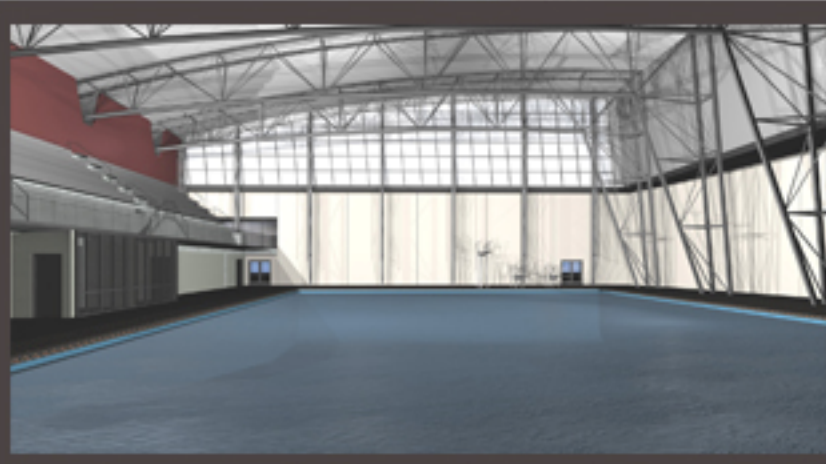
Design and Construction Team

Owner - YMCA of York and York County
General Contractor - N/A (Not Constructed)
Architect - Nutec Design Associates, Inc.
Structural - Nutec Design Associates, Inc.
MEP - JDB Engineering, Inc.
Geotechnical - GTS Technologies, Inc.
Fire Protection - JDB Engineering, Inc.



Architecture

- Multi-level, state-of-the-art natatorium complex
- 53-foot high natatorium with raised seating
- 12-foot deep indoor swimming pool
- Vast lobby with 23'-0" ceiling height
- Precast concrete ramp to upper level
- Facade: metal wall panels, precast concrete panels, and glazed curtain walls
- Two large sloped/curved glazed aluminum curtain walls enclosing indoor pool area (each 123'-11" long and 21'-0" tall at highest point)
- Standing seam metal roof panels
- Plant climbing system on outside faces of exterior walls
- Designed to achieve LEED Silver Certification



MEP

- Two large PoolPak dehumidification units
- Dedicated ventilation unit with heat recovery for the locker and toilet areas
- 12,000 CFM rooftop unit with a dehumidification cycle, including hot-gas reheat, for the entrance and eating areas
- Two air cooled condensers with 6 fans each
- Primary service is 13.2 kV, 3 phase, 3 wire
- Main distribution panelboard 277/480 V, 3 phase, 4 wire, 1200A
- Sound and paging system
- Lighting includes incandescent, fluorescent, and HID fixtures with occupancy sensors

Structural System

- Triangular HSS trusses spanning 130'-0"
- Columns for trusses are triangular, tapered, and spaced 30'-0" on center
- Long span deck to span between trusses
- HSS wind column trusses, 3'-0" deep, 51'-0" tall
- HSS columns supporting upper levels vary from HSS6x6 to HSS18x18
- Precast concrete grandstand supported by sloped W27 beams and HSS columns
- 12" hollow core precast concrete planks with 2" lightweight concrete topping supported by W-shape beams at concourse level
- Lobby HSS trusses spanning about 41'-0" and spaced 15'-0" on center
- Isolated spread footings at various depths

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Executive Summary

This thesis study investigated a redesign of the entire roof structural system of the Farquhar Park Aquatic Center. The original design for the natatorium was over budget and was therefore never constructed. The main goal of this study was to explore various structural systems in an attempt to develop a design that better met the financial needs of the owner while still maintaining a pleasing architectural appearance. The structural system of the original design for the natatorium is composed of curved, triangular shaped steel HSS trusses with tapered columns that span 130'-0" over the indoor pool area. New truss configurations were designed using a king post truss system, steel space frame, and glulam truss system. After the truss systems were designed, they were compared in terms of cost, architectural impact, and feasibility, and a final design was chosen. The glulam trusses were determined to be the best option for the alternate roof system. The glulam structural system offered architectural integrity and a competitive cost while the king post truss system lacked architectural freedom and the steel space frame was determined to be too costly. Laminated decking was then designed for the trusses using a two-span continuous layup. It was later determined that achieving diaphragm action with the required three-inch nominal decking is often difficult. Therefore, 3/8" plywood was designed for the given wind and seismic loads and was to be attached to the top of the decking to provide the roof diaphragm with the ability to transfer lateral forces to the lateral force resisting frames. Connections for the glulam truss members were then designed using 3/4" diameter bolts and steel side plates. The bolted metal side plates worked well since all of the truss members were designed to have the same width. Final connections were quite large, with twenty-four bolts being required for bottom chord splice connections and twenty-eight bolts being required for top chord connections.

Since the glulam trusses were designed to only take gravity loads, new lateral force resisting systems were designed. Wood braced frames were added to the perimeter in the East/West direction, while other wood braced frames were designed to replace original steel braced frames in the North/South direction along the west end of the natatorium. Steel moment frames and steel braced frames near the precast concrete grandstand were redesigned as reinforced concrete moment frames. Wind columns that transfer lateral loads to the roof diaphragm were also redesigned using wood. Wind loads were recalculated to account for changes in building height and shape due to the glulam truss configurations, and seismic loads were updated to account for the increased weight of the building. The wood roof structural system was found to be much heavier than the original steel system, and the concrete moment frames weighed much more than the steel moment frames used in the original design, thus increasing the seismic loads on the building. Direct shear values and torsional shear values were calculated and appropriately applied to the lateral force resisting frames. SAP2000 was used to model the frames and obtain member forces. The final designs for the lateral systems met the story drift requirements for wind and seismic loads. An overturning check and foundation check were also performed to account for the new lateral loads and building weight. The original foundations were found to have adequate capacity to carry the increased building loads.

An architectural depth was studied due to the introduction of the new truss system into the indoor pool area. Changes in building height and in the shape of the roof were investigated, as well as changes to the overall appearance of the building, both internally and externally. In addition, several room layouts were changed to accommodate new column locations. Since the building is a natatorium, a building enclosure breadth study using material covered in AE 542 (Building Enclosures) was also implemented to investigate how the design of the building accounts for moisture-related and thermal-related problems that often arise with indoor pool environments.

The MAE course-related study involved a continuation of the building enclosure analysis using information addressed in AE 537 (Building Failures) concerning moisture-related problems with buildings. Pressure treatment of wood and problems with wood trusses were also investigated. The Building Enclosure breadth study using information from AE 542 could also count toward the MAE requirements. This study included glass capacity design calculations as well. Extensive use of AE 597A (Computer Modeling) was also necessary to model the proposed trusses and proposed lateral force resisting systems in SAP2000.

Acknowledgements

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The entire AE faculty and staff

A special thanks to my friends, family, and classmates for their much needed support.

Building Design Summary

The Farquhar Park Aquatic Center is a 37,000 square foot multi-level, state-of-the-art natatorium complex designed by Nutec Design Associates, Inc., a full-service architectural and engineering firm located in York, PA. The facility is located in the city of York and features a 53-foot high natatorium with raised seating, a 12-foot deep indoor swimming pool with diving platforms, a 3,600 square foot single story masonry bath house, and a large outdoor swimming pool, as can be seen in Figure 1. The complex was intended to be used by the YMCA of York, but the original design was never constructed due to cost and budget concerns. The natatorium contains an entry level, a concourse level, and a gallery level. The main entrance opens up into an expansive 24-foot high lobby that spans from one end of the building to the other. The lobby provides access to concessions, men's and women's toilets, and corridors that connect the main lobby to the indoor swimming pool area. The entry level also contains men's and women's lockers and showers, a team room, offices, storage rooms, timer room, utility room, dish room, and trophy display case.



Figure 1 – Aerial View of Natatorium Complex

Concrete stairs near the main entrance lead up to the concourse level which houses a mechanical room and a team store. A long precast concrete ramp also connects the ground floor to the second floor. The floor of the concourse level sits about 10 ½' above the ground level and consists of 12" precast hollow core concrete planks, as can be seen in Figure 2. Visitors can overlook the lobby below behind a 3 ½' guardrail. A precast L-shaped concrete balcony spans the entire length of the pool and provides access to the grandstand seating area.

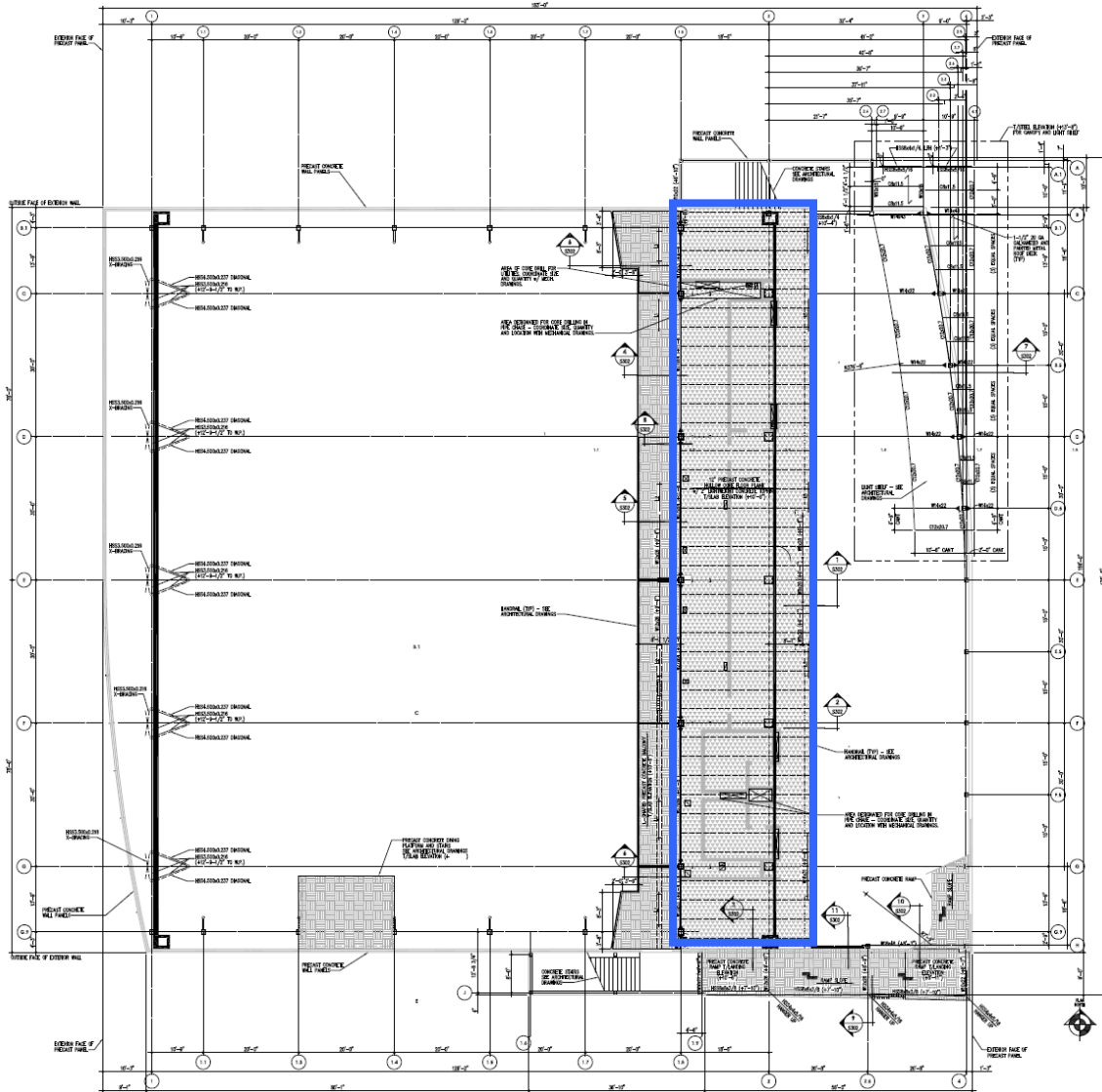


Figure 2 - Concourse Level Framing Plan (12" precast concrete hollow core floor planks are shown in blue – they span 27'-0" and run almost the entire length of the building)

The natatorium's curved roof spans about 130'0" and is supported by large trusses, creating a very open space. The lower roof above the lobby sits about 14' below the lowest point of the curved roof and contains most of the mechanical units. Trusses spaced at 15'-0" on-center support the roof and units. The east-facing and west-facing exterior walls of the natatorium are both slightly curved. At each end of the indoor swimming pool area is a large, curved glazed aluminum curtain wall made of Solera-T glazing. These two curtain walls are each 123'-11" long, 21'-0" tall at their highest points, and 8'-0" tall at their shortest points. Precast concrete panels are primarily used as the façade along with a mix of metal wall panels and glazed curtain walls, as can be seen in Figure 3.

Nutec Design Associates designed the facility to comply with certain LEED credits for the project to achieve LEED Silver Certification. Thermal shading effects were provided by a façade plant climbing system that helped to reduce indoor air temperatures. Another green feature was the natural daylighting provided by the large glass curtain walls enclosing the indoor swimming pool area. Other requirements were related to certain materials and ensuring that they are environmentally friendly.



Figure 3 – View of Main Entrance of Natatorium (showing precast concrete panels, metal wall panels, and glazed curtain walls)

Structural System Overview

Foundation

The geotechnical evaluation was performed by GTS Technologies, Inc. on September 30, 2005. The study included five boring tests, only one of which hit water and revealed a water level 12'-0" below existing site grades. The recommended allowable bearing pressure from GTS Technologies for compacted structural fill was 2500 psi. A shallow foundation system consisting of isolated spread footings at various depths was used. Most of the foundations were located about 2'-0" below finished floor elevation, however a few along the west side of the natatorium were located about 15'-0" below finished floor elevation in order to get below the pool structure. This can be seen in Figure 4. Footings range in size from 4'-6"x4'-6"x1'-0" up to 19'-0"x19'-0"x2'-0". Larger foundations were required to handle the loads carried by the trusses spanning across the indoor pool.

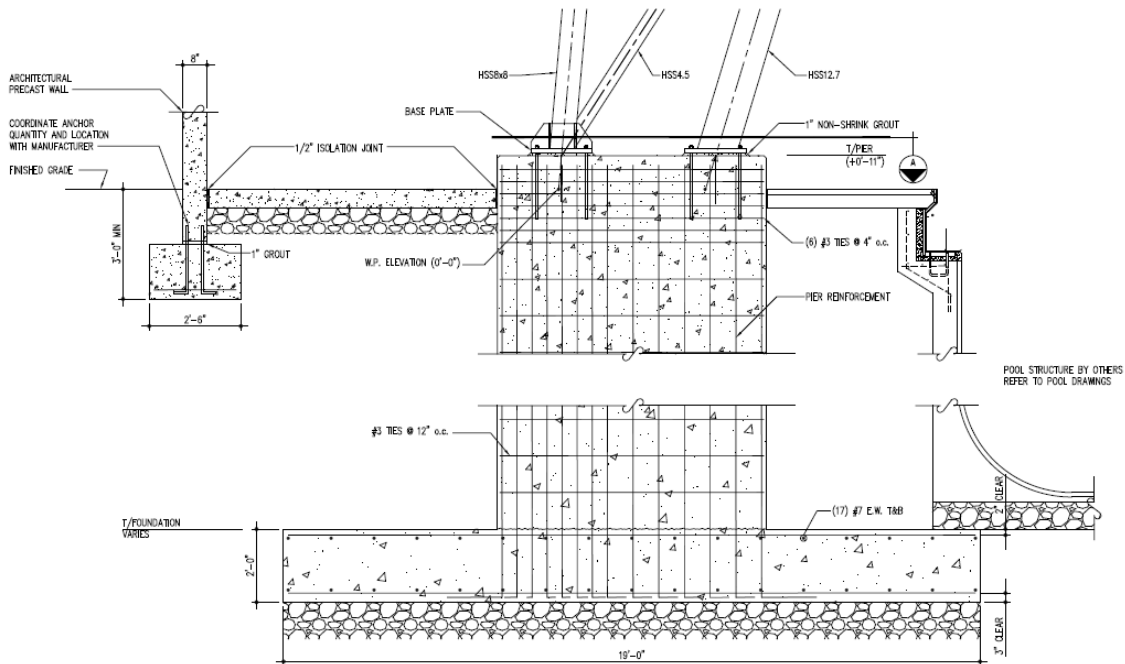


Figure 4 – Detail of Pier Supporting Large Tapered Truss Column

Concrete with a compressive strength of 4,000 psi was used for the footings. Reinforcement in the footings consists of #5, #6, and #7 bars, while reinforcement in the piers consists of #6 and #8 bars, with the #8 bars only being used in the large, deep piers supporting the tapered truss columns. A typical pier detail is shown in Figure 5. Strip footings were 2'-6" wide for interior walls and 2'-0" wide for exterior walls. Geotechnical reports indicate that exterior footings shall be embedded a minimum of 36 inches below final grade for frost protection. Foundations were to be placed on a geotextile layer to minimize the loss of aggregate materials into the subgrade. Due to the proximity of Willis Creek Run and the fact that water was found in one boring test, the geotechnical report suggests that the bottom layer of the pool slab be designed to include a 12-inch No. 57 aggregate drainage layer and pressure release valves to prevent potential floatation due to ground water when the pool is drained.

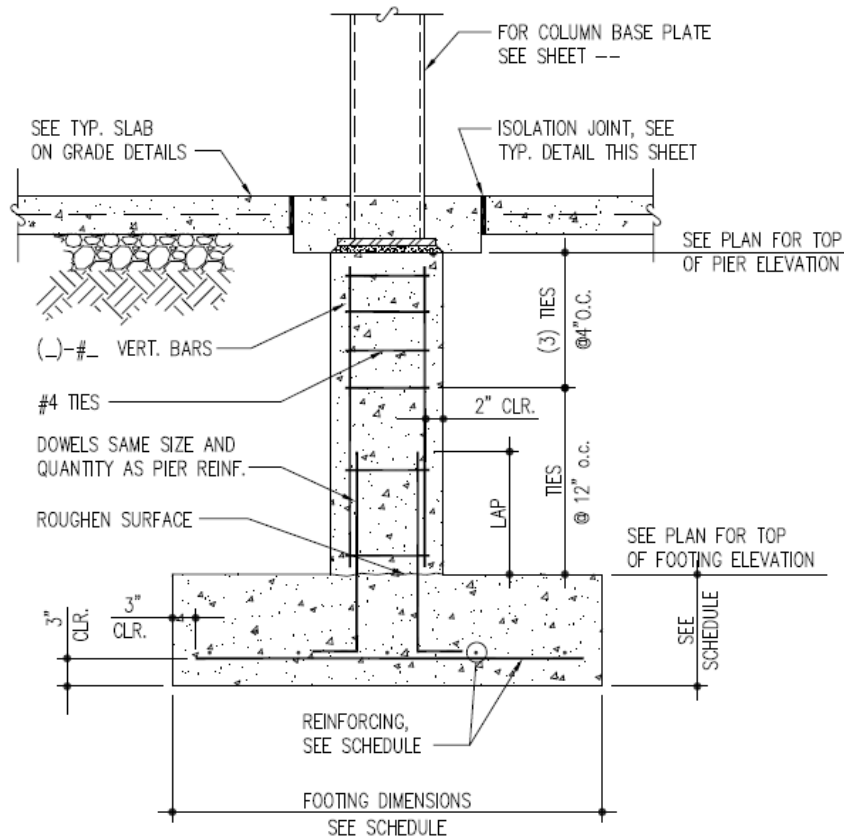


Figure 5 – Typical Pier Detail

Superstructure

The ground floor consists of a 4" concrete slab-on-grade with 6x6 W2.0xW2.0 W.W.F. on 4" crushed stone base and a compressive strength of 4,000 psi. The concession area sits on a recessed concrete slab, and a portion of the floor slab near the pool structure becomes 8" thick with #4 bars at 12" on-center L.W. and #5 bars at 12" on-center S.W. HSS columns in the lobby run along the east wall and support the roof trusses above the lobby. The entry level also contains 12" CMU walls with #5 bars at 32" on-center that are grouted solid full height. These walls enclose parts of the bathrooms, locker rooms, offices, team room, storage rooms, and utility room and are located beneath the grandstand seating area. A floor plan of the entry level is shown in Figure 6. Precast concrete columns help support the 8" precast concrete ramp that runs from the ground floor up to the concourse level. The ramp is also supported by W-shape beams, HSS columns, and hangers.

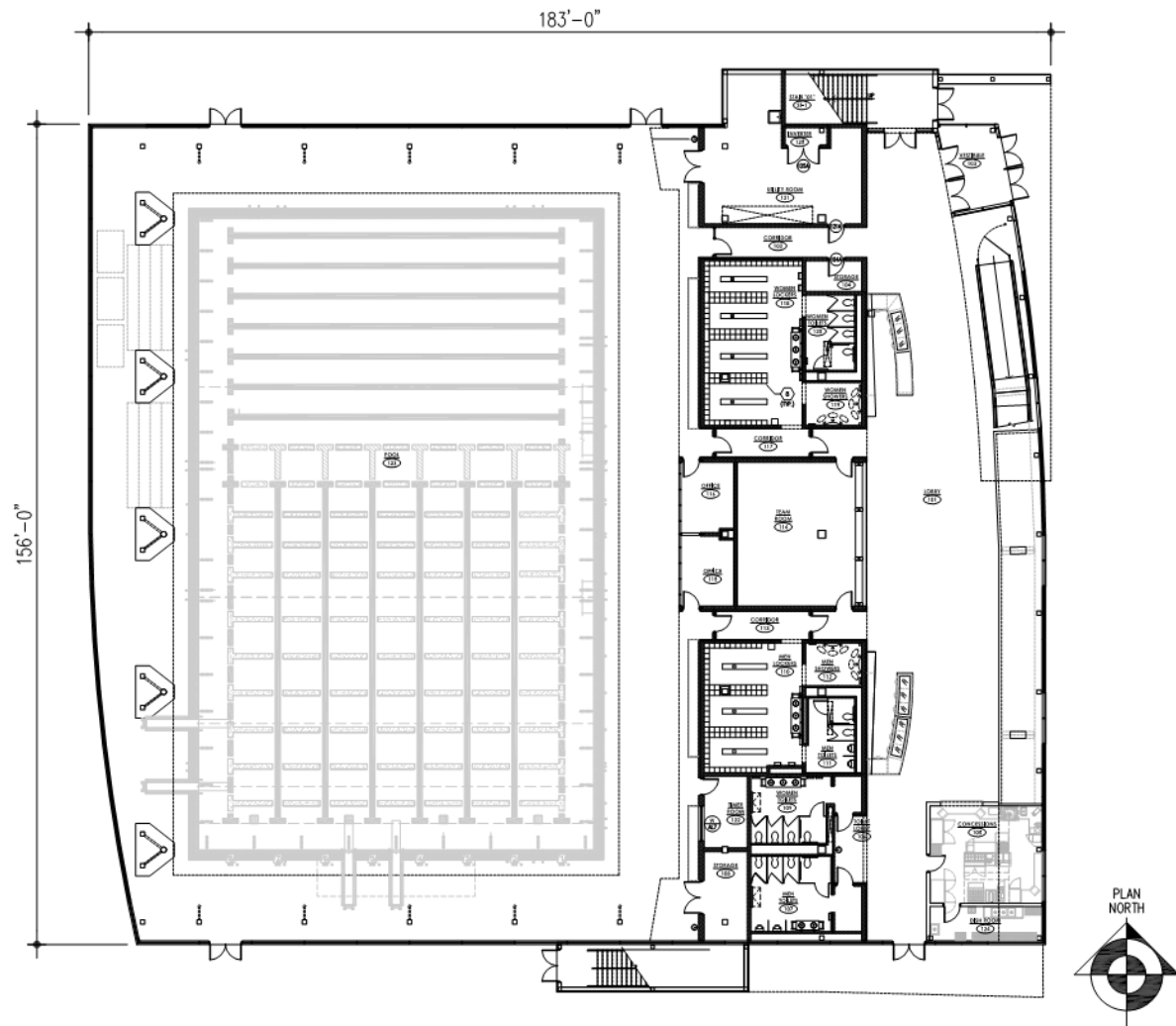


Figure 6 – Entry Level Floor Plan

Triangular HSS trusses spanning 130'-0" support the large curved roof above the indoor swimming pool area and are shown in Figure 7. The columns for these trusses are triangular, tapered, and spaced 30'-0" on center. Both the trusses and the supporting columns are made up of HSS members. Long span deck was used to span between the trusses. The other ends of the large trusses are supported by HSS18x18x5/8 columns. HSS wind column trusses run along the north and south walls in the indoor pool area as well. The trusses are 3'-0" deep and vary in height with the tallest at 51'-2 1/4" above finished floor elevation. The wind column trusses connect into the main roof diaphragm. The rest of the high roof framing primarily consists of HSS6x6 and HSS8x8 members.



Figure 7 – Rendering of Indoor Pool Area Showing Large Curved Trusses

The precast concrete grandstand seating area that runs from the concourse level to the gallery level is supported by sloped W27x94 beams that frame into the HSS18x18x5/8 members that also support the large curved trusses. The floor system of the concourse level consists of 12” precast concrete hollow core floor planks with 2” lightweight concrete topping, as is shown in Figure 8. Top of slab elevation is 10’-6”. The precast concrete balcony is supported by a 12” CMU wall, and additional strength is provided by a 12” bond beam with two continuous #5 bars. A canopy and light shelf near the main entrance and lobby are slightly higher than the concourse level and are supported by cantilevered W14x22 and W14x43 beams. Additional framing is provided by C8x11.5 beams and curved C12x20.7 beams. Moment connections allow the W14 beams to cantilever from the supporting HSS10x10 columns.

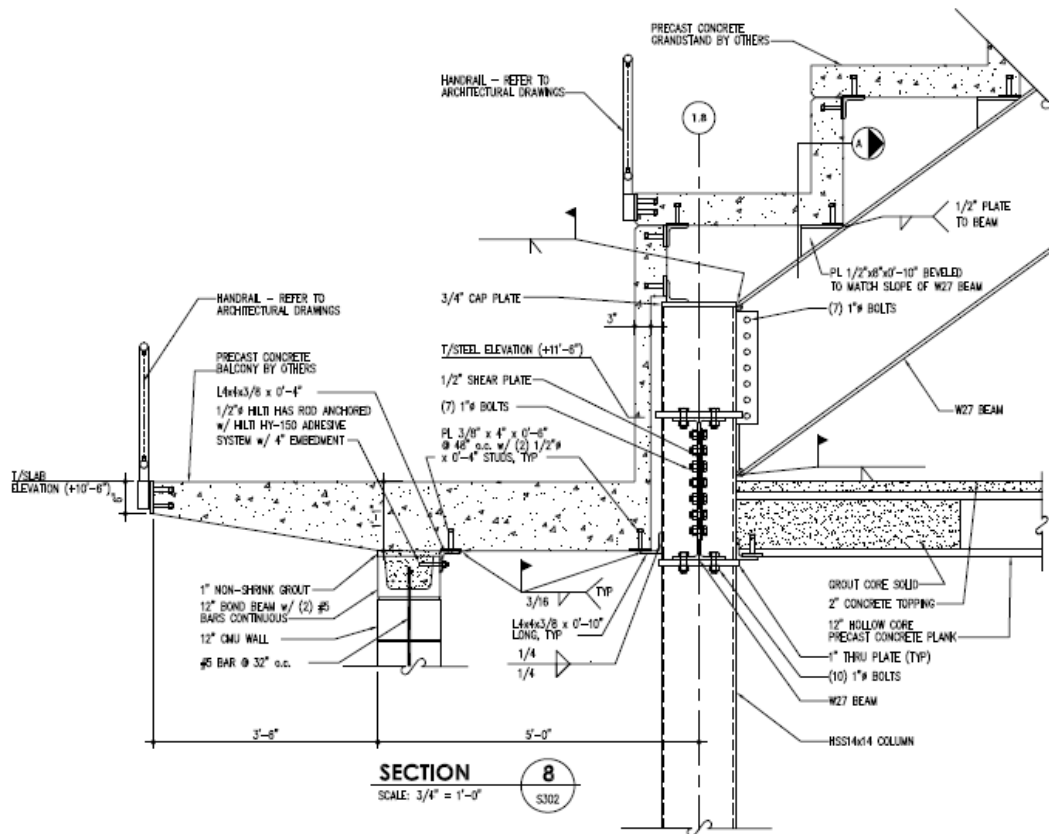


Figure 8 – Section Showing the 12” Hollow Core Precast Concrete Planks, the Precast Concrete Balcony, and the W27x94 Beams Supporting the Concrete Grandstand

The gallery level has HSS roof trusses spanning about 41'-0" and spaced 15'-0" (and 2'-5" deep) supporting 6" 18 GA acoustical long span metal roof deck with 18 GA perforated cover and polyencapsulated acoustical batt insulation. The trusses are 2'-5" deep, slightly sloped, and also support the mechanical unit framing above. The top of steel elevation for the mechanical unit support framing is 28'-0", and the framing consists of W8, W10, and C8 beams.

Lateral System

The large truss columns and mezzanine moment frame take the lateral load in the East/West direction, while the braced tapered truss columns, a braced frame between the pool and lobby, and a steel moment frame at the east side of the lobby handle the lateral load in the North/South direction. Seismic loads due to the concourse level floor system and precast concrete balcony are resisted by another steel moment frame. Some of this seismic load goes into the CMU walls as well, but the steel moment frame provides most of the lateral support. The wind columns are designed to simply take the wind force in the North/South direction and transfer it to the roof diaphragm. A mezzanine level framing plan is shown in Figure 9, and a roof framing plan is shown in Figure 10. The

wind columns transfer roughly half the load to the ground or base connection and the other half of the load to the high roof diaphragm. The roof diaphragm transfers the load to the large trusses over the indoor pool, which in turn sends part of the load to the five braced tapered truss columns and the rest of the load to the braced frame between the pool and lobby. The large truss columns are laterally braced by HSS3.500x0.216 X-bracing. The two chords of the truss columns are offset by four feet at the base, providing a rather rigid support that can handle high lateral loads. The large trusses and supporting truss columns can be seen in Figure 11, and the wind columns can be seen in Figure 12.

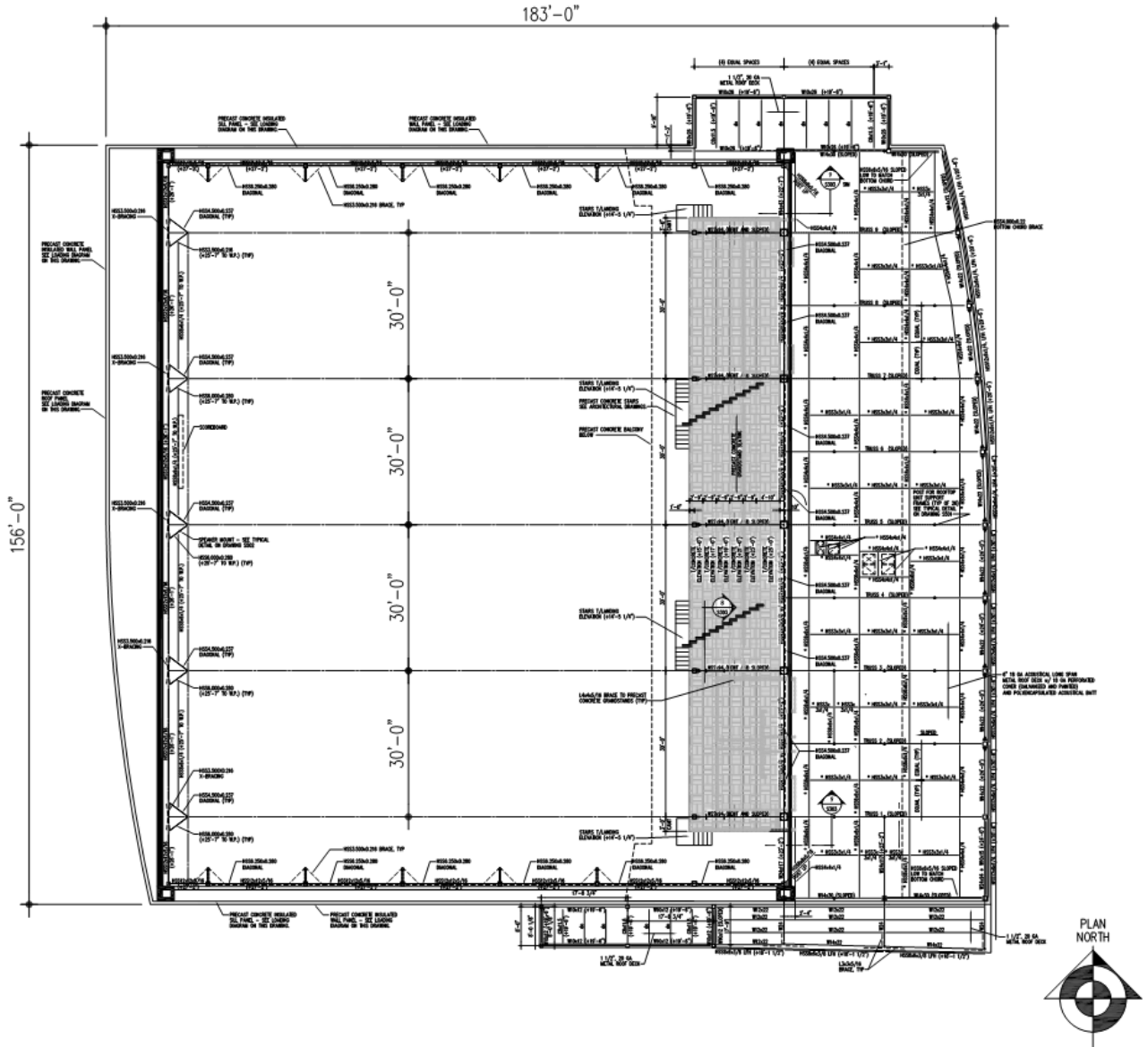


Figure 9 – Gallery/Mezzanine Level Framing Plan (the shaded portion is the grandstand seating area)

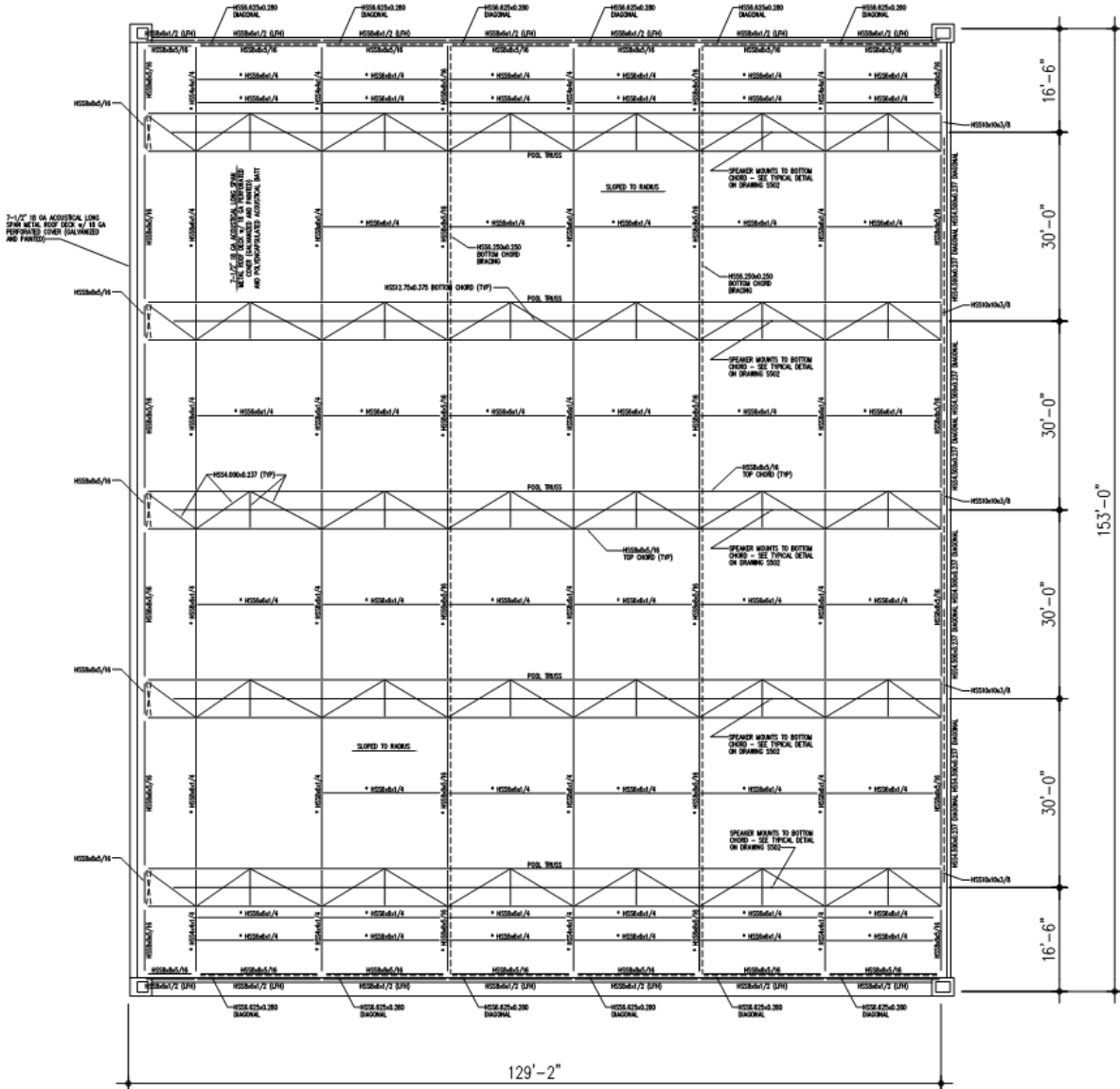


Figure 10 – Roof Framing Plan (including the five large trusses above the pool area spaced 30'-0" on center and additional framing)

