Farquhar Park Aquatic Center

York, PA

Final Report

Jason Kukorlo

Structural Option

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March 7, 2010

Farquhar Park Aquatic Center
York, Pennsylvania

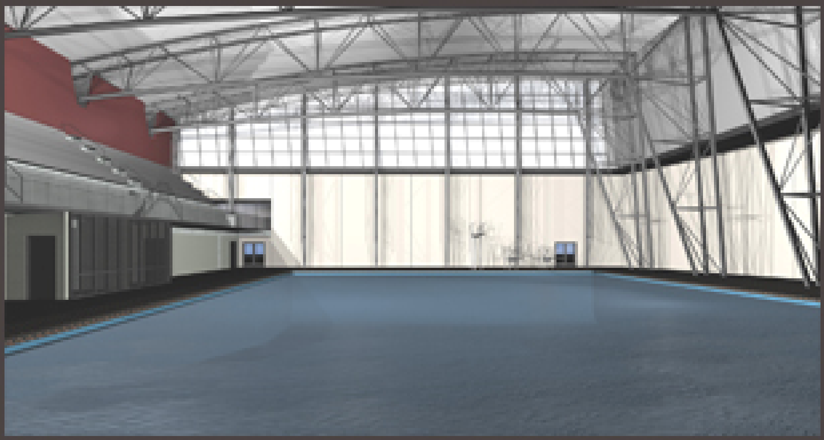
Project Information
Occupancy - Assembly
Size - 36,987 SF
Stories - One story (53'-0") with raised seating
- Entrance, concourse, and gallery level
Construction Dates - Project Not Constructed
Cost - $13 million
Delivery Method - Design-Bid-Build

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.

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

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

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 continous 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 ¾" 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

The author wishes to thank the following firms and individuals for their assistance and generosity with this thesis project:

Nutec Design Associates, Inc.
David Atkins
Douglas Shively

YMCA of York and York County

The Pennsylvania State University
Dr. Linda Hanagan
Professor M. Kevin Parfitt
Professor Robert Holland
Dr. Andres Lepage
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 than 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 – Arial 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.

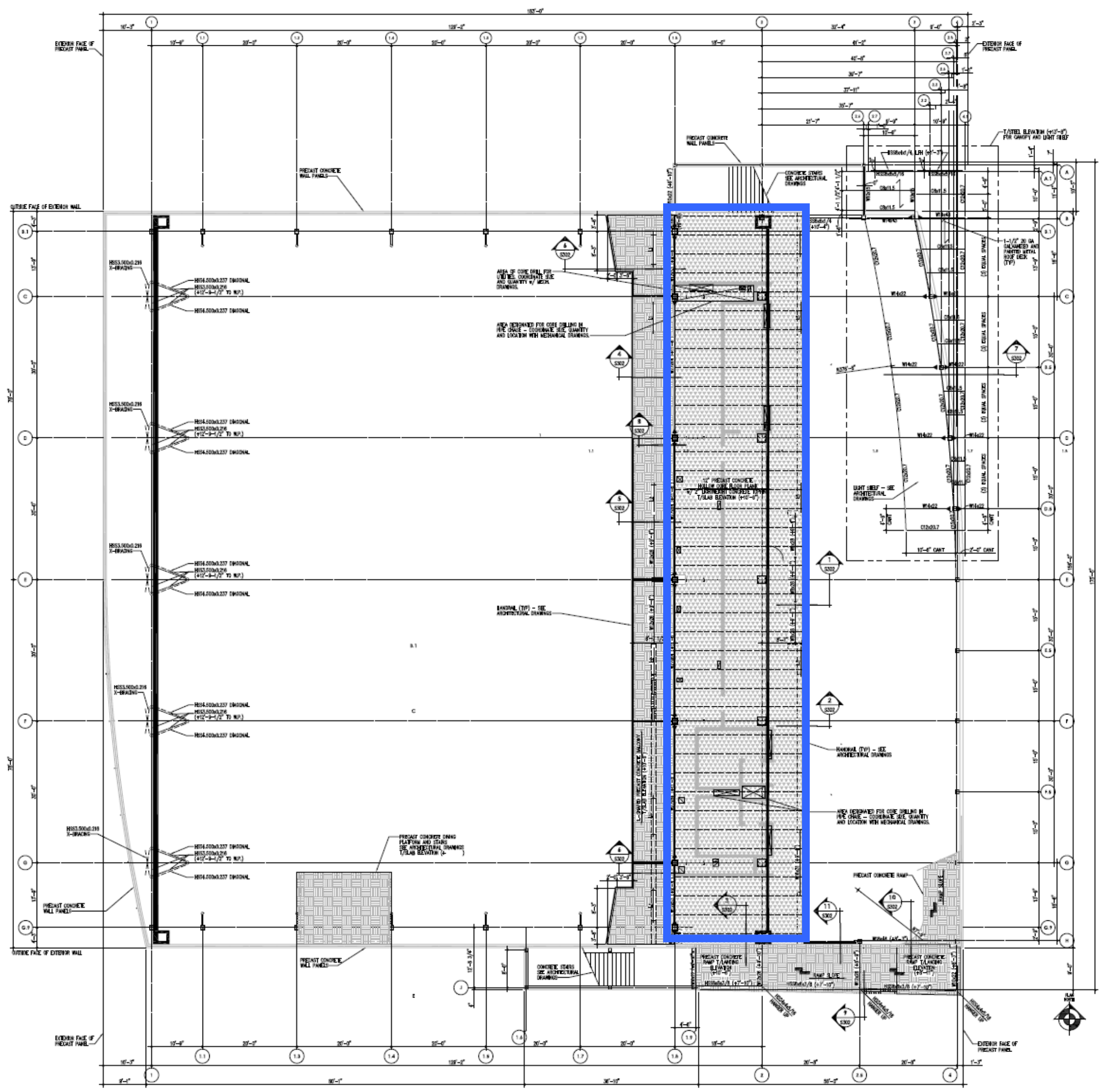


Figure 2 - Concourse Level Framing Plan (12" precast concrete hollow core floor planks are shown in blue – they span 27'-0" and run almost the entire length of the building)

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.

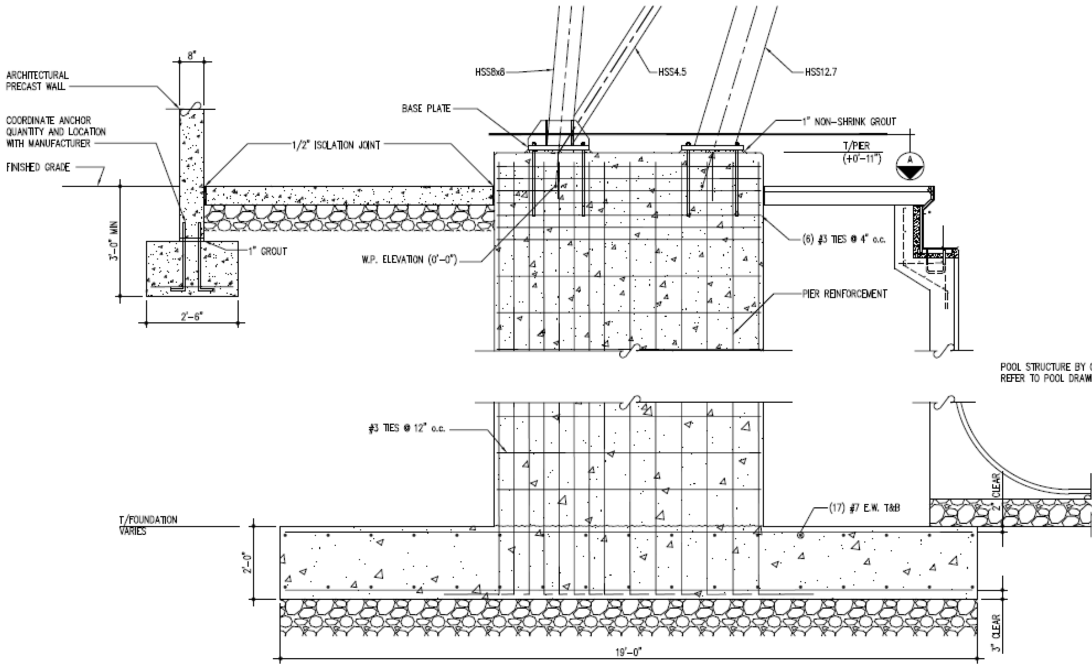


Figure 4 – Detail of Pier Supporting Large Tapered Truss Column

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.

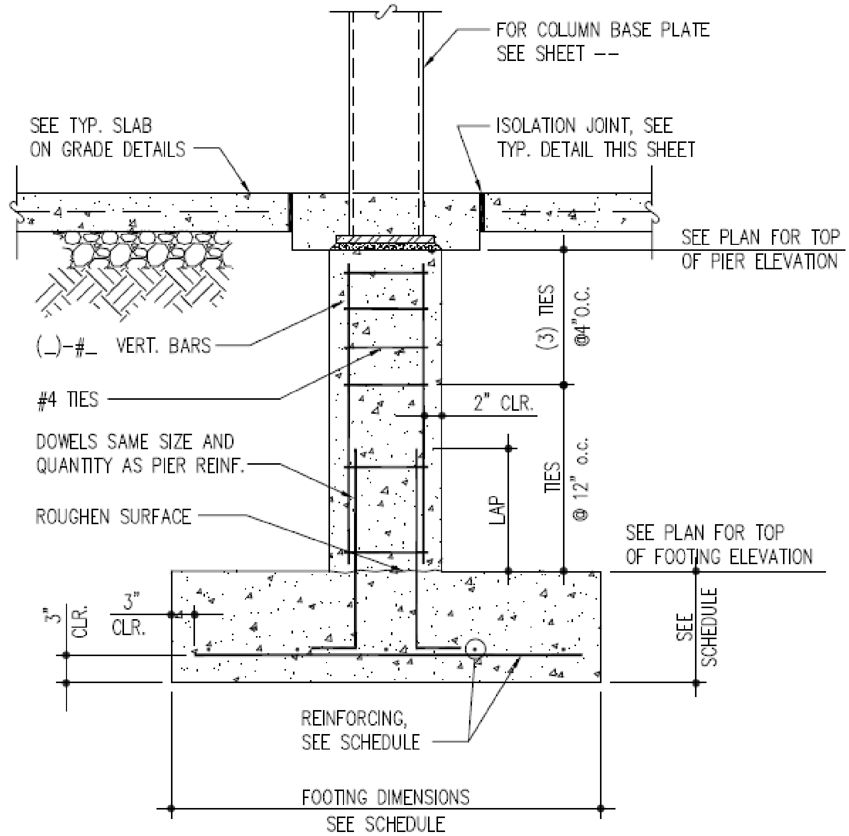


Figure 5 – Typical Pier Detail

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.

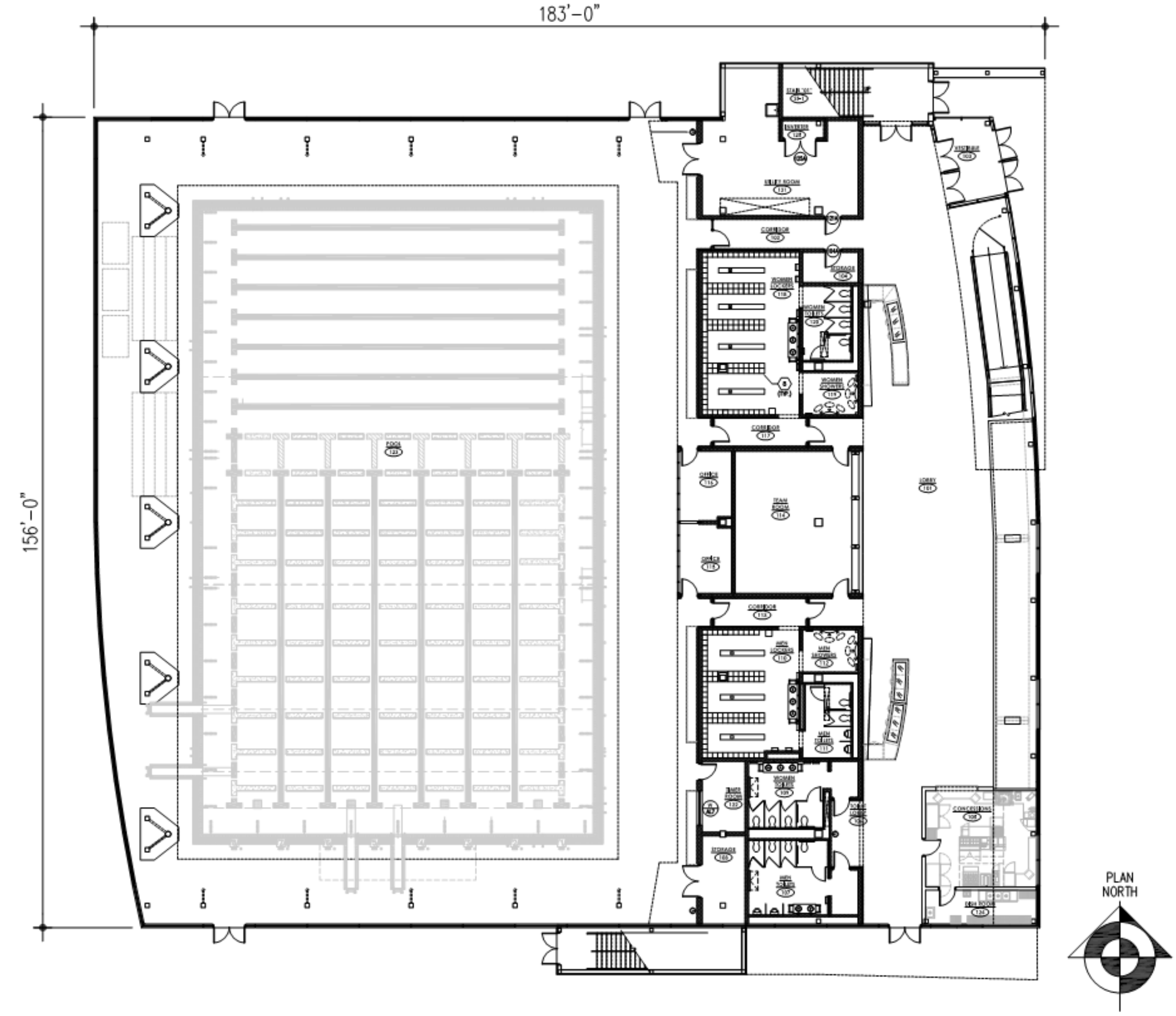


Figure 6 – Entry Level Floor Plan

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 ¼" 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.

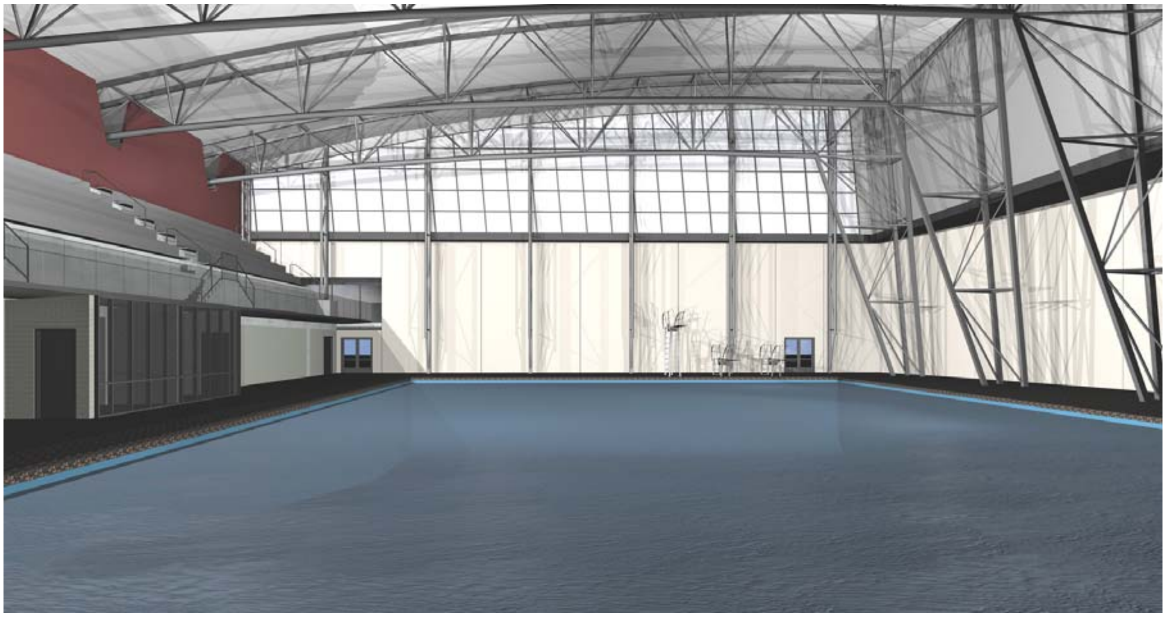


Figure 7 – Rendering of Indoor Pool Area Showing Large Curved Trusses

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.

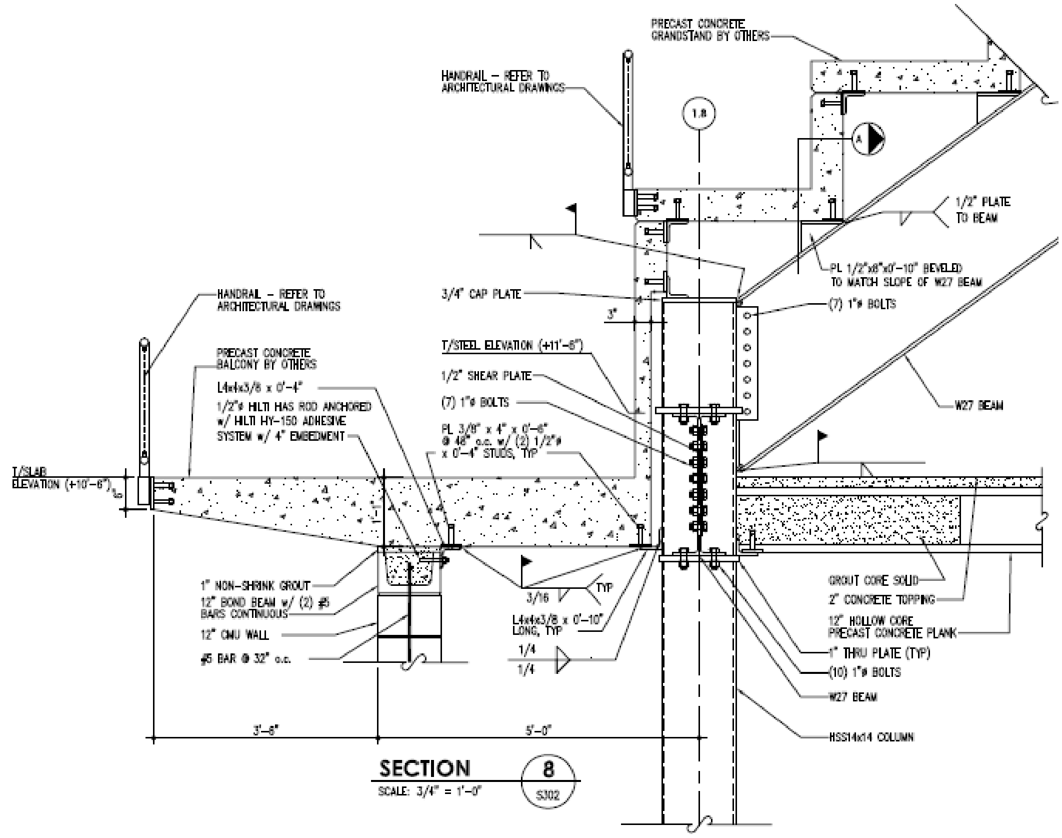


Figure 8 – Section Showing the 12" Hollow Core Precast Concrete Planks, the Precast Concrete Balcony, and the W27x94 Beams Supporting the Concrete Grandstand

The gallery level has HSS roof trusses spanning about 41'-0" and spaced 15'-0" on center (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.

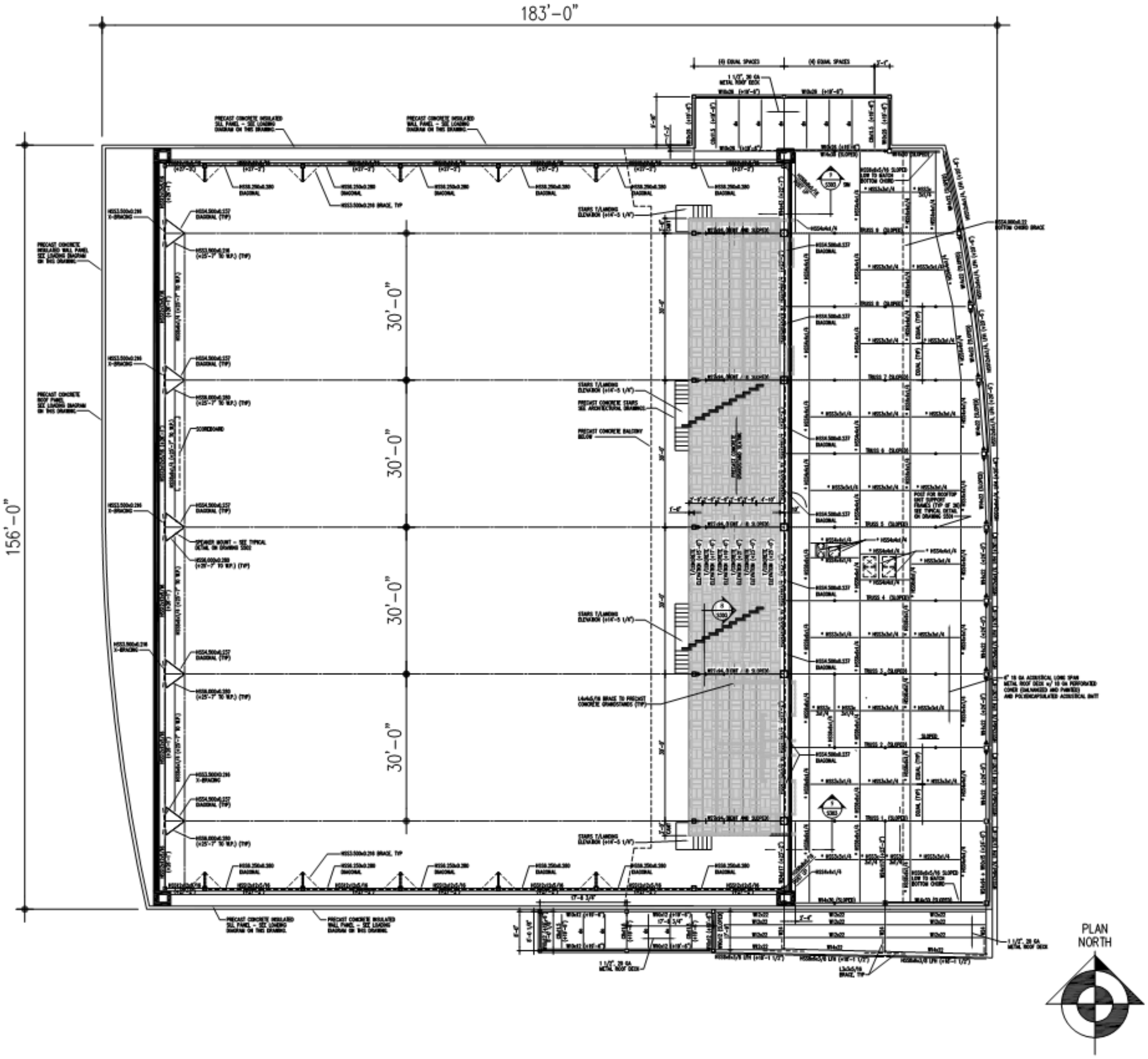


Figure 9 – Gallery/Mezzanine Level Framing Plan (the shaded portion is the grandstand seating area)