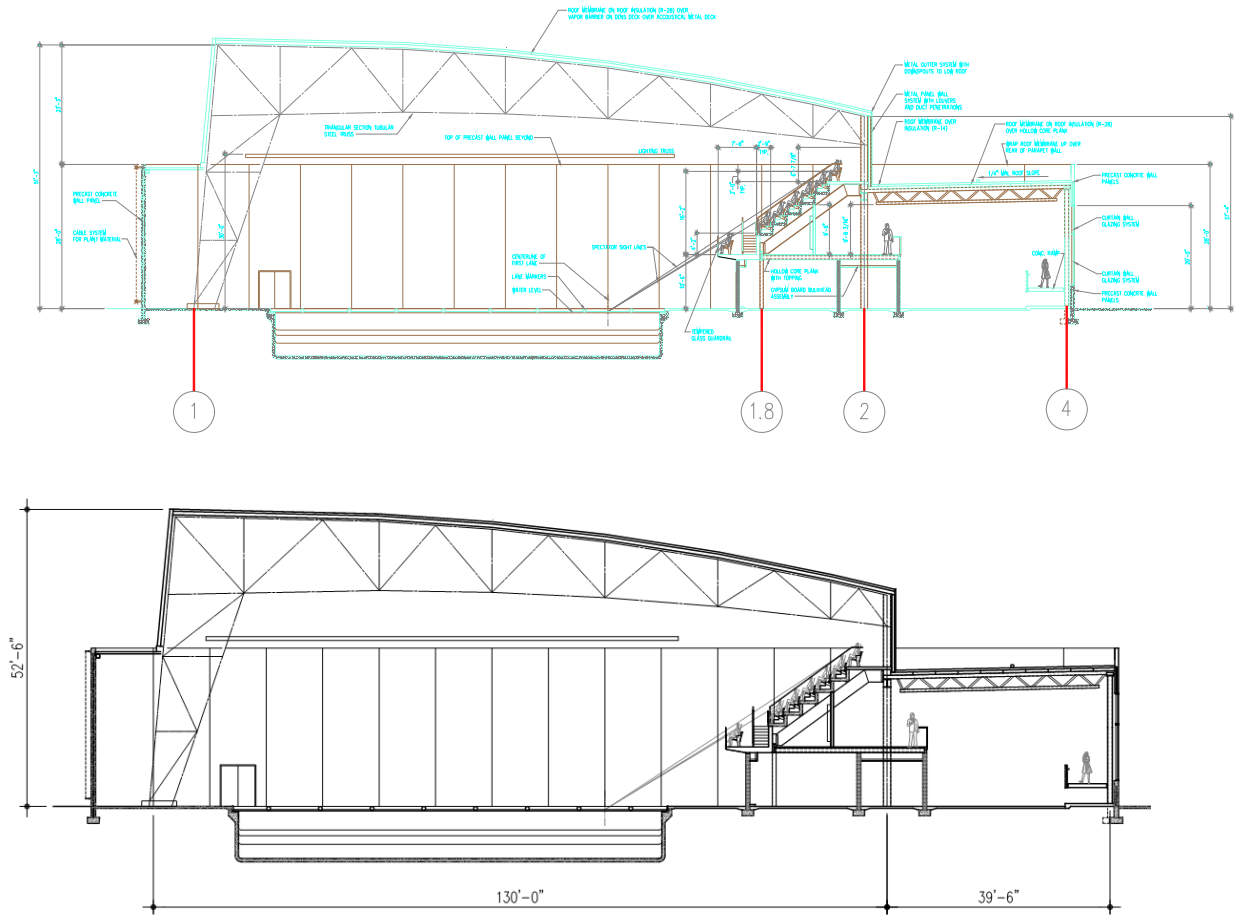


Figure 11 – Cross Section Through Center of Building (Looking North); Top Figure Shows Column Lines Mentioned Throughout Thesis Report

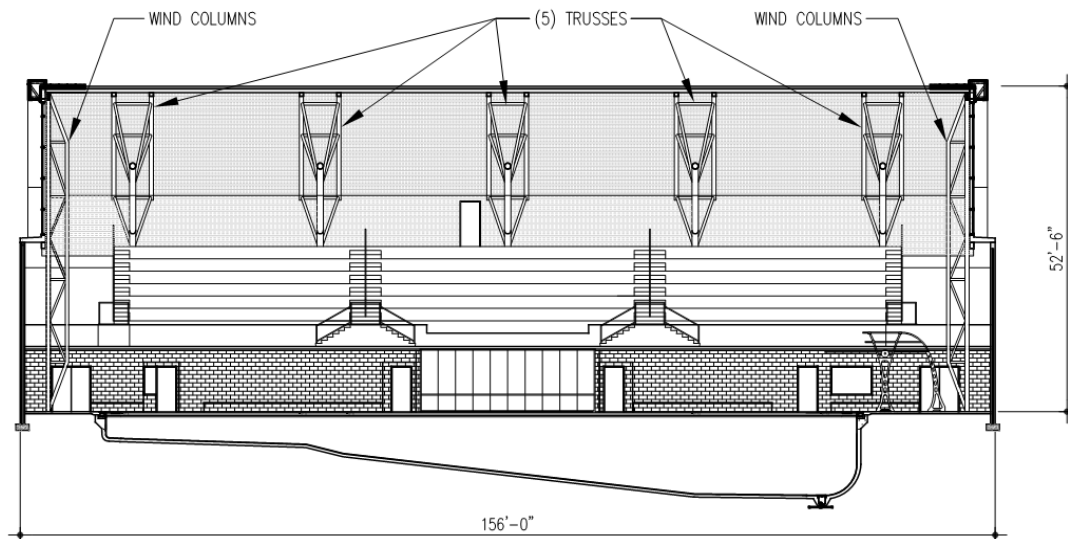


Figure 12 –Cross Section Through Indoor Pool Area Showing the Wind Columns (Looking East)

Problem Statement

The original design for the Farquhar Park Aquatic Center natatorium was over budget and hence was never constructed. The natatorium was actually built as a less expensive pre-engineered building that better met the financial needs of the YMCA. Although the original structural system that was proposed was fancy, it is evident that it did not work for the purpose of the project. A YMCA is focused on providing for the community; therefore it did not really make sense to design a structurally complicated, expensive building for the natatorium complex. Money spent by the YMCA should be spent on the people, not on an overly-extravagant building (particularly in a place like York, PA). The overall goal of this thesis is to investigate potential solutions for the design of the natatorium that provide a “happy medium” in between the original design and the building that was finally constructed. The final design will attempt to incorporate alternative structural systems while still maintaining the architectural integrity of the original design.

Proposed Solution

The current structural system of the original design for the Farquhar Park Aquatic Center natatorium is composed of curved, triangular shaped steel HSS trusses with tapered columns that span 130'-0" over the indoor pool area. The proposed thesis will include a redesign of the entire roof structural system, which will have strong architectural impacts as well. New truss configurations will be designed using a king post truss system, wood trusses or glulam members, and a modified space frame. After the proposed truss systems are designed, they will be compared in terms of cost, feasibility, and architectural impact and a final design will be chosen. In the event that the new trusses only take gravity loads, a new lateral force resisting system composed of perimeter braced frames will be designed. It will be crucial to ensure that lateral loads applied to the roof actually get transferred to these perimeter braced frames. In addition, the existing concourse level floor system, balcony, and grandstand seating area will be redesigned as an entirely precast structure. Nitterhouse Concrete Products, Inc. will be contacted to investigate the feasibility and design of this precast system. Also, the current steel HSS columns that support the east end of the large trusses will be redesigned as concrete columns. Concrete moment frames may also be used to replace the existing steel braced frames at the grandstand seating area in the North/South direction. A final foundation check will be performed to verify that the existing foundation can adequately carry all loads present with the proposed system.

An architectural depth will be studied due to the introduction of a new truss system into the indoor pool area. Changes in building height and in the shape of the roof will be investigated, as well as effects on the lighting of the space. The overall appearance of the building, both internally and externally, will be affected by each new truss design. Plus, room layouts may need to change due to changes in column locations. A second breadth topic will relate to an analysis of the building enclosure. Material covered in AE 542 (Building Enclosures) will be used to investigate how the design of the building accounts

for moisture-related and thermal-related problems due to the fact that the building is a natatorium. The MAE course-related topic will be a continuation of the building enclosure analysis by including information addressed in AE 537 concerning moisture-related problems with buildings. Necessary changes to building elements to account for these problems will also be made. Extensive use of AE 597A (Computer Modeling) will also be necessary to model the proposed trusses and proposed lateral force resisting systems in SAP2000.

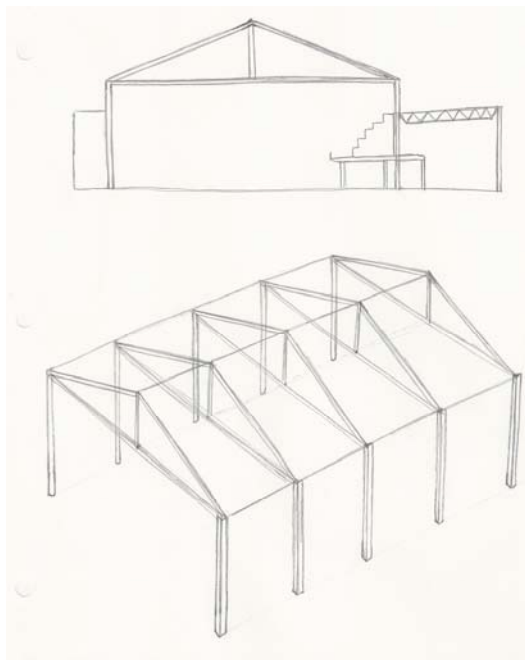
Structural Depth

Gravity System Study

King Post Truss Design

The first alternate roof system design that was investigated was a king post truss system. A king post truss is a rather simple system that typically consists of two diagonal members that extend from the ends of the bottom chord and meet at the apex of the truss. A vertical member called the king post connects the apex to the tie beam, or bottom chord of the truss. The diagonal members, or king post braces, are said to be in compression while the king post and bottom chord are said to be in tension. King post trusses are typically used for situations with shorter spans. Longer spans usually require a more sophisticated truss. Sometimes a queen post truss, which essentially has two king posts, is used to span longer distances.

For the Farquhar Park Aquatic Center, numerous king post truss configurations with varying heights were investigated. Sketches were initially made, and then truss shapes were put into SAP2000 to determine appropriate dimensions for a desired architectural appearance. Dead loads, snow loads, and roof live loads were considered and appropriately applied to the models of the trusses in SAP2000, and the resulting axial loads in each member were determined from the program. All members were modeled as pinned at the ends. Members were sized using the AISC Steel Construction Manual.



Figures 13 (left) and 14 (right) – Preliminary Sketch of Potential King Post Truss System Design (left); Image of a Basic King Post Truss from www.precraftedhomes.com (right)

One of the goals with the king post truss system was to design a truss that did not appear too shallow, yet not too deep. It was recognized that a large depth would be required due to the large span, and this required depth needed to be determined in order to determine the feasibility of using a king-post truss system for the natatorium. The first king post truss design had a traditional triangular shape with two diagonal members for the top chord, a bottom chord, a king post, and two diagonal web members extending from the bottom of the king post member to the midpoints of the top chord members. Additional vertical members were added from the midpoints of the top chord members to the bottom chord, splitting each diagonal top chord member into two separate members. This was necessary in order to decrease the large unbraced lengths of these members. Plus, as one entire member each diagonal top chord would have been almost 67'-0" long, which was too excessive. Depths of 5'-0" to 10'-0" were found to be too shallow, so a truss depth of 15'-0" was determined to be a minimum. With an initial truss depth of 15'-0" and truss spacing of 30'-0" (to match the spacing of the original design), the resulting tensile force in the bottom chord from SAP2000 was 343 kips. Not only did this truss configuration lack architectural appeal due to its plain shape for such a long span, but the forces in the members were also considerably large.

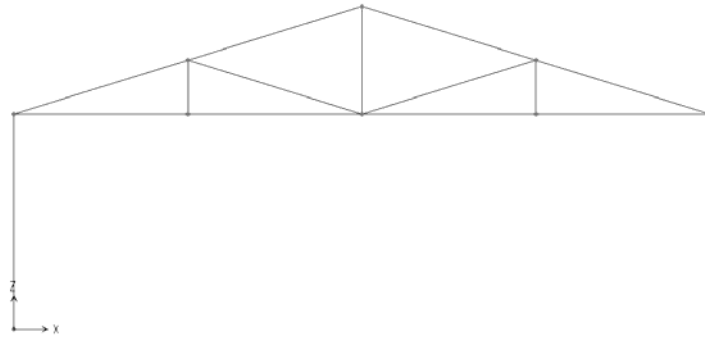


Figure 15 - SAP2000 Model of Triangular-Shaped Steel King Post Truss with 15'-0" Depth

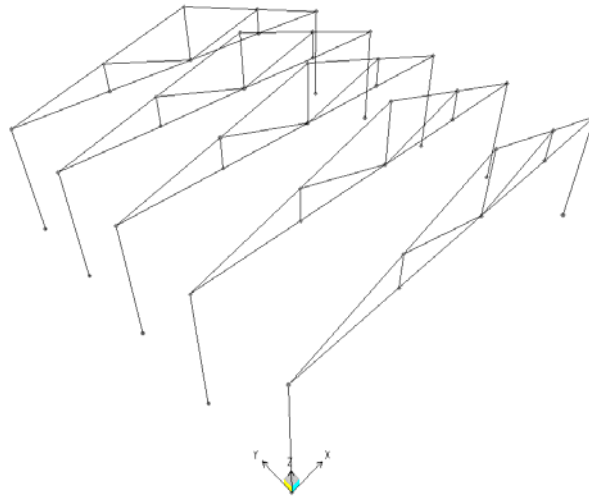
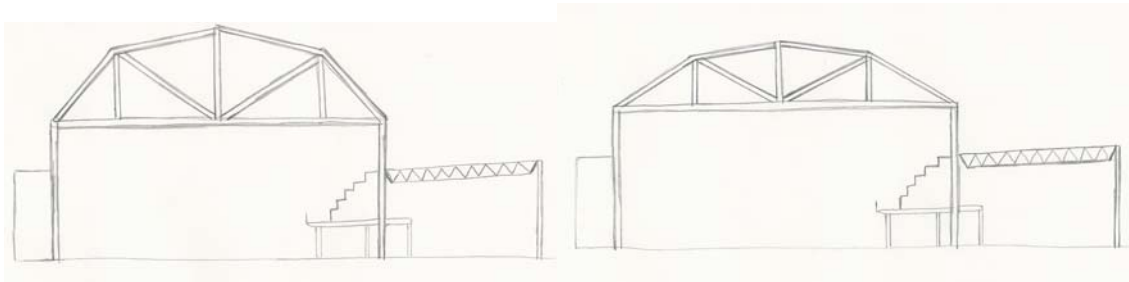


Figure 16- SAP2000 Model of Triangular-Shaped Steel King Post Truss with 15'-0" Depth Spaced 30'-0" o.c.

To add more architectural interest to the truss shapes, the joints of the top chord at midspan between the far ends of the bottom chord and the apex of the truss were raised 4'-0". This added more of a curve to the shape of the truss and created a "modified" king post truss configuration, as can be seen in Figure 19 below. In addition, the resulting bottom chord tensile force decreased to 228 kips with the trusses still spaced at 30'-0" o.c. Members of this truss were designed with HSS members using the AISC Steel Construction Manual. Using the lightest sections for each member resulted in an HSS12x12x1/4 top chord, HSS8x8x1/4 bottom chord, HSS 5 1/2 x 5 1/2 x 1/8 diagonal web members, and HSS2x2x1/8 vertical web members. This resulted in a weight of 9,322 lb for one truss, or 46,611 lb for five total trusses. Calculations are found in Appendix A. Although this configuration was more architecturally pleasing and resulted in decreased bottom chord forces, the member forces still seemed rather high. It was determined that an even deeper truss would be required to achieve more of a curved shape and decreased member forces.



Figures 17 and 18 - Preliminary Sketches of Potential Steel King Post Truss Configurations

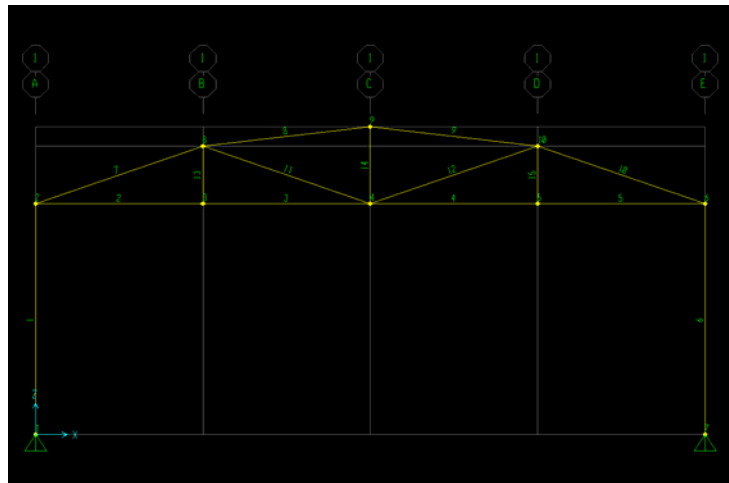


Figure 19 - SAP2000 Model of King Post Truss with 15'-0" Depth and More Curved Appearance

The depth of the truss was increased to 20'-0", with the upper top chord members extending 5'-0" below the apex. This resulted in a bottom chord force of 174 kips and a maximum top chord force of 204 kips with the trusses spaced at 30'-0". It was determined that the sizes of the members for this truss would be very similar to those of the previous truss.

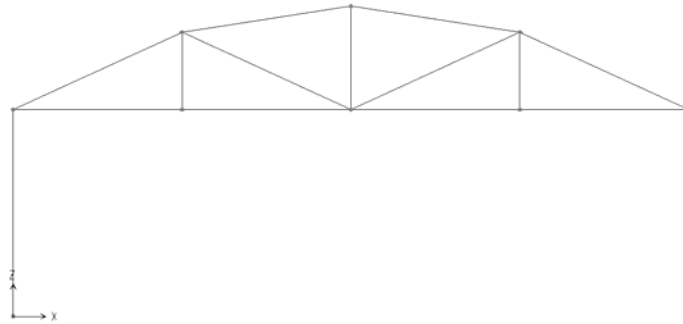


Figure 20 - SAP2000 Model of King Post Truss with 20'-0" Depth

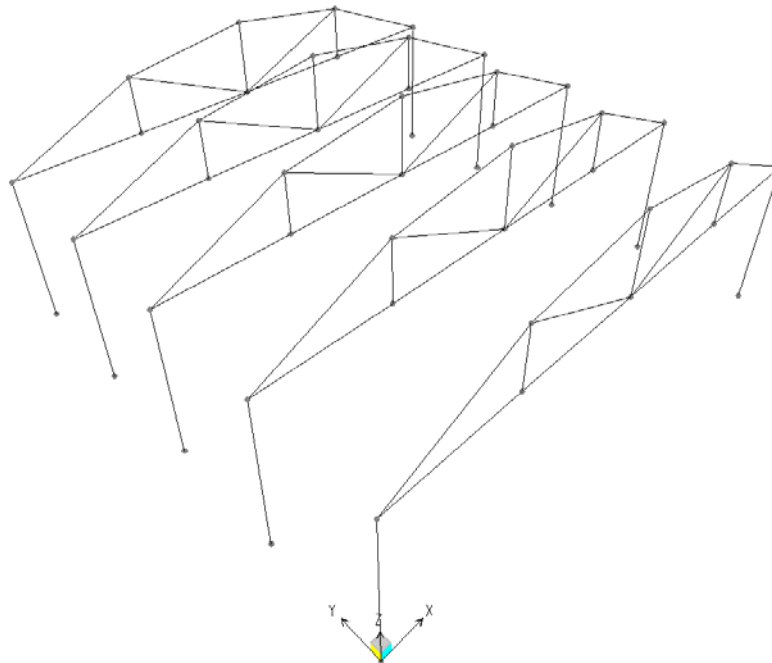


Figure 21 - SAP2000 Model of Steel King Post Trusses with 20'-0" Depth Spaced 30'-0" o.c.

Overall, it was decided that a modified king post truss design could possibly work structurally for the purposes of the natatorium and result in a decreased cost as compared to the original curved and tapered steel HSS trusses. However, the king post truss designs seemed too basic and lacked architectural style. The typical shape of a king post truss limited the architectural design options for this type of system.

Space Frame Design

The second roof system that was investigated was a steel space frame. Space frames can offer many advantages over other types of roof systems. Space frames are fairly light weight and can span very long distances to create large column-free spaces. They are

very strong for their weight and can accommodate concentrated loads. Space frames are also very redundant systems, which means that failure of one member will most likely not result in failure of the entire structure. The openness of the frame allows for other services, such as electrical and mechanical equipment, to be installed more easily within the structural depth of the frame. Space frames can also be pre-assembled to allow project acceleration. Space frames typically come in modules that can be easily assembled together on site. Architecturally, the frame can be left exposed without a ceiling to add texture and style to the space. They offer a great deal of design freedom and can be formed into almost any shape. However, one of the disadvantages of space frames is that they can be rather expensive. The joints are often the most expensive element of the space frame. It seems as though space frames are only cost effective if they are absolutely needed for a given situation.

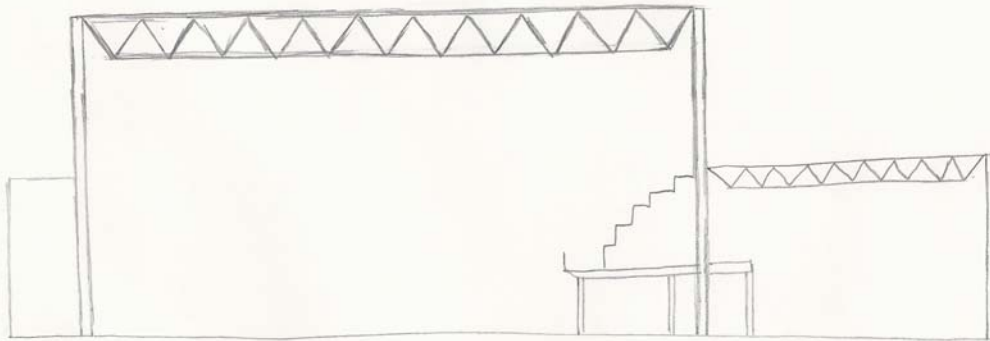


Figure 22 - Preliminary Sketch of Space Frame for the Natatorium

Typical module sizes for space frames are 4', 5', 8', and 12'. The depth of a space frame usually falls in the range of span/12 to span/20. For the Farquhar Park Aquatic Center, this would result in a space frame depth of 8 to 13 feet using the longer span of approximately 156' in the North/South direction. Several space frames were designed and modeled in SAP2000 using various depths and module sizes. First, a space frame with 4'-0" modules and a 10'-0" depth was investigated. Then, a space frame with 4'-0" modules and a 5'-0" depth was created to examine the architectural effects of a shallow frame. The 4'-0" modules seemed too small for the large area the space frame was covering, so 8'-0" modules were analyzed with a space frame depth of 8'-0". The 8'-0" modules appeared to be most appropriate for the natatorium project. A module size of 12'-0" seemed too large architecturally for the indoor swimming pool space.

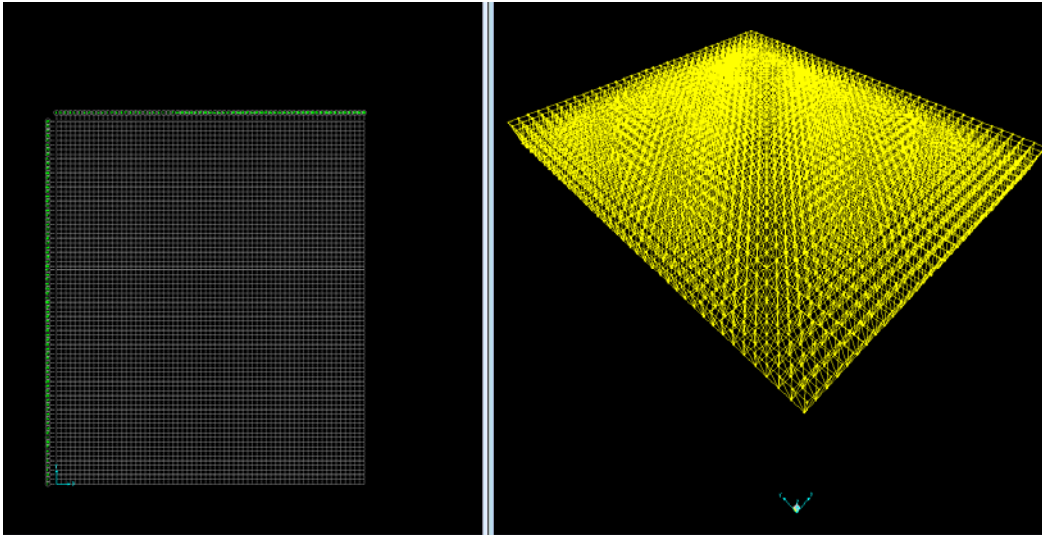


Figure 23 - Space Frame with 4'-0" Modules and 10'-0" Depth

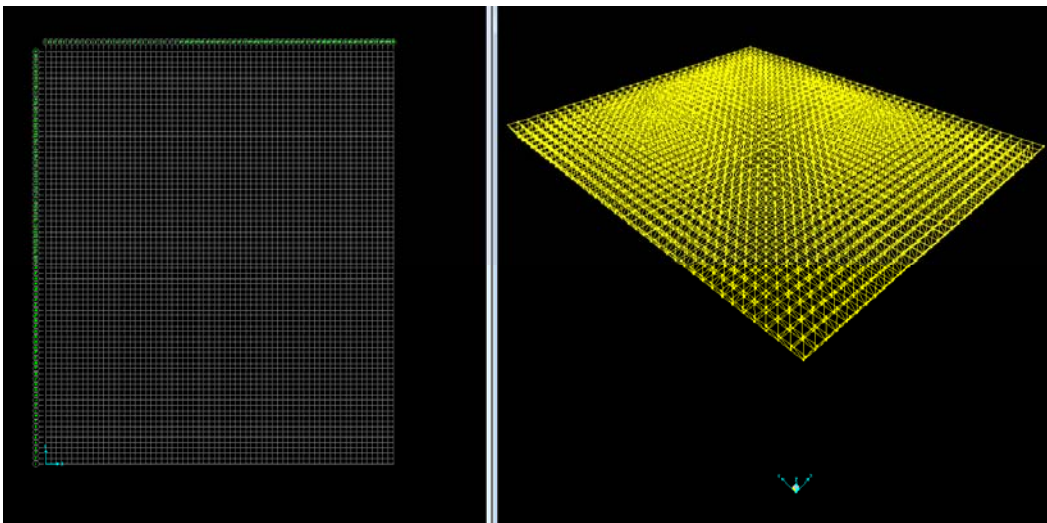


Figure 24 - Space Frame with 4'-0" Modules and 5'-0" Depth

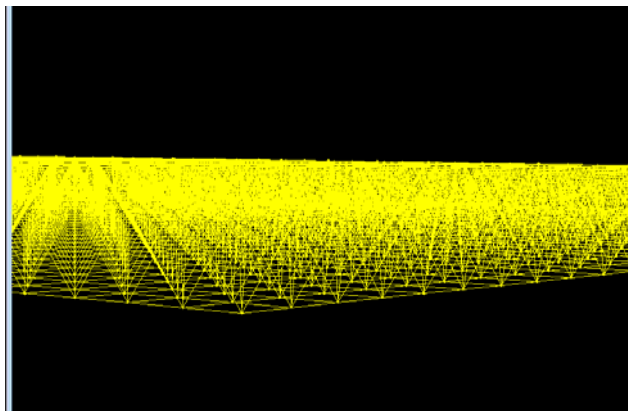


Figure 25 - View to Show Depth of Space Frame with 4'-0" Modules and 5'-0" Depth

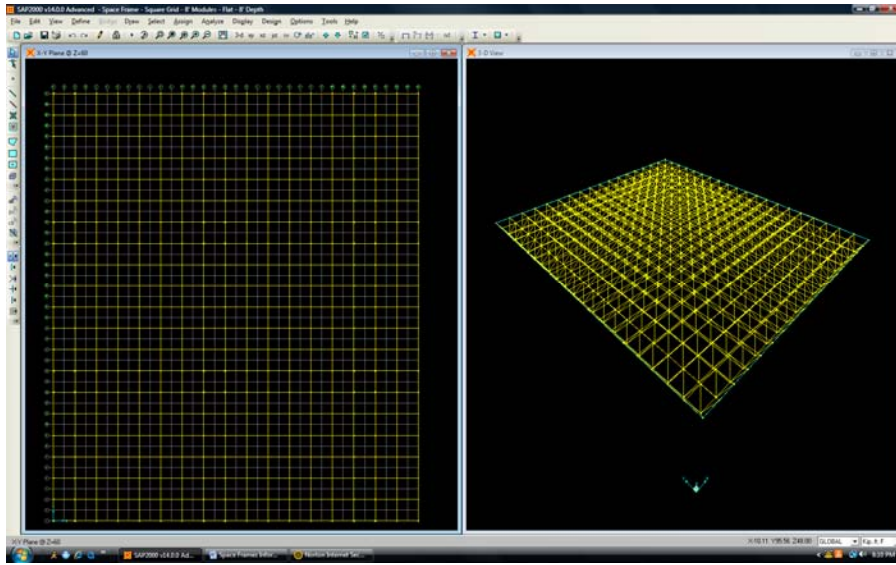


Figure 26 - Space Frame with 8'-0" Modules and 8'-0" Depth

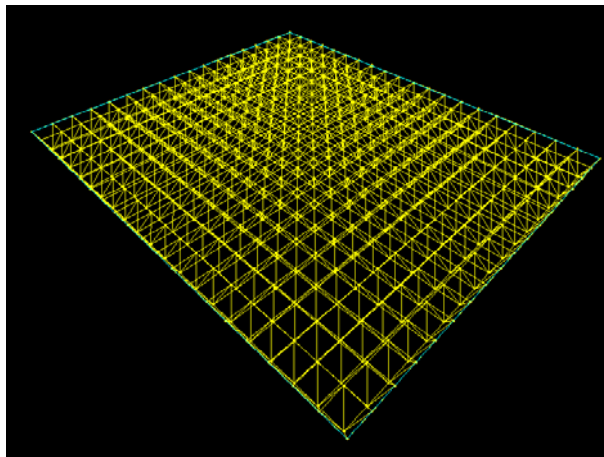


Figure 27 - Space Frame with 8'-0" Modules and 8'-0" Depth

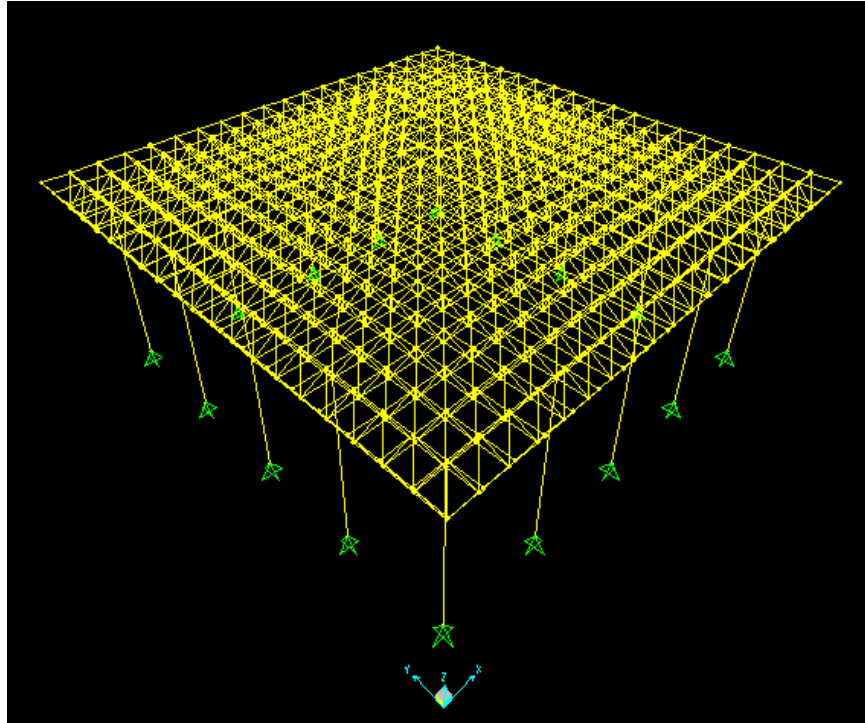


Figure 28 - Space Frame with 8'-0" Modules and 8'-0" Depth with Supporting Columns

The space frame design with 8'-0" modules and depth of 8'-0" was analyzed further in SAP2000 by applying the appropriate dead, snow, and roof live loads to the frame. Loads were applied as concentrated loads to the joints of the space frame. The final design resulted in 760 top members that were each 8'-0" long, 684 bottom members that were also each 8'-0" long, and 1,444 diagonal members that were each nearly 10'-0" long. This resulted in a total of 2,888 members and 23,104 linear feet of steel for the entire space frame. In addition, this configuration contained roughly 3,000 joints. Using the AISC Steel Construction Manual, it was found that an HSS4.000x0.291 (12.3 lb/ft) would work for the largest resulting compressive force and 10'-0" unbraced length. From this result, a rough estimate of the weight of the space frame was made assuming an average of 10 lb/ft for all members. This resulted in a total weight of 231,040 lb for the space frame $[(23,104 \text{ ft})(10 \text{ lb/ft}) = 231,040 \text{ lb}]$. Therefore, the steel space frame required roughly five times as much steel, by weight, than the steel king post truss system with trusses spaced 30'-0" o.c. Overall, the space frame weighed about 11.85 psf while the steel king post truss system weighed about 2.39 psf.