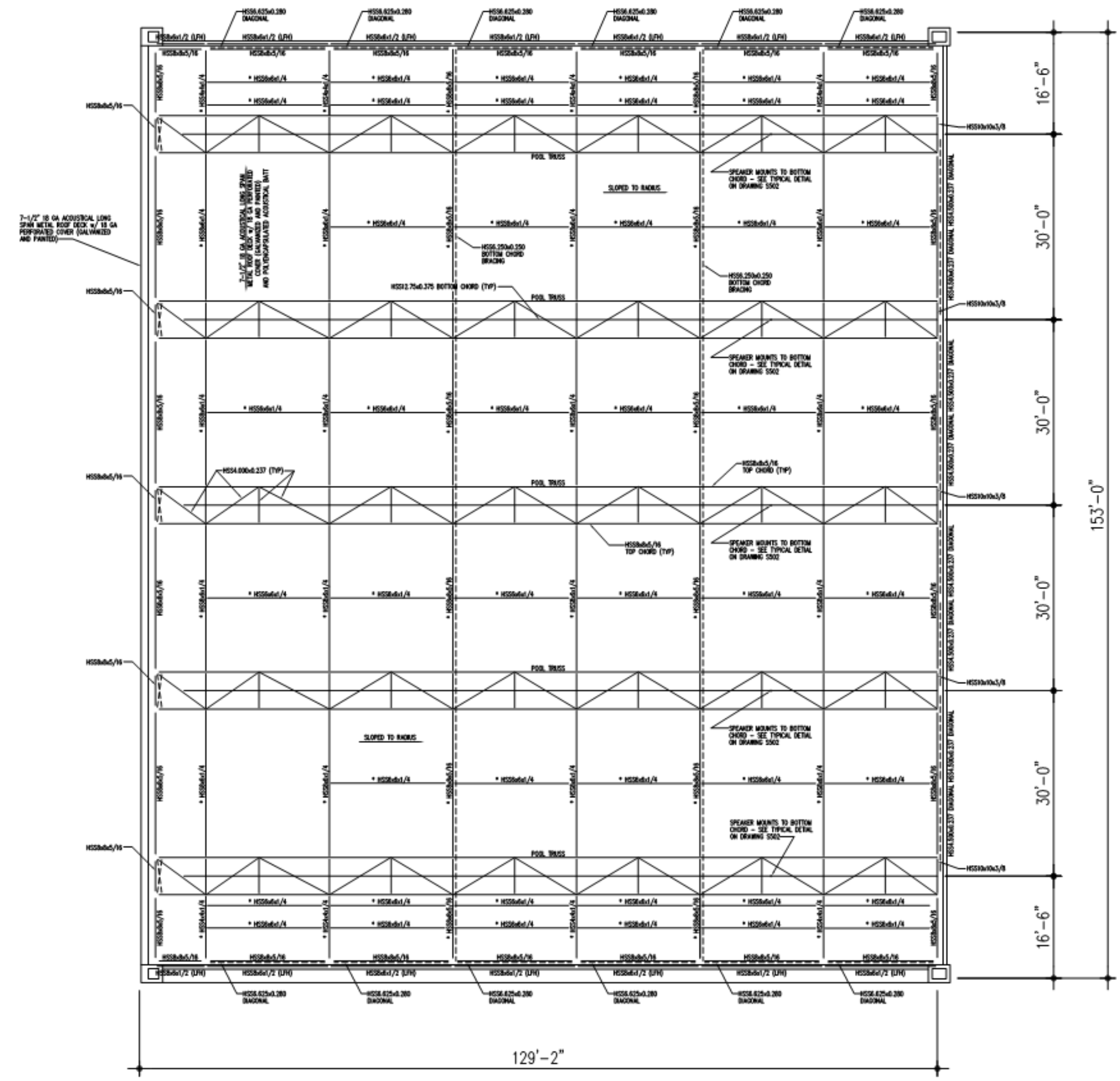


Figure 10 – Roof Framing Plan (including the five large trusses above the pool area spaced 30'-0" on center and additional framing)

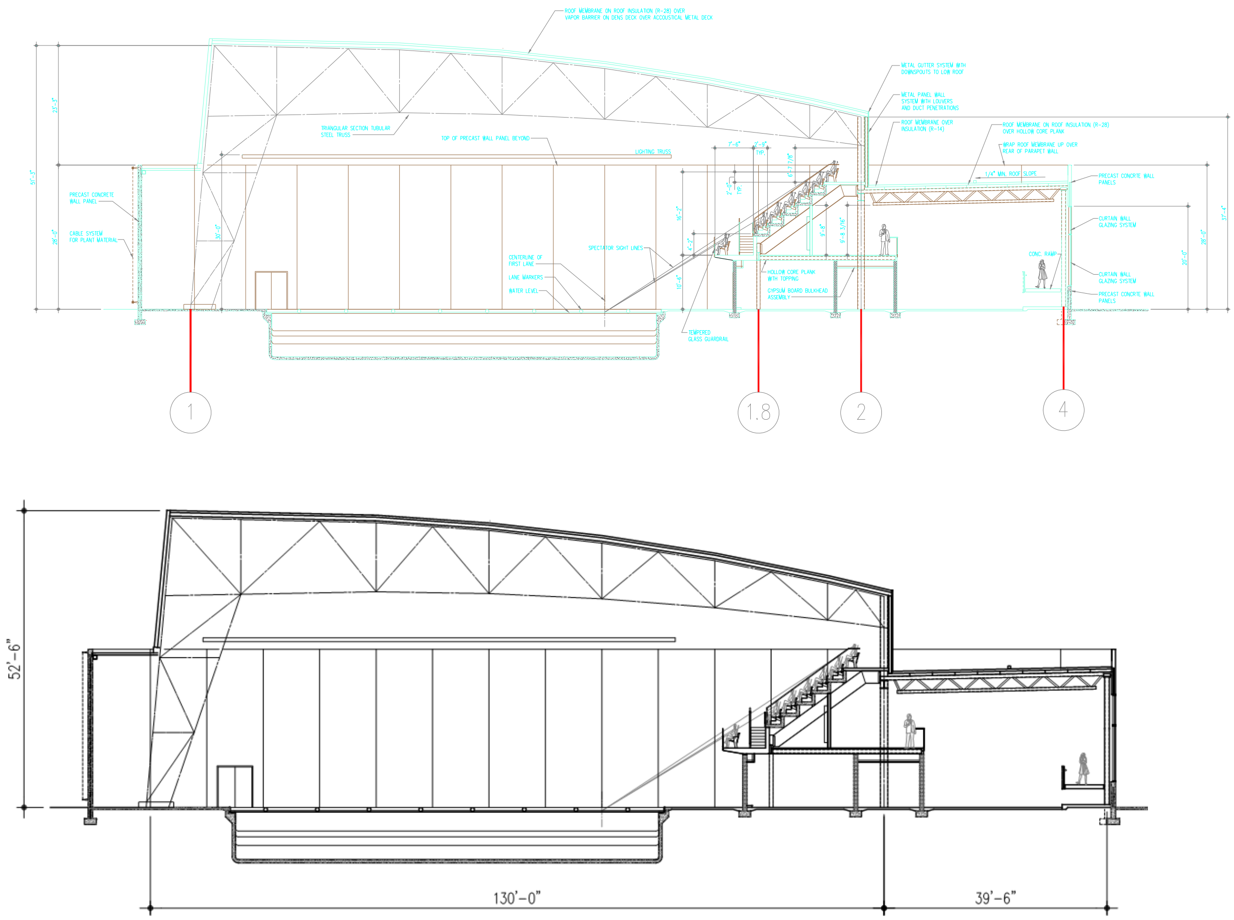


Figure 11 – Cross Section Through Center of Building (Looking North); Top Figure Shows Column Lines Mentioned Throughout Thesis Report

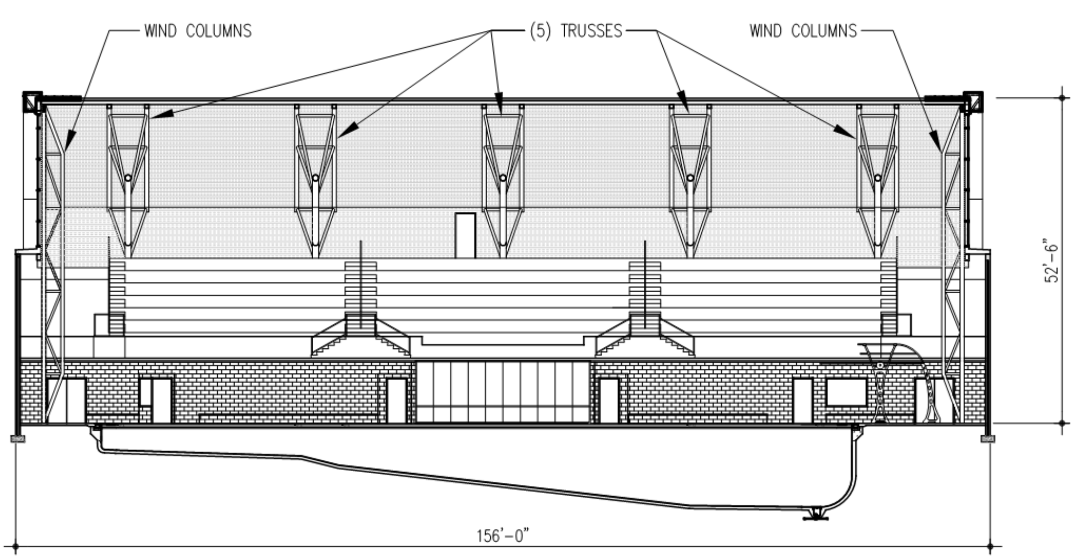


Figure 12 –Cross Section Through Indoor Pool Area Showing the Wind Columns (Looking East)

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.

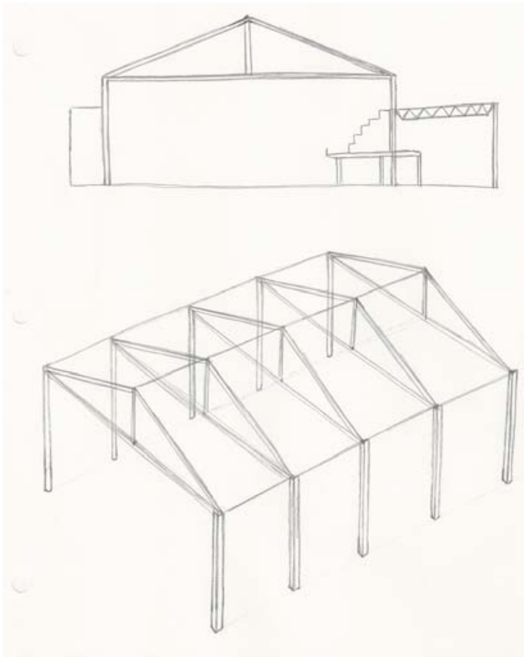
Structural Depth

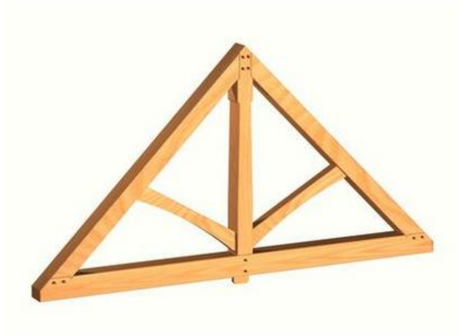
Gravity System Study

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.

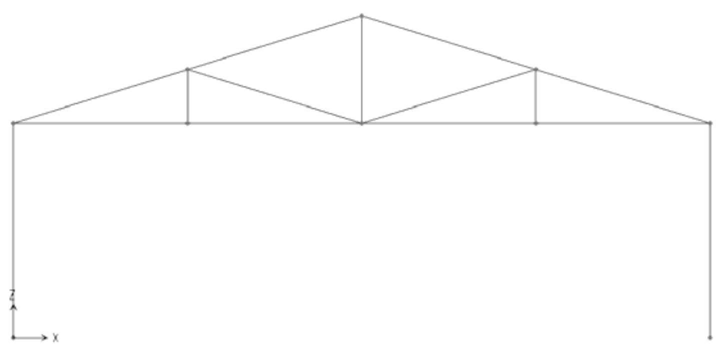


Figure 15 - SAP2000 Model of Triangular-Shaped Steel King Post Truss with 15'-0" Depth

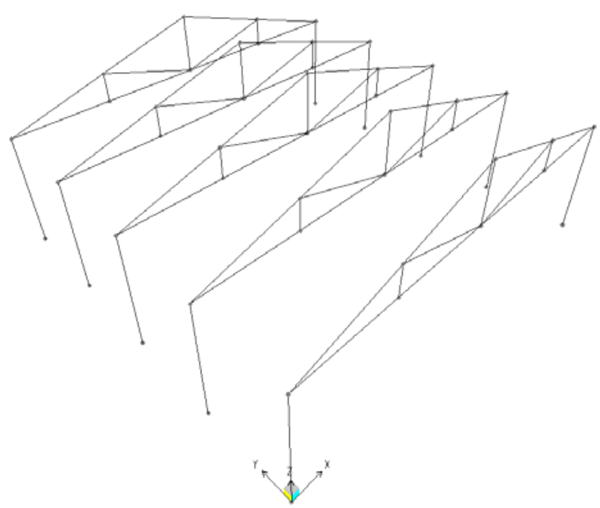
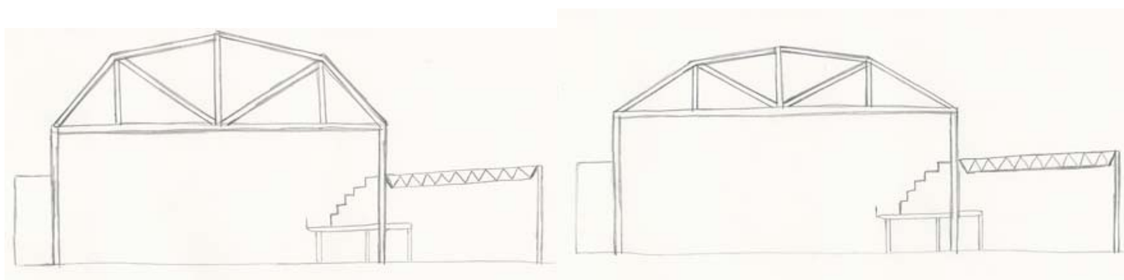


Figure 16- SAP2000 Model of Triangular-Shaped Steel King Post Truss with 15'-0" Depth Spaced 30'-0" o.c.