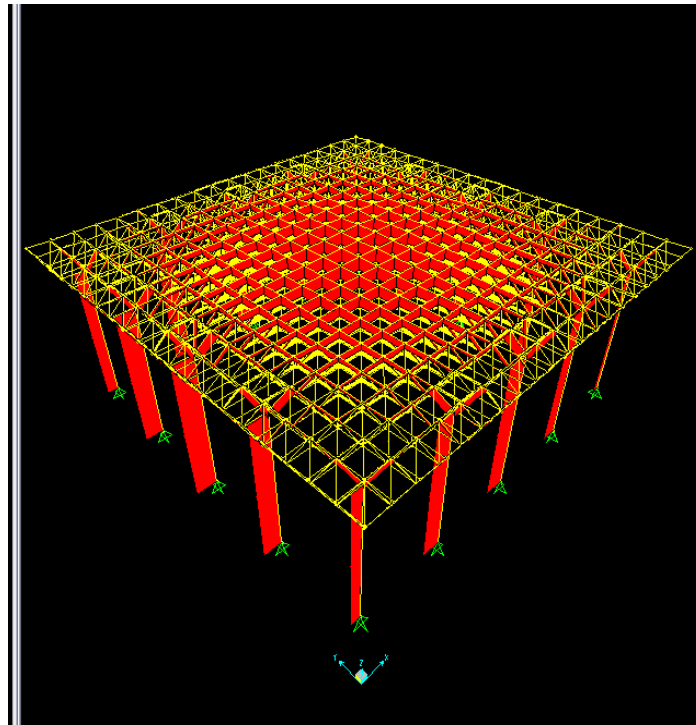


Figure 29 - Image from SAP2000 Showing Axial Forces for Space Frame with 8'-0" Modules and 8'-0" Depth (red indicates higher axial forces)

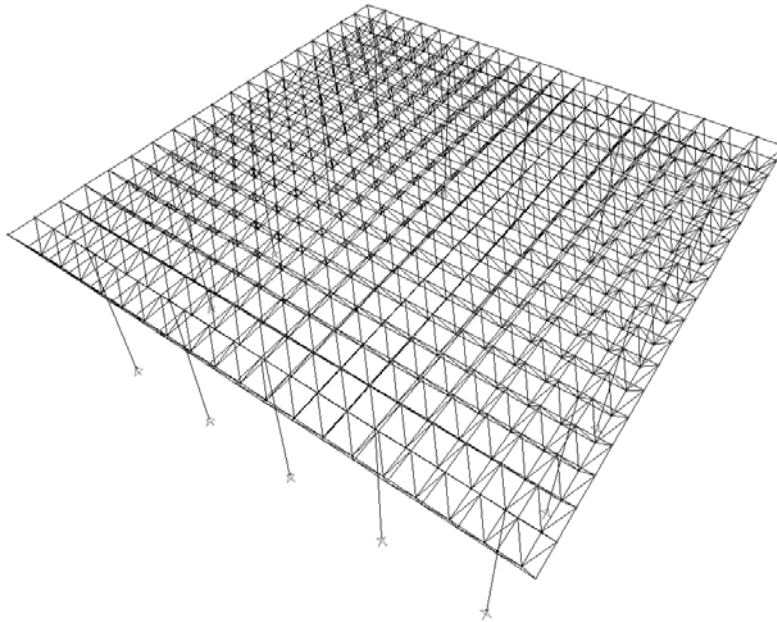


Figure 30 – Additional Image from SAP2000 of Space Frame with 8'-0" Modules and 8'-0" Depth

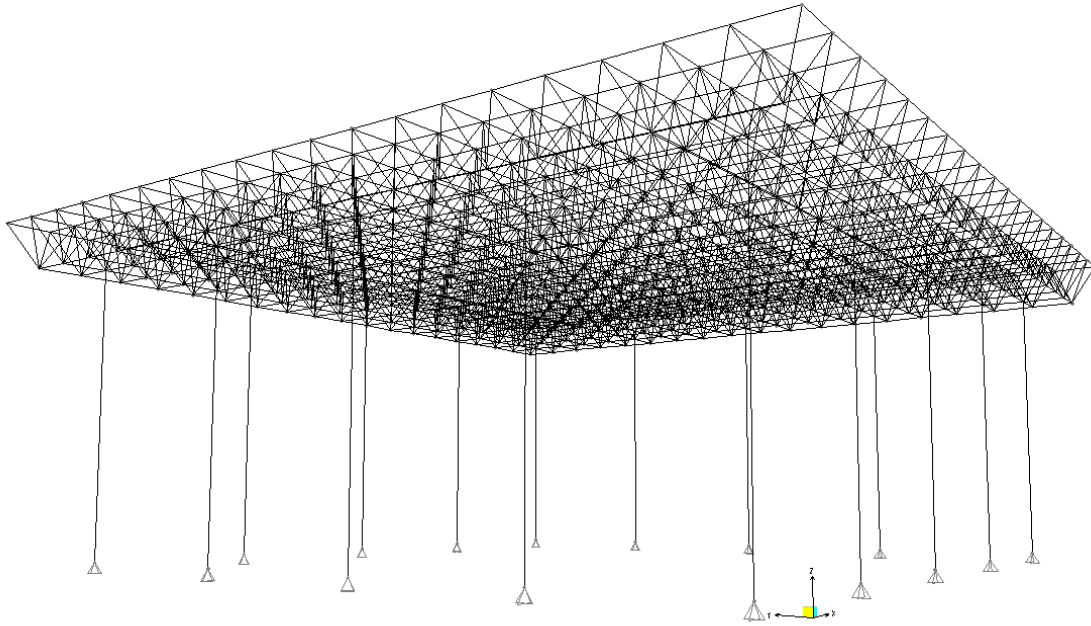


Figure 31 – Additional Image from SAP2000 of Space Frame with 8'-0" Modules and 8'-0" Depth

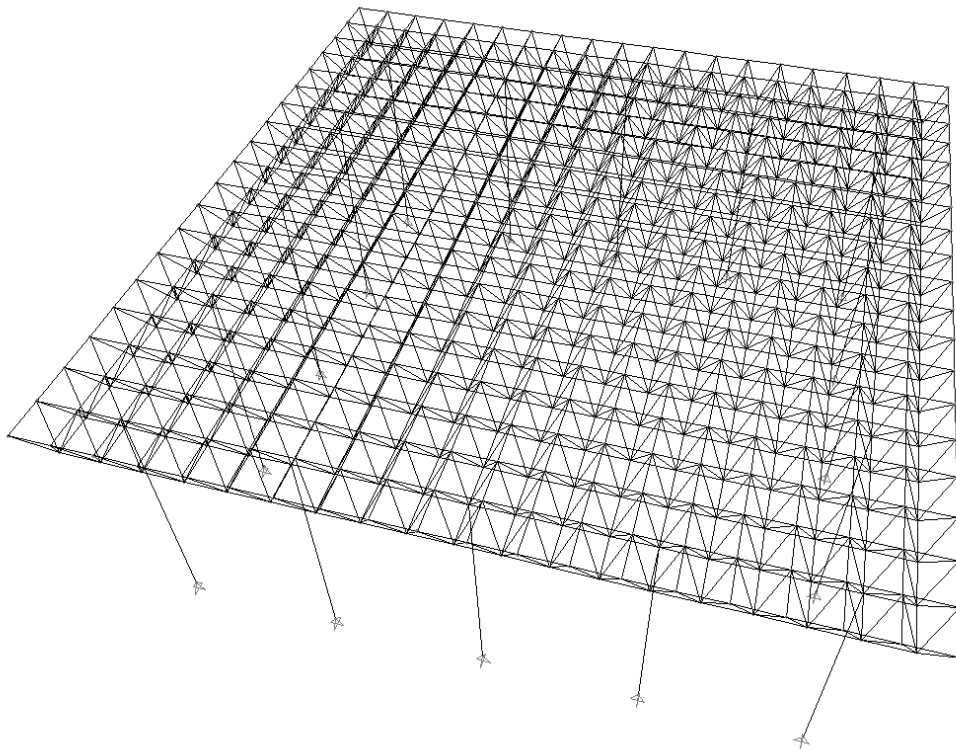


Figure 32 – Additional Image from SAP2000 of Space Frame with 8'-0" Modules and 8'-0" Depth

All space frame designs that were investigated for the natatorium were basically flat. A curved space frame would have been more architecturally appealing, especially for the roof shape from the exterior of the building. However, this would have driven up costs even more due to a more complex configuration. The flat space frame with 8'-0" modules and 8'-0" depth would have been too expensive in the first place due to the excessive number of joints required. This design weighed nearly three times as much as the original truss system, which weighed approximately 84,000 lb and went over budget in the first place. A space frame with 4'-0" modules would have had even more joints than the design with the 8'-0" modules. As mentioned before, using 12'-0" modules seemed almost too large for the indoor pool space, although this would have resulted in fewer joints. Overall, the space frame design would have been too expensive for the Farquhar Park Aquatic Center. Plus, the less costly flat space frame configuration with no curves was rather plain architecturally and would have resulted in a basic flat roof shape when viewed from the exterior. Although space frames offer many advantages, they are generally quite expensive and are not very cost effective unless absolutely needed.

Glulam Trusses

The final roof system that was designed for the Farquhar Park Aquatic Center consisted of wood trusses. It was determined that using glulam members would be most appropriate due to the long 130'-0" span and rather large resulting forces in the members. It was also recognized early in the design process that trying to maintain the 30'-0" truss spacing of the original design was unrealistic and resulted in extremely high loads for wood members. The first wood truss designs were analyzed at a 15'-0" spacing and then a 10'-0" spacing, which still resulted in high, but manageable, forces in the members. With the trusses at a spacing of 10'-0" or 15'-0", the column locations of the original design would still not have to change since they were spaced at 30'-0" o.c. The trusses that would not directly land on a column would bear on a beam spanning between the columns. However, it was later determined that using trusses spaced at 8'-0" o.c. would work best since 8'-0" is a more common dimension in wood construction for components such as roof boards. Any possible way to make the construction process easier would help alleviate the overall cost of the project. The smaller spacing also helped by reducing the member forces. One of the drawbacks of using the 8'-0" spacing was that the locations of the columns on which the east ends of the trusses bear had to move. This required that rooms on the ground floor and concourse level be properly laid out to accommodate the new column locations. Please see the Architectural Breadth for the column relocation study.

It was decided that the truss members would be designed using Southern Pine glulam ID #50. Southern Pine is one of the best species of wood for pressure treatment because it absorbs the pressure treatment fluid better than other species of wood. Pressure treatment will be required for the wood trusses due to the harsh natatorium environment and is discussed in more detail in the M.A.E. Breadth, which is a continuation of the Building Enclosure breadth using information from AE 537. Southern Pine glulam ID #50 also

has rather high strength characteristics, which was deemed to be beneficial for the high anticipated member loads. Several sketches were made of various truss configurations, and these were later modeled in SAP2000. Loads that were applied to the top chord of the truss included the weight of laminated wood decking as well as other roofing dead loads, snow loads, and roof live load. A distributed dead load of 10 psf was applied to the bottom chord as well to account for the weight of speakers and any lighting fixtures mounted to the bottom chord. The appropriate loads were applied to the models in SAP2000, and the program was used to obtain the resulting member axial forces. The 2005 National Design Specification for Wood Construction was used to design the members. Resulting member forces, load combinations, and member design calculations are found in Appendix A.

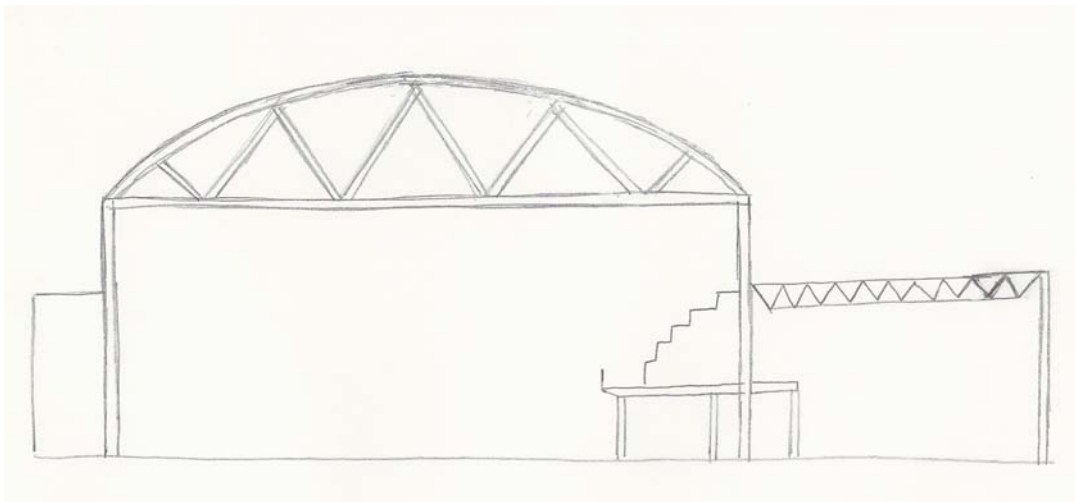


Figure 33 - Preliminary Sketch of Potential Wood Truss Configuration

The first truss configurations that were designed had shapes much like those of the steel king post trusses that were designed. However, as was determined with the king post truss system, these configurations lacked much architectural style. Trusses were designed using a large depth of 20'-0" to help minimize the axial forces in the top and bottom chords, especially since the members were going to be wood. Even with the 20'-0" depth, the resulting member forces were still very high due to the 130'-0" span. The initial wood truss designs were modified by adding in more and more web members to reduce the extensive unbraced lengths of the top chord and to add more of a curved shape to the roof. Truss designs that were investigated are shown below in Figures 34 to 40. Trusses were initially designed at a spacing of 10'-0" o.c., which resulted in an average force in the top and bottom chords of 45,000 – 50,000 lb each. Members for the final selected truss shape, which separated the top chord into ten members, were designed using sawn lumber and glulam members for a preliminary comparison of which of the two would be best for the trusses. Designing using sawn lumber resulted in either a 6x8 or 4x10 Select Structural Southern Pine bottom chord and either an 8x12 No. 1 Southern Pine, 8x10 Dense Select Structural Southern Pine, or 6x24 Dense Select Structural Southern Pine top chord. Due to the large sizes and small chance of finding members of

these sizes and required lengths, it was determined that designing using glulam members would be the best option.

Loads Applied to Top Chord of Glulam Trusses	
DEAD	PSF
Zinc Standing Seam Metal Roof Panels	1.5
1/2" Moisture Resistant Gypsum Board	2.5
4 1/2" Rigid Insulation	6.75
3" Decking	7.6
Superimposed	5
Assumed Self Weight	5
Total	28.35
Use	30
LIVE	
L_r	20
SNOW	
S	23.1

Table 1 - Loads Applied to Top Chord of Glulam Trusses

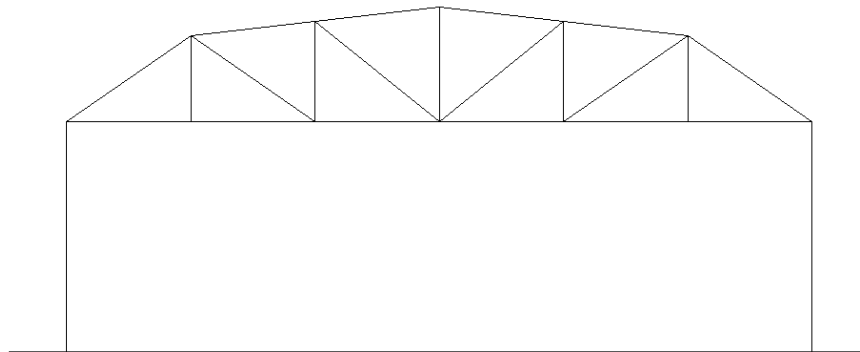


Figure 34 - Wood Truss Configuration with Top Chord Separated into 6 Members

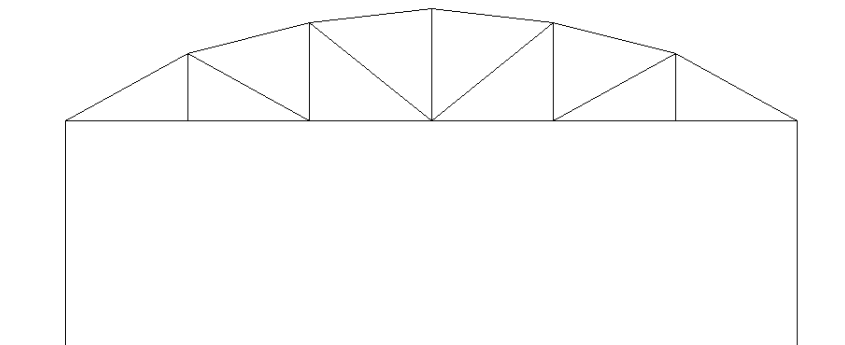


Figure 35 - Modified Wood Truss Configuration with Top Chord Separated into 6 Members

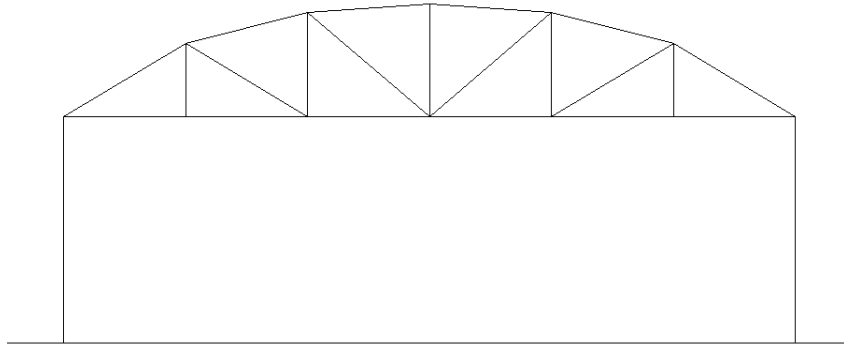


Figure 36 - Another Modified Wood Truss Configuration with Top Chord Separated Into 6 Members

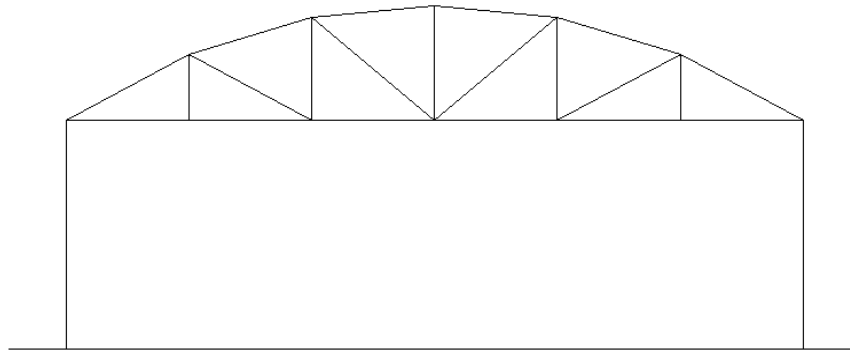


Figure 37 - Another Modified Wood Truss Configuration with Top Chord Separated Into 6 Members
(Curved Shape is Slightly Different than Previous Design)

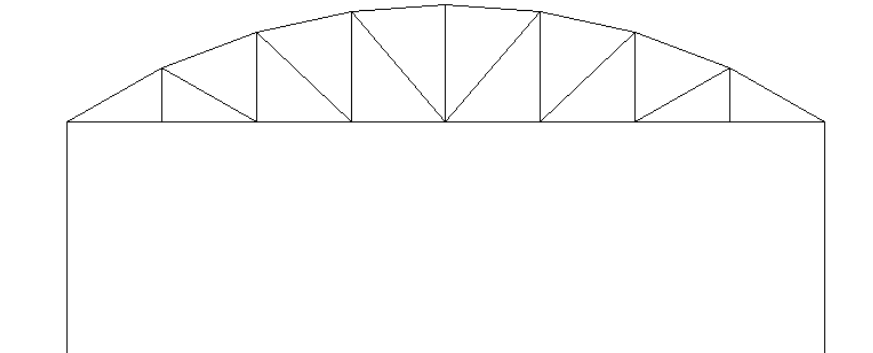


Figure 38 - Wood Truss Configuration with Top Chord Separated Into 8 Members

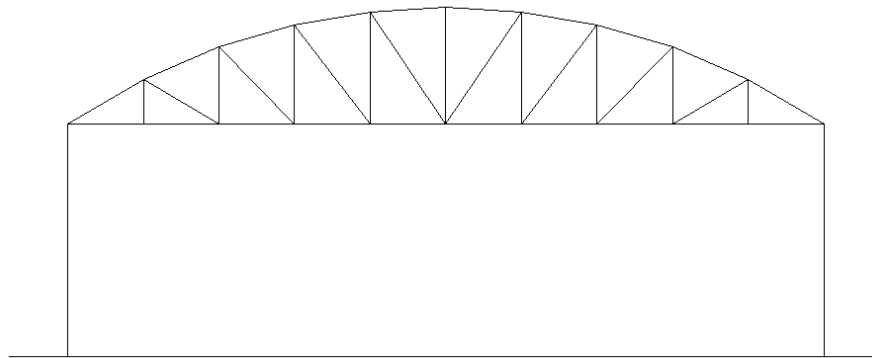


Figure 39 - Wood Truss Configuration with Top Chord Separated Into 10 Members

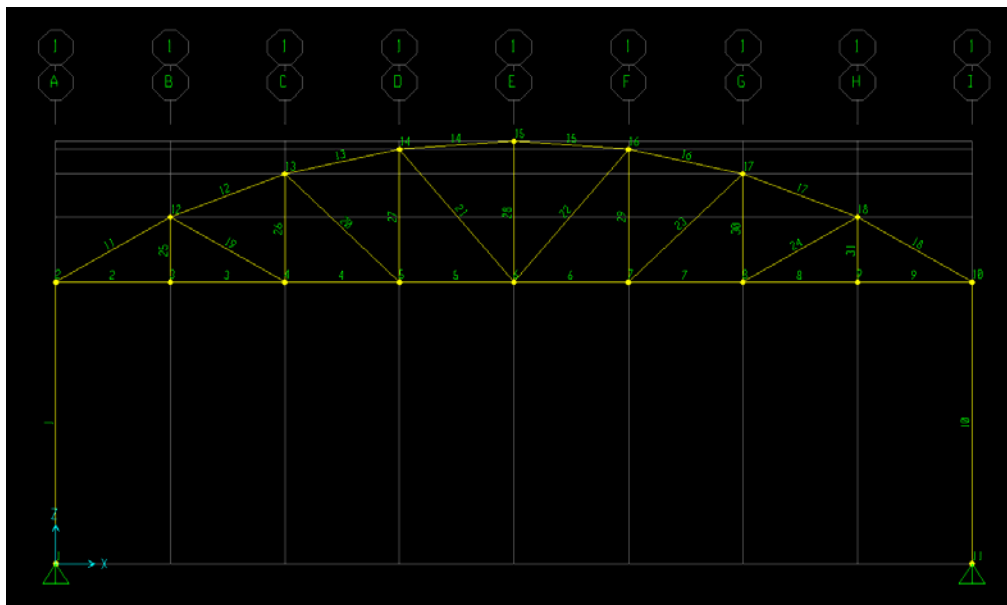


Figure 40 - SAP2000 Model of Glulam Truss Configuration with Top Chord Separated Into 8 Members

The final glulam truss configuration with trusses spaced at 8' o.c. resulted in a bottom chord tensile axial force of approximately 50,000 lb and a top chord compressive axial force that was also about 50,000 lb. These were rather high forces, even with the trusses at the smaller 8'-0" o.c. spacing. All truss members were designed to be the same width so that bolted metal side plates could easily be attached to the sides of the members. It was determined that bolted metal side plates would be the best option for connecting members of this size and for the high loads being transferred being the members. The final glulam truss design resulted in a 6 3/4" x 12 3/8" top chord, a 6 3/4" x 8 1/4" bottom chord, 6 3/4" x 6 7/8" web members, and 6 3/4" x 15 1/8" columns supporting the west ends of the trusses. Calculations are found in Appendix A. Design of the wood columns is discussed in the next section. All members are Southern Pine glulam ID #50. The bottom chord is spliced at three locations, which breaks up the bottom chord into four members. The top chord is broken up into ten individual members, so connections are

required at each joint where the top chord members meet. Web members also connect into each of these joints. The trusses bear on glulam columns on the west side and will bear on a concrete moment frame on the east side. The design of connections and of the concrete moment frame is discussed in later sections.

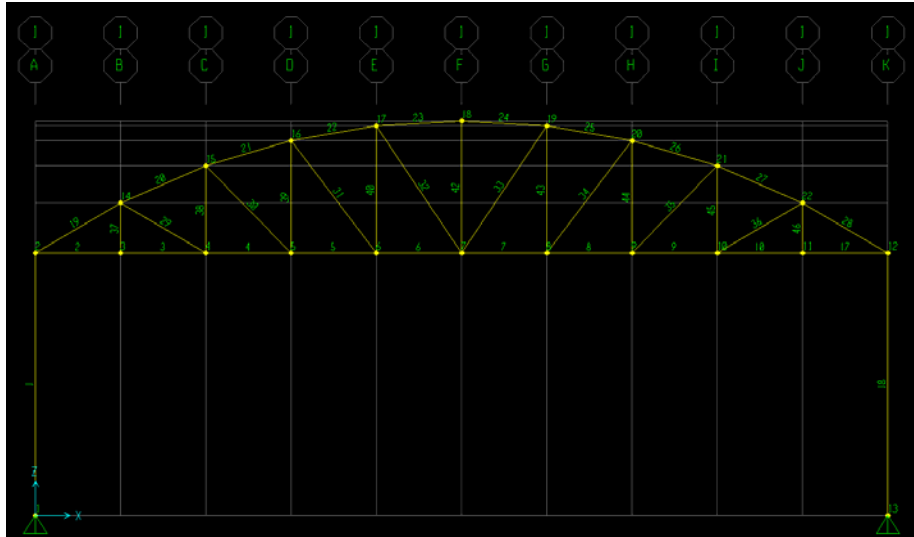


Figure 41 - SAP2000 Model of the Final Selected Glulam Truss Configuration

SUMMARY	
Top Chord	6 3/4" x 12 3/8"
Bottom Chord	6 3/4" x 8 1/4"
Web Members	6 3/4" x 6 7/8"
West Column	6 3/4" x 15 1/8"
All members are Southern Pine, Glulam I.D. #50	

Table 2 - Summary of Member Sizes of Final Glulam Truss Configuration

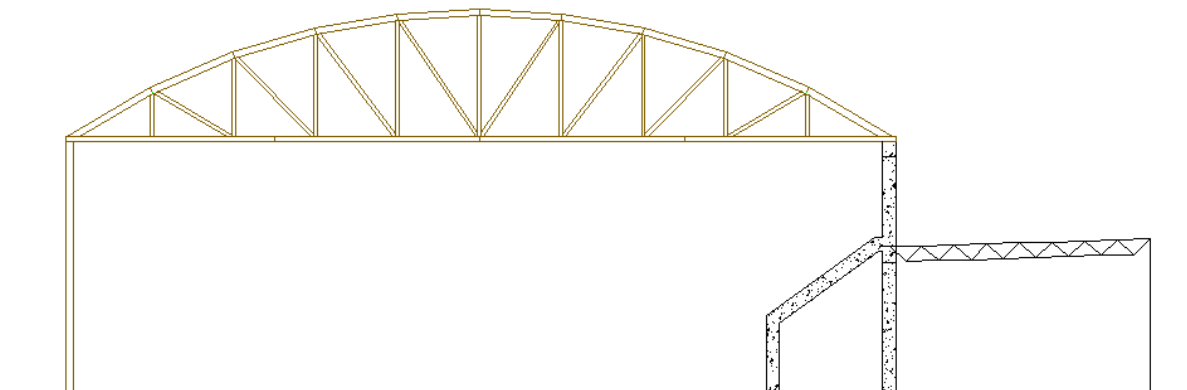


Figure 42 - Final Glulam Truss Configuration

The glulam truss system was generally found to be a more cost effective solution than the steel space frame and more architecturally pleasing than both the steel space frame and the steel king post truss system. The main problem encountered later in the design process was determining how the trusses would be transported to the job site. The 20'-0" depth at the midspan of the truss made this section too large to be transported on the road since it would not clear any overpasses or else would be imposing on other traffic lanes if laid diagonally. The wood trusses would be cheaper if they are able to be fabricated off-site. If designed again it may be beneficial to look into using a shallower depth for the trusses if transportation becomes a problem.

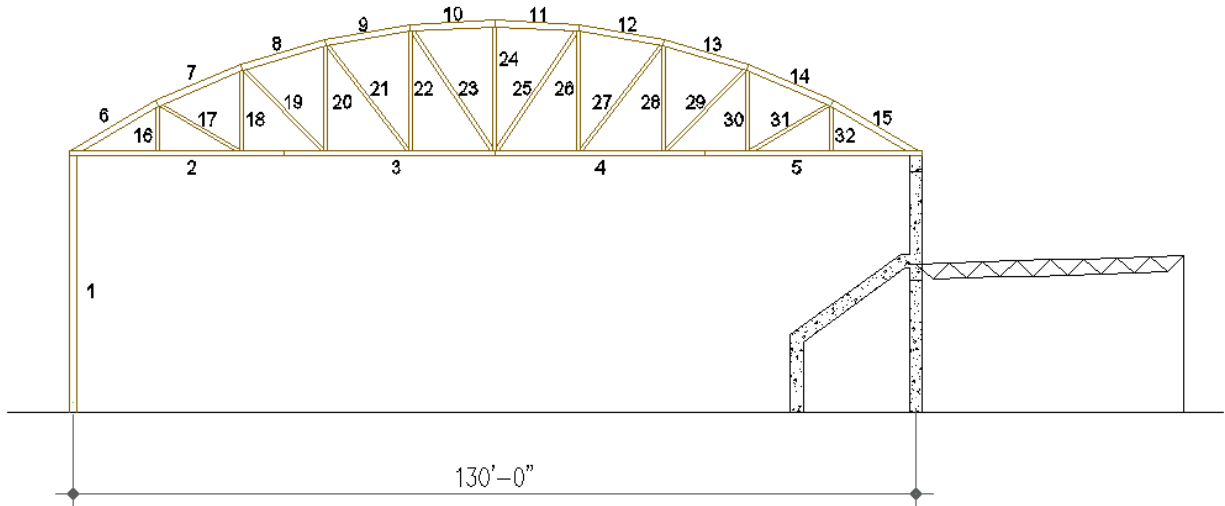


Figure 43 - Member Labels for Final Glulam Truss

Final Glulam Truss Member Lengths			
Member #	Length	Member #	Length
1	40'-0"	17	15'-1"
2	32'-6"	18	13'-2 11/16"
3	32'-6"	19	18'-6 1/2"
4	32'-6"	20	17'-0 7/16"
5	32'-6"	21	21'-5 3/16"
6	15'-1"	22	19'-3 3/16"
7	14'-1 3/4"	23	23'-2 15/16"
8	13'-6 9/16"	24	20'-0"
9	13'-2 1/4"	25	23'-2 15/16"
10	13'-0 1/4"	26	19'-3 3/16"
11	13'-0 1/4"	27	21'-5 3/16"
12	13'-2 1/4"	28	17'-0 7/16"
13	13'-6 9/16"	29	18'-6 1/2"
14	14'-1 3/4"	30	13'-2 11/16"
15	15'-1"	31	15'-1"
16	7'-7 3/4"	32	7'-7 3/4"

Table 3 - Final Glulam Truss Member Lengths (Coordinated with Figure Above)

Comparison

The three roofing systems that were investigated were compared in terms of cost, feasibility, and architectural impact. As mentioned above, the steel space frame system was determined to be a costly system for the Farquhar Park Aquatic Center. An estimation showed that the space frame weighed approximately three times as much as the original truss system and about five times as much as the alternate steel king post truss system. It also lacked architectural integrity due to the fact that a flat design had to be implemented in order to keep costs relatively low. Even with the plain, flat roof design, the cost would still be too high due to the excessive number of required members and connections. The steel king post truss system was expected to be lower in cost than the original design since the king post trusses had a much simpler configuration than the curved and tapered HSS trusses used in the original design. However, the king post truss system was too plain and, like the space frame system, did not provide much architectural freedom. The glulam truss system was determined to be the best option for the alternate roof system in terms of cost, feasibility, and architectural impact. With this system it was possible to develop a truss configuration with a nicely shaped curve without causing the cost of the system to skyrocket. The glulam truss system would also provide a relatively competitive cost compared to the steel king post truss system. Labor costs with wood construction are usually relatively low. Also, since the east ends of the trusses bear on the concrete moment frame, this eliminated the need for additional footings at this location. However, additional footings will be required under the columns that support the west ends of the trusses since the number of trusses was increased from 5 in the original design to 19 in the alternate glulam truss design. These footings may be much smaller than those used to support the west ends of the original trusses and truss columns, although they are greater in number. Further analysis is required to compare footings costs. Overall, the glulam truss system best met the goals of this thesis by providing a pleasing architectural appearance but keeping costs reasonable by not getting too fancy.

Comparison of Three Alternate Roof Systems			
	Cost	Feasibility	Architectural Impact
Steel King Post Trusses	Competitive	High	Poor
Steel Space Frame	High	Poor/Moderate	Poor/Moderate
Glulam Trusses	Competitive	High	High

Table 4 - Comparison of the Three Alternate Roof Systems that were Investigated

The glulam truss system was compared to the original design in terms of cost using RS Means Building Construction Cost Data (2009). A summary of the estimated costs are shown below in Tables 5 and 6. It was estimated that the new glulam truss wood system would be approximately \$100,000 cheaper than the original steel truss system. The trusses themselves were estimated to be nearly the same cost, but the laminated deck was found to be much less costly than the long-span metal roof deck used in the original design. The weight of the glulam truss roof system was determined to be about 544 kips, while the roof structural system of the original design was about 257 kips. While the weight of the roof structural system more than doubled, the glulam truss roof system was

still found to cost less than the original roof system. The glulam trusses also would have most likely been cheaper had curved top chord members been used. This would have eliminated the required number of splice connections for the top chord by maybe separating the top chord into three or four members instead of ten.

These tables also include the estimated costs of the new concrete moment frames and the original steel moment frames. The concrete moment frames are discussed in more detail in a later section. The cost estimation comparison only takes into account the parts of the building that changed, which was the roof structural system and the replacement of steel braced frames with concrete moment frames. The additional cost of the wood braced frames that were added was not included in this comparison. The design of these braced frames is discussed in a later section. This comparison does not take into account the cost of the moment connections for the original system nor any bolts or connections that were required for the original design. Also, weight per linear foot values for HSS18x18x5/8 shapes could not be found in the AISC Steel Construction Manual, so the weight of an HSS16x16x5/8 was used instead. Therefore, the estimated cost of the original system will be slightly higher than that calculated. The cost of special wood connections that may be required for the wood lateral system may be expensive and were not taken into account as well. Calculations for the cost comparison are found in Appendix A.

Estimated Cost Comparison	
	Cost (\$)
Wood Roof System	
Metal Side Plates	14,212.00
Laminated Roof Deck	113,770.80
Plywood Sheathing	17,643.60
Glulam Trusses	242,011.14
High-Strength Bolts	148,845.24
TOTAL	536,482.78
Steel Roof System (Original Design)	
Galvanizing of Trusses	15,253.27
Galvanizing of Metal Roof Deck	35,773.92
Metal Roof Deck	369,298.80
Steel Trusses	250,208.90
TOTAL	670,534.89
Concrete Moment Frames	
Formwork for Beams	22,381.23
Formwork for Columns	11,938.20
Columns	39,951.08
Beams	77,852.52
Reinforcing for Beams	23,215.57
Reinforcing for Columns	9,056.67
TOTAL	184,395.27
Steel Moment Frames (Original Design)	
Beams	56,433.24
Columns	45,960.09
Moment Connections	
TOTAL	102,393.33

Table 5 - Estimated Costs of Alternate Design versus Original Design

Total Overall Estimated Costs	
	Cost (\$)
Alternate Structural System	
TOTAL	720,878.05
Original Structural System	
TOTAL	772,928.22

Table 6 - Total Overall Estimated Costs of Alternate Design versus Original Design

Wood Decking

Wood structural panels, such as plywood, are usually used to span between closely spaced roof beams or trusses. Lumber sheathing is used to span longer distances. Due to the 8'-0" spacing of the glulam trusses, lumber sheathing was required for this alternate design. Lumber sheathing is available as solid decking or laminated decking. It was determined that laminated decking would be more cost effective if 3" or thicker decking is required. Nominal three and four inch decking is adapted well for use with glued laminated arches or trusses and can provide a pleasant all-wood appearance. The decking can also be erected quickly and easily. Timber decking can span from 3 – 20 feet, and the layup of the decking affects its capacity. Shown below are diagrams from WCD 2 – Tongue and Groove Roof Decking showing typical layups of tongue-and-groove decking.

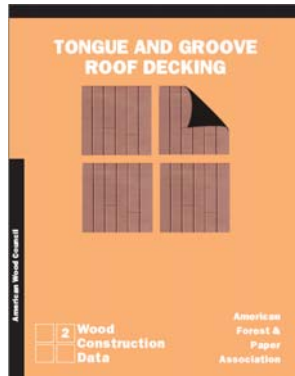


Figure 44 – WCD 2: Tongue and Groove Roof Decking

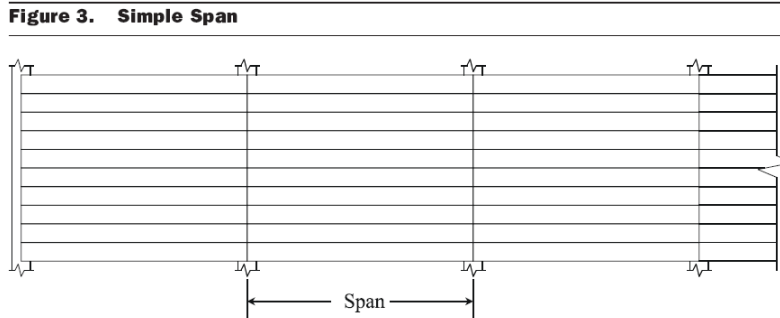


Figure 45 - Simple Span Layup (Image from WCD 2 – Tongue and Groove Roof Decking)

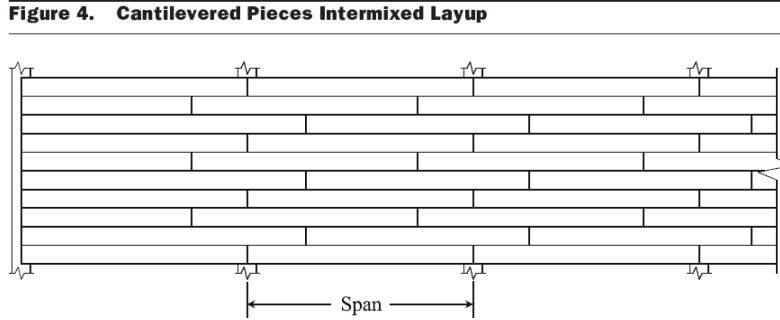


Figure 46 - Cantilevered Pieces Intermixed Layup (Image from WCD 2 – Tongue and Groove Roof Decking)

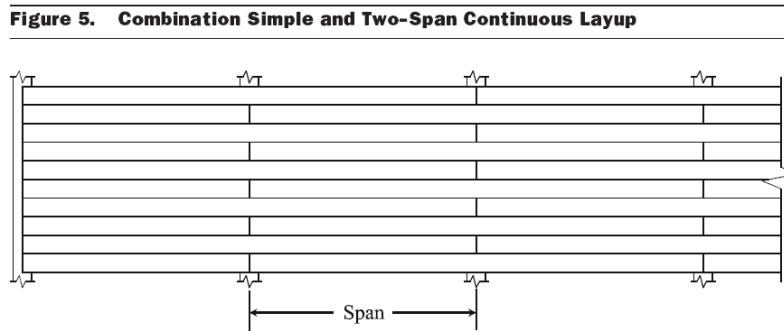


Figure 47 - Combination Simple and Two-Span Continuous Layup (Image from WCD 2 – Tongue and Groove Roof Decking)

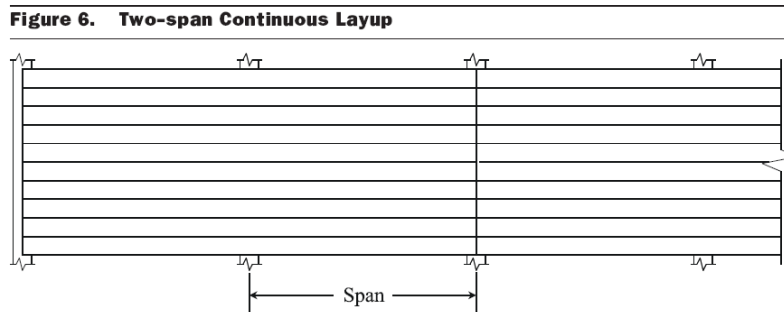


Figure 48 - Two-Span Continuous Layup (Image from WCD 2 – Tongue and Groove Roof Decking)

First the required thickness of heavy timber, or solid, roof decking was determined. Load tables from AITC 112*-81 Standard for Tongue-and-Groove Heavy Timber Roof Decking were used to determine the required thickness of decking. Table 3 from this Standard gives bending stress values and modulus of elasticity values for various species of wood to be used with the load tables. Southern Pine was selected to be used to match the Southern Pine trusses. Since the decking would be used where the moisture content will exceed 19% for an extended period of time, bending stress values were multiplied by a factor of 0.86 and modulus of elasticity values by a factor of 0.97. A two-span continuous layup was chosen for the decking. For nominal two inch decking, the

allowable roof load was limited by deflection. It was determined that nominal two inch decking would barely work for the two-span continuous layup. The two-span continuous layup has the highest capacity out of all the layups. The allowable roof load to meet the $L/240$ deflection criteria for this layup is 53.65 psf. The appropriate C_D factor must be applied to the given values for different load combinations. For the load combination $D + L_r$, the value of 53.65 psf will apply since $C_D = 1.0$ for this load combination. The total load from the controlling load combination $D + L_r$ was 50 psf, which works but is very close to the allowable deflection limit. Therefore, it was decided that nominal three inch decking should be provided due to any possible uncertainties in the calculated loads. The required nailing schedule for three and four inch decking is given from this AITC Standard as follows: “Each piece should be toenailed at each support with one 40d nail and face nailed with one 60d nail. Courses shall be spiked to each other with 8 in. spikes at intervals not to exceed 30 in. through predrilled edge holes and with one spike at a distance not exceeding 10 in. from each end of each piece.” Calculations are found in Appendix A.