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 ½ x 5 ½ 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

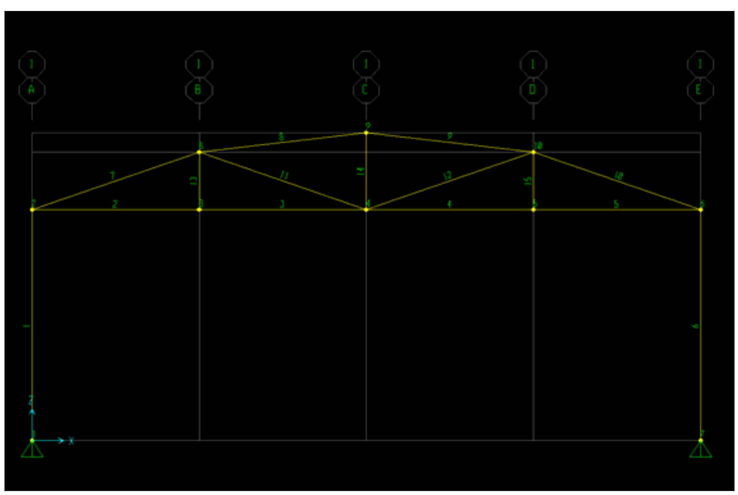


Figure 19 - SAP2000 Model of King Post Truss with 15'-0" Depth and More Curved Appearance

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.

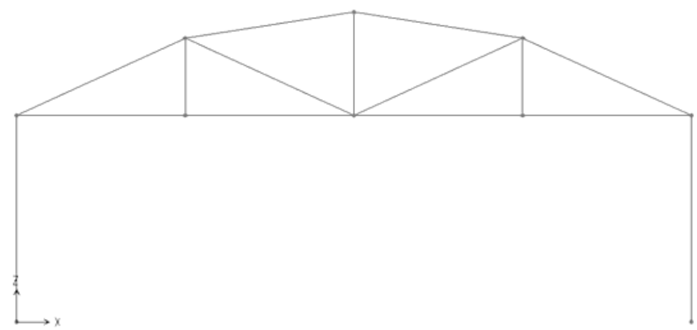


Figure 20 - SAP2000 Model of King Post Truss with 20’-0” Depth

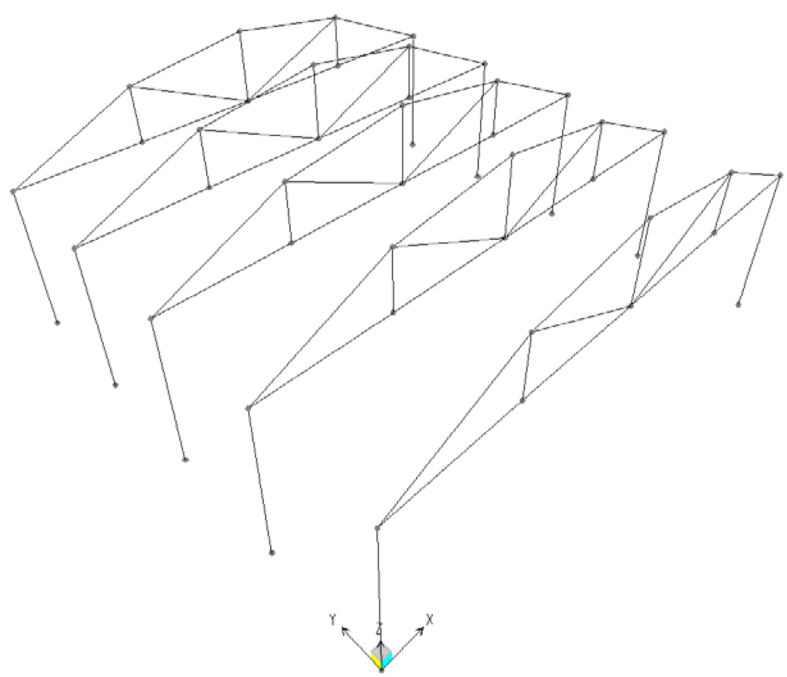


Figure 21 - SAP2000 Model of Steel King Post Trusses with 20’-0” Depth Spaced 30’-0” o.c.

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.

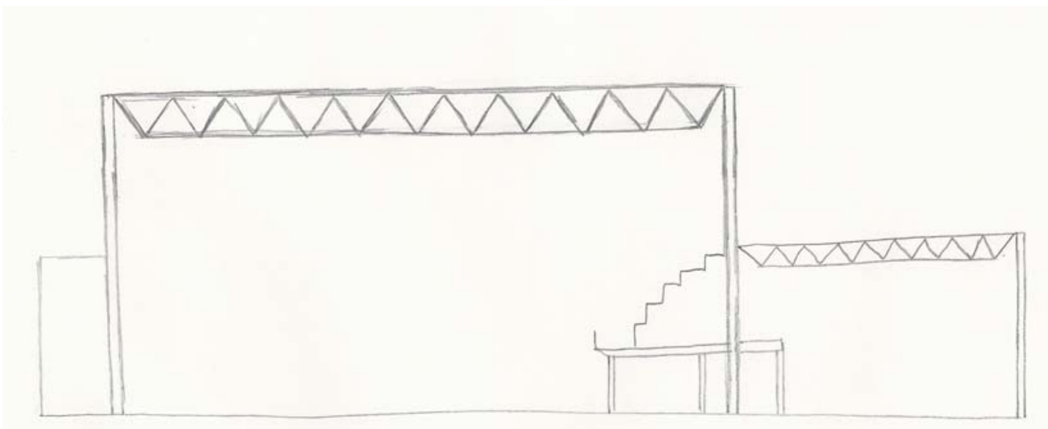


Figure 22 - Preliminary Sketch of Space Frame for the Natatorium

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 to 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.