Next, the required thickness of laminated decking was determined. Span-load tables from Section 7 of the Timber Construction Manual were used. Table 7.9 from this Section provided values for Southern Pine, so Southern Pine was again selected as the species for the decking. This table gave allowable uniformly distributed total roof load values limited by deflection for controlled random layup decking. The smallest size given for Southern Pine, with an actual size of $2 \frac{3}{16}'' \times 5 \frac{3}{8}''$, had a capacity of 136 psf for the deflection limit of $L/240$. The actual load for the controlling load combination of $D + L_r$ was well within this limit. The footnotes at the bottom of the table also state that the actual size for Southern Pine is $2 \frac{1}{4}'' \times 5 \frac{3}{8}''$. Therefore, it was determined that nominal three inch Southern Pine laminated decking would be used with an actual size of $2 \frac{1}{4}'' \times 5 \frac{3}{8}''$. In addition, it was decided that the laminated decking would be used instead of the heavy timber, or solid, roof decking. The Southern Pine laminated decking would better match the appearance of the Southern Pine glued-laminated trusses. Plus, the laminated decking would generally be cheaper than solid decking due to the thicker required decking size.

Diaphragm

It is sometimes difficult or costly to obtain diaphragm action from three inch tongue-and-groove decking alone. Sometimes adhesives can be applied on top of the tongue-and-groove joints to help achieve diaphragm resistance. Certain nailing schedules can also be applied to the tongue-and-groove joints, but this can result in increased labor costs. The most common method to obtain diaphragm action when using three inch tongue-and-groove decking is to install plywood or another structural panel over the decking. The decking provides the required blocking, and the requirements for nailing of panel edges basically stay the same as if the panels were being installed over joists. For this design, plywood was designed to provide diaphragm resistance and would be nailed on top of the tongue-and-groove decking. ANSI / AF&PA SDPWS-2005 “Special Design Provisions for Wind and Seismic” was used to determine the required thickness of plywood for the

design wind and seismic loads applied to the building. The required thickness for seismic loads was found to govern. The final design consisted of 3/8" Structural I plywood with all edges supported and nailed into three inch minimum nominal framing, 8d common nails at 6-in. o.c. at boundary and continuous panel edges, 6-in. o.c. at other panel edges (blocking is provided by the tongue-and-groove decking), and 12-in. o.c. in the field. Calculations are found in Appendix A.

Chords were also designed for the required diaphragm forces. The axial forces in the chords were determined by resolving the diaphragm moment into a couple for both the longitudinal direction and the transverse direction. For the longitudinal direction, the wood members at the top of the braced frames were designed to function as the chord members. Seismic loads controlled the design and resulted in a 3 1/2" x 5 1/2" member using Southern Pine glulam ID #50. For the transverse direction, the wood members at the top of the braced frames in the North/South direction were designed to act as the chord members. Seismic loads also controlled the design of these members, which resulted in 6 3/4" x 8 1/4" members using Southern Pine glulam ID #50.

Wood Columns

The columns supporting the west end of the trusses were steel in the original design. For the alternate wood design, it was decided that glulam columns would be used to match the glulam roof trusses. The columns were designed to take all the roof loads, although SAP2000 models showed that a large portion of this load was carried by the braces. The columns were also designed to take lateral wind loads that were applied to the west façade. The columns were assumed to be pinned at the top and bottom, and the resulting moment due to wind load was rather large due to the 40'-0" unbraced length of the column. The design resulted in 6 3/4" x 15 1/8" columns using Southern Pine glulam ID #50. Calculations for the design of the glulam columns are found in Appendix A.

Wood Truss Member Connections

Bolted metal side plate connections were designed to connect the members of the glulam trusses. This was considered to be the best design option due to the large member forces in the top and bottom chord. Connections were designed using the 2005 National Design Specification for Wood Construction. The load combination $D + L_r$ controlled all connection designs. Connections were designed using 1/4" steel side plates. Nominal design values for 3/4" bolts in double shear for a 6 3/4" thick Southern Pine glulam member with 1/4" steel side plates and load applied parallel to grain were provided in Table 11I of the NDS. A wet service factor of 0.7 was applied to the connection designs due to the high moisture levels in the natatorium. All edge distance, end distance, and spacing requirements were met for all connections to obtain a geometry factor of one. Bottom chord heel connections and splice connections both resulted in (24) 3/4" diameter bolts arranged in two rows. The spacing between bolts in a row was 3", and the spacing between rows of bolts was 2 7/8". Six inch steel plates were used to architecturally allow

a portion of the glulam members to be seen around the edges of the plates instead of making the plates cover the entire depth of the bottom chord. Due to the large number of required bolts for the top chord and bottom chord connections, the use of 4-inch diameter shear plate connectors was investigated. However, the design resulted in a required fifteen 4-inch diameter shear plates using a geometry factor of 1.0, which requires a 9" spacing between the shear plates in a row for parallel to grain loading. This would result in an unrealistically large connection. Therefore, the final connection used the (24) 3/4" diameter bolts arranged in two rows.

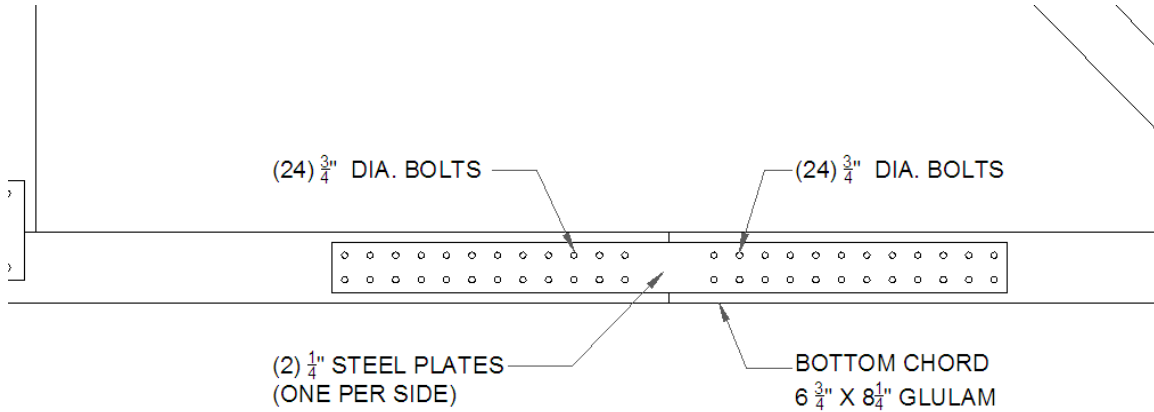


Figure 49 - Typical Bottom Chord Splice Connection

Top chord connections resulted in (28) 3/4" diameter bolts arranged in two rows. These connections were designed for the highest top chord force, and the same connection was used for all top chords. The forces in the top chords were all relatively close in magnitude, so it was valid to use the same connection for all top chord connections. Plus, this would create a more pleasing architectural appearance if all of these connections are the same. Eight inch steel plates were used instead of six inch plates due to the larger depth of the top chord. Architecturally, it was desired to keep approximately the same percentage of wood clearance around the edges of the plates as that for the bottom chord connections.

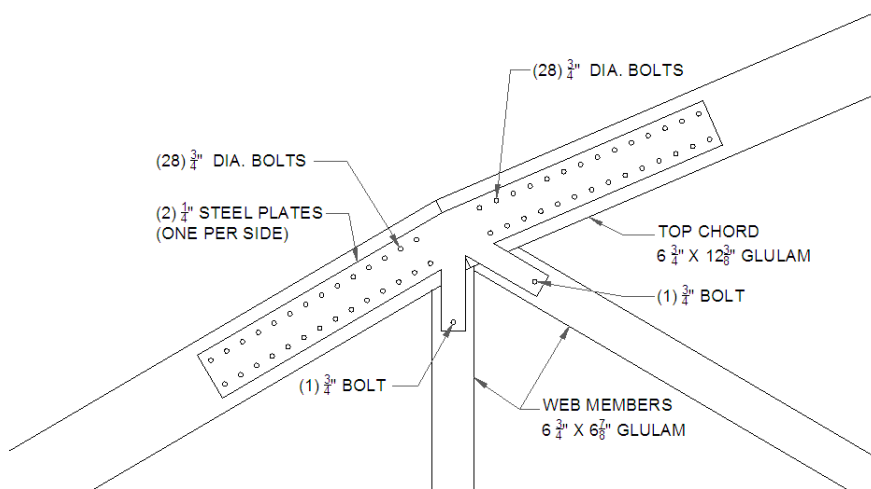


Figure 50 - Typical Top Chord Connection

The resulting axial forces in the web members were very small compared to the forces in the top and bottom chord. All forces in the web members were around 1,000 lb or less. Therefore, this permitted the use of (1) $\frac{3}{4}$ " diameter bolt connections to connect the web members to the chords. Several sources suggest using overlapping plates at connections such as this to essentially maintain a pinned connection, but a single plate for the entire connection was chosen for this design. The use of full single plates is also more common and has a more appealing appearance architecturally.

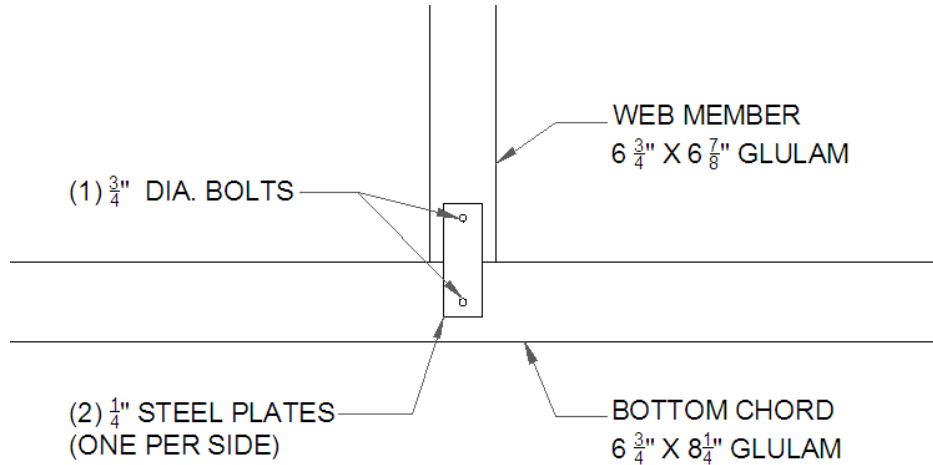


Figure 51 - Typical Vertical Web Member Connection to Bottom Chord

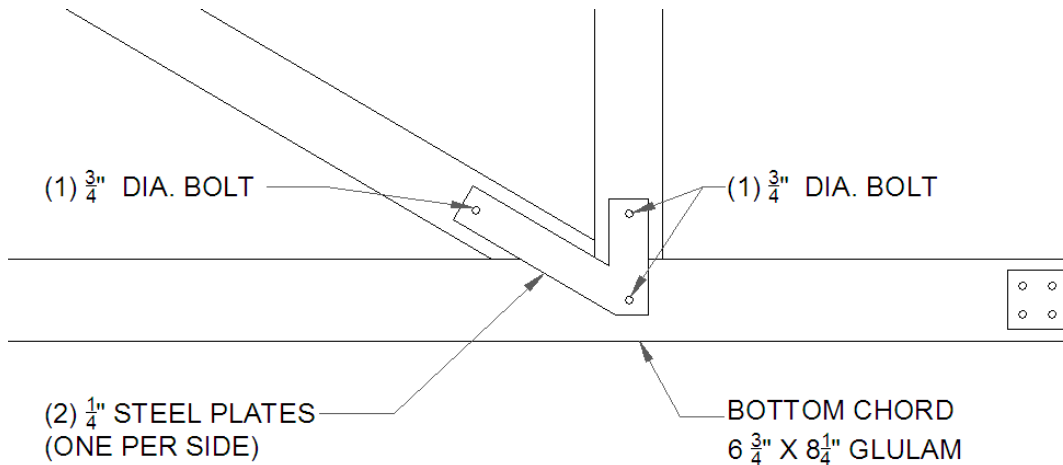


Figure 52 - Typical Web Member Connection to Bottom Chord

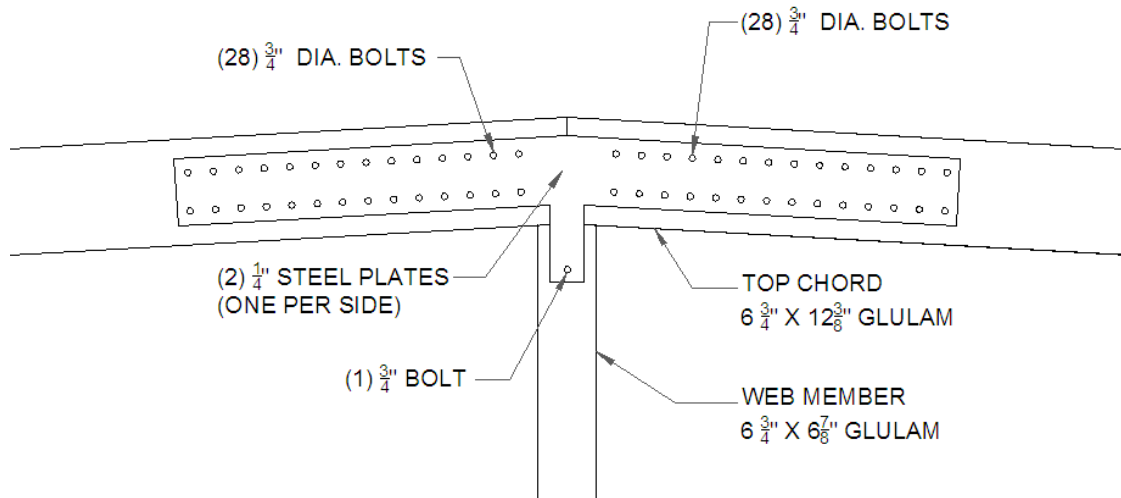


Figure 53 - Top Chord Connection

More advanced connections may be required at the heel connections where the trusses and braces in the North/South direction meet at the top of the wood columns. Special saddle-type connections may need to be investigated in which the truss would rest in the saddle while lateral bracing members can frame into the sides of the glulam column by steel angles. All members would most likely be bolted.

Wood Truss Connection to Concrete Moment Frame at Column Line 2

The east ends of the steel trusses of the original design for the Farquhar Park Aquatic Center were supported by steel HSS columns. The east ends of the glulam trusses of the alternate design must frame into the new concrete moment frame at column line two. The design of this moment frame is discussed in more detail in a later section. All lateral forces perpendicular to and parallel to the moment frame that are to be resisted by the frame must be properly transferred from the roof diaphragm to the concrete moment frame. A typical connection detail is shown below. Lateral forces perpendicular to the moment frame are transferred from the wood plate to the concrete beam or concrete column by anchor bolts. The same occurs for lateral forces parallel to the concrete moment frame. The strength of the anchor bolts is governed by the capacity of the bolt parallel to grain or perpendicular to grain in the wood plate, or by its capacity in the concrete.

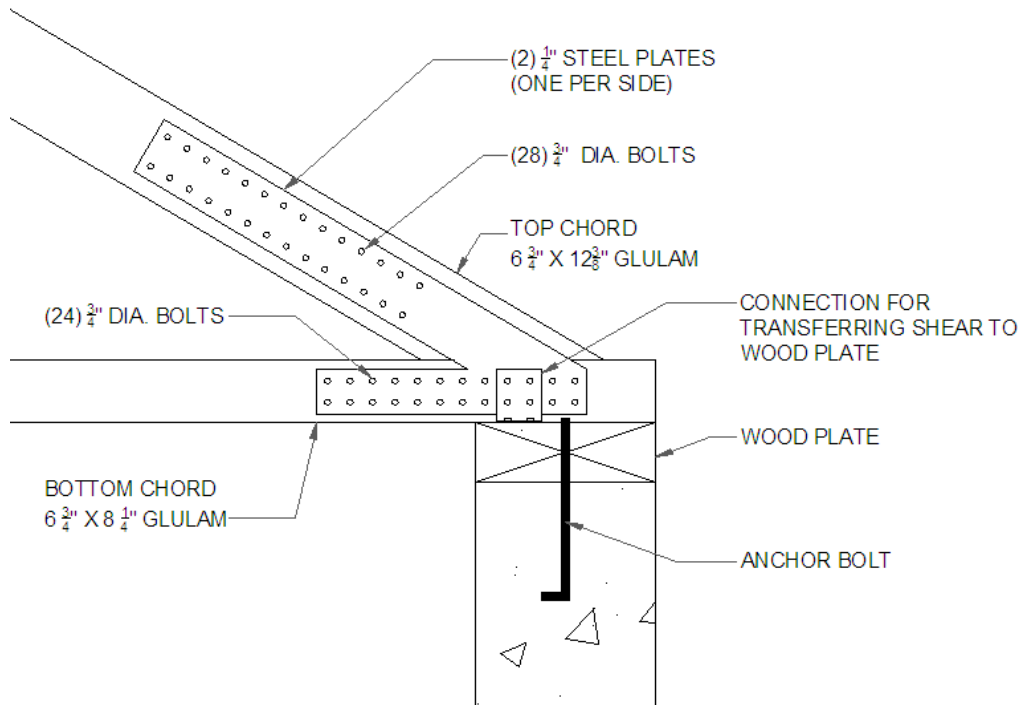


Figure 54 - Typical Connection of Glulam Truss to Concrete Beam or Column of Concrete Moment Frame at Column Line 2

Lateral System Study

Wind Loads

Method 2 – Analytical Procedure of ASCE 7-05 Section 6.5 was used to determine wind loads. Wind loads had to be re-calculated from those used in Technical Report 3 due to changes in the building height and changes in the applicable wind load equations due to the switch from steel moment frames to concrete moment frames at column lines one and two. Although the updated wind loads resulted in lower wind pressures as compared to those from the Technical Report 3, the base shear in the North/South direction slightly increased due to the larger wall surface area that resulted from the glulam truss configuration. The tops of the columns along column line 2 were also raised from about 37'-0" to 40'-0" above ground level, which added surface area to the North and South facades. The base shear in the East/West direction for the alternate design was considerably less than that from Technical Report #3 due to the change in roof shape. The curved roof shape using the glulam trusses resulted in wind uplift loads on the roof, while the original design had to account for horizontal wind load for basically the entire height of the building due to the nearly vertical west wall. Variables used in the wind calculations are located in Table 7 and wind loads are noted in Tables 8, 9, and 10. Calculations for wind loads are found in Appendix B.

Wind Variables			ASCE 7-05 Reference
Basic Wind Speed	V	90 mph	Figure 6-1 (p. 33)
Wind Directionality Factor	K_d	0.85	Table 6-4 (p. 80)
Importance Factor	I	1.15	Table 6-1 (p. 77)
Exposure Category		C	Sec. 6.5.6.3
Topographic Factor	K_{zt}	1.0	Sec. 6.5.7.1
Velocity Pressure Exposure Coefficient Evaluated at Height z	K_z	Varies	Table 3 (p. 79)
Velocity Pressure at Height z	q_z	Varies	Eq. 6-15
Velocity Pressure at Mean Roof Height h	q_h	22.904	Eq. 6-15
Equivalent Height of Structure	z	36	Table 6-2
Intensity of Turbulence	I_z	0.197	Eq. 6-5
Integral Length Scale of Turbulence	L_z	508.78'	Eq. 6-7
Background Response Factor (North/South)	Q	0.9272	Eq. 6-6
Background Response Factor (East/West)	Q	0.8636	Eq. 6-6
Gust Effect Factor (North/South)	G	0.858	Eq. 6-4
Gust Effect Factor (East/West)	G	0.85	Eq. 6-4
External Pressure Coefficient (Windward)	C_p	0.8	Figure 6-6 (p. 49)
External Pressure Coefficient (N/S Leeward)	C_p	-0.5	Figure 6-6 (p. 49)
External Pressure Coefficient (E/W Leeward)	C_p	-0.4654	Figure 6-6 (p. 49)

Table 7 - Wind Variables

"Building 1" - Wind Loads (North/South Direction) B=183'-0", L=156'-0"															
Level	Height Above Ground - z (ft)	Story Height (ft)	K _z	q _z	Wind Pressure (psf)				Total Pressure (psf)	Force (k) of Windward Only	Force (k) of Total Pressure	Story Shear Windward (k)	Story Shear Total (k)	Moment Windward (ft-k)	Moment Total (ft-k)
					Windward	Leeward	Side Walls	Roof							
4	60.0	20.0	1.13	22.90	15.72	-9.04	-12.66	-16.28	24.76	0.00	0.00	0.00	0.00	0.00	0.00
3	40.0	15.3	1.04	21.08	14.47	-9.04	-12.66	-16.28	23.51	41.03	66.68	41.03	66.68	1641.29	2667.10
2	24.7	14.2	0.937	19.00	13.04	-9.04	-12.66	-16.28	22.08	27.43	46.46	68.47	113.13	676.67	1145.92
1	10.5	10.5	0.85	17.23	11.83	-9.04	-12.66	-16.28	20.87	21.32	37.63	89.79	150.76	223.88	395.07
sum(Story Shear (Windward))=89.78 k					sum (Story Shear (Total))=150.77 k										
sum(Moment (Windward))=2541.84 ft-k					sum (Moment (Total))=4208.09 ft-k										

Table 8 - Wind Loads to Indoor Pool Area – N/S direction (these loads are applied to the braced frame at column line 1 and the moment frame at column line 2)

*Wind load at Level 4 gets applied to Level 3 for lateral force resisting system

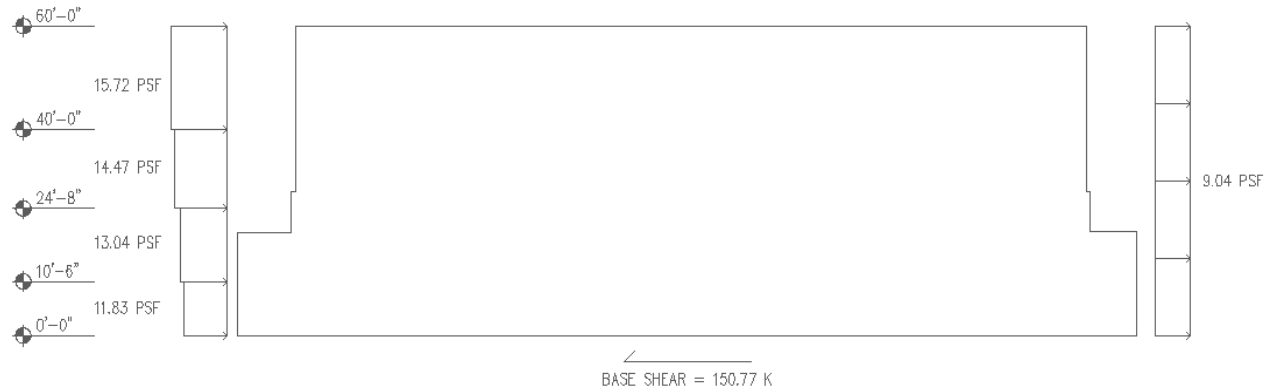


Figure 55 - "Building 1" Wind Loads – North/South

"Building 4" - Wind Loads (North/South Direction) B=183'-0", L=156'-0"															
Floor	Height Above Ground - z (ft)	Story Height (ft)	K _z	q _z	Wind Pressure (psf)				Total Pressure (psf)	Force (k) of Windward Only	Force (k) of Total Pressure	Story Shear Windward (k)	Story Shear Total (k)	Moment Windward (ft-k)	Moment Total (ft-k)
					Windward	Leeward	Side Walls	Roof							
3	40.0	15.3	1.04	21.08	14.47	-9.04	-12.66	-16.28	23.51	0.00	0.00	0.00	0.00	0.00	0.00
2	24.7	14.2	0.937	19.00	13.04	-9.04	-12.66	-16.28	22.08	8.41	14.21	8.41	14.21	207.55	350.61
1	10.5	10.5	0.85	17.23	11.83	-9.04	-12.66	-16.28	20.87	0.00	0.00	8.41	14.21	0.00	0.00
sum(Story Shear (Windward))=8.41 k					sum (Story Shear (Total))=14.21 k										
sum(Moment (Windward))=207.55 ft-k					sum (Moment (Total))=350.61 ft-k										

Table 9 - Wind Loads to Lobby Area – N/S direction (these loads are applied to the braced frame at column line 2 and the moment frame at column line 4)

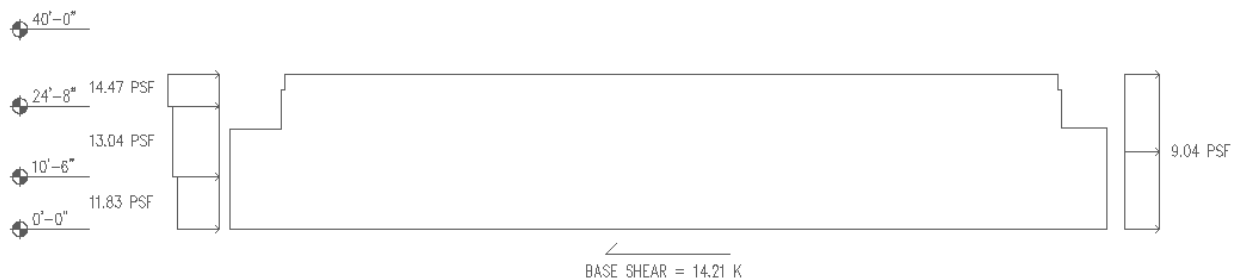


Figure 56 - "Building 4" Wind Loads – North/South

Wind Loads (East/West Direction) B=156'-0", L=183'-0"															
Floor	Height Above Ground - z (ft)	Story Height (ft)	K _z	q _z	Wind Pressure (psf)				Total Pressure (psf)	Force (k) of Windward Only	Force (k) of Total Pressure	Story Shear Windward (k)	Story Shear Total (k)	Moment Windward (ft-k)	Moment Total (ft-k)
					Windward	Leeward	Side Walls	Roof							
4	60.0	20.0	1.13	22.90	15.58	-8.34	-12.54	-16.13	23.91	0.00	0.00	0.00	0.00	0.00	0.00
3	40.0	15.3	1.04	21.08	14.33	-8.34	-12.54	-16.13	22.67	16.97	26.85	16.97	26.85	678.96	1074.08
2	24.7	14.2	0.937	19.00	12.92	-8.34	-12.54	-16.13	21.26	30.95	51.49	47.93	78.34	763.54	1270.03
1	10.5	10.5	0.85	17.23	11.72	-8.34	-12.54	-16.13	20.05	21.58	44.89	69.51	123.23	226.58	471.35
sum(Story Shear (Windward))=69.50 k					sum (Story Shear (Total))=123.23 k										
sum(Moment (Windward))=1669.08 ft-k					sum (Moment (Total))=2815.46 ft-k										

Table 10 - Wind Loads to Entire Building – E/W direction (these loads are applied to the perimeter braced frames in the E/W direction)

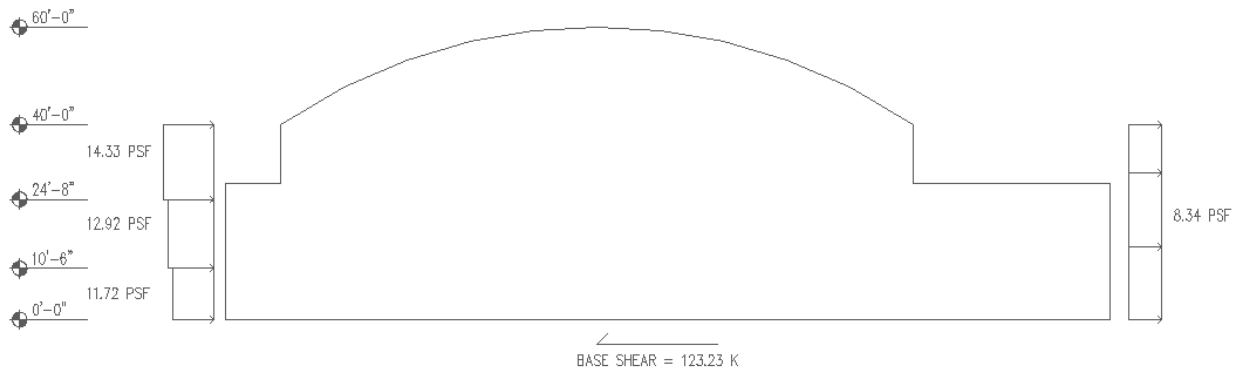


Figure 57 - Wind Loads on Entire Building (East/West)

Seismic Loads

Seismic loads were determined using ASCE 7-05. Seismic loads had to be recalculated for the alternate design due to changes in the weight of the building. The weight of the glulam truss roof system was considerably heavier than the original steel roof structure, and the weight of the concrete moment frames was much heavier than the weight of the original steel moment frames. Values of R and C_s also changed due to changes in the building's lateral force resisting systems. For Technical Report 3, an R-value of 3 was used for "Steel systems not specifically detailed for seismic resistance, excluding cantilever column systems." For the wood braced frames of the alternate design, an R-value of 4 was used for "Light-framed wall systems using flat strap bracing", which was the closest category from Table 12.2-1 (ASCE 7-05) that applied. For the concrete moment frames, an R-value of 3 was used for "Ordinary Reinforced Concrete Moment Frames". After performing the seismic load calculations, it was determined that the C_s value for the wood braced frames was higher than the C_s value for the concrete moment frames. Therefore, the higher, more conservative C_s value was applied to all lateral force resisting frames throughout the entire building. Variables used in the seismic calculations are located in Table 11 and seismic loads are noted in Tables 13, 14, 15, and 16. Calculations for seismic loads are found in Appendix B.