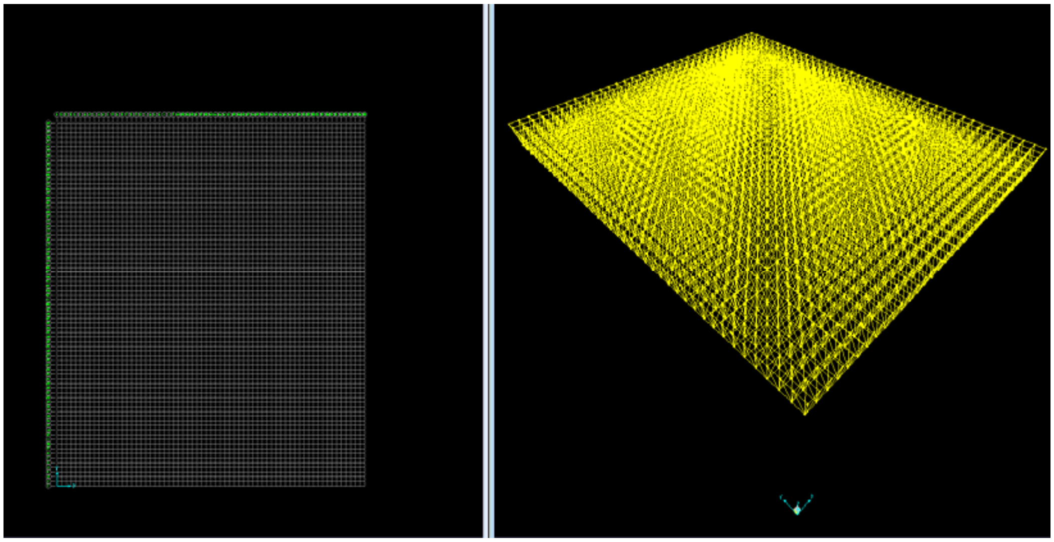


Figure 23 - Space Frame with 4'-0" Modules and 10'-0" Depth

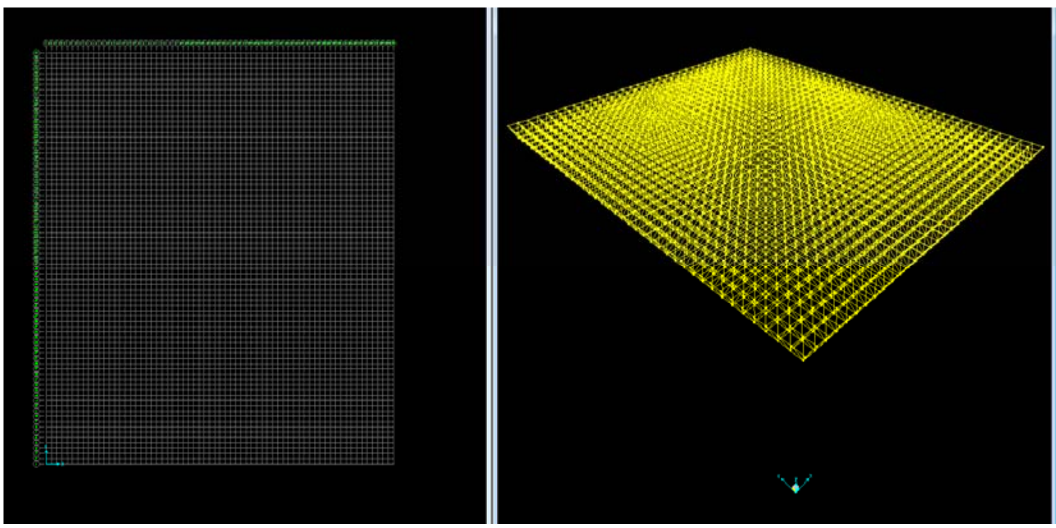


Figure 24 - Space Frame with 4'-0" Modules and 5'-0" Depth

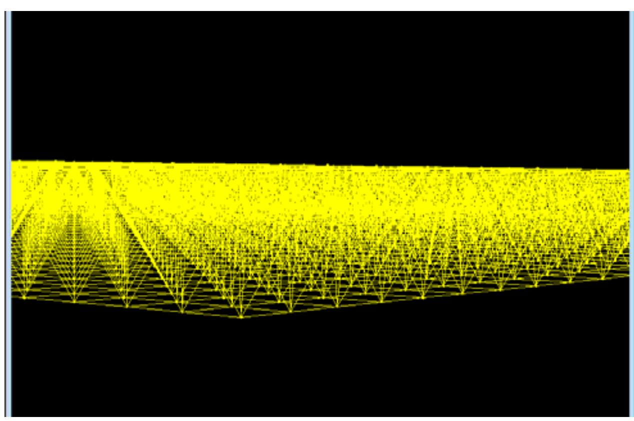


Figure 25 - View to Show Depth of Space Frame with 4'-0" Modules and 5'-0" Depth

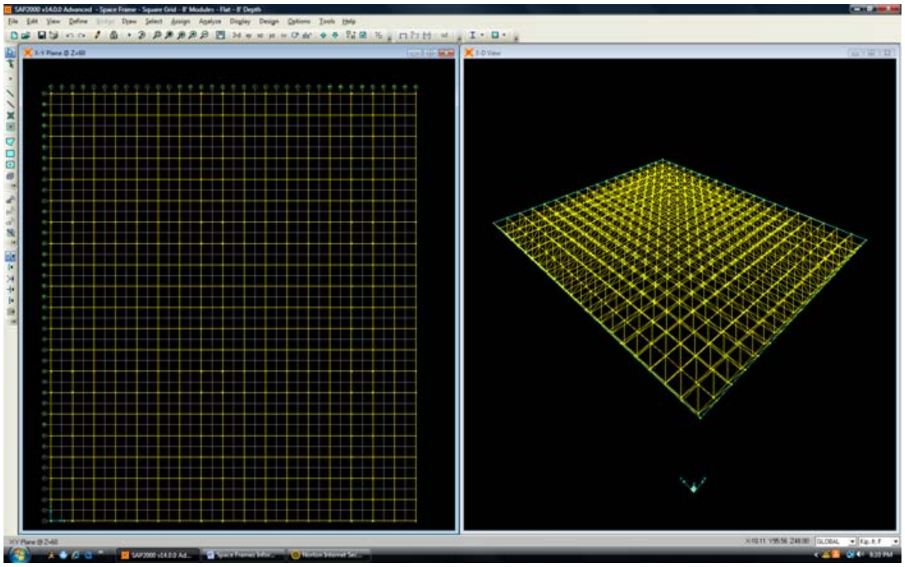


Figure 26 - Space Frame with 8’-0” Modules and 8’-0” Depth

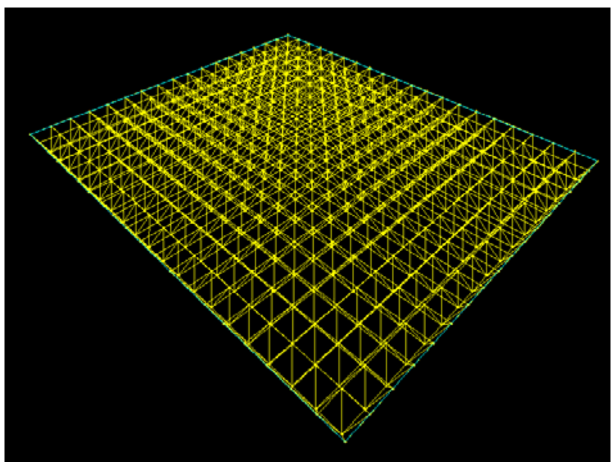


Figure 27 - Space Frame with 8’-0” Modules and 8’-0” Depth

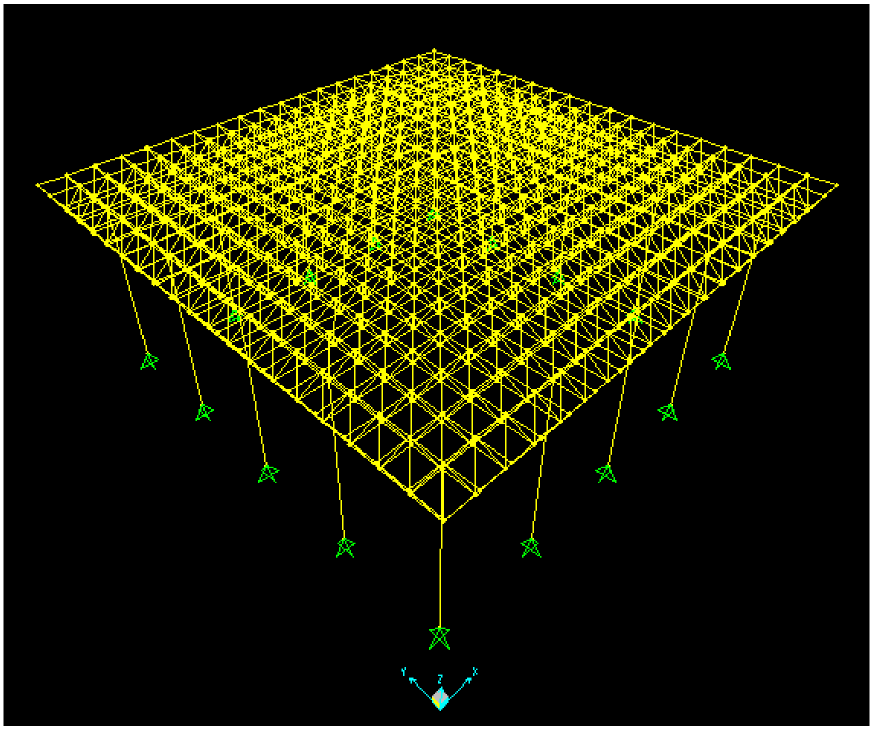


Figure 28 - Space Frame with 8'-0" Modules and 8'-0" Depth with Supporting Columns

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 ft)(10 lb/ft) = 231,040 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.

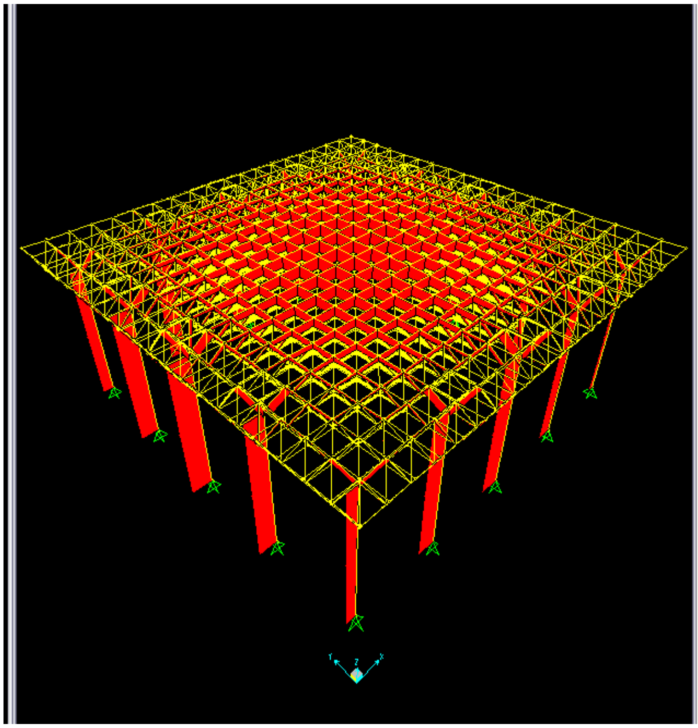


Figure 29 - Image from SAP2000 Showing Axial Forces
for Space Frame with 8’-0” Modules and 8’-0” Depth
(red indicates higher axial forces)

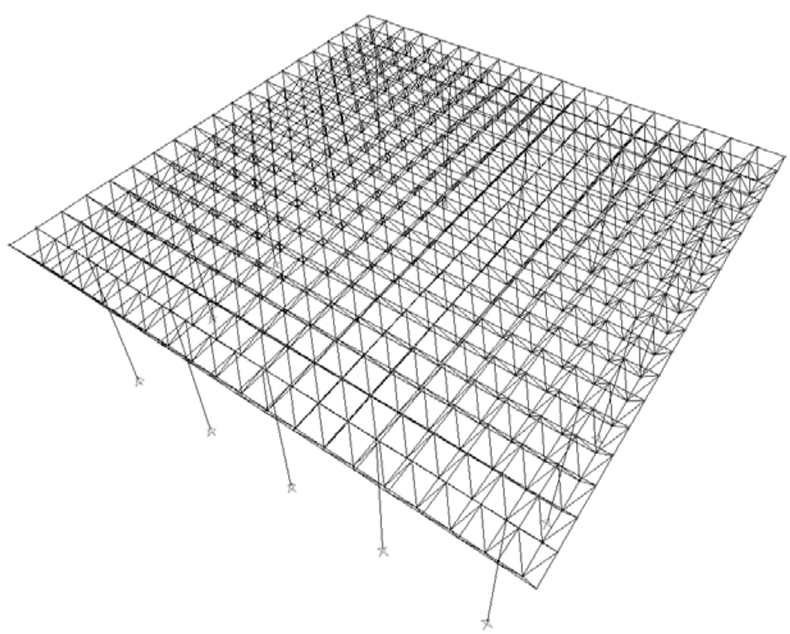


Figure 30 – Additional Image from SAP2000 of Space Frame with 8’-0” Modules and 8’-0” Depth

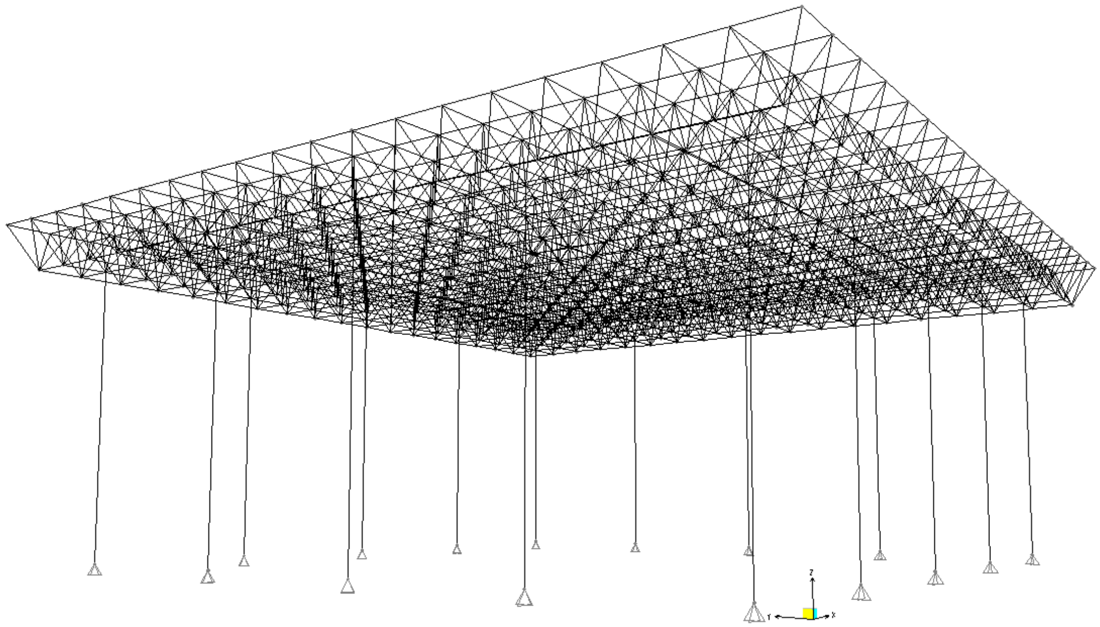


Figure 31 – Additional Image from SAP2000 of Space Frame with 8'-0" Modules and 8'-0" Depth

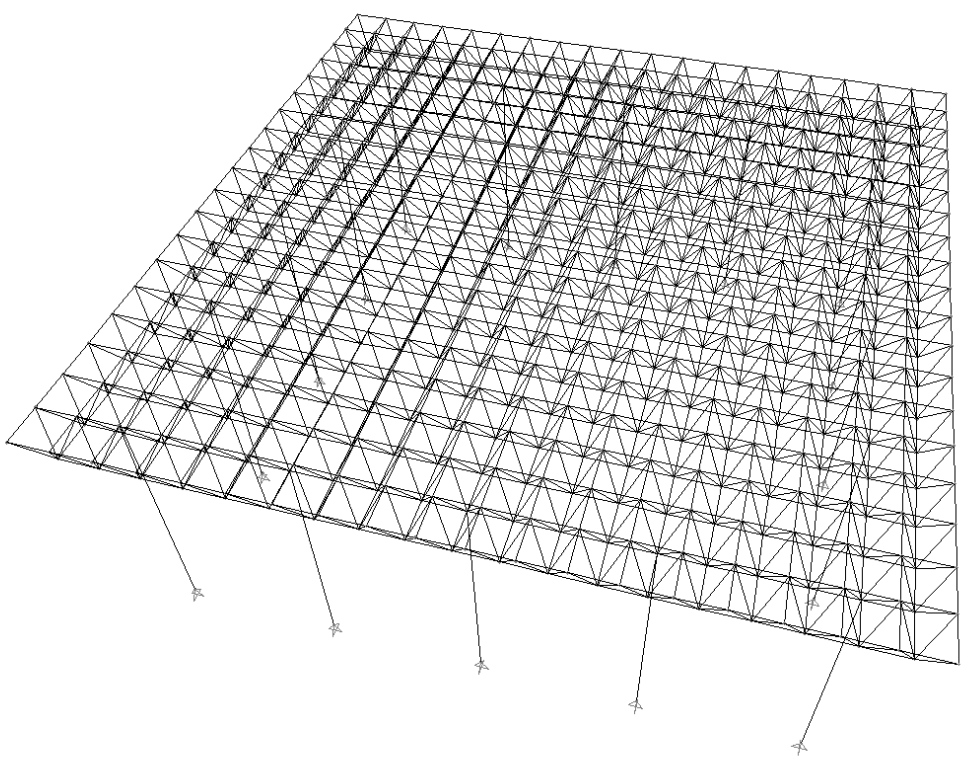


Figure 32 – Additional Image from SAP2000 of Space Frame with 8'-0" Modules and 8'-0" Depth

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.

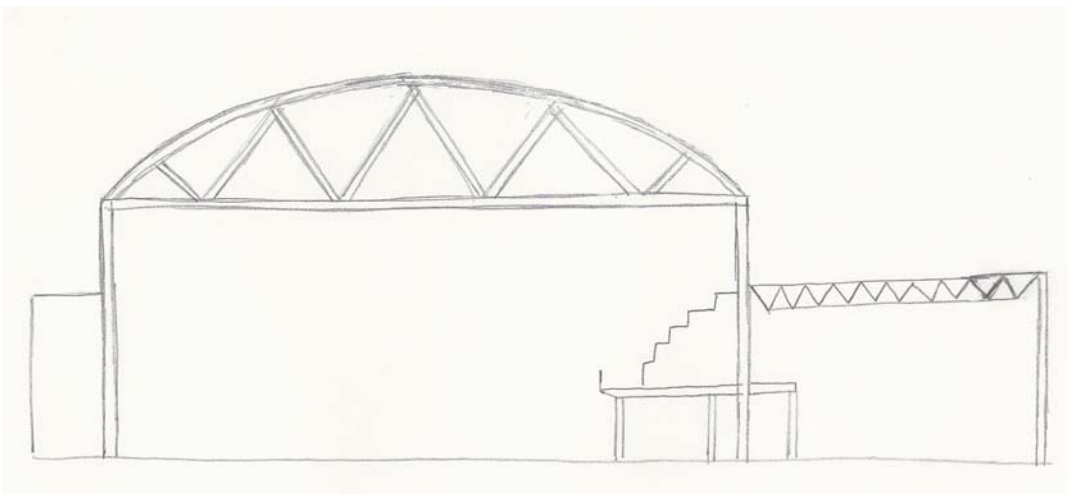


Figure 33 - Preliminary Sketch of Potential Wood Truss Configuration

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

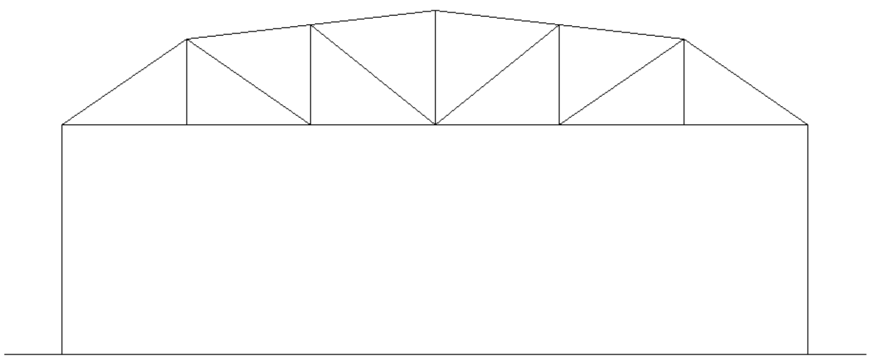


Figure 34 - Wood Truss Configuration with Top Chord Separated Into 6 Members

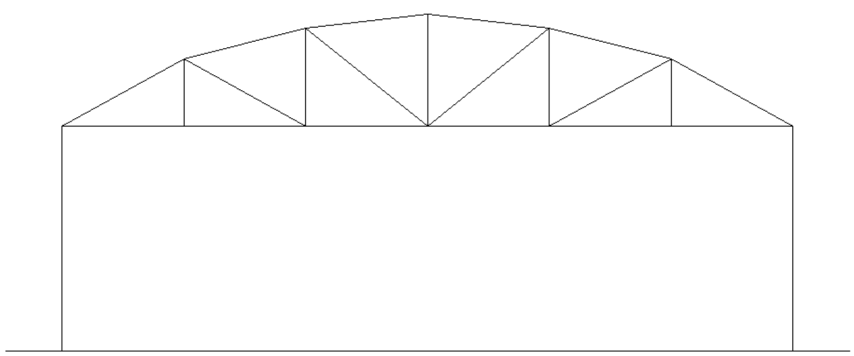


Figure 35 - Modified Wood Truss Configuration with Top Chord Separated Into 6 Members

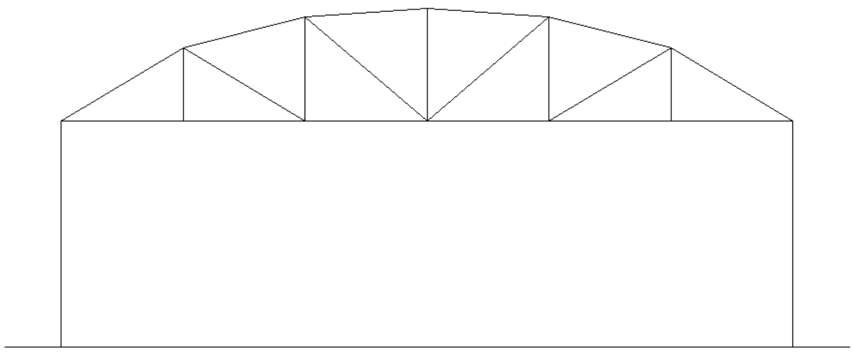


Figure 36 - Another Modified Wood Truss Configuration with Top Chord Separated Into 6 Members

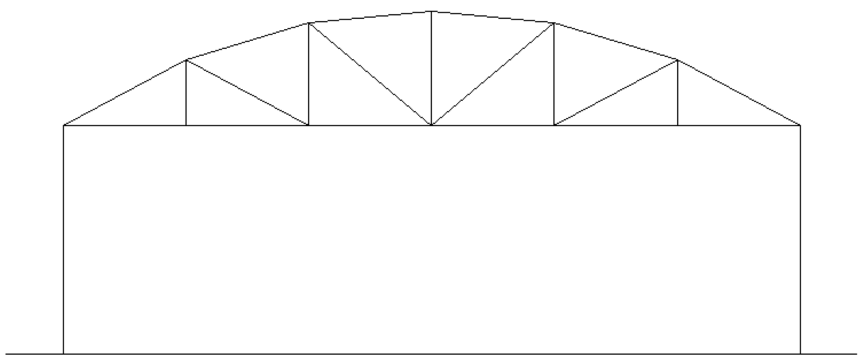


Figure 37 - Another Modified Wood Truss Configuration with Top Chord Separated Into 6 Members
(Curved Shape is Slightly Different than Previous Design)

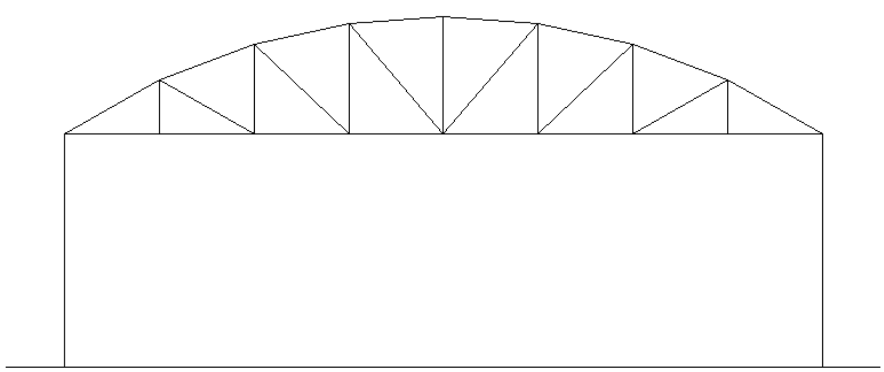


Figure 38 - Wood Truss Configuration with Top Chord Separated Into 8 Members

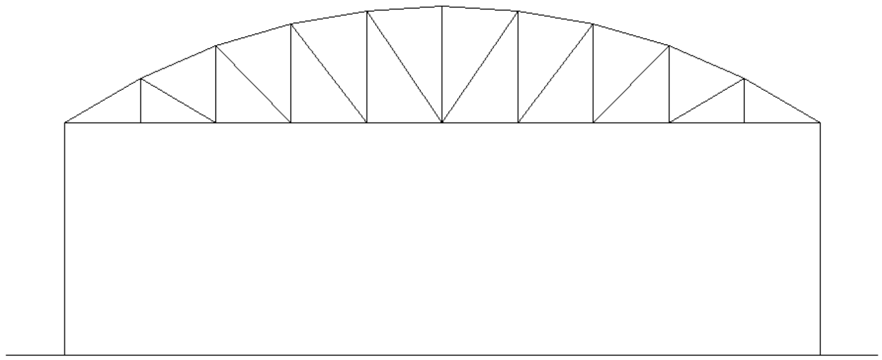


Figure 39 - Wood Truss Configuration with Top Chord Separated Into 10 Members

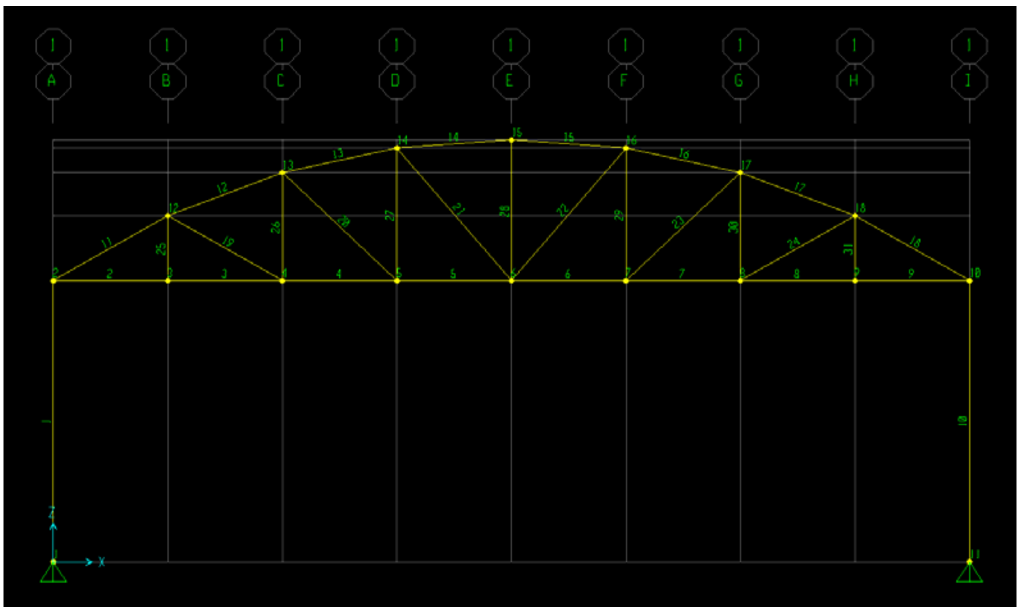


Figure 40 - SAP2000 Model of Glulam Truss Configuration with Top Chord Separated Into 8 Members

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 ¾" x 12 3/8" top chord, a 6 ¾" x 8 ¼" bottom chord, 6 ¾" x 6 7/8" web members, and 6 ¾" 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.

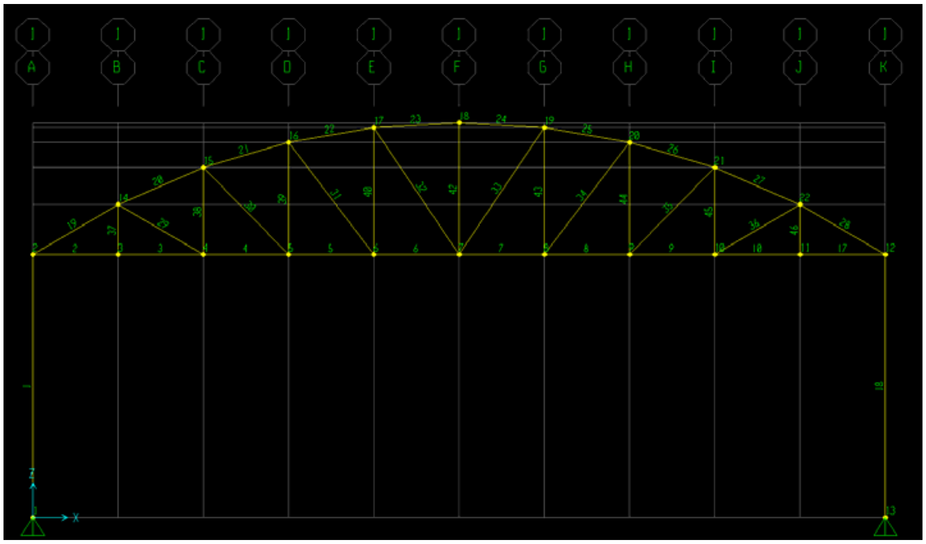


Figure 41 - SAP2000 Model of the Final Selected Glulam Truss Configuration

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

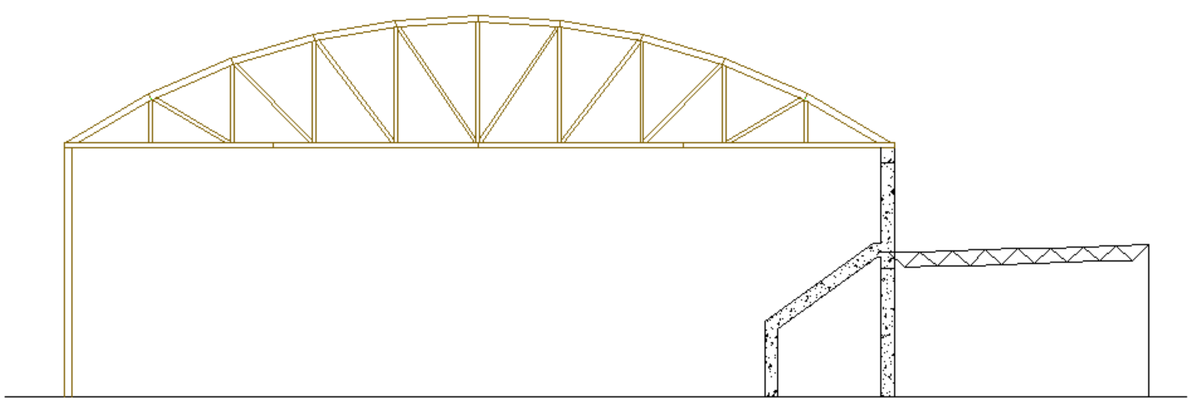


Figure 42 - Final Glulam Truss Configuration

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.