Seismic Design Variables			ASCE Reference
Site Classification		C	
Occupancy Category		III	
Structural System		Steel Systems Not Specifically Detailed for Seismic Resistance, Excluding Cantilever Column Systems	Table 12.2-1
Spectral Response Acceleration, Short Period	S_S	0.2	Figure 22-1
Spectral Response Acceleration, 1-Second Period	S_1	0.054	Figure 22-2
Site Coefficient	F_a	1.2	Table 11.4-1
Site Coefficient	F_v	1.7	Table 11.4-2
MCE Spectral Response Acceleration, Short Period	S_{MS}	0.24	Eq. 11.4-1
MCE Spectral Response Acceleration, 1-Second Period	S_{M1}	0.0918	Eq. 11.4-2
Design Spectral Acceleration, Short Period	S_{DS}	0.16	Eq. 11.4-3
Design Spectral Acceleration, 1-Second Period	S_{D1}	0.0612	Eq. 11.4-4
Seismic Design Category	SDC	A	Table 11.6-1
Response Modification Coefficient	R	3	Table 12.2-1
Importance Factor	I	1.25	Table 11.5-1
Approximate Period Parameter	C_t	0.02	Table 12.8-2
Building Height (above grade)	h_n	60 ft	
Approximate Period Parameter	x	0.75	Table 12.8-2
Approximate Fundamental Period	T_a	0.4312	Eq. 12.8-7
Long Period Transition Period	T_L	6 sec	Figure 22-15
Calculated Period Upper Limit Coefficient	C_u	1.7	Table 12.8-1
Fundamental Period	T	0.4312	
Seismic Response Coefficient	C_s	0.044353	Eq. 12.8-2
Structure Period Exponent	k	1.0	

Table 11 - Seismic Design Variables

Level	Elevation
3	40'-0"
2	24'-8"
1	10'-6"

Table 12 - Elevations Corresponding to the Levels Used in the Lateral Analysis

“Building 1”

Building 1 - Level 1			
Weights of Building Components		Center of Mass	
Component	Weight	x (ft)	y (ft)
Large Truss Columns	4.078 kips	1.1510	78.0000
Wind Columns	7.785 kips	51.9010	78.0000
Precast Concrete Panels	484.223 kips	31.5959	80.8546
Total=	496.085 kips	31.6643	80.7863

Building 1 - Level 2			
Weights of Building Components		Center of Mass	
Component	Weight	x (ft)	y (ft)
Large Truss Columns	4.877 kips	1.1510	78.0000
Wind Columns	9.078 kips	51.9010	78.0000
Conc. Moment Frame at C.L. 2	156.000 kips	130.0000	78.0000
Precast Concrete Panels	458.031 kips	30.0784	81.5370
Precast Concrete Sills	112.577 kips	60.4948	78.0000
Total=	740.563 kips	55.8277	80.1876

Building 1 - Level 3			
Weights of Building Components		Center of Mass	
Component	Weight	x (ft)	y (ft)
Large Wood Truss Columns	2.535 kips	1.1510	78.0000
Large Wood Trusses	87.718 kips	66.1510	78.0000
Wind Columns	9.353 kips	51.9010	78.0000
Conc. Moment Frame (C.L. 2)	119.000 kips	130.0000	78.0000
Roofing	374.400 kips	66.1510	78.0000
Total=	593.006 kips	52.7936	78.0000

Seismic Loads - "Building 1"										
Level	Story Weight w_x	Height h_x (ft)	h_x^k	$w_x h_x^k$	C_{vx}	Lateral Force F_x	Story Shear V_x	Moments M_x (ft-k)		
3	593.01	40.00	40.00	23720.24	0.503	40.79	0.00	1631.40		
2	740.56	24.67	24.67	18267.22	0.387	31.41	40.79	774.76		
1	496.09	10.50	10.50	5208.89	0.110	8.96	72.19	94.04		
sum($w_x h_x^k$)=		47196.35		sum(F_x)=V=			81.15 kips		sum(M_x)=	2500.20
Total Weight of "Building 1" (Above Grade) =				1829.65 kips						

Table 13 - Seismic Loads – “Building 1”

*Seismic force at Level 4 is applied to Level 3 for lateral force resisting system

$$V = C_s W = (0.044353)(1829.65 \text{ kips}) = 81.15 \text{ kips}$$

$$C_{vx} = w_x h_x^k / \text{sum}(w_i h_i^k)$$

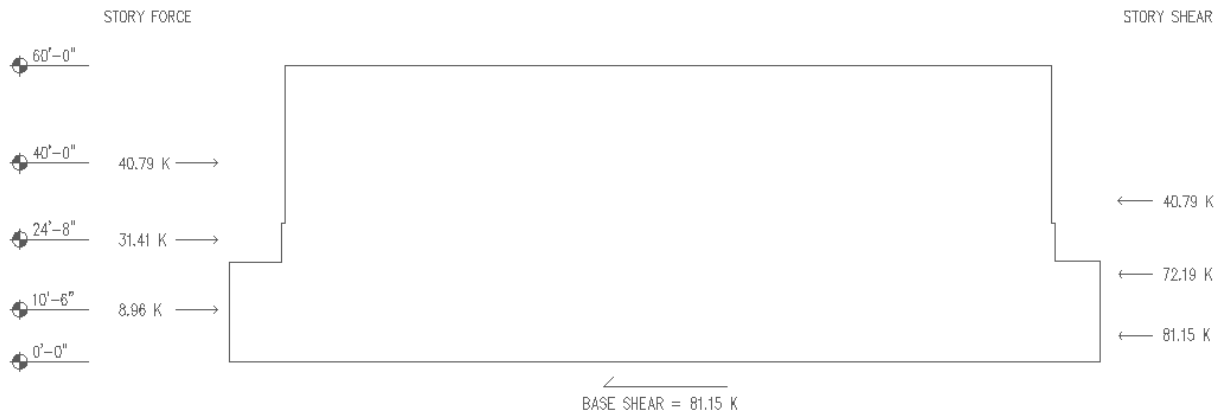


Figure 58 - “Building 1” Seismic Loads

“Building 2”

Building 2 - Level 1			
Weights of Building Components		Center of Mass	
Component	Weight	x (ft)	y (ft)
Concrete Grandstand	130.314 kips	113.1518	78.0000
(2) Stairs at Grandstand	30.382 kips	109.5729	78.0000
Concrete Beams (Bent and Sloped)	19.172 kips	166.1094	78.0000
Balcony	162.813 kips	107.1264	78.0000
Conc. Moment Frame (C.L. 1.8)	61.659 kips	111.3594	78.0000
Total=	404.340 kips	112.6943	78.0000

Building 2 - Level 2			
Weights of Building Components		Center of Mass	
Component	Weight	x (ft)	y (ft)
Concrete Grandstand	215.967 kips	123.7292	78.0000
Interior Walls	113.812 kips	126.4783	70.0919
Total=	329.779 kips	124.6779	75.2708

Seismic Loads - "Building 2"									
Level	Story Weight w_x	Height h_x (ft)	h_x^k	$w_x h_x^k$	C_{vx}	Lateral Force F_x	Story Shear V_x	Moments M_x (ft-k)	
3	0.00	40.00	40.00	0.00	0.000	0.00	0.00	0.00	
2	329.78	24.67	24.67	8134.55	0.657	21.39	0.00	527.73	
1	404.34	10.50	10.50	4245.57	0.343	11.17	21.39	117.24	
sum($w_x h_x^k$)=		12380.12		sum(F_x)=V=		32.56 kips		sum(M_x)=	644.97
Total Weight of "Building 2" (Above Grade) =				734.12 kips					

Table 14 - Seismic Loads – “Building 2”

$$V = C_s W = (0.044353)(734.12 \text{ kips}) = 32.56 \text{ kips}$$

$$C_{vx} = w_x h_x^k / \text{sum}(w_i h_i^k)$$

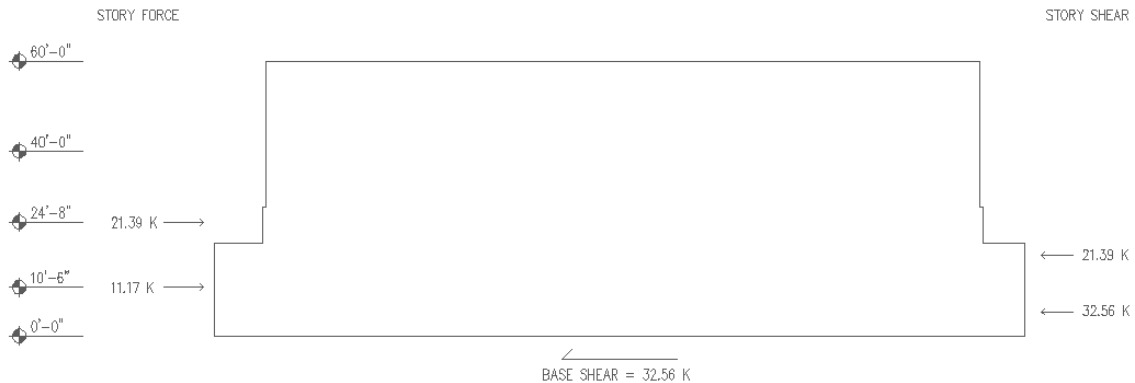


Figure 59 - “Building 2” Seismic Loads

“Building 3”

Building 3 - Level 1			
Weights of Building Components		Center of Mass	
Component	Weight	x (ft)	y (ft)
Precast Concrete Planks	427.386 kips	125.4010	78.0000
Concrete Stairs and Landing (North)	26.048 kips	130.4713	160.7292
Concrete Stairs and Landing (South)	20.123 kips	114.8116	-4.5521
Precast Concrete Ramp	82.596 kips	198.1712	10.0257
Interior Walls from Ground Level	342.366 kips	123.1031	77.7234
Interior Walls from Level 2	191.021 kips	130.6473	68.5422
Total=	1089.540 kips	125.7531	78.2569

Seismic Loads - "Building 3"								
Level	Story Weight w_x	Height h_x (ft)	h_x^k	$w_x h_x^k$	C_{vx}	Lateral Force F_x	Story Shear V_x	Moments M_x (ft-k)
3	0.00	40.00	40.00	0.00	0.000	0.00	0.00	0.00
2	0.00	24.67	24.67	0.00	0.000	0.00	0.00	0.00
1	1089.54	10.50	10.50	11440.17	1.000	48.32	0.00	507.41
sum($w_x h_x^k$)= 11440.17		sum(F_x)=V= 48.32 kips				sum(M_x)= 507.41		
Total Weight of "Building 3" (Above Grade) = 1089.54 kips								

Table 15 - Seismic Loads – “Building 3”

$$V = C_s W = (0.044353)(1089.54 \text{ kips}) = 48.32 \text{ kips}$$

$$C_{vx} = w_x h_x^k / \text{sum}(w_i h_i^k)$$

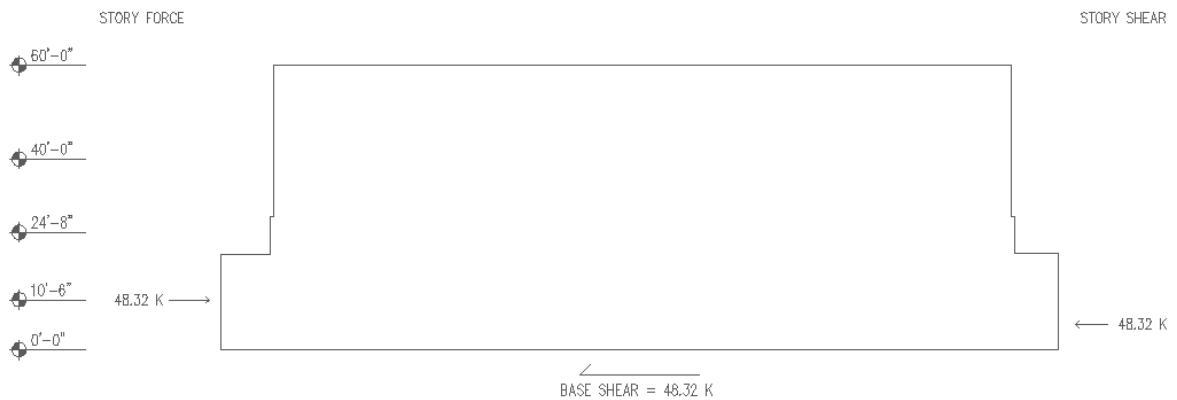


Figure 60 - “Building 3” Seismic Loads

“Building 4”

Building 4 - Level 2			
Weights of Building Components		Center of Mass	
Component	Weight	x (ft)	y (ft)
Roofing Above Lobby	337.055 kips	152.6354	78.0000
Trusses Above Lobby	22.230 kips	150.3677	76.7767
Gallery Level Framing	51.671 kips	144.9739	56.2096
Canopy Framing	8.618 kips	165.1920	132.4399
Columns in Lobby	8.260 kips	157.7642	66.9078
Precast Concrete Panels	265.228 kips	166.9367	79.0722
Mechanical Unit Support Framing	19.089 kips	149.2219	78.5808
Mechanical Units	48.500 kips	146.5257	76.8963
	760.650 kips	151.5494	75.1941

Seismic Loads - "Building 4"								
Level	Story Weight w_x	Height h_x (ft)	h_x^k	$w_x h_x^k$	C_{vx}	Lateral Force F_x	Story Shear V_x	Moments M_x (ft-k)
3	0.00	40.00	40.00	0.00	0.000	0.00	0.00	0.00
2	760.65	24.67	24.67	18762.70	1.000	33.74	0.00	832.18
1	0.00	10.50	10.50	0.00	0.000	0.00	33.74	0.00
sum($w_x h_x^k$) =		18762.70	sum(F_x) = V =		33.74 kips	sum(M_x) =		832.18
Total Weight of "Building 4" (Above Grade) =					760.65 kips			

Table 16 - Seismic Loads – “Building 4”

$$V = C_s W = (0.044353)(760.65 \text{ kips}) = 33.74 \text{ kips}$$

$$C_{vx} = w_x h_x^k / \text{sum}(w_i h_i^k)$$

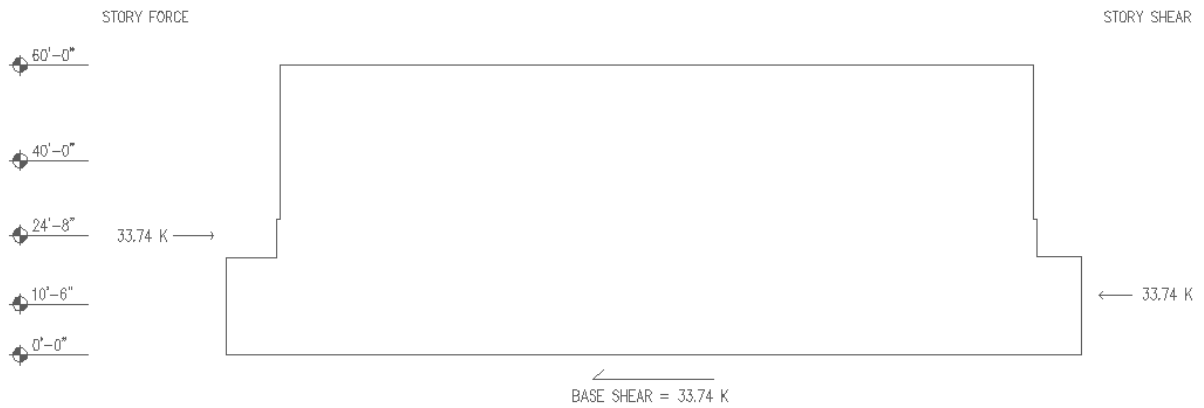


Figure 61 - “Building 4” Seismic Loads

Distribution of Loads

Since the diaphragm above the indoor pool area for the alternate design was wood, the diaphragm was considered to be a flexible diaphragm. With flexible diaphragms, loads are distributed based on tributary area. Therefore, for lateral loads applied to the large indoor pool area (“Building 1”) in the North/South direction, half of these loads were distributed to the wood braced frame at column line 1 and the other half of these loads was distributed to the concrete moment frame at column line 2. For lateral loads in the East/West direction, each of the five concrete moment frames in the East/West direction received loads based on tributary area. Since these frames were evenly spaced, the load distributed to each frame was almost the same. Small differences in load occurred at the outer columns. Due to symmetry, the two perimeter wood braced frames in the East/West direction each received a much smaller load than that applied to the concrete moment frames. Although the roof above the indoor pool area became a flexible diaphragm, the roof above the main lobby remained a rigid diaphragm. With a rigid diaphragm, loads are distributed based on the relative stiffnesses of the lateral force resisting frames.

Wood Braced Frame at Column Line 1

A wood braced frame at column line 1 was designed as an alternate to the originally designed steel braced frames at the same location that were part of the tapered steel trusses that spanned over the indoor pool area. Wood was chosen for these frames to architecturally match the glulam trusses and laminated decking. Several braced frame configurations were designed and compared to determine the one that best suited the space architecturally. Frames with two, three, and four X-braces in elevation were considered. Various patterns of different locations of the X-bracing were also investigated and are shown below in Figures 62 to 65. The final selected configuration is shown below in Figure 66.

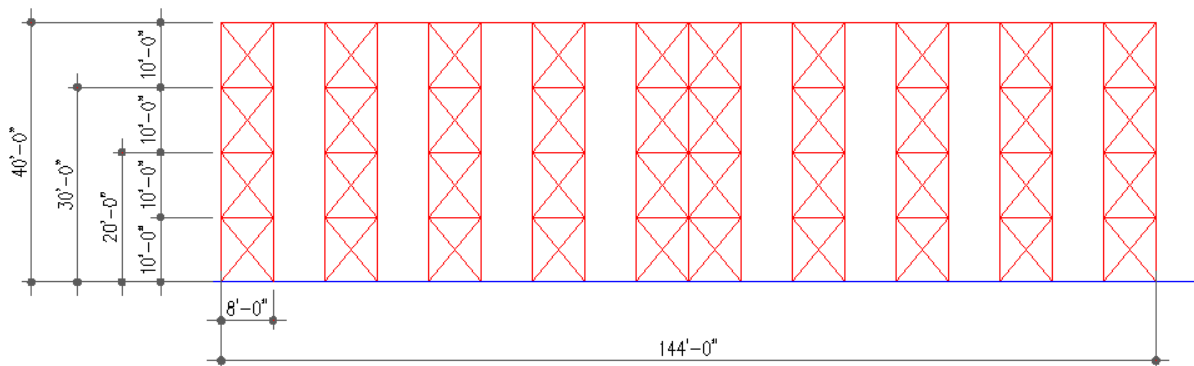


Figure 62 - Potential Column Line 1 Braced Frame Configuration with 4 X-Braces Vertically

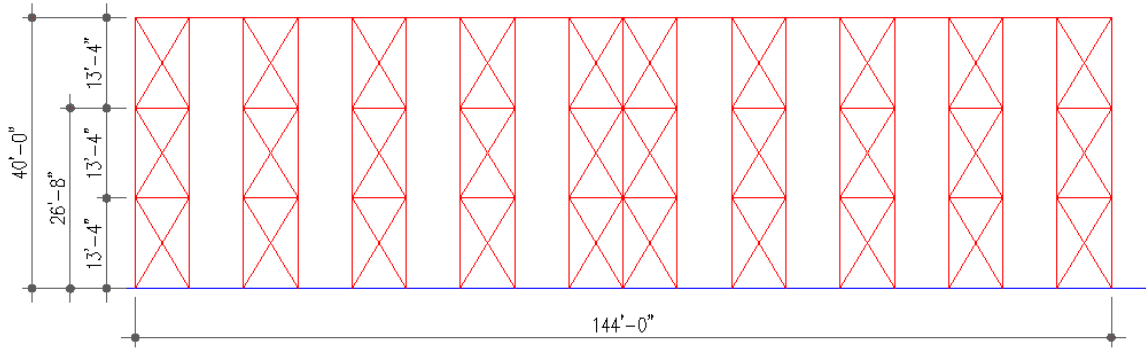


Figure 63 - Potential Column Line 1 Braced Frame Configuration with 3 X-Braces Vertically

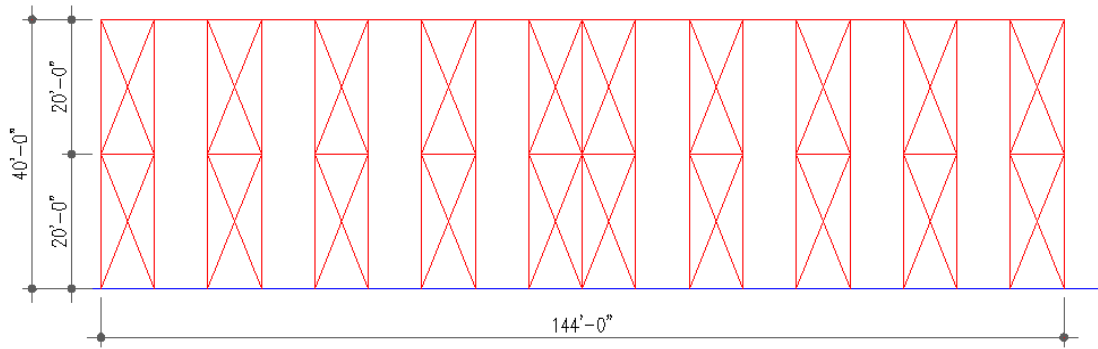


Figure 64 - Potential Column Line 1 Braced Frame Configuration with 3 X-Braces Vertically

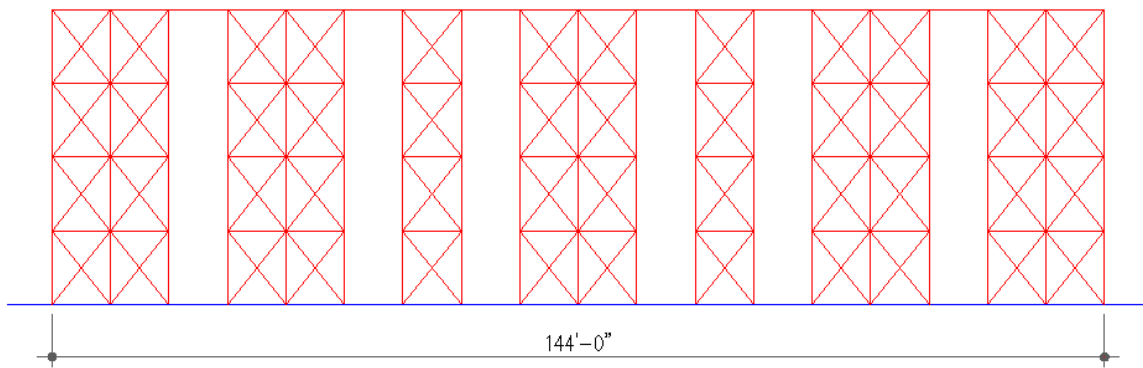


Figure 65 - Potential Column Line 1 Braced Frame Configuration with 4 X-Braces Vertically and Different Layout of Locations of Braced Frames

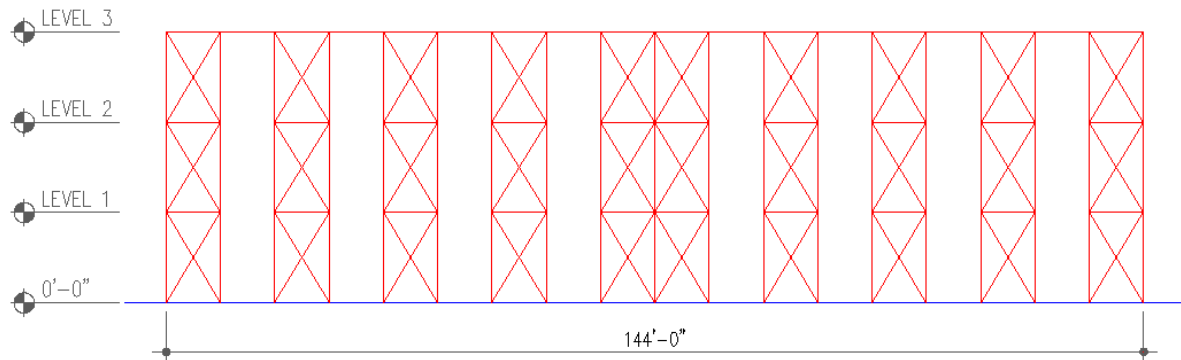


Figure 66 - Final Selected Braced Frame Configuration at Column Line 1

The final selected braced frame configuration consisted of ten braced frames that were spaced apart in an even pattern along column line 1 with two of the frames right beside each other in the middle. The main intent of the braced frames was to brace every column along column line 1. These columns were 40'-0" high and already had to bracing in the East/West direction. If these columns were not braced in the North/South direction, the resulting column sizes would have been considerably large, especially since glulam was used for the columns. The columns were already 15 1/8" deep due to the 40'-0" unbraced length in the East/West direction. The visual appearances of using two, three, and four X-braces in the vertical direction were also considered, and the three X-brace design was determined to be most appropriate for the space. This configuration provided a desirable height-to-width ratio of the X-braces, and the three level also match up very closely with the level of the other lateral force resisting frames used throughout the building.

SAP2000 was used to model the new braced frame. All members of the braced frame were assumed to be pinned at the ends. The diagonal members are also connected where they intersect each other to reduce their unbraced length for bending about the y-axis from 15.55' to 7.77', which helped to reduce the required member size. The controlling load combination $D + 0.75W + 0.75 S$ resulted in a maximum compressive force in the diagonal members of 17.121 kips. A 3 1/2" x 6 7/8" member using Southern Pine glulam ID #50 was calculated as having sufficient capacity for the diagonal members. This size was used for all diagonal members for ease of construction and architectural consistency. The diagonal members would be bolted to the glulam columns, but special brackets or attachment equipment may be necessary since these braced members are much thinner than the 15 1/8" face of the glulam column that they are framing into. Further investigation is required for these connections. Calculations for the design of the diagonal members are found in Appendix B.

Concrete Moment Frame: Column Line 1.8

The steel moment frames along column line 1.8 and column line 2 were replaced by concrete moment frames in the alternate design. The proposal for this project stated that

this area of the building would be redesigned as completely precast, but it was not recognized at that time that it is not feasible to design moment frames using precast concrete. After speaking with John Jones of Nitterhouse Concrete Products, it was realized that moment connections with precast columns and beams are possible but are not very cost effective. Therefore, the concrete moment frames for the Farquhar Park Aquatic Center alternate system were designed using standard reinforce concrete instead of precast units.

The columns along column line 1.8 do not take a great deal of axial force since they only really support one floor, although they support part of the concrete grandstand and concrete balcony in addition to the precast concrete planks at the concourse level. The columns do resist considerably high moments, however, which resulted in rather large column sizes. High moments were due to heavy floor dead and live loads and well as significant seismic loads since a majority of the weight of the building is located in this area. The superimposed live loads were high for the area that these concrete columns were supporting due to the mechanical room and grandstand live load. The self weight dead loads of the grandstand, balcony, and precast planks were also rather high. Pattern loading was considered and produced large moments in the exterior columns of the moment frame. Seismic loads created additional moments in the columns and beams. This moment frame does not resist wind loads.

Columns and beams were designed using ACI 318-08. PCA Column was also used to study interaction diagrams and check the capacity of the columns for the required axial forces and moments. Reinforced concrete column design aids from the textbook “Reinforce Concrete Design and Mechanics” by Wight and MacGregor were used as well. Calculations are found in Appendix B. The columns at column line 1.8 were found to not be slender, so the diagrams from PCA Column did not include slenderness. A clear cover of 2.25” was used instead of 1.5” due to the corrosive natatorium environment. The final design resulted in 24”x24” columns with (12) #8 bars and 24”x26” beams with (5) #8 bars and (5) #7 bars for negative-moment reinforcement and (5) #7 bars for positive-moment reinforcement. Story drifts due to seismic loads were determined to be well within the required limits and are found in Appendix B. The 24”x24” column size matched the width of the designed columns at column line 2 and the width of the sloped concrete beams. Details are shown below in Figures 67, 68, and 69. ACI Code Section 12.11.2 requires that at least one-fourth of the positive-moment reinforcement used at mid-span must be continuous through interior supports and fully anchored at exterior supports. More detailed development, anchorage, and splicing requirements require further investigation.

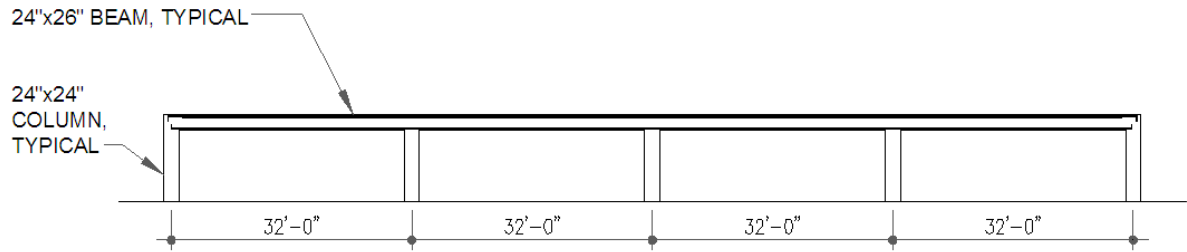


Figure 67 - Concrete Moment Frame at Column Line 1.8

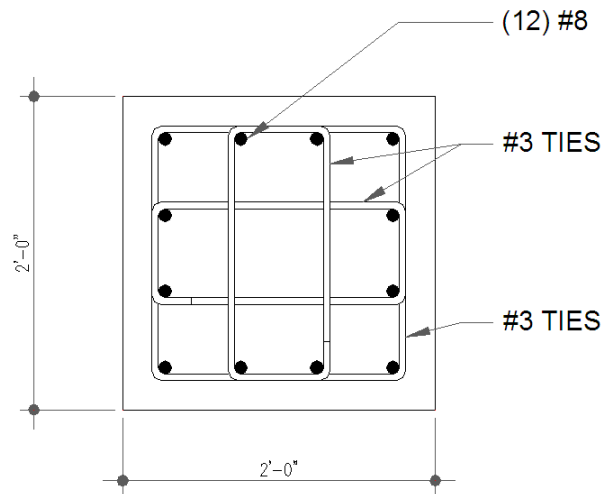


Figure 68 - Detail of Typical Reinforced Concrete Column at Column Line 1.8

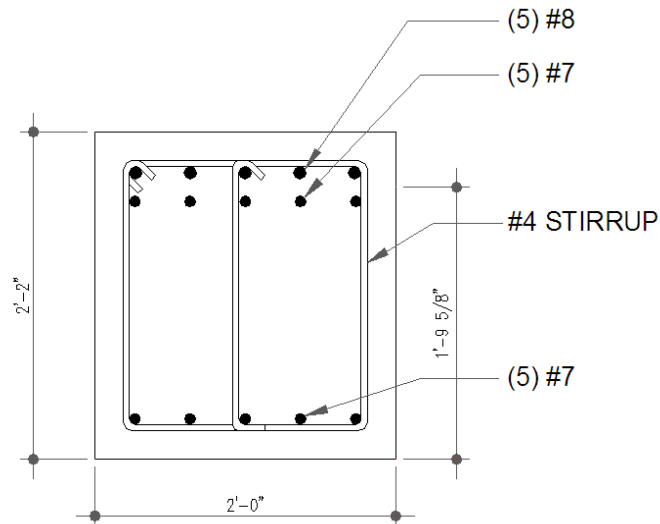


Figure 69 - Detail of Typical Reinforced Concrete Beam at Column Line 1.8

Concrete Moment Frame: Column Line 2

The concrete moment frame along column line 2 was designed in the same manner as the frame along column line 1.8. High moments resulted in the beams and columns due to large roof loads and live loads applied to the frame. The glulam roof trusses spaced at 8' o.c. bear on the top concrete beams and the tops of the columns. The moment frame also carried about half of the self weight of the precast grandstand as well as superimposed loads applied to the grandstand. Live load patterns were also considered, which resulted in significant moments in the columns and beams. ACI 318-08 was used to design the columns and beams. PCA Column was also used to produce interaction diagrams and check the capacity of the designed columns. Reinforced concrete column design aids from the textbook "Reinforce Concrete Design and Mechanics" by Wight and MacGregor were also used. A clear cover of 2.25" was again used due to the corrosive natatorium environment. Calculations are found in Appendix B.

The design resulted in 24"x24" columns with (12) #8 bars and 24"x30" beams with (10) #7 bars for negative moment (8) #6 bars for positive moment. Story drifts due to wind and seismic loads were determined to be within the required limits. Seismic story drifts were well within the allowable limits while story drifts due to wind were close to the limit of $H/400$. Deflection limits due to wind loads were one of the main reasons why the columns had to be so large.

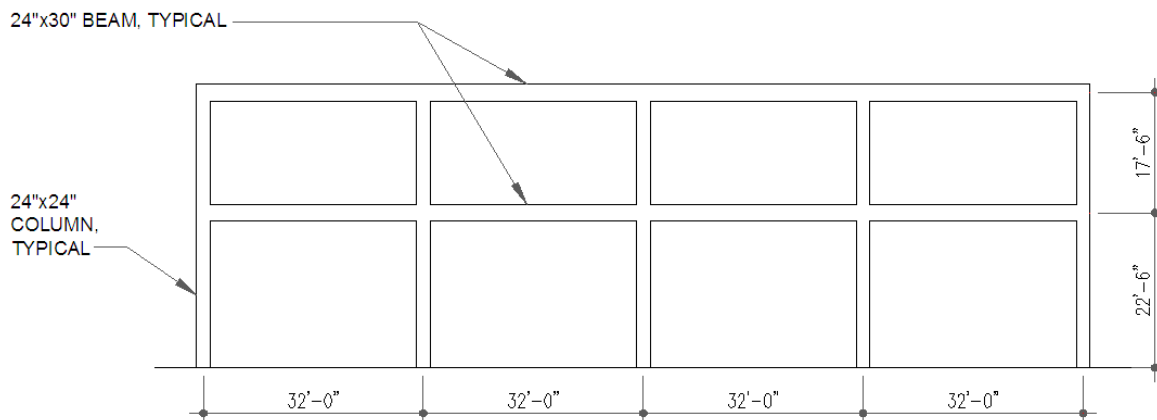


Figure 70 - Concrete Moment Frame at Column Line 2

Concrete Moment Frame – East/West Direction

The original East/West lateral system design for the Farquhar Park Aquatic Center featured five steel moment frames with bent and sloped W27 beams spanning between column lines 1.8 and 2. The steel HSS trusses and supporting tapered columns also helped to resist lateral loads. For the alternate design, perimeter braced frames were originally going to be used to resist all lateral loads in the East/West direction. However, this design was susceptible to having uplift problems due to the significant resulting

lateral forces to be resisted by each frame and would have probably required large, complex, and expensive connections at the bases of the columns. Therefore, concrete moment frames were designed to resist lateral loads in the East/West directions. The five frames matched the configuration of the original steel moment frames in the East/West direction. Perimeter wood braced frames were also designed and are described in the next section.

The concrete moment frames in the East/West direction consisted of sloped concrete beams spanning between the concrete columns designed at column line 1.8 and column line 2. Wind load and seismic loads were applied to the frame. The columns, which were already designed for lateral loads in the North/South direction, had to be checked for lateral loads in the East/West direction as well. The 24"x24" columns with (12) #8 bars were found to have sufficient capacity. The design of the sloped concrete beams resulted in 24"x26" beams with (7) #7 bars for negative-moment reinforcement and (4) #7 bars for positive moment reinforcement. Details are shown below in Figures 71 and 72. Story drifts for seismic loads were found to be well within the drift limits, but story drifts due to wind were very close to the H/400 limit. Again, deflection played a major role in requiring a large column size at column line 2.

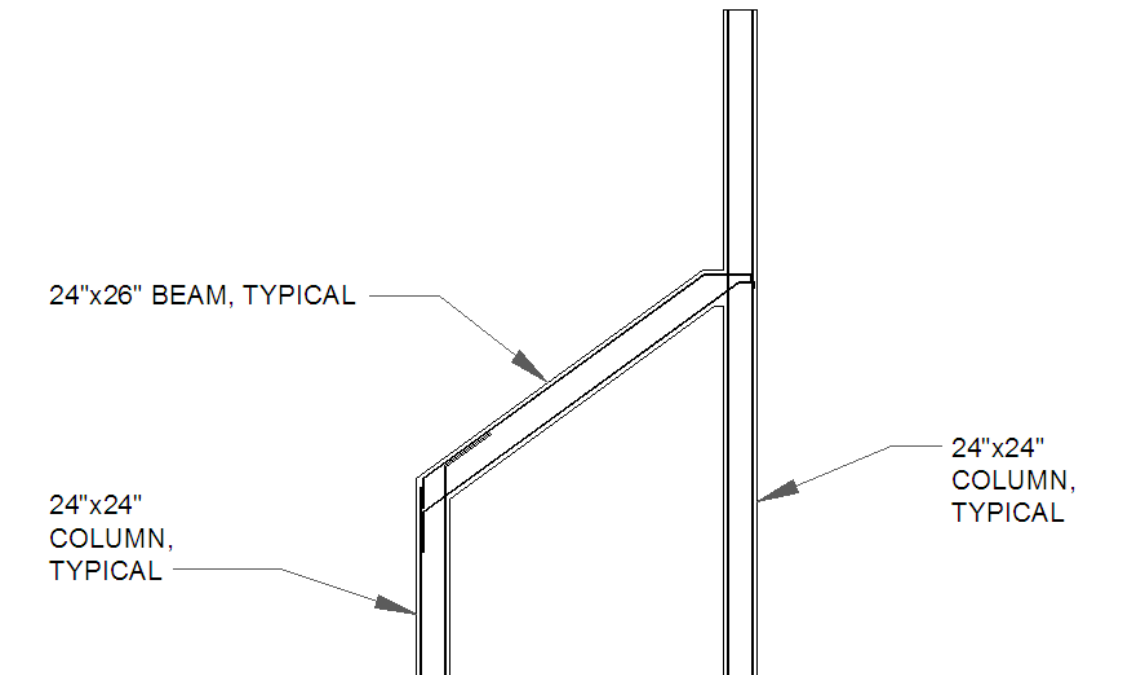


Figure 71 - Concrete Moment Frame in East/West Direction

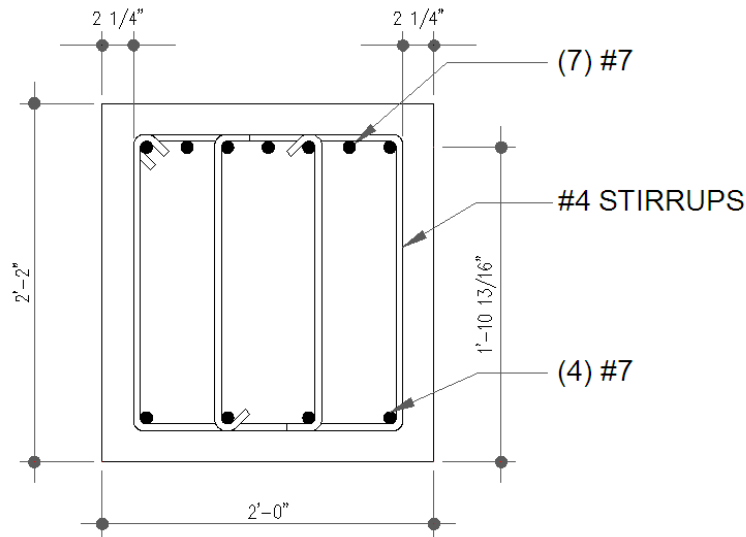


Figure 72 - Midspan Detail of Typical Sloped Reinforced Concrete Beam of Moment Frame in East/West Direction

Wood Braced Frames – East/West Direction

Wood braced frames were added along the North and South edges of the indoor pool area to help alleviate some of the lateral load applied by the East/West concrete moment frames. Due to the flexible wood roof diaphragm, lateral loads were distributed to the lateral force resisting frames based on tributary area. The frames did not take significant lateral loads, which was beneficial because it limited the size of the required members. In addition, gravity loads were neglected since most, if not all, of the gravity loads were carried by the glulam trusses. Architectural considerations were taken into effects when examining different configurations for the braced frames. The final configuration used two separate frames along both the North and South sides with three X-braces vertically for each frame as shown in Figure 73 below. Members were designed using Southern Pine glulam ID #50 to match the glulam trusses and braced frames in the North/South Direction at column line 1. The diagonal members were designed to be the same width as the columns for ease of bolted metal side plate connections. The 2005 National Design Specification for Wood Construction was used to design the members. The final design resulted in 6 3/4" x 6 7/8" Southern Pine glulam ID #50 members for all diagonal and horizontal members. The frames were very stiff and the story drifts were well within the limits for wind and seismic loads.

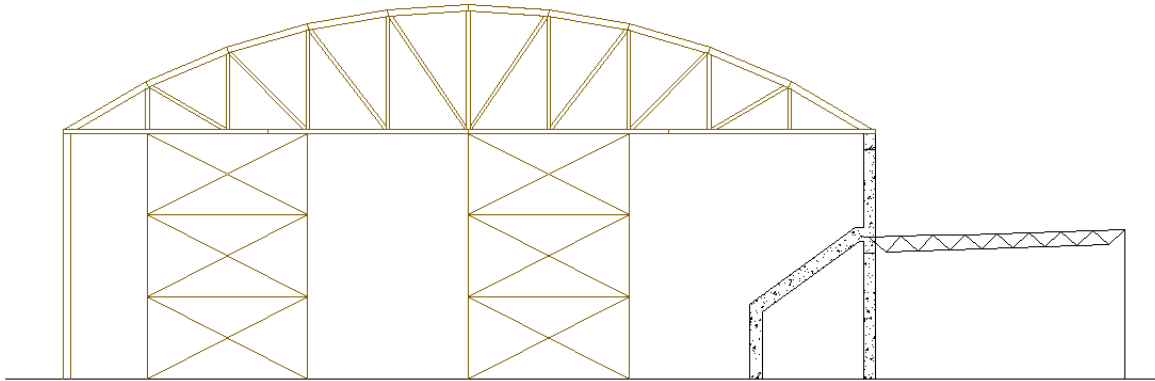


Figure 73 – Elevation Showing Wood Braced Frames in East/West Direction

Wind Columns

The original design for the natatorium used steel HSS wind columns to transfer the lateral load in the North/South direction to the roof diaphragm. These wind columns were redesigned in wood to match the rest of the wood structural system in the indoor pool area. Wood wind columns are spaced at 26'-0" o.c. as compared to the 20'-0" spacing of the original steel wind columns. A total of ten wind columns were used, with the tallest being 60'-0" to reach the top of the glulam trusses. The wind columns are used solely to transfer lateral loads to the roof diaphragm. The wood braced frames in the East/West direction frame into the outer or compression chord of the wind columns. A truss configuration was used to match the original design. Plus, using a single wood member to span 60'-0" would have resulting in very high moments and thus very large required members. A model of the truss was created using SAP2000, and the appropriate lateral loads were applied. The 2005 National Design Specification for Wood Construction was used to design the members. The final design resulted in 6 3/4" x 6 7/8" Southern Pine glulam ID #50 members. The truss members may be connected together using toothed metal plate connectors. The wood braced frames in the East/West direction were designed so that the 6 7/8" dimension matched the 6 7/8" dimension of these wind column members for the use of bolted metal side plate connections. Calculations are found in Appendix B.