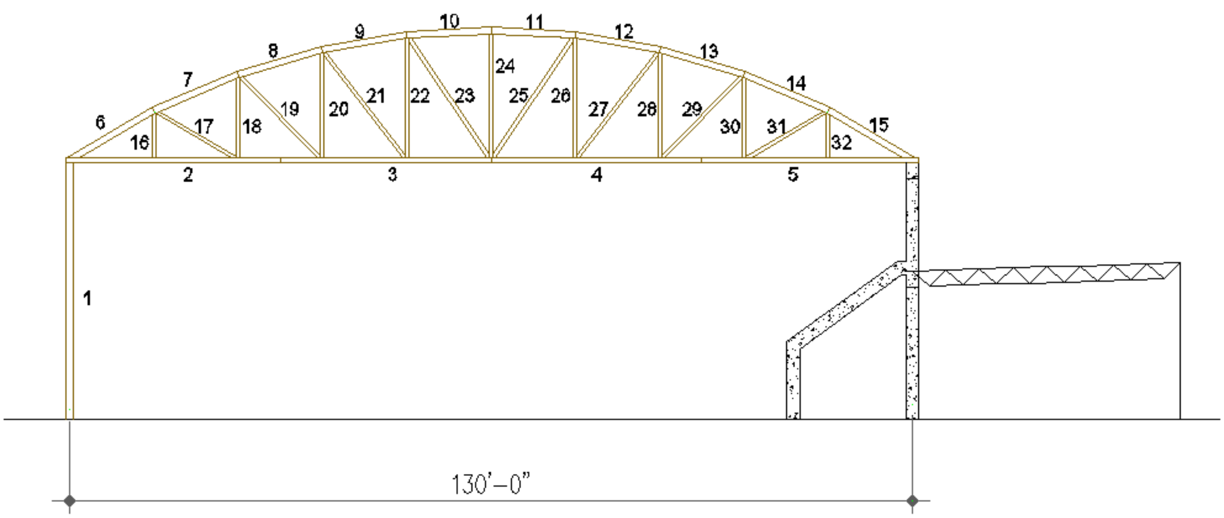


Figure 43 - Member Labels for Final Glulam Truss

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 weighted 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 originals 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 ($)
Alterntate 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.

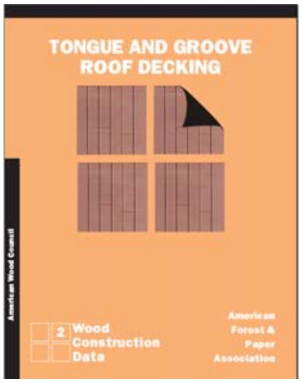


Figure 44 – WCD 2: Tongue and Groove Roof Decking

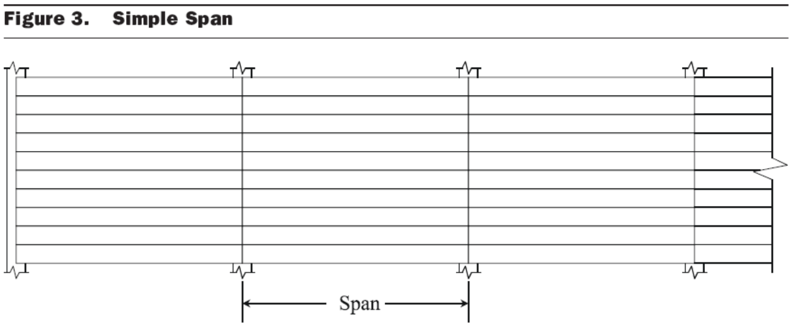


Figure 45 - Simple Span Layup (Image from WCD 2 – Tongue and Groove Roof Decking)

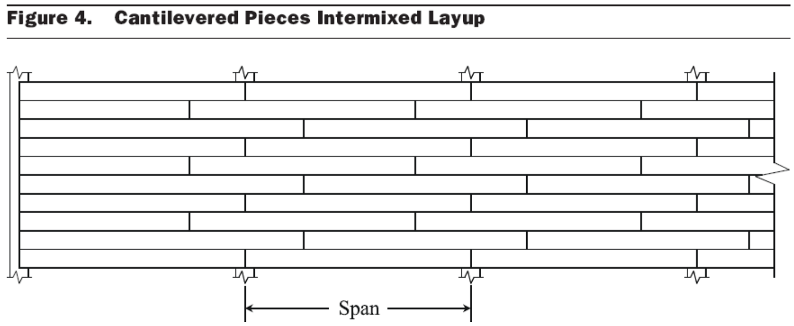


Figure 46 - Cantilevered Pieces Intermixed Layup (Image from WCD 2 – Tongue and Groove Roof Decking)

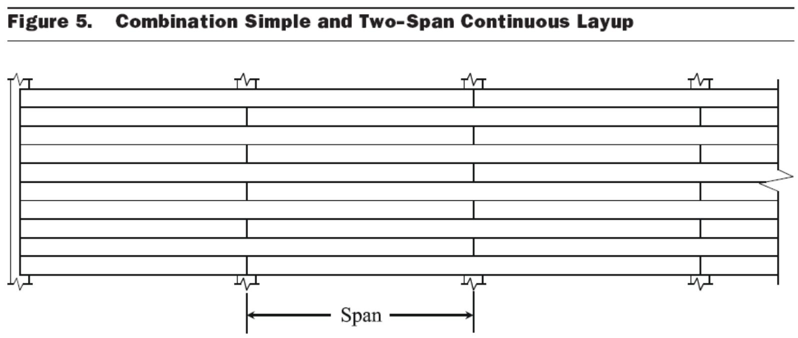


Figure 47 - Combination Simple and Two-Span Continuous Layup (Image from WCD 2 – Tongue and Groove Roof Decking)

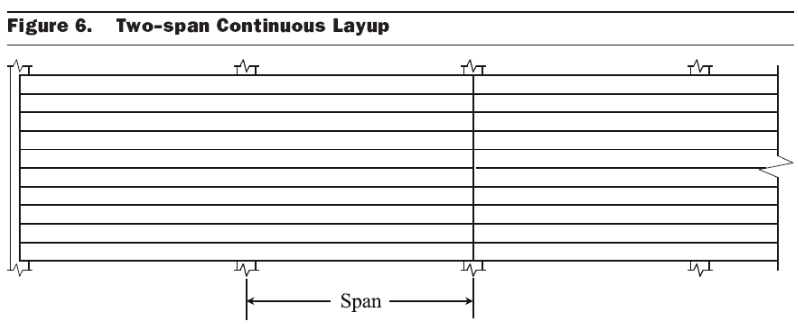


Figure 48 - Two-Span Continuous Layup (Image from WCD 2 – Tongue and Groove Roof Decking)

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 3/16" x 5 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 ¼" x 5 3/8". Therefore, it was determined that nominal three inch Southern Pine laminated decking would be used with an actual size of 2 ¼" x 5 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 ½" x 5 ½" 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 ¾" x 8 ¼" 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 is 6 ¾" 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 ¼" steel side plates. Nominal design values for ¾" bolts in double shear for a 6 ¾" thick Southern Pine glulam member with ¼" 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) ¾" 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) ¾" diameter bolts arranged in two rows.

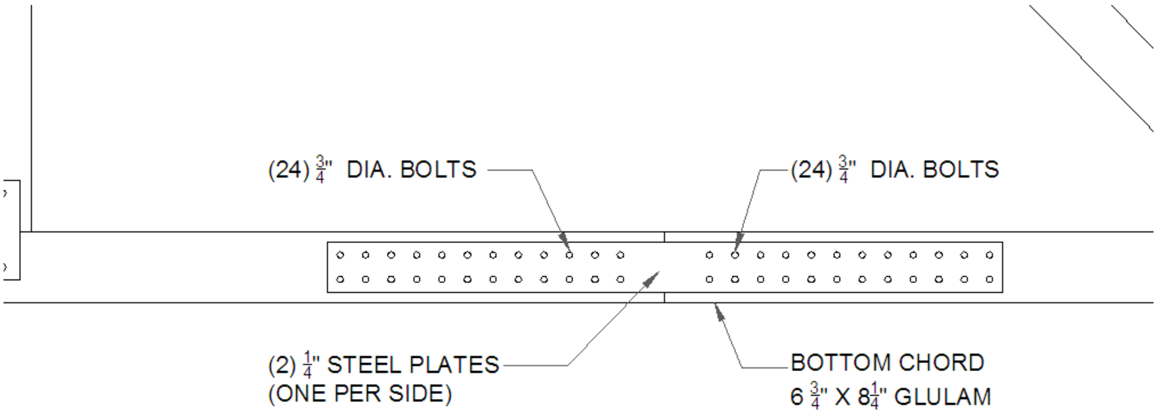


Figure 49 - Typical Bottom Chord Splice Connection

Top chord connections resulted in (28) ¾" 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.

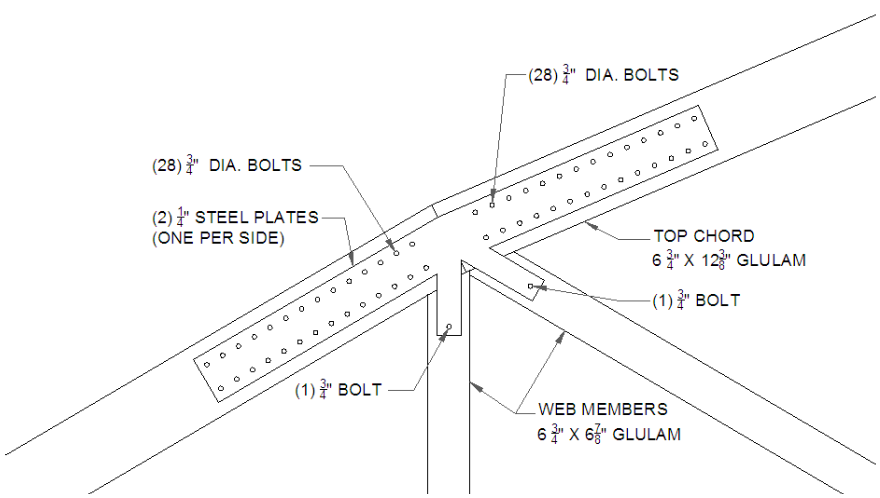


Figure 50 - Typical Top Chord Connection

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) ¾" 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.