Overtopping Check

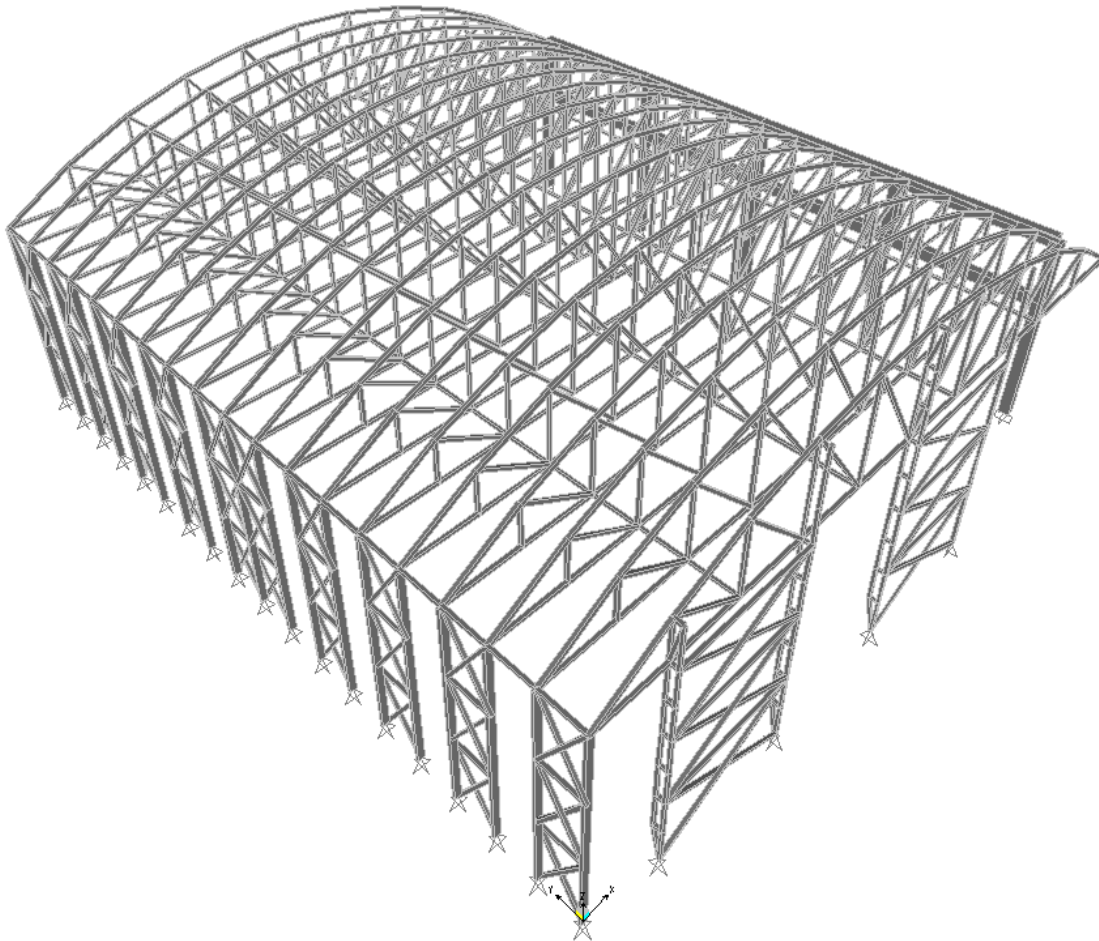
The lateral forces applied to the building cause overturning moments at the bases of the lateral force resisting frames. These overturning moments cause tensile, or uplift, forces in members of the lateral force resisting frames. Wind uplift forces also contribute to these forces. The dead weight of the building, along with superimposed loads, resists these upward forces caused by the overturning moments. The worst case of overturning occurred at the wood braced frame in the East/West direction. Special connections at the bases of the wood braced frames may be required to resist overturning forces. A more detailed check of overturning effects at several locations was performed, and calculations

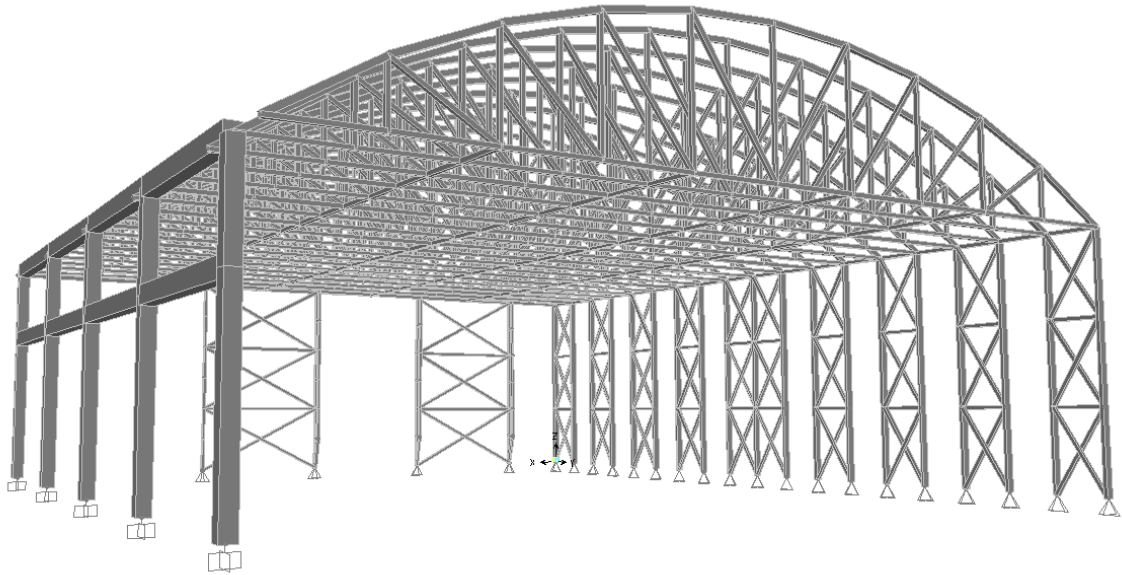
are found in Appendix B. Overturning at the concrete moment frame at column line 1.8 was not a concern because the frame only takes seismic loads, and the uplift force due to seismic loads was only about two kips. Overturning was not found to be a concern with any other lateral force resisting systems as well.

Foundation Check

The capacities of the foundations used in the original design for the Farquhar Park Aquatic Center must be checked accounting for the new weight of the building. The weight of the wood roof structure and concrete moment frames caused the overall building weight to increase. The capacity of a critical footing at column line 2 was checked and found to be more than sufficient to handle the increased loads of the building. Calculations are founding Appendix B.

3D Models





Architectural Breadth

An architectural breadth was studied due to significant changes in the shape of the natatorium's roof and appearance of the façades. In addition, the concrete moment frames designed as substitutes for the original steel moment frames caused a change in column sizes and locations. In the original design, the steel columns along column line 1.8 were HSS14x14 members while the columns along column line 2 were HSS18x18 members. The alternate concrete moment frame design consisted of 24"x24" columns both at column line 1.8 and column line 2. The spacing of the columns also changed from 30'-0" in the original design to 32'-0" in the alternate design. This caused the outermost columns to move 4'-0" outward and the inner columns next to the middle columns to move 2'-0" outward. The increase in column size and change in column location deemed that a room layout study be implemented. Some rooms required little layout changes and easily accommodated the new columns spatially, while other rooms required considerable layout changes. Maintaining coordination of building systems due to the new room layouts was also vitally important. Several mechanical openings through both the ground level and concourse level required relocation, and it was pertinent to ensure that the openings still lined up with each other. In the images shown below, the blue-colored columns are the original columns that were resized and relocated, and the magenta-colored columns are the new relocated concrete columns for the alternate design.

Room Layouts to Accommodate Column Relocations

GROUND LEVEL:

#1)

New Column Locations with Original Room Layouts:

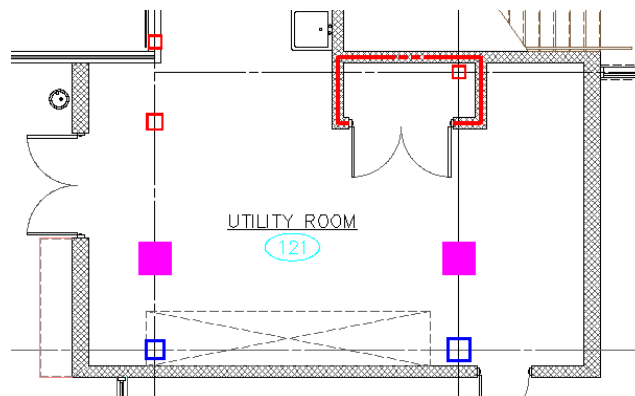


Figure 74 – Utility Room (Ground Level)

Solution:

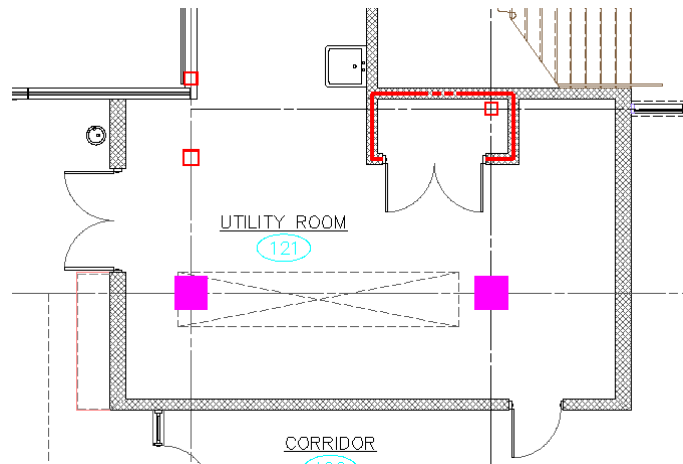


Figure 75 – Utility Room (Ground Level)

The new column locations in the utility room caused little or no problems. The original steel columns, shown in blue in Figure 74 above, were located along the south edge of the room, while the new columns, shown in magenta, were located toward the middle of the room. Having columns in the middle of the room should be fine, especially since it is only a utility room. No plans of the layout of the utility rooms were found to ensure that these columns were not in the way of anything, but it may be wise to investigate this further. The columns allow plenty of clearance space for circulation and make as much usable space as possible available. The mechanical opening was relocated as well due to re-layouts of the concourse level floor plan above, but did not cause any problems because it is actually located in the ceiling of the ground level utility room. The opening is shown in Figures 74 and 75 above just for reference. In addition, the original drawings showed the mechanical opening in the room and even showed a column running right through it. Therefore, it was assumed that this could be done with the new concrete columns as well.

#2)
New Column Locations with Original Room Layouts:

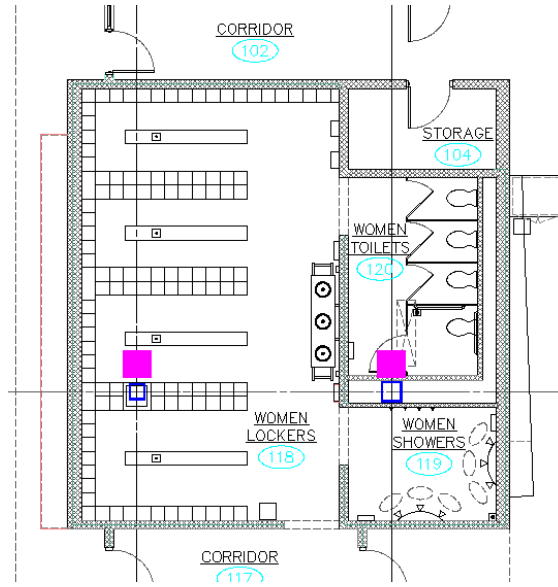


Figure 76 – Women's Locker Room (Ground Level)

Solution:

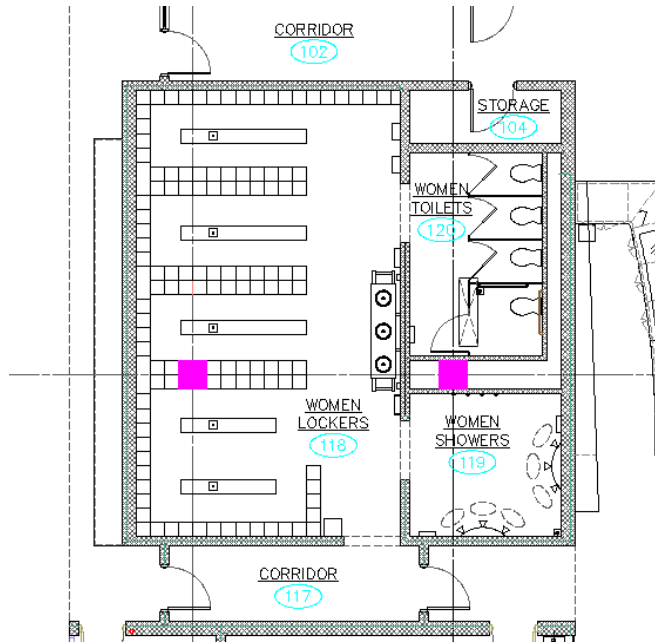


Figure 77 – Women's Locker Room (Ground Level)

For the women's locker rooms, locker layouts were reorganized so that one of the islands in the middle lined up with the column. The 24" width of the column fit in perfectly with the 24" width provided by the 12"x12" lockers. The southernmost area of lockers in the

women's locker room became larger than the original design due to the shifting of the southernmost locker island. Another bench was added to this area due to the increased in open space. Additional lockers were added near the doorway to make up for the ones that were lost due to the new locker layout. It also seemed that adding these lockers would also help block the view of anyone in the lobby who might happen to be able to see in if corridor door is open. According to the original drawings, it appears that there is no door between the corridor and women's locker room. Overall, the re-layout added three lockers to the total amount in the room.

#3)
New Column Locations with Original Room Layouts:

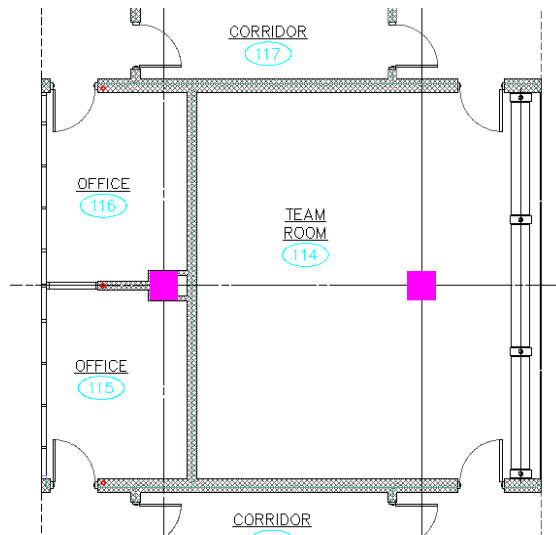


Figure 78 – Team Room and Offices Room (Ground Level)

Solution:

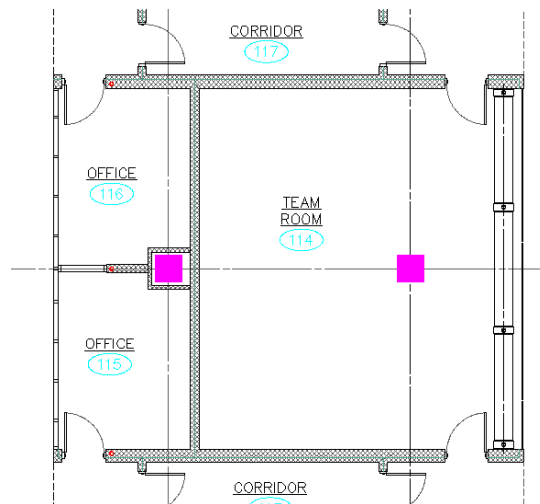


Figure 79 – Team Room and Offices Room (Ground Level)

The columns in the team room and between the two offices did not change location, only size. The new size of the column in the team room did not raise any issues. The original column was already located toward the middle of the room, so the larger column size did not disrupt any other systems or services around it. For example, if the column had been located alongside a wall then the larger size may have required relocation of the wall, but this was not the case. The increase in column size for the column between the offices required that the 4" CMU walls surrounding the column be shifted slightly outward to apply a gap between the column and walls. Overall, this was a minor modification that barely affected the office spaces since the 4" CMU walls in the corner only moved a few inches.

#4)

New Column Locations with Original Room Layouts:

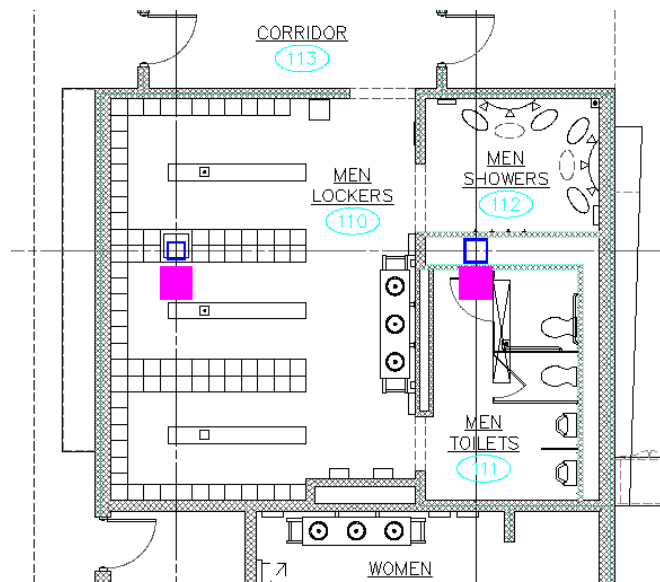


Figure 80 – Men's Locker Room (Ground Level)

Solution:

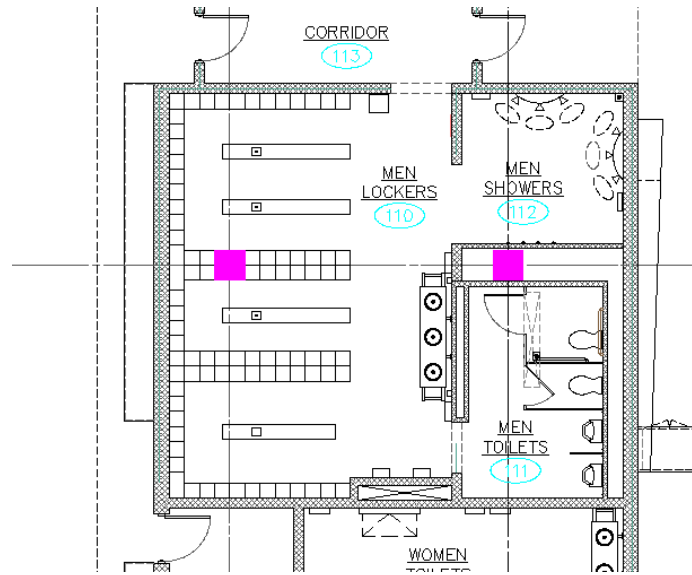


Figure 81 – Men's Locker Room (Ground Level)

The solution to the relocation and increased size of the column in the men's locker room was very similar to that for the women's locker room. One of the locker islands was moved to line up with the concrete column. Again, the 24" column width fit in perfectly with the 24" width provided by the rows of two lockers back to back. The men's locker room is actually a good bit smaller than the women's locker room. The locker re-layout resulted in one less locker than the original design, but more lockers could be added by the door as was done with the women's locker room. This would also help to block the potential view of anyone in the lobby. A second bench was added to the space that increased in area due to the relocation of the locker island. The alternate layout still allows an ample amount of clearance between the benches and the lockers. In addition, the length of the southernmost bench was shortened to allow for more clearance between the bench and the protruding wall.

The new location of the column in the men's locker room toilets caused it to end up in the way of the handicap bathroom and would not allow the handicap stall door to swing open. Therefore, keeping the north wall of the locker room stationary, the entire row of stalls was shifted south in an effort to keep the column enclosed by the walls on either side. As a result, the men's locker room as a whole became larger. More lockers were put in the locker room due to the additional space that became available. It was necessary to line up the mechanical duct shaft from the concourse level above, which had already been moved to accommodate new layouts on the concourse level. Overall, the alternate layout accommodated the new column locations and maintained a continuous mechanical penetration from the ground level to the concourse level. However, this layout moved the south wall of the locker room into the women's toilets area and decreased the amount of available space in the room.

#5)
New Column Locations with Original Room Layouts:

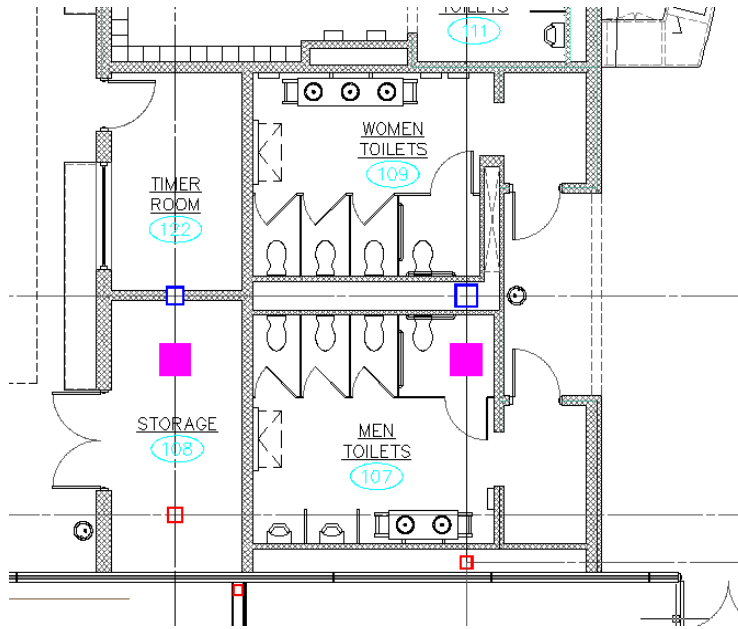


Figure 82 – Men's Restroom, Women's Restroom, Timer Room and Storage Room (Ground Level)

Solution:

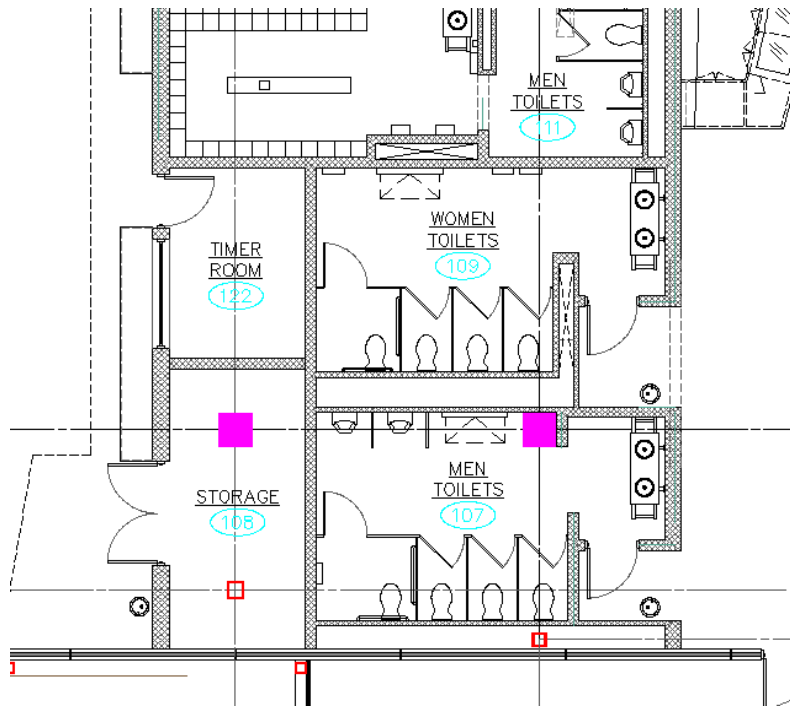


Figure 83 – Men's Restroom, Women's Restroom, Timer Room and Storage Room (Ground Level)

The men's and women's toilets areas at the south end of the natatorium were the most difficult to re-layout. The column along column line 2 ended up right in the middle of the handicap stall of the men's restroom, creating a major problem. The amount of available space in the men's restroom and adjacent women's restroom had also decreased since, as mentioned in #4 above, the wall from the men's locker room had been moved south to accommodate other room layouts and hence landed right in the middle of the sink area of the original design of the women's bathroom. This required that the women's restroom area be shifted toward the men's restroom area. For the re-layout, the urinals were switched to the north side of the room and the stalls were moved to the south side of the room. A new location for the sink was also investigated. The sink would not fit in between the urinals and column and was eventually moved to the entrance area. The entrance door was switched to face south and basically matched the entryway of the women's restroom. The changing station was moved to the north wall to fill the space between the column and urinals. A water fountain was also added a water fountain near the men's restroom entrance. In addition, the handicap stall was moved to the west wall so that the wall that extended north into the path between the sinks and the toilet area could be pulled back to allow for more circulations space. The concrete column fits in nicely along the newly relocated walls, and the new layout efficiently uses the limited available space.

For the women's toilet area, the newly relocated wall from the men's locker room to the north ended up on top of the sink in the original design of the women's restroom. This took away a major amount of the available space in this room. Therefore, due to the limited amount of space it was deemed necessary to change the sink unit to a sink with two bowls instead of three bowls and attempt to move it to the entrance area of the restroom. It was also necessary to make sure the mechanical opening lined up with the one from the concourse level above. This was achieved in conjunction with the re-layout in #4 discussed above. The handicap stall was moved to the west end of the bathroom so that the stall door did not swing out into the entrance path since the width of this area got cut a short. The wall that extended into the space between the newly relocated sink and toilets was pulled back to allow more space for people to get to the toilet area. The entire set of stalls was moved southward to create more room in this women's bathroom. This hence moved the south wall into the men's bathroom space, which was discussed above. The water fountain was moved slightly, and as mentioned above, another one was added by the new entrance to the men's restroom.

The timer room became a little smaller due to the relocation of the south wall of the men's locker room. It was necessary to move the window of the timer room to the south to keep it centered on the wall to the south of the entrance door into the room. The south wall of the timer room was not moved to allow enough space in the storage room. Resulting gap between the new column in the storage room and the north wall of the storage room was only 2'-8", making it undesirable to take away any more of this space so that it could still be usable.

CONCOURSE LEVEL (UPPER LEVEL):

#6)

New Column Locations with Original Room Layouts:

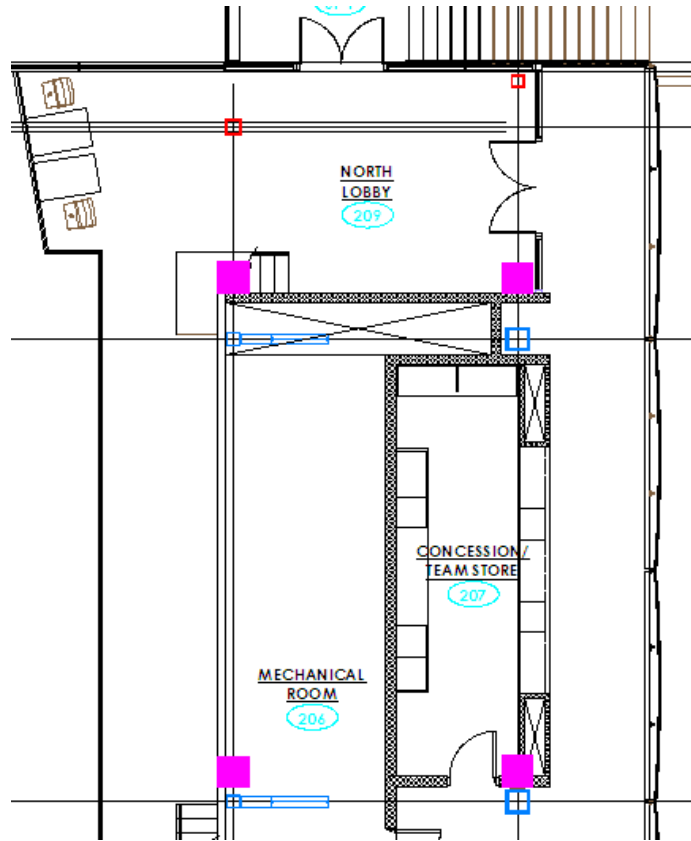


Figure 84 – Concession/Team Store and Mechanical Room (Concourse Level)

Solution:

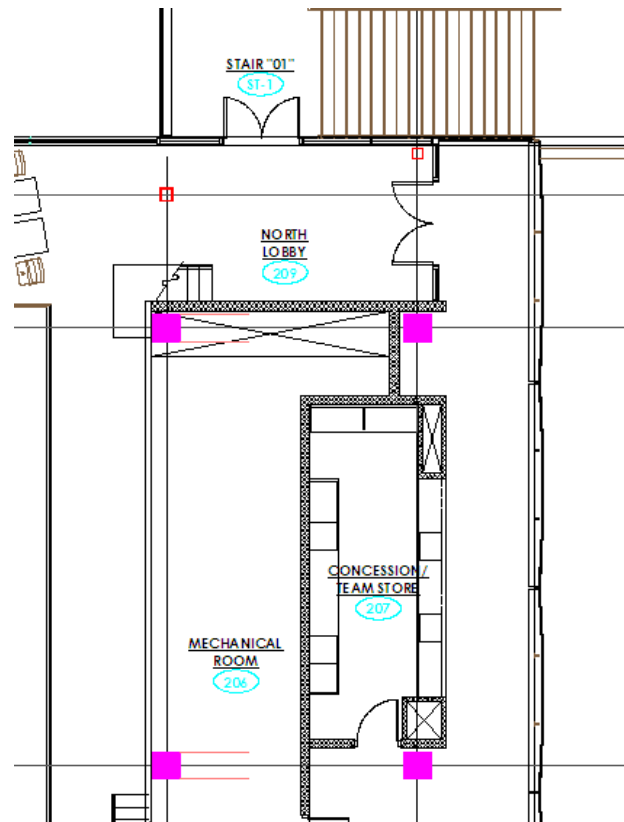


Figure 85 – Concession/Team Store and Mechanical Room (Concourse Level)

The relocated northernmost columns of the concrete moment frames ended up to the outside of the CMU wall at the edge of the grandstand seating area. In an effort to keep a similar layout to the original design, this CMU wall was moved north to keep the columns and sloped concrete beam at this location to remain enclosed from the north lobby. It would have appeared architecturally unpleasing to see the columns and sloped beams just to the outside of the CMU wall when the wall was intended to block the view of the structure from the lobby spaces. The North and South lobbies decreased in area slightly, with their width in the North/South direction decreasing by a couple feet to accommodate the relocation of the outer columns supporting the grandstand seating area. The small set of steps to get from the balcony to the grandstand also had to be shifted slightly due to the new locations of the columns. If the steps had not been moved, they would have been located right in the middle of the 24"x24" concrete columns along column line 1.8 and the sloped concrete beams. Therefore, the length of the grandstand seating area increased in the North/South direction.

The concession/team store decreased in area slightly, and the door leading into the concession/store room moved in the North direction a few feet due to the new column location. The shape of this mechanical opening by this door was changed so that it would not take away counter space of the concession/team store. The newly changed opening maintained the same size as the original opening. This mechanical opening could probably have just been shifted in the north direction, keeping the same shape and size and taking away a small portion of the counter space. To accommodate this slight

decrease in floor space, the store could gain additional area at the north end due to the shifting of the large duct shaft space in that area. Three mechanical openings were affected by the floor plan re-layout. All openings were kept the same shape and size except for one in the concession/team store area that was mentioned above. Coordination of all relocated mechanical openings was coordinated with the ground level layouts, as previously discussed. After checking with the mechanical drawings, it was evident that moving the duct shaft a couple feet here and there was permitted and did not really cause any problems.

#7)

New Column Locations with Original Room Layouts:

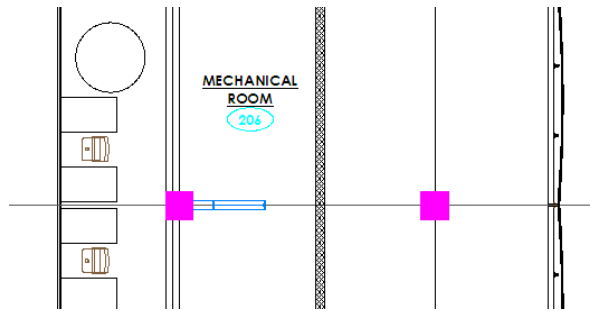


Figure 86 –Mechanical Room (Concourse Level)

Solution:

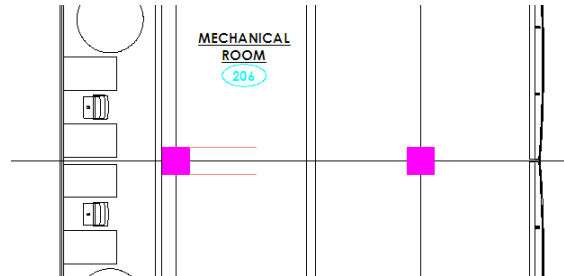


Figure 87 –Mechanical Room (Concourse Level)

The new locations of the columns located in the middle of the concrete moment frame raised little or no concerns. The west, bottom edge of the concrete grandstand had to be shifted a few inches to clear the larger column along column line 1.8. This occurred for all new column locations along column line 1.8. The locations of the concrete columns in the mechanical room are fine since they are located along the edge of the mechanical room to allow for the maximum use of the area. The sloped concrete beams take away some of this available space though, but this occurred with the original design as well.

#8)
New Column Locations with Original Room Layouts:

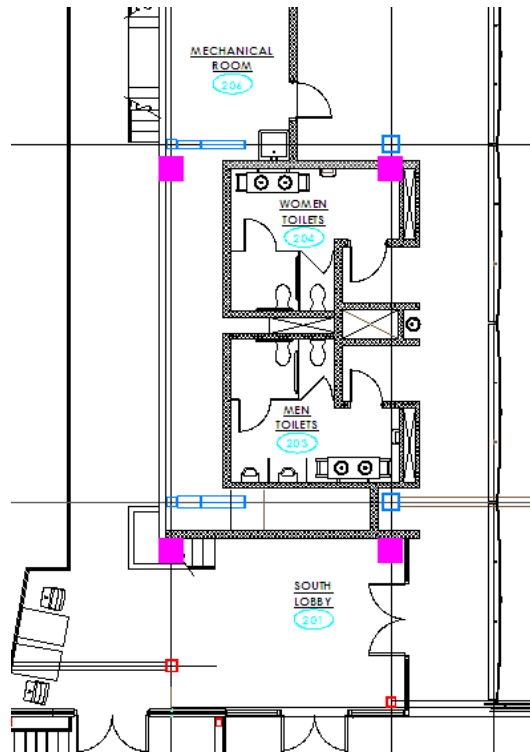


Figure 88 – Women's Restroom, Men's Restroom, Mechanical Room, and South Lobby (Concourse Level)

Solution:

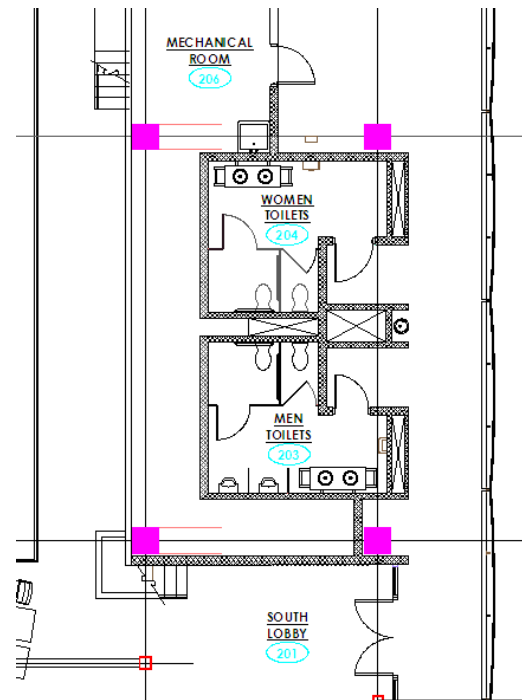


Figure 89 – Women’s Restroom, Men’s Restroom, Mechanical Room, and South Lobby (Concourse Level)

The men’s and women’s restrooms were shifted southward to accommodate the new locations of the columns. The layouts of these rooms were kept the same. All mechanical openings that were relocated were coordinated with the ground floor layouts, as previously discussed. For the southernmost column, the same layout solution that applied to the northernmost columns was implemented here as well. The sink in the mechanical room should have enough clearance at its new location with the sloped beam now overhead. The new layout provides about 12-15 additional SF of space for the mechanical room.

New Roof Shape and Appearance of Façade in Elevation View

The original design for the Farquhar Park Aquatic Center featured curved and tapered steel HSS trusses that spanned 130’-0” over the indoor swimming pool area. Several alternate roof system shapes and designs were investigated for this thesis project. One of the main goals was to develop a more cost effective alternate system but still maintain the architectural integrity of the original design. Three alternate roof systems were investigated: a steel king-post truss system, a steel space-frame system, and a wood truss system. With the steel king-post truss system, the possible configurations were rather plain compared to the original truss system, which limited the design options for this type of system architecturally. King-post trusses are typically just triangular in shape, which did not seem to fit the profile of the original design. The space frame designs that were investigated were somewhat limited architecturally as well. Space frames can have unique shapes, such as curves that span long distances, but using a curved space frame would have been too complex and too costly for the natatorium. Typical space frames

are basically flat with depths that usually vary between four and twelve feet. The final space frame design that best suited the intentions of the natatorium was flat as well, which offered little architectural expression. The wood truss system offered much more architectural flexibility while still maintaining a rather competitive price. Several curved glulam truss configurations were investigated. The final depth of the trusses was rather large due to the long 130'-0" span over the indoor pool. Wood trusses with shallower depths were investigated, although the designs resulted in considerably high axial forces in the truss members. In addition, the trusses that were shallower almost appeared flat and lacked architectural style. In addition, deeper trusses offered a more pleasing architectural appearance by maintaining a larger curve, especially with the long span. The final glulam truss design had a depth of 20'-0". Out of the three roof systems investigated, the wood trusses seemed to best meet the goals of the project.

Wood structural systems, in general, offer a warm, pleasant architectural appearance. The laminated decking to be used with the glulam truss system also provides a V-groove between adjacent members that provides an attractive architectural look. Decking is often left exposed from below for architectural purposes. Architectural considerations were also applied to the design of the alternate lateral system. The use of wood for the braced frames and columns was chosen in order to match the appearance of the wood trusses. Various lateral system patterns and configurations to account for architectural appearance were discussed in previous sections.

New Design with Wood Trusses (above) and Original Design (below):

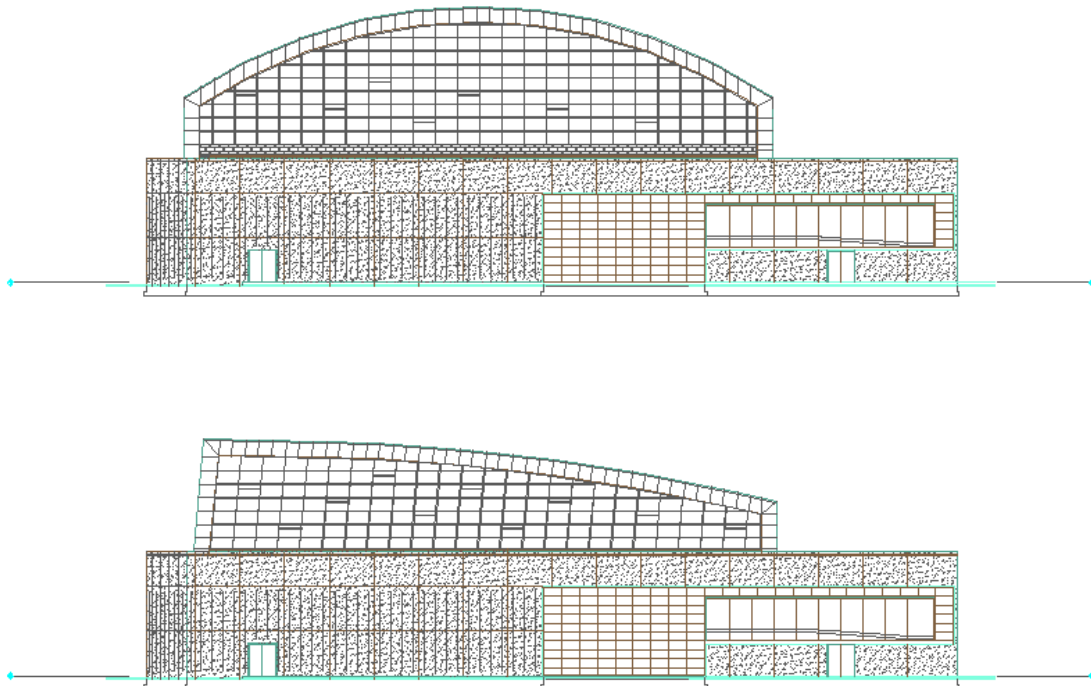


Figure 90 - Elevation with New Roof Shape and Façade (above); Elevation with Original Roof Shape and Façade (below)

The new glulam truss design also expanded the area of the large glass curtain wall facades on the North and South faces of the building. This raised a concern as to how this would affect the thermal performance of the indoor pool area, especially since the one façade faces south. These large glass facades consist of Solera-T translucent insulating glazing units that provide high thermal performance and transmit diffuse light. The Solera-T glazing units are discussed in more detail in the Building Enclosure Breadth. Therefore, using a more expansive glass façade on the North and South faces should have minimal impacts on the thermal performance of the building. The large glass façade will also allow more daylighting into the indoor pool space, which may help to decrease overall lighting and electrical costs for the natatorium. The mullions of the original façade were slanted to match the slope of the west face of the roof system. For the alternate design with the glulam trusses, the mullions were oriented vertically and a short stone base was added to the bottom of the façade.

Building Enclosure Breadth Study (AE 542)

What makes this building unique is the fact that it is a natatorium. Natatoriums are often considered to be one of the most difficult types of buildings to design. Poor natatorium design can haunt an owner due to the inherent moisture and thermal problems that can arise and potentially ruin the building.

Exterior Wall Systems

The exterior wall system is, by far, the single most expensive part of a building enclosure. Building envelopes must be properly designed to account for moisture infiltration and prevent condensation within the envelope system. Moisture generally travels from areas with higher moisture content to areas with lower moisture content, from higher pressures to lower pressures, and from higher temperatures to lower temperatures. Condensation must absolutely be avoided in a natatorium. Condensation forms whenever moisture in the air touches a surface that is cooler than the ambient dew point temperature. If condensation forms within a wall or roof system, it can allow mold and mildew to grow and can cause the building materials to deteriorate. Condensation that forms in the winter can also freeze and cause considerable damage to building systems. The building envelope of the natatorium must be able to perform properly year-round at a relative humidity of 50% to 60%. Building components that form thermal bridges must be avoided at all costs. Components with low R-values such as windows and emergency exit doors must be blanketed with warm air to prevent condensation from forming.

The first major step in performing a condensation analysis is determining locations in a wall or roof system where condensation may form. Then the designer can ensure that the vapor barrier is properly placed to prevent moisture from reaching building components that are at a temperature below the dew point temperature. It is pertinent that vapor retarders be sealed or taped at all the seams. The most important element in protecting a building structure from moisture damage is the vapor retarder. The H.A.M. (Heat, Air, and Moisture) Toolbox was used to analyze various wall and roof systems used in the Farquhar Park Aquatic Center. The program produces temperature gradients throughout a wall system and identifies where the dew point temperature lies within the wall. The Natatorium Design Manual by Seresco Technologies, Inc. was used to establish typical natatorium design conditions.

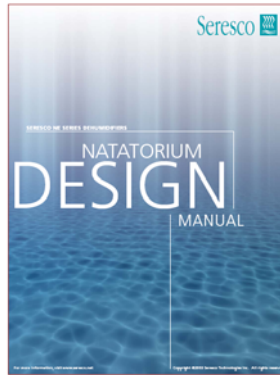


Figure 91 – The Natatorium Design Manual by Seresco Technologies, Inc.

Typical Natatorium Design Conditions		
Pool Type	Air Temperature, °F	Water Temperature, °F
Competition	75 to 85	76 to 82
Diving	80 to 85	84 to 88
Elderly Swimmers	84 to 85	85 to 90
Hotel	82 to 85	82 to 86
Physical Therapy	80 to 85	90 to 95
Recreational	82 to 85	80 to 85
Whirlpool/spa	80 to 85	102 to 104

Table 17 - Typical Natatorium Design Condition (from the Natatorium Design Manual by Seresco Technologies, Inc.)

The Farquhar Park Aquatic Center is a natatorium for the YMCA of York and York County. The natatorium is used for swimming competitions as well as for recreation. As seen in Table 17, indoor pools used for competition are typically kept at an indoor air temperature of 75°F to 85°F, while indoor pools used for recreational purposes are usually kept at an indoor air temperature of 82°F to 85°F. Therefore, for the H.A.M. Toolbox program an analysis was performed for various indoor air temperatures; each system was analyzed for an indoor air temperature of 75°, 80°F, and 85°F. Since the relative humidity of natatoriums is typically in the range of 50% to 60%, an analysis was performed for a relative humidity of 50% and 60% as well. The H.A.M. Toolbox has summer and winter design conditions built into the program. The outdoor summer and winter design conditions for Philadelphia, PA, which is fairly close to the location of the natatorium in York, PA, were used for the analysis. Four different roof systems were analyzed, and the materials for each system are listed below starting with the outermost material. Roof System #1 was the roof system above the indoor swimming pool. Roof System #2 was located near the top of the wind columns. Roof System #3 was the nearly vertical, but slightly sloped, portion of the roof on the West façade of the natatorium.

Roof System #4 covered the indoor pool area between the large truss columns and the west wall of the natatorium; the space is mostly about 10' wide and spans the entire length of the building in the North/South direction. Wall systems were not really investigated since most of the building is enclosed by insulated precast concrete panels, which cannot be modeled in the H.A.M. Toolbox and are described in more detail below. The outputs from the H.A.M. Toolbox are also shown below. Printouts from H.A.M. for all six design conditions are shown for roof system #1, but since the results for the other roof systems were very similar, only a few outputs were selected to be shown for these systems.

Roof System #1:

- Roof membrane
- Roof insulation (R-28)
- Vapor barrier
- DensDeck
- Acoustical metal deck (not modeled in H.A.M.)

Results: For all winter cases, the dew point always occurred in the rigid insulation. Since moisture cannot actually condense inside rigid insulation, it must condense on one of the surfaces of the insulation. In each case, the moisture would condense on the outer surface of the rigid insulation. Therefore, the moisture would condense on the roof membrane, which was beneficial as long as the membrane is considered to act as a vapor barrier. If so, the system was properly designed for winter conditions. For summer conditions, the dew point was located in the rigid insulation for outside air temperatures of 75°F and 80°F. In this case, the moisture would therefore condense on the inside surface of the rigid insulation since the warm moist air from outside would be moving toward the inside surface of the wall system. The vapor barrier was located right next to the inside surface of the rigid insulation, thus prevented moisture from reaching the dens deck and acoustical metal deck. The designers were definitely aiming to keep moisture off of the acoustical metal deck. The dew point was not located in the roof system for an outside air temperature of 85°F. Therefore, the roof system was properly designed for summer conditions as well.

Roof System #2:

- Zinc standing seam metal roof
- Vapor barrier
- ½" moisture resistant gypsum wall board
- 4 ½" rigid insulation
- Vapor barrier
- ½' moisture resistant gypsum wall board

Results: The results were very similar to those from Roof System #1. For all winter cases, the dew point was located in the rigid insulation. However, in this roof system configuration the moisture would condense on the inner surface of the ½" moisture resistant gypsum wall board since the vapor barrier is located on the outside face of this

outer layer of gypsum board. It seems that typically the vapor barrier would be located on the inside face of this outer layer of gypsum board to stop the condensed moisture from reaching the gypsum board. However, since the gypsum board is moisture resistant, perhaps the gypsum board will still perform properly if water condenses on it. For summer conditions, the dew point was located in the rigid insulation for outside air temperatures of 75° and 80°F but was not located in the roof system for 85°F. The vapor barrier was properly located because it would prevent condensed moisture from reaching the inner layer of gypsum board. Therefore, the roof system was properly designed for summer conditions.

Roof System #3:

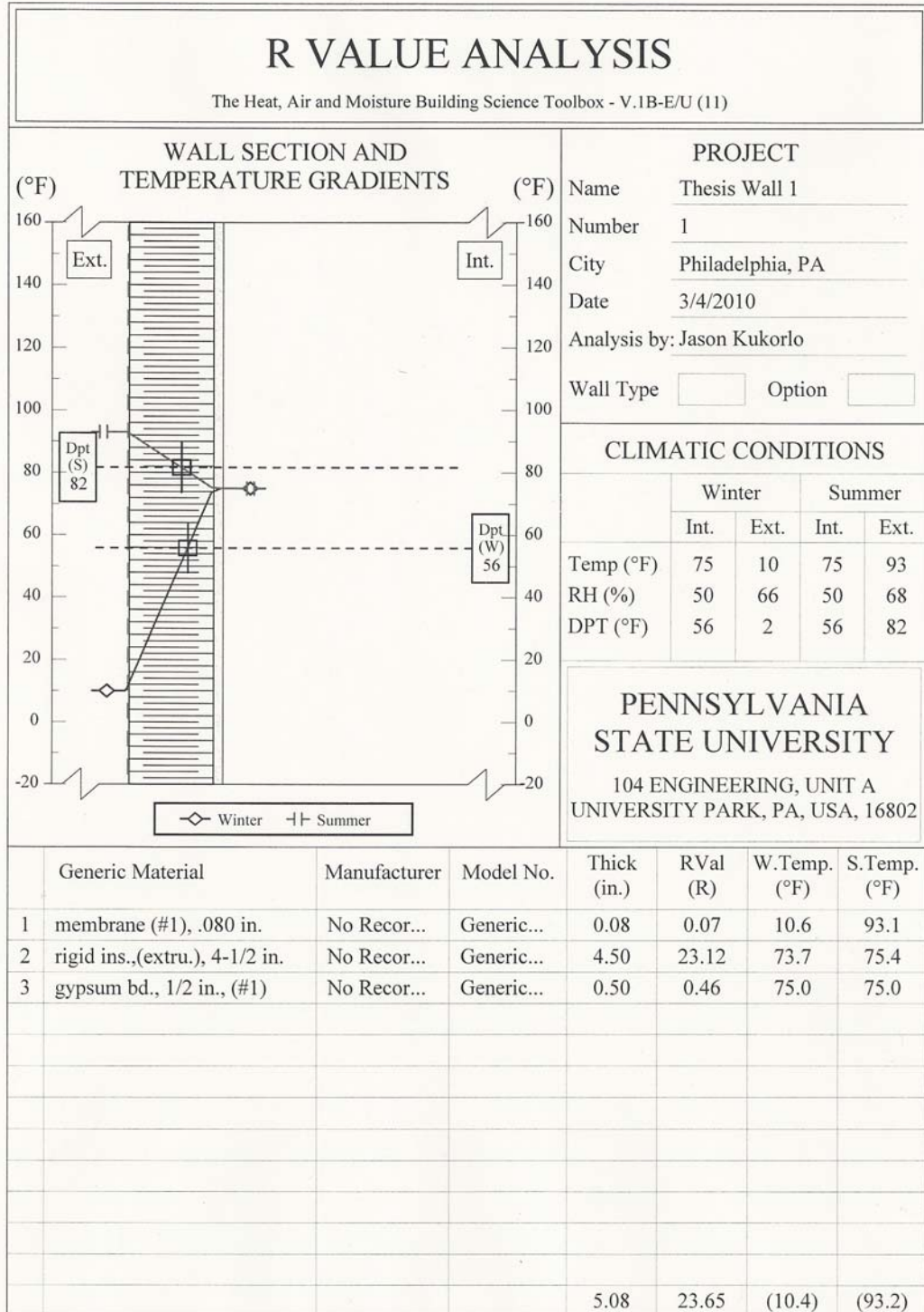
- Zinc flat lock panel
- Vapor barrier
- ½" moisture resistant gypsum wall board
- 1 ½" rigid insulation
- ½' moisture resistant gypsum wall board

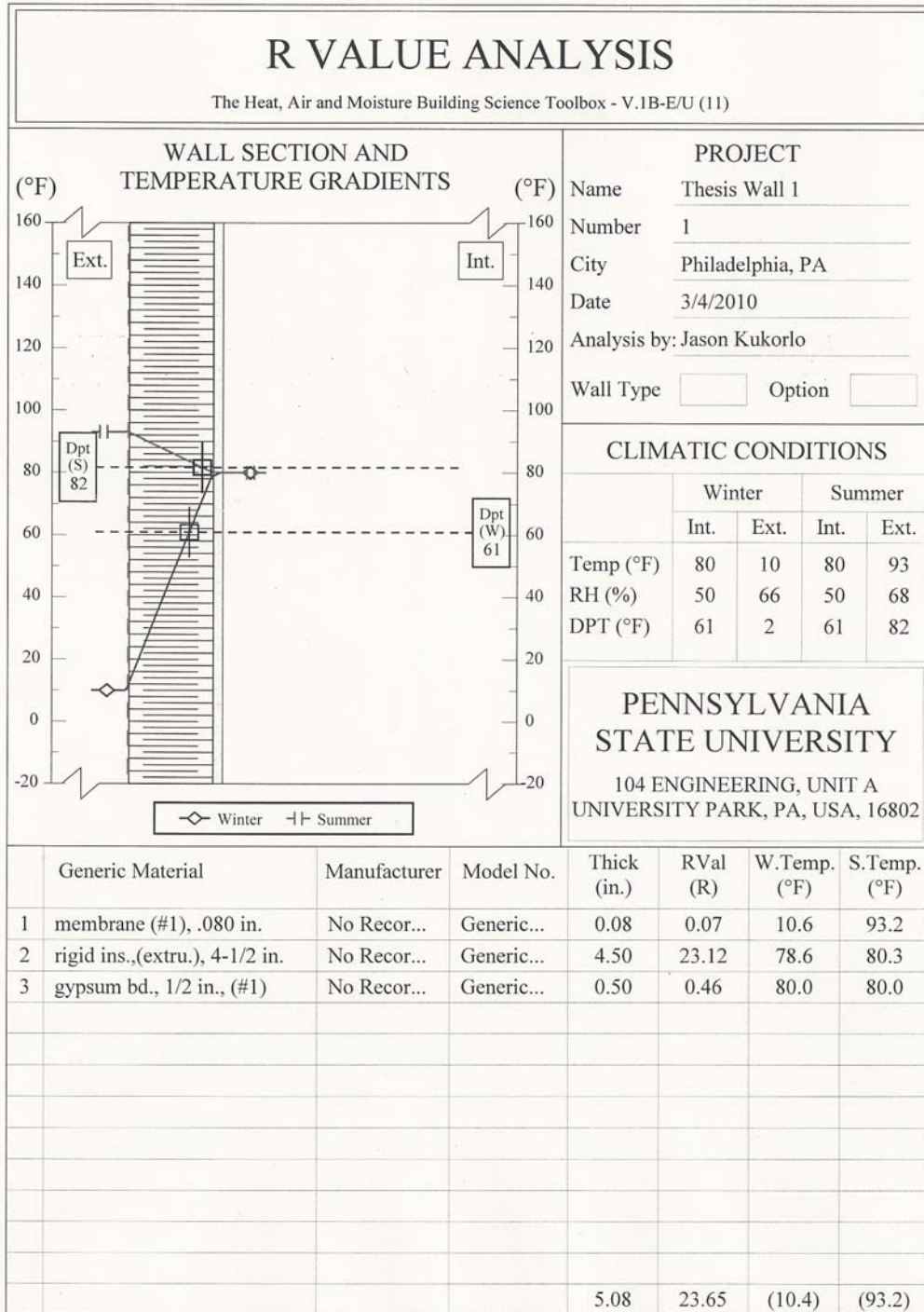
Results: Again, the results were very similar to those from Roof System #1 and Roof System #2. For all winter conditions, the dew point was located in the rigid insulation. Like Wall System #2, the moisture would then condense on the inner surface of the outer layer of gypsum wall board. However, since the gypsum wall board is moisture resistant it should not be negatively affected if moisture condenses on it. For summer conditions, the dew point was located in the rigid insulation for outdoor air temperatures of 75°F and 80°F, and the dew point was not located in the roof system for 85°F. Much like the situation for winter conditions, the condensed moisture would condense on the outer surface of the inner layer of moisture resistant gypsum wall board. As long as this gypsum board is not negatively affected by condensed moisture, it is properly located because it prevents moisture from condensing on the inside layer of the wall system that is exposed to the interior of the building.

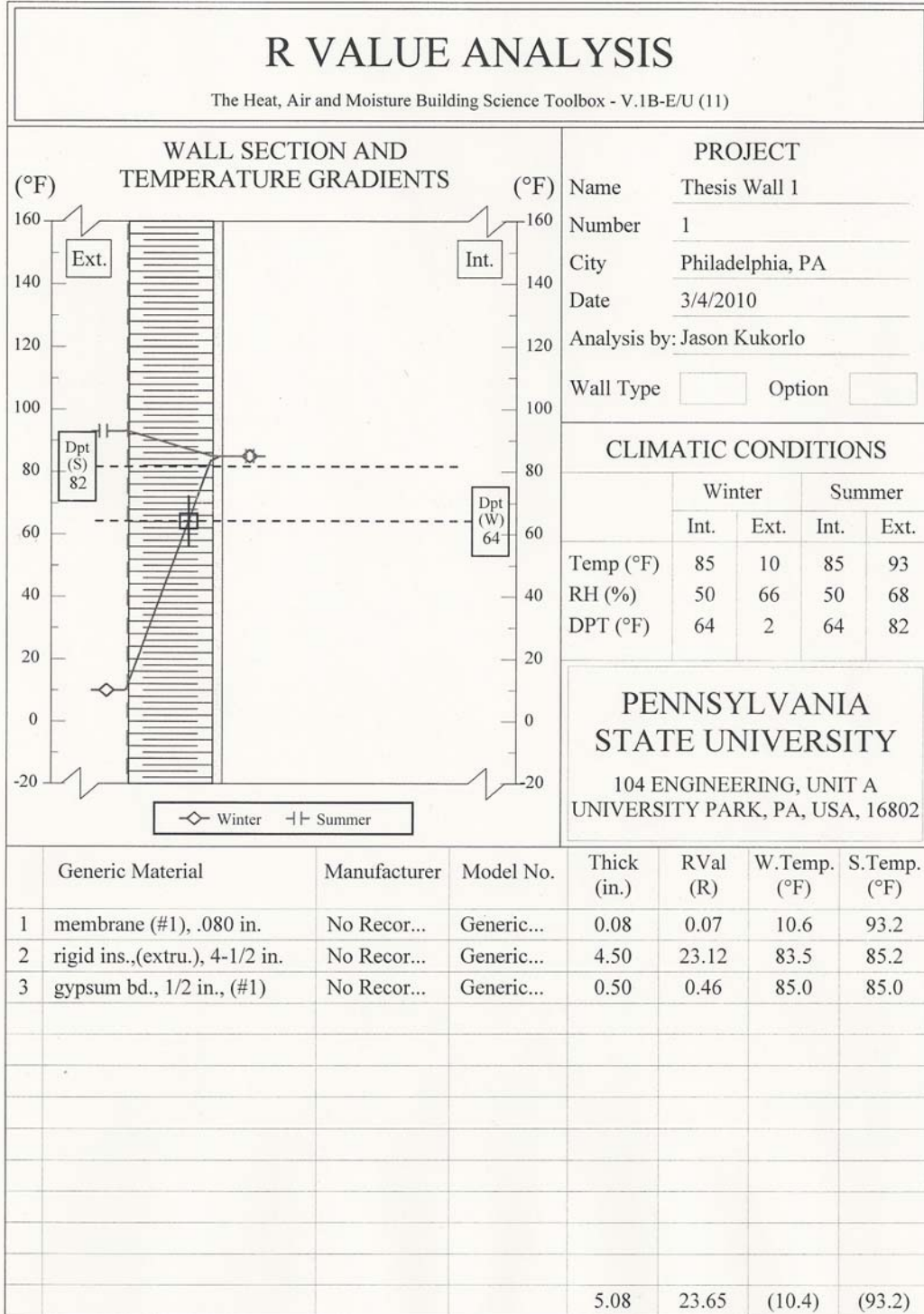
Roof System #4:

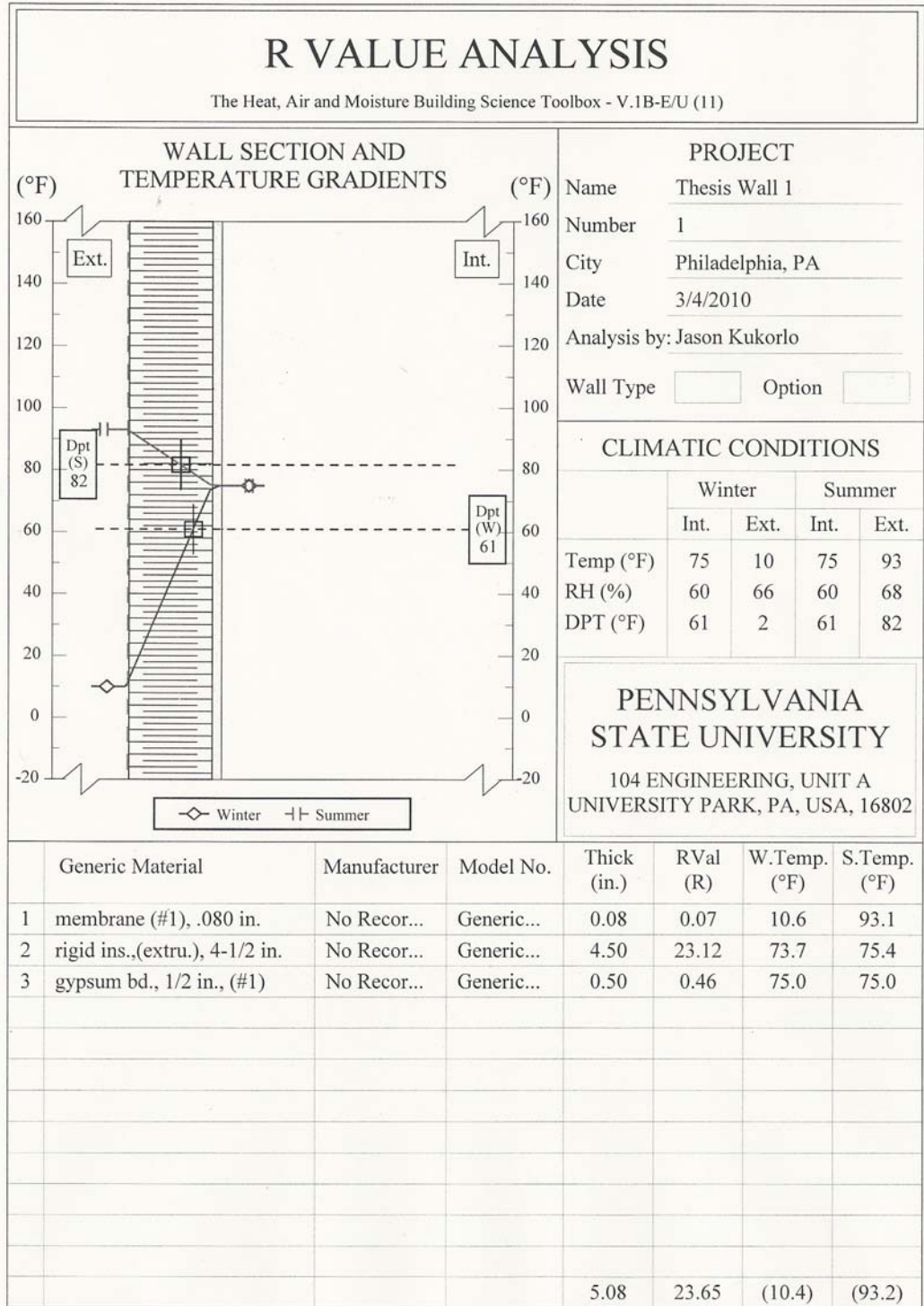
- Fully adhered membrane roofing system
- Vapor barrier
- 4 ½" insulation
- Precast concrete plank

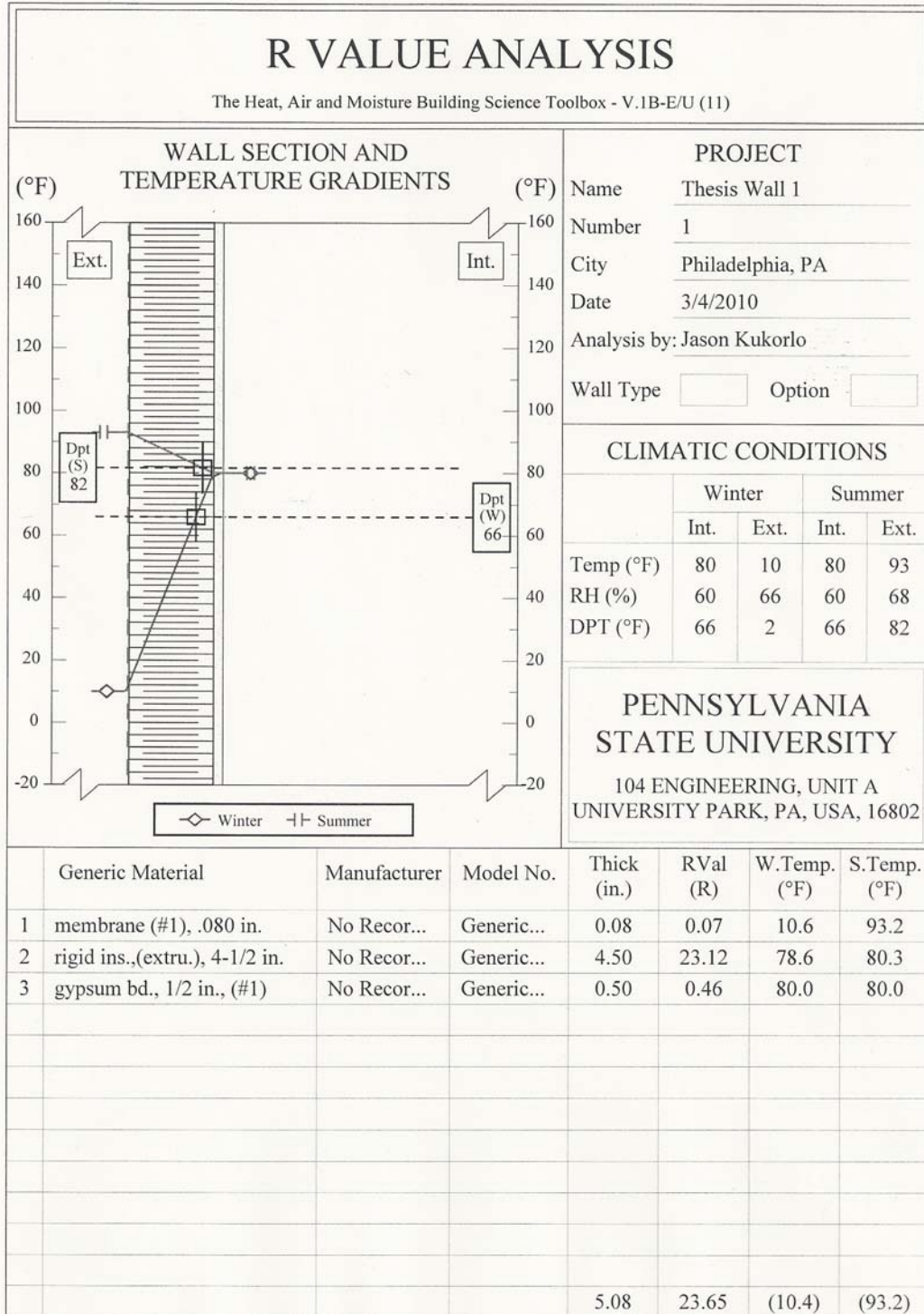
Results: For all winter conditions, the dew point was located in the rigid insulation. The vapor barrier was properly located since it would prevent condensed moisture from reaching the fully adhered membrane roofing system. For summer conditions, the dew point was located in the rigid insulation for outdoor air temperatures of 75°F and 80°F and was not located in the roof system for 85°F. The moisture would then condense on the outer surface of the precast concrete planks. Therefore, the roof system was properly designed for winter and summer conditions.

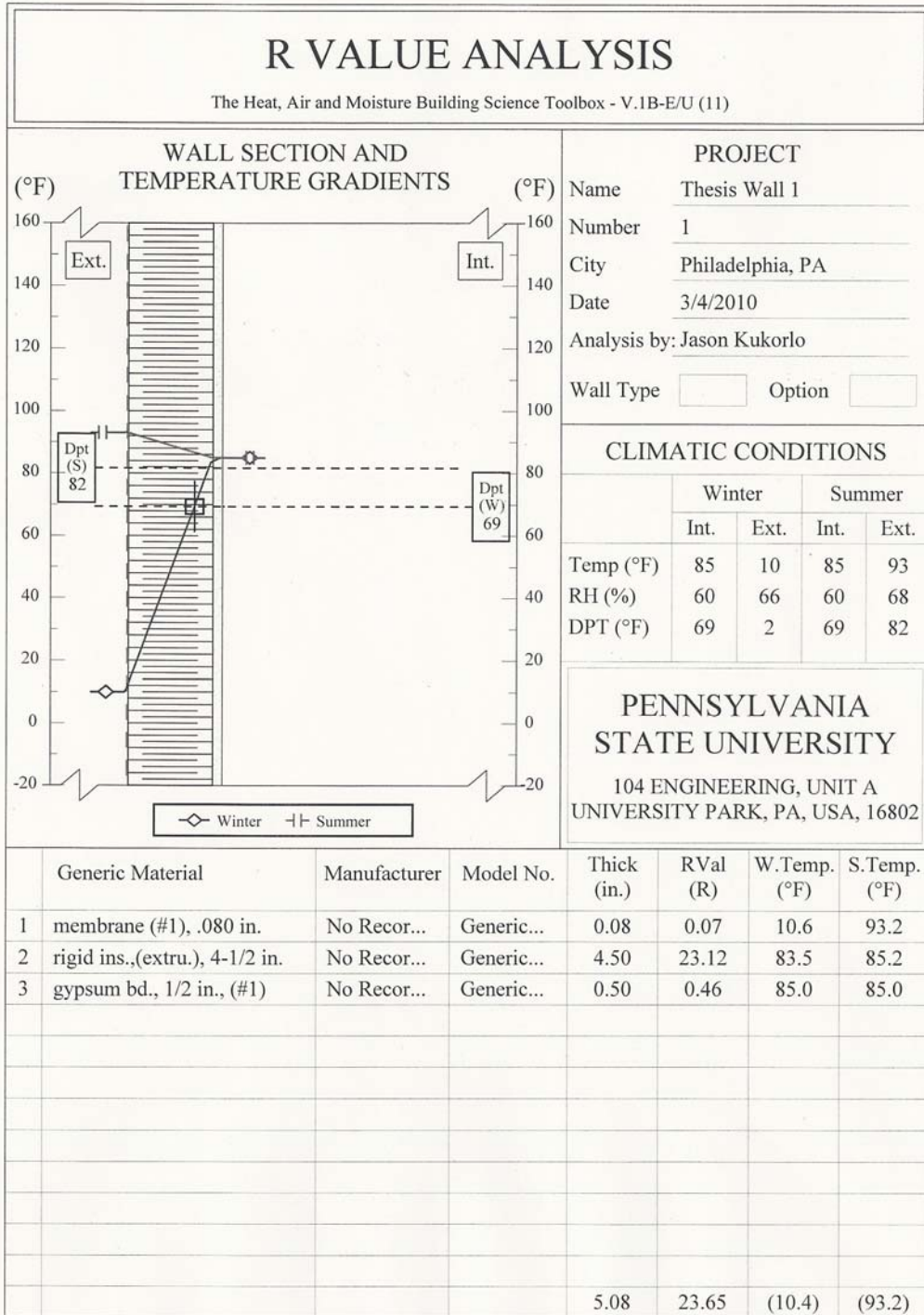


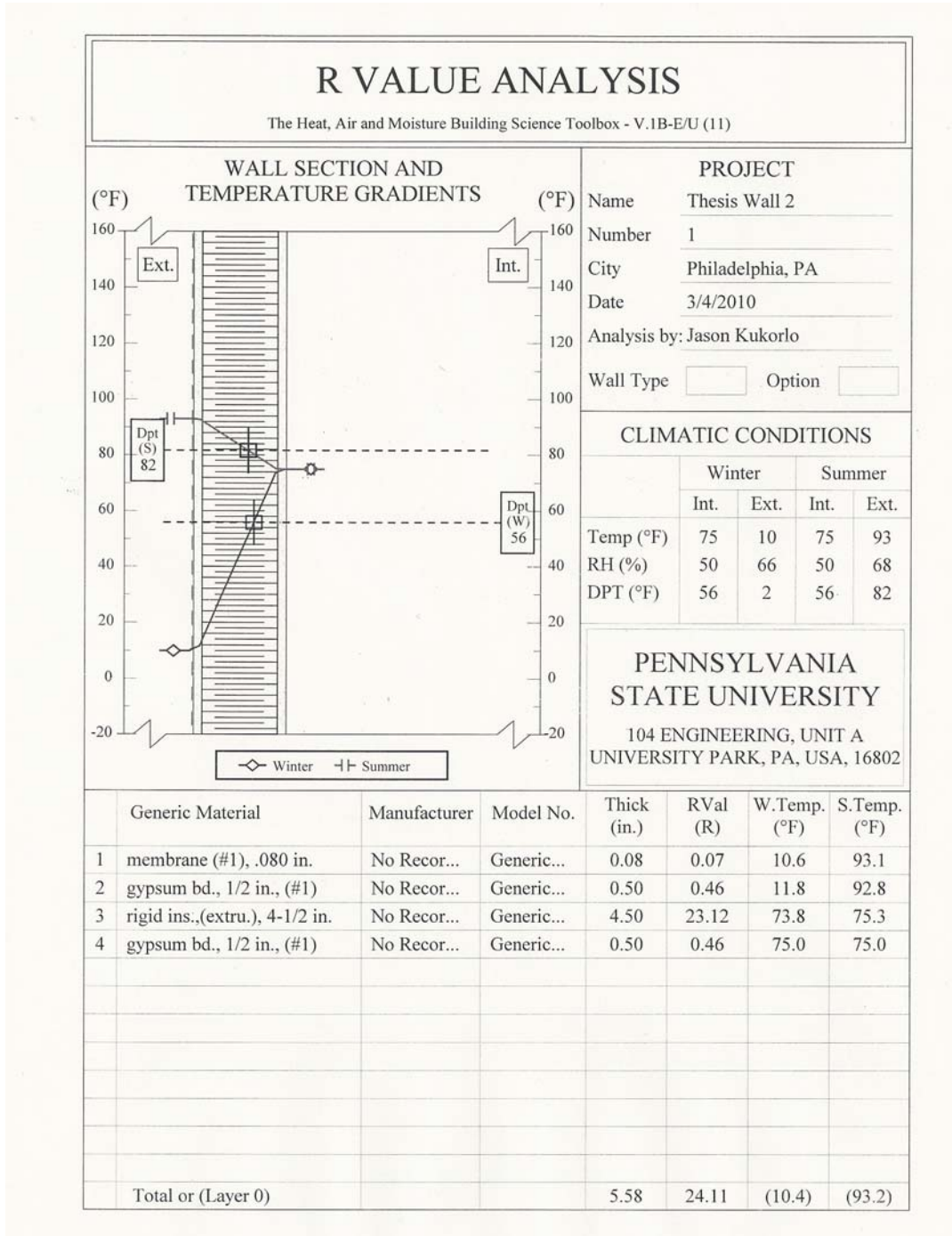


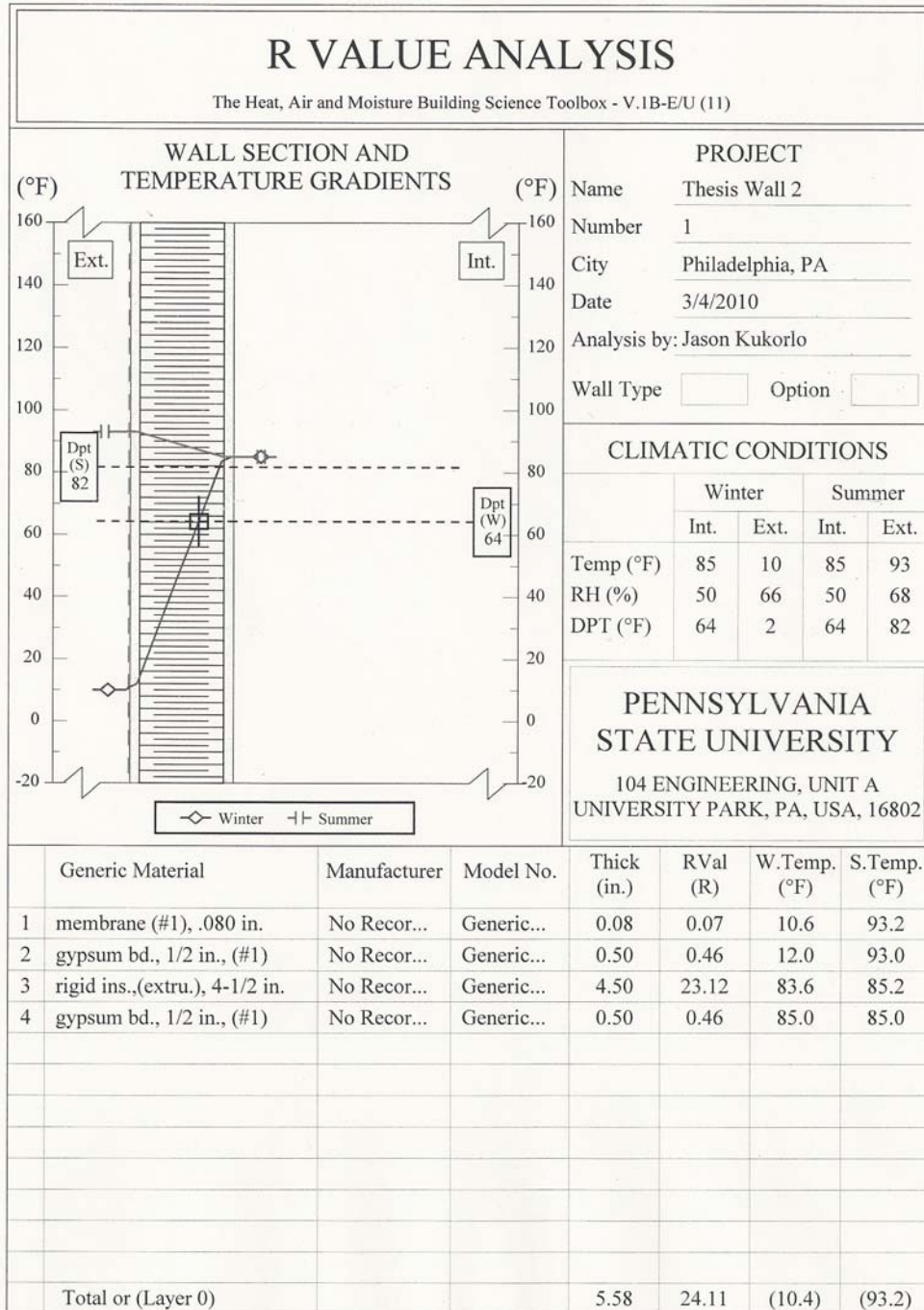


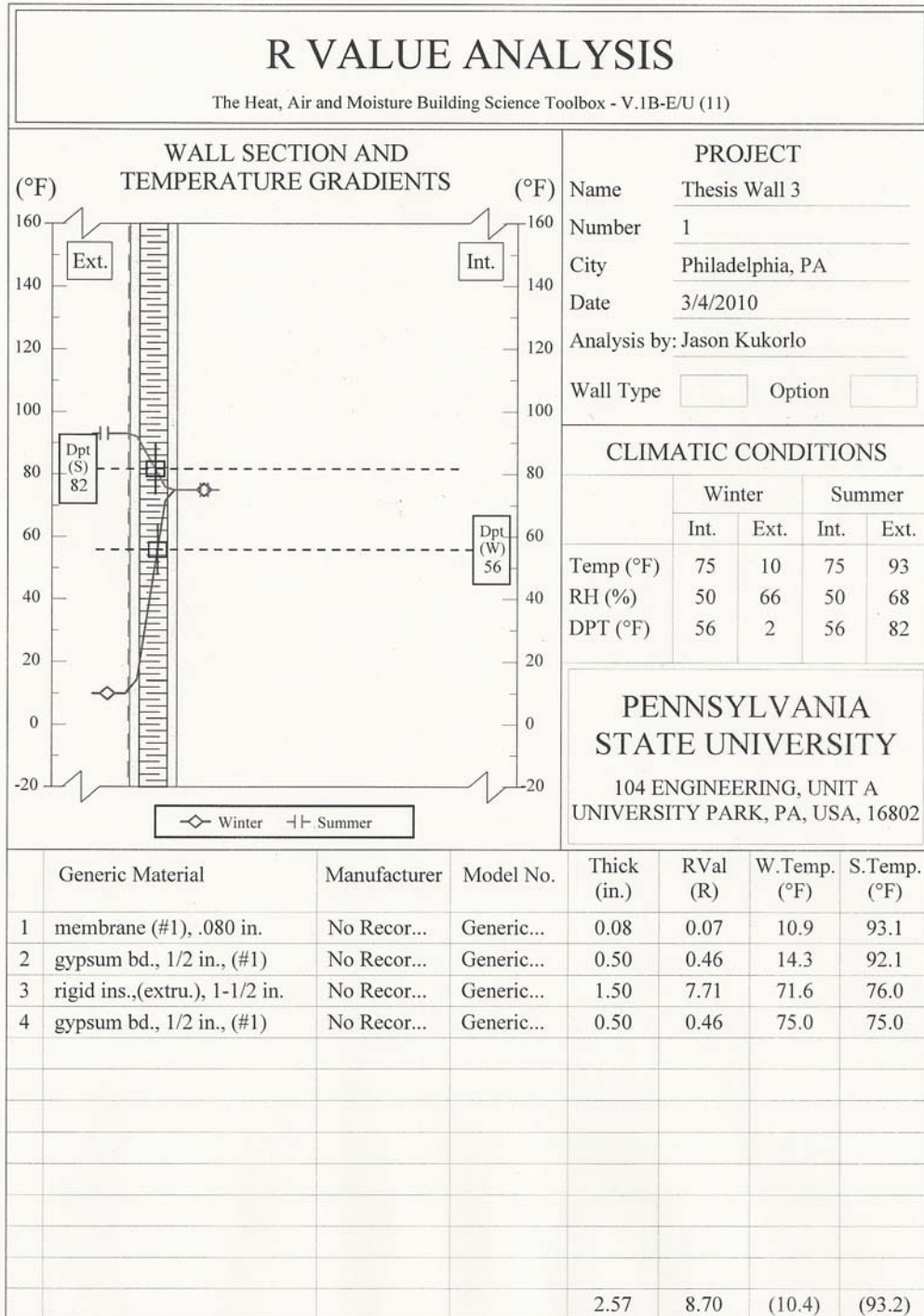












DensDeck

DensDeck is a roof board that provides excellent resistance to moisture. The roof system above the indoor pool of the Farquhar Park Aquatic Center uses DensDeck as the layer above the acoustical metal deck. DensDeck can be used as a membrane support layer or a roof underlayment and maintains high strength throughout cycles of dampness and drying. The roof board has fiberglass mats to resist mold growth and therefore help to provide a longer-lasting roof system. It has been shown to perform better than wood fiber and perlite in terms of resisting moisture absorption. Water can often destroy other roof boards and cause severe losses of strength, but DensDeck resists moisture absorption and retains its strength. DensDeck has a solid gypsum core treated with special processes, making DensDeck the only gypsum core roof board that is moisture resistant. The material has outstanding performance in high humidity situations, which definitely applies to the Farquhar Park Aquatic Center. When exposed to high humidity, DensDeck has been shown to absorb only about three percent of the moisture the wood fiberboard absorbed and about ten percent absorbed by perlite. Therefore, DensDeck was an wise choice to use for the roof system of the natatorium.

Precast Concrete Insulated Wall Panels

Precast concrete insulated wall panels surround the entire indoor pool area and most of the rest of the building. The wall panels were provided by Nitterhouse Concrete Products, Inc. After speaking with John Jones from Nitterhouse who worked on the project with Nutec, it was discovered that the precast wall panels were one of the main topics of discussion during meetings with Nutec as compared to the other precast components of the building. Most of the insulated wall panels used on the Farquhar Park Aquatic Center were 8" thick and about 10'-0" wide. Condensation cannot form within the precast wall panels since they are air-tight, hence making the panels mold and mildew resistant. Therefore, the insulated wall panels provided an excellent solution for the building enclosure of the natatorium, providing great condensation and moisture control. The panels are also architecturally pleasing in appearance and are available in many finishes. The insulated wall panels are very durable and strong and typically have a 2-4 hour fire rating. Successful thermal performance can be achieved by the panels, and they can be provided with a range of R-values. In addition, the panels attenuate sound transmission well and hence minimize noise transfer through the walls. These positive acoustical properties help to keep noise either in or out of the natatorium. The precast concrete insulated wall panels are also provided at competitive prices.

Solera-T Insulated Translucent Glazing Units

The large glass curtain walls enclosing the indoor pool area were recognized as a possible area of concern for the thermal performance of the wall system and the overall energy usage of the building due to their large size and the fact that the walls face North and South. The curtain walls are composed of Solera-T insulated translucent glazing units by Advanced Glazing Ltd. These units consist of two lites of glass with a high thermal performance translucent insulating core and are designed to fit into most curtain wall

systems. Solera T units diffuse the natural sunlight and allow a comfortable level of light deep into the space. The glazing units used on the natatorium have a panel thickness of 2 3/4" and a maximum U-factor of 0.25. The core material of the units consists of a semi-rigid interlocked acrylic insulating honeycomb with a light-diffusing cloth membrane. Moisture and pressure equilibrium are maintained within the glazing units by a stainless steel capillary tube vent. To prevent moisture from the inside of the building into the intra-frame cavity, the unit must be properly sealed on the interior. The intra-frame cavity must be drained and vented to the outside to prevent the buildup of humid air from the inside, which applies to the Farquhar Park Aquatic Center. This also maintains pressure equilibrium and allows any standing water to properly drain. To prevent condensation from forming on interior surface during winter conditions and hence improve thermal and energy performance, it is recommended that thermally broken frames be used with the Solera-T units. Structural calculations were going to be performed on these large glass curtain walls due to their expansive size, but the glass design methods learned in AE 542 are not applicable due to the special Solera-T units. However, information regarding the structural performance of these units was found. The honeycomb material used as the core of the glazing units is very stiff. Calculations show that a 96"x48" panel is capable of supporting loads of up to 500 psf normal to its surface when simply supported at ends separated by the 96" dimension. This exceeds, by far, the structural capacity of monolithic lites of glass and can span large areas with only the corners supported. Overall, the Solera-T glazing units provide appropriate moisture and thermal control for the Farquhar Park Aquatic Center. The units do not allow excessive amounts of heat in, and they admit diffuse light instead of direct, glaring light. In addition, the glazing units provide excessive strength and are structurally capable of spanning the extensive areas that they cover.

Fenestration Systems

Most modern architectural glass sheets are produced by casting a layer of molten glass of the desired thickness on a bed of molten tin in a process known as the "float process." The three main types of glass are annealed glass, heat-strengthened glass, and fully tempered glass. Annealed glass is not given any heat treatment to improve its strength and breaks into large, sharp shards when it fails.

Heat-strengthened glass is heated to about 1500°F and then cooled quickly to increase the strength of the glass in tension. This type of glass typically breaks into smaller fragments than annealed glass does and usually stays in its opening, although it can break into large shards as well. Heat-strengthened glass is approximately twice as strong as annealed glass and can handle higher wind loads and, in heat-absorbing glass, higher thermal stresses. The surface precompressive stress of heat-strengthened glass is relatively low and typically between 3,500 psi and 7,500 psi. Heat-strengthened glass is also less likely to fail from spontaneous breakage due to nickel sulfide if the residual surface compression is less than 7,500 psi. In addition, the appearance of this type of glass is often slightly distorted due to the heat treating process. Ceramic-coated heat-strengthened glass was used on the north façade of the Farquhar Park Aquatic Center

near the main entrance of the building. This type of glass was also used in multiple areas on the East façade of the building next to panels of low-E insulating glass units.

Fully tempered glass is heated to a higher temperature than heat-strengthened glass and cooled much more rapidly, hence increasing the surface precompressive stress in the glass to more than 10,000 psi. This type of glass breaks into small dice-like cubes, which is a much safer mode of failure than the large sharp shards of annealed glass. Fully tempered glass is about four times as strong as annealed glass but has a less pleasant architectural appearance because the heating process produces waves and visual distortions in the glass. Plus, nickel sulfide particles in the glass can cause spontaneous breakage of fully tempered glass. Nickel sulfide particles in the glass can expand when subjected to heat and hence cause a crack that propagates. Fully tempered glass is in the natatorium where safety glass is required, such as with the glass guardrails on the precast concrete balcony.

Laminated glass units (LGUs) consist of two or more plies of glass bonded together with a plastic interlayer, often polyvinyl butryal (PVB). The PVC attenuates sound transmission and helps prevent the transmission of ultraviolet rays through the glass unit. Low-E glass has a reflective or low-emissivity coating that reflects infrared radiation and visible light, hence improving the thermal performance of the glass. Insulating glass units (IGUs) are made up of two or more lites of glass with a concealed air cavity in between. A dessicant is commonly used to keep the air space dry. The air cavity helps to attenuate sound transmission as well as reduce heat gains and heat losses. Solar-control low-E insulating glass units were used on the South façade of the Farquhar Park Aquatic Center near the dish room, concession area, and lobby. They were also used on various parts of the East façade of the building beside sections of ceramic-coated, heat-strengthened glass. These units consist of a fully tempered outdoor lite and an annealed indoor lite. Laminated glass and insulating glass units also typically provide better acoustic performance than other types of glass.

In addition to playing a key role toward building aesthetics, a building's glazing system can have a significant impact on the building's thermal performance. Heat losses and gains through glass are important in terms of a building's peak electricity demand and energy use. Daylighting provided by glass facades can also have a large effect on a building's energy consumption. The thermal performance of a glazing unit is controlled by solar radiation (transmission, absorption, and reflectance) as well as the U-value of R-value of the glass. The low-E insulating glass units used on the Farquhar Park Aquatic Center have a maximum visible light transmittance of 50 percent, a winter nighttime U-factor of 0.33, a summer daytime U-factor of 0.33, a solar heat gain coefficient of 0.40, and a maximum outdoor visible reflectance of 17 percent.

The resistance of a glazed perimeter to intruding moisture controls the moisture protection of glazing. Both wet glazing and dry glazing systems are used to prevent moisture infiltration through glass units. Wet glazing uses a gunable sealant at the glass perimeter and is generally more expensive than dry glazing, although it provides better

moisture protection than dry glazing. Dry glazing uses rubber gaskets to create moisture seals and depends on the compression of the gasket to keep out air and water.

Proper design of the glass components of a building is essential. One of the main goals of glass design is to keep the façade from breaching. It is much cheaper to replace glass than to fix problems due to the loss of building operations. Glass-to-frame contact can be avoided by using appropriate setting blocks at the bottom glass edge and side blocks, or anti-walk pads, along the vertical glass edges. Glass strength capacity calculations for wind loads were performed below for two glass curtain wall panels used on the Farquhar Park Aquatic Center.

Glass Strength Calculations

Strength calculations for two glass curtain wall panels were performed using ASTM E 1300: Standard Practice for Determining Load Resistance of Glass in Buildings. Both panels were solar-control low-E insulating-glass units with an outer lite of 1/4" fully tempered monolithic glass and an inner lite of 1/4" annealed monolithic glass. The first glass panel was one of a series of similar glass panels located on the South façade of the building enclosing the main entrance lobby. The non-factored load for each lite was 24.66 psf, which was based on the length of 110", width of 60", and 1/4" thickness. Both the inner lite and outer lite were checked for short duration loads and long duration loads. The lower of these four values controlled the capacity of the insulating glass unit. The governing strength of the IGU was 24.66 psf based on the load resistance of the inner lite for long duration loads. The maximum wind load in the North/South direction at the location of the insulating glass unit was 13.04 psf, hence the unit had sufficient capacity (24.66 psf) to carry the wind load. Calculations for glass strength are found in Appendix C.

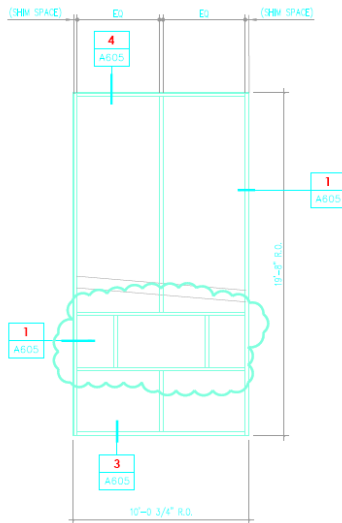


Figure 92 – Glass Panel Used for Glass Calculations

The second glass panel was located on the east façade and enclosed a portion of the concession area in the ground floor lobby. This insulating glass unit was rather large, with a height of 150" and width of 60". Due to these dimensions, the non-factored load for the IGU dropped to 15.675 psf. It was clear that increasing the dimensions of a glass panel has a significant effect on the load-carrying capacity of the glass unit. In this case, the governing strength of the insulating glass unit was 15.675 psf based on the load resistance of the inner lite for long duration loads. The capacity of the inner lite for long duration loads also controlled the strength of the first IGU that was analyzed. The maximum wind load in the East/West direction at the locating of the insulating glass unit was 12.92 psf. Therefore, the IGU did have sufficient capacity to withstand the load, although the unit did not have much additional strength above the required wind force. A larger safety factor may be preferred.

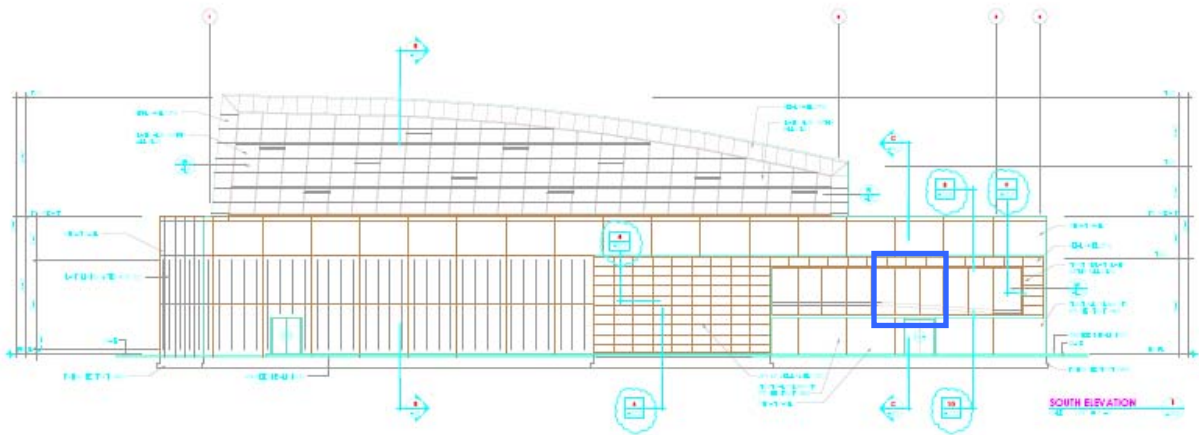


Figure 93 – Elevation of South Façade of Natatorium Showing Glass Panel Used for Second Glass Calculations

Façade or Building Enclosure Continuation using AE 537 (MAE Breadth)

Pressure Treated Wood

Pressure treated wood is wood that has been chemically preserved to prevent moisture decay and attack from termites and other insects. The pressure treatment process forces chemical preservatives into the wood by placing the wood in a closed cylinder and applying pressure. The preservative is bonded to the wood fiber by a “fixation” process. There are three general classes of wood preservatives used for pressure treatment. Waterborne preservatives are typically used for agricultural, residential, industrial, commercial, recreational, and marine applications. Creosote and creosote/coal-tar mixtures are commonly used for utility poles, railroad ties, pilings, guardrail posts and timbers used in marine structures. Oil-borne preservatives such as Pentachlorophenol, or Penta, and Copper Naphthenate are most often used for industrial applications, including utility poles. The pressure-treatment process is the most effective method for protecting wood use exposed to marine environments and allows deeper penetration of the chemical preservatives into the wood.

The wood structural system designed for the Farquhar Park Aquatic Center will need to be pressure treated due to the corrosive natatorium environment. Waterborne preservatives are most often preferred for marine building applications and would hence most likely be used for the glulam trusses, columns, and lateral bracing systems located in the indoor swimming pool area. These types of preservatives are paintable, clean, and odorless. In addition, waterborne preservatives are EPA-registered for both interior and exterior use without a sealer. Waterborne preservatives that are typically used include Chromated Copper Arsenate (CCA), Copper Azole (CA), Alkaline Copper Quat (ACQ), Sodium Borates, and Micronized Copper Quat (MCQ).

Wood preservatives penetrate sapwood, the outer living portion of a tree, more easily than heartwood, the inner dead portion of a tree. Southern pine is often used in pressure treating due to its high percentage of sapwood. This is one of the main reasons that the glulam trusses for the Farquhar Park Aquatic Center were chosen to be designed using southern pine. Stainless steel is often recommended as one of the best materials to use to prevent corrosion problems, but it is usually more expensive and more difficult to obtain. Hence, it seemed that looking into a wood structural system instead of another steel system would be more cost effective and therefore provide a better means of meeting the overall financial goals of this project.

In December of 2003, Chromated Copper Arsenate (CCA) was discontinued for general consumer and residential use. CCA had been used successfully for years, but the public negatively viewed the use of the preservative in recent years due to the presence of arsenic in the chemical. The public became concerned about the environmental impacts of CCA and the effects it could have on people, in particular children. The use of CCA is still permitted for poles, piles, saltwater marine exposure, permanent wood foundations, and in engineered wood products like structural composite lumber, plywood, and glulam timber. Most new formulations of pressure treating preservatives are copper based,

which tend to have more corrosive effects on steel connectors, anchors, and fasteners than wood treated with CCA. Galvanic corrosion, or galvanic compatibility, occurs when two different types of metal are put in contact with each other causing one to deteriorate and the other to basically remain unaffected. The presence of moisture causes the magnitude of damage to increase. With wood pressure-treated with copper-based preservatives, a chemical reaction occurs between the copper and the metal used as a connector or for flashing. During this process, the copper is mostly unaffected while the other metal tends to corrode. Chemical formulations of preservatives often vary from product to product, so chemical preservatives must be properly selected for each given application to try to avoid corrosion problems. The use of stainless steel is one of the best solutions for galvanic corrosion since it is closer to copper on the galvanic scale. Zinc or galvanized coatings also solve the problem with success. The most highly recommended fastener types for compatibility with copper-based preservatives include stainless steel, hot-dip galvanized, corrosion-resistant polymer coated products, copper, and silicon bronze. It is suggested that any flashing used with these connections or wood treated with copper-based preservatives be stainless steel, copper, or coated copper.

Former CCA-treated wood was produced at a lower cost than the newer copper-based preservatives, and basically all wood treated with CCA was given the same amount of preservative. The copper-based preservatives are used to treat wood based on various exposure and retention levels due to the higher overall production costs of these preservatives. Retention levels, or ratings, are broken up into three categories: Decking, Above Ground – Exterior, and Ground Contact. “Above Ground” is the standard for outdoor exposure, and “Ground Contact” involves the highest level of treatment. The glulam trusses used for the Farquhar Park Aquatic Center would most likely fall into the “Ground Contact” rating due to the highly corrosive indoor pool environment.

Preservative-treated wood is recommended in situations where wood is in contact with the ground, below water, or exposed to weather. It is also used when wood is in contact with or imbedded in concrete, such as the glulam trusses of the Farquhar Park Natatorium design that sit on the beams and columns of the concrete moment frame at column line 2. It is crucial that the products used for the pressure treatment of the glulam trusses, braced frames, and decking be carefully selected to prevent corrosion of the bolted metal side plate connections and any other metal fasteners used in conjunction with the wood structural system.

Common Problems with Metal-Plate-Connected Wood Trusses

Metal-plate-connected wood trusses are commonly used in short-span residential applications and long-span industrial structures. Typical spans for commercial buildings can be around 80 feet. These trusses are often shop-fabricated, which reduces labor costs in the field. Although they provide a relatively cost effective structural system, the trusses are very flexible and unstable until they are properly braced and set in place. The most common problem leading to failure of metal-plate-connected wood trusses during construction is missing or improper temporary truss bracing. Many of the same problems involved with metal-plate-connected wood trusses apply to the large glulam trusses

designed for the Farquhar Park Aquatic Center, even though these trusses use bolted metal side plates instead of toothed metal plates. Care must be taken when the glulam trusses that span 130'-0" are erected during the construction process.

The trusses are very long and slender and provide little or no resistance to out-of-plane bending. Therefore, it is crucial that adequate lateral bracing be provided to ensure out-of-plane stability of the trusses. Common bracing problems include both insufficient temporary bracing that can lead to failure during construction and insufficient permanent bracing that can cause collapse of the structure while it is in service. Collapses of long-span trusses can even occur several years after the trusses are erected.

Storage of the trusses on site is another common source of failure. Sites are often not perfectly flat surfaces for the trusses to lie on, and out-of-plane bending of the trusses while laying on an uneven surface can put additional stresses into the truss members and connections that were not originally accounted for in the design process. While short-span light-weight trusses can often be lifted into place by hand, long-span trusses such as those designed for the natatorium must be lifted by crane. The Truss Plate Institute provides guidelines for the proper design, handling, and erection of these trusses.

Wood trusses are most prone to collapse between the time the trusses are set in place and the time the sheathing is nailed down. It is pertinent that temporary bracing be properly placed to provide lateral support until the roof sheathing that provides the diaphragm action is placed. Often times spacers used to maintain equal distances between the trusses during the erection process is the only source of lateral bracing support provided until the roof sheathing is set in place. Additional permanent bracing of the trusses may also be required. It is common to provide diagonal lateral bracing over several trusses.

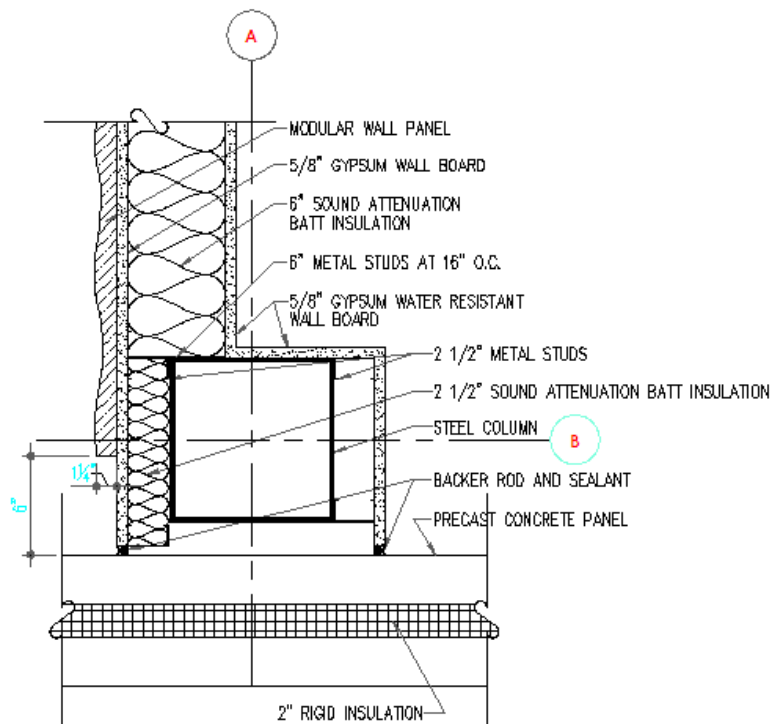
The design of the glulam trusses for the Farquhar Park Aquatic Center provides additional permanent bracing members connecting the bottom chords of the trusses. This lateral bracing spans the entire length of the roof in the North/South direction and is spaced at 26'-0" o.c. Temporary lateral bracing must be provided during the erection of these large trusses to avoid lateral bending and potential catastrophic failure of the trusses. Extra care must be taken since the trusses are also located 40'-0" in the air over the open indoor pool space below.

Waterproofing and Detailing

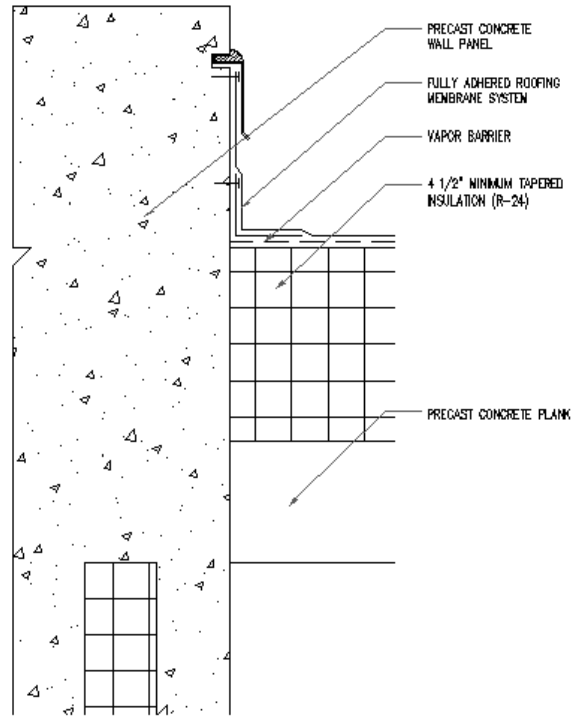
A large portion of building problems and construction claims occur at the roof and façade. Improper detailing, lack of detailing, and misunderstanding of the behavior of a wall system are some of the most common sources of problems. The building envelope is the most expensive part of the building, typically accounting for about 20% of the cost of the building as compared to about 5-6% for structural steel. Most problems occurring at facades are moisture related. A presentation entitled "Fundamental Wall Waterproofing Concepts" by Simpson Gumpertz and Heger pointed out three cardinal rules in waterproofing:

- Successfully integrate backup waterproofing and flashing to provide a watertight wall system.
- Provide watertight flashing to direct water out of the wall system.
- Assume water will penetrate exterior surfaces and provide redundancy (i.e. backup waterproofing membrane to collect this leakage)

Shown below is a detail showing the intersection of a stud wall and precast concrete panels at a corner column location. It can be seen that the backer rod and sealant are properly located between dissimilar materials. The 5/8" gypsum water resistant wall board and the precast concrete panels expand and contract as different rates, so using a backer rod and sealant to separate the two materials from touching each other is crucial to avoid wall performance problems at these joints.

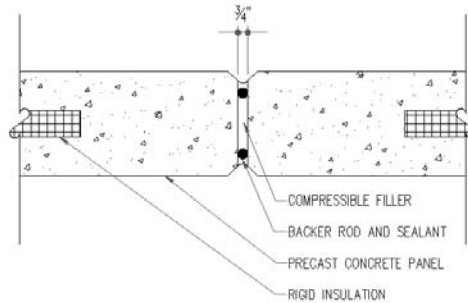


The parapet flashing detail pictured next shows the correct shape of the sealant at the top of the flashing. A convex sealant profile will typically fail, so a concave sealant profile is often suggested for optimum performance.



11 **PARAPIT FLASHING DETAIL**
 A501 SCALE: 3" = 1'-0"

The ideal depth-to-width ratio of sealants is 1/2:1 to 1:1. For the Farquhar Park Aquatic Center, backer rod and sealant details between the precast insulated concrete wall panels used for a majority of the building envelope were investigated. Most panels were 9'-11 1/4" with a 3/4" expansion joint in between the panels. When measured in AutoCAD, the typical panel joint detail showed that the depth of the sealant ranged from about 3/8" to 3/4", when fell into the recommended depth-to-width ratio of 1/2:1 to 1:1.



TYPICAL PANEL JOINT

In high-humidity environments such as that of a natatorium, it is often desirable to separate other parts of the building from the indoor pool area to prevent the spread of

moisture from this area to other regions of the building. The overall layout of the Farquhar Park Aquatic Center seems to deal with this quite well. On the ground floor, the expansive main lobby is separated from the indoor pool area by three corridors that are each 32'-0" deep. Two of the corridors even have two doors. These separations between the pool and lobby area can help prevent humid air from the pool area from easily penetrating into the lobby. At the concourse level, the indoor pool area is separated from the ramp, stairs, and essentially the open main lobby space by multiple sets of doors. Again, this helps to mitigate the spread of hot, humid air throughout the rest of the building. There is basically no direct continuous open path from the indoor pool area to the lobby for air to travel, helping to create a more comfortable lobby space for visitors.

Conclusion

The structural depth study that investigated alternate roof systems for the Farquhar Park Aquatic Center determined that a Southern Pine glulam truss system provided both an architecturally pleasing yet cost effective solution. A cost analysis using RS Means Building Construction Cost Data showed that the laminated decking for the wood system was much cheaper than the long-span metal deck used for the original design. The trusses themselves were estimated to be about the same cost as the original steel trusses. Even though the weight of the wood roof system was more than that of the roof system with the curved steel trusses, the wood roof system overall was found to cost less than the steel system using cost estimates from RS Means. The glulam trusses also provided the ability to achieve a curved roof shape at a competitive price, hence maintaining architectural style in the design. In addition, Southern Pine is often used for pressure treated wood due to its ability to absorb pressure treatment chemicals better than other species of wood. Therefore, using Southern Pine provided an excellent solution for the glulam truss system since pressure treatment would be required due to the natatorium environment.

The bolted connections of the wood trusses made up a large portion of the overall wood system cost. This cost could be decreased by perhaps using a curved top chord instead of using several straight individual members as was done in the structural depth study. This would eliminate several of the large top chord connections by maybe only using three or four top chord members and hence only three or four top chord spliced connections. Overall, however, the glulam truss system design was found to meet the goals of the thesis project by providing a cost effective solution that still maintain architectural integrity.

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