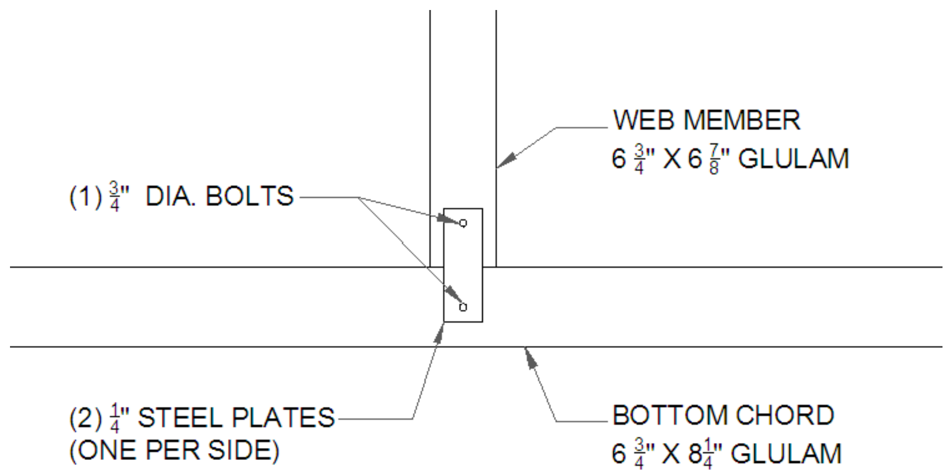


Figure 51 - Typical Vertical Web Member Connection to Bottom Chord

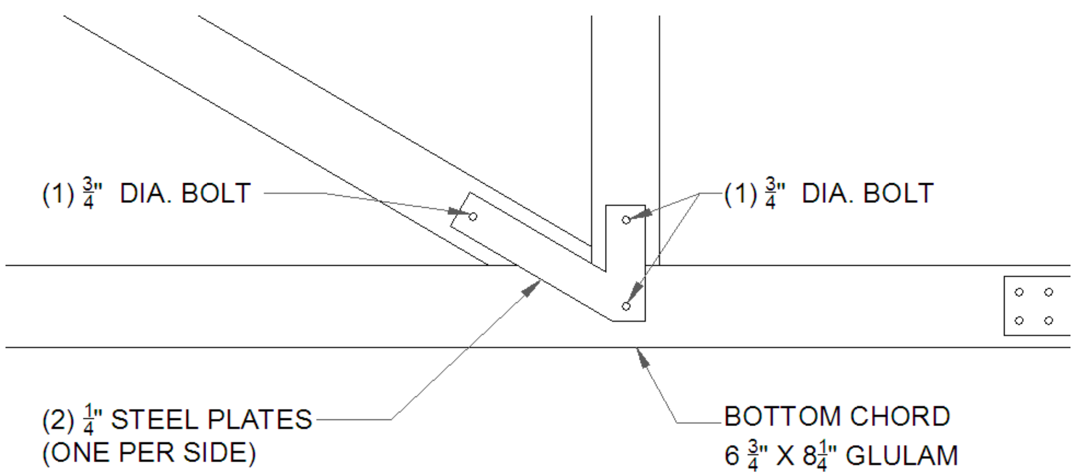


Figure 52 - Typical Web Member Connection to Bottom Chord

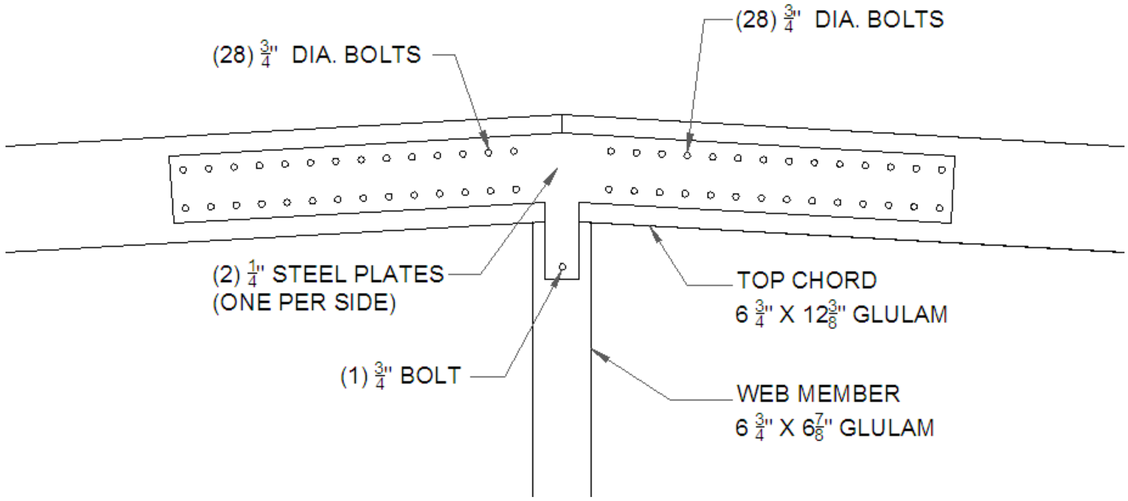


Figure 53 - Top Chord Connection

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.

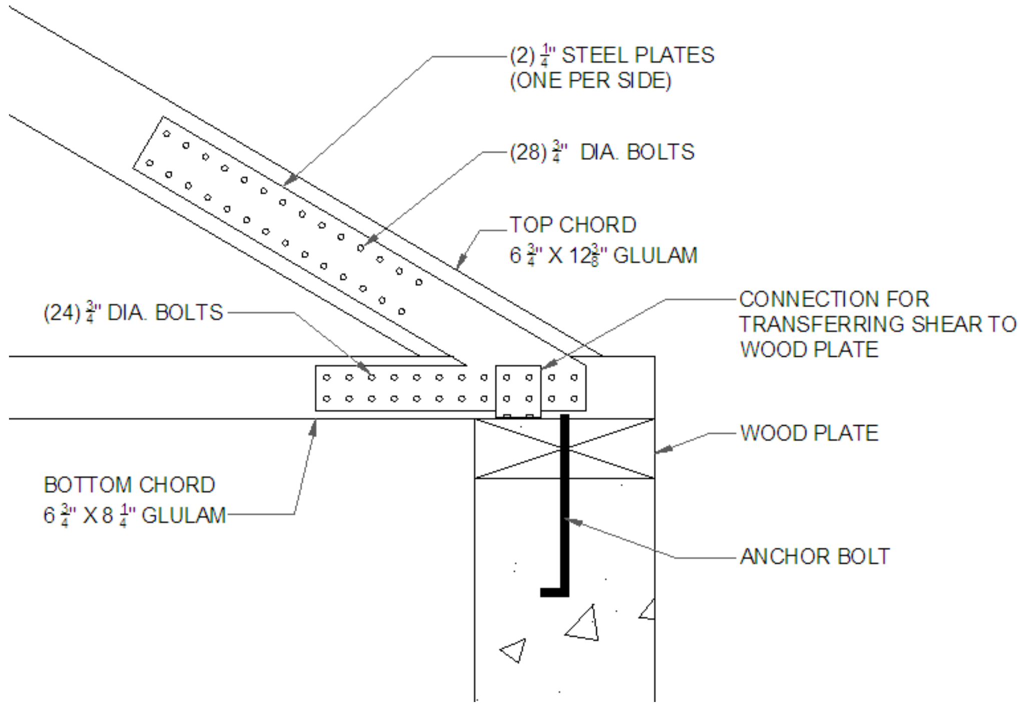


Figure 54 - Typical Connection of Glulam Truss to Concrete Beam or Column of Concrete Moment Frame at Column Line 2

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

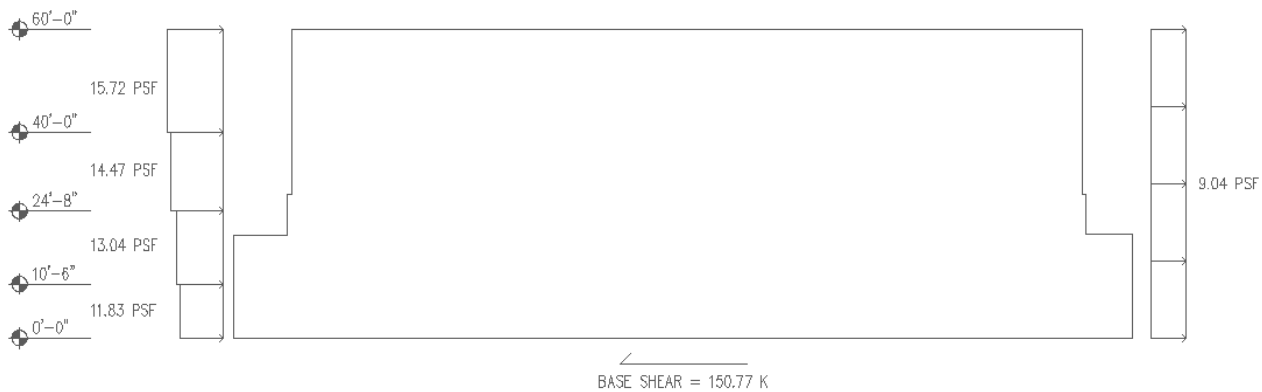


Figure 55 - "Building 1" Wind Loads – North/South

"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 moment frame at column line 2 and the moment frame at column line 4)

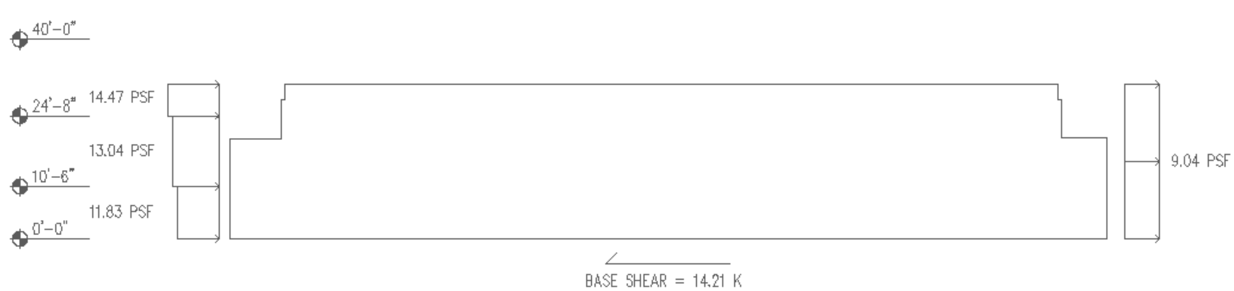


Figure 56 - "Building 4" Wind Loads – North/South

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)

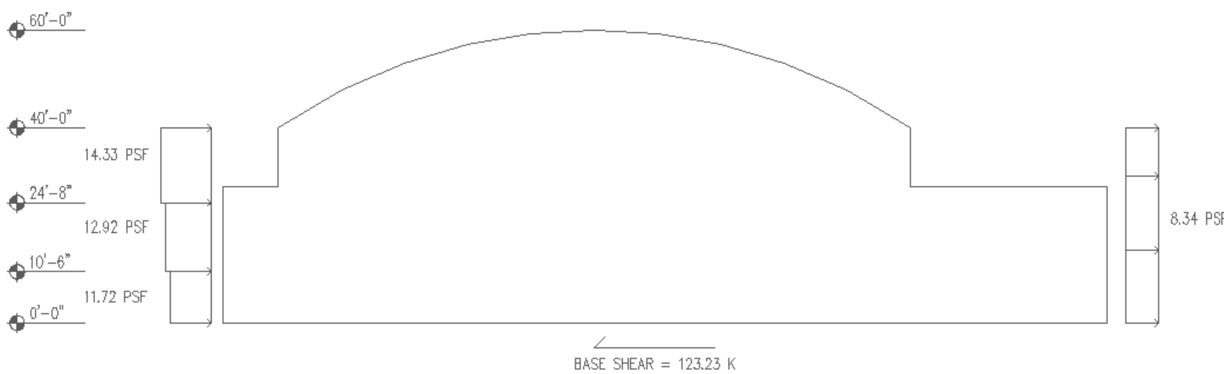


Figure 57 - Wind Loads on Entire Building (East/West)

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

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

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

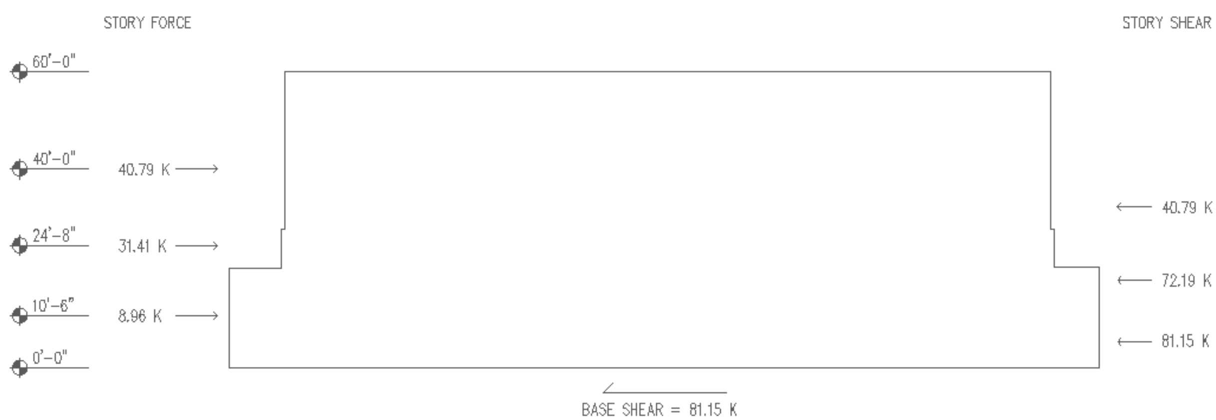


Figure 58 - "Building 1" Seismic Loads

"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**	**$w_xh_x^k$**	**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($w_xh_x^k$)=	**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 = CsW = (0.044353)(734.12 kips) = 32.56 kips

$C_{vx} = w_xh_x^k/\text{sum}(w_ih_i^k)$

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**	**$w_xh_x^k$**	**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
$sum(w_xh_x^k)$=	**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"

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

$C_{vx} = w_xh_x^k/sum(w_ih_i^k)$

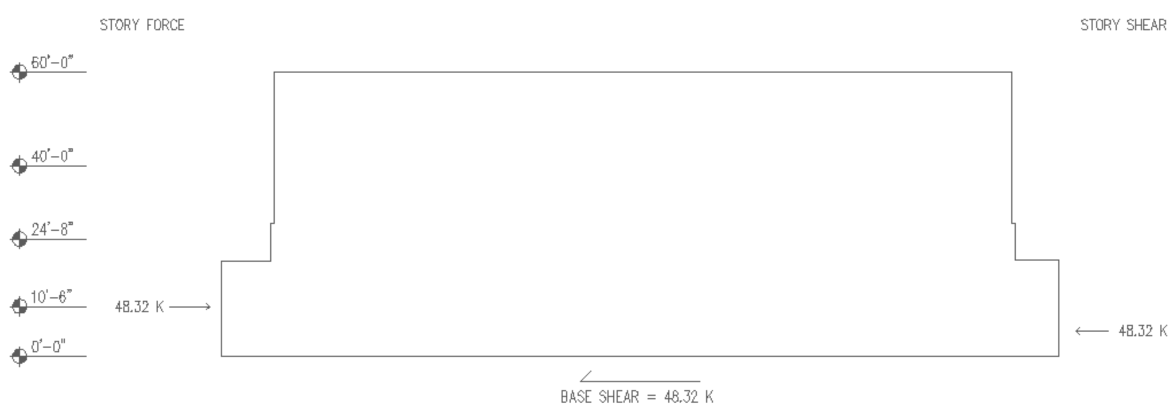


Figure 60 - "Building 3" Seismic Loads

"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"

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

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

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.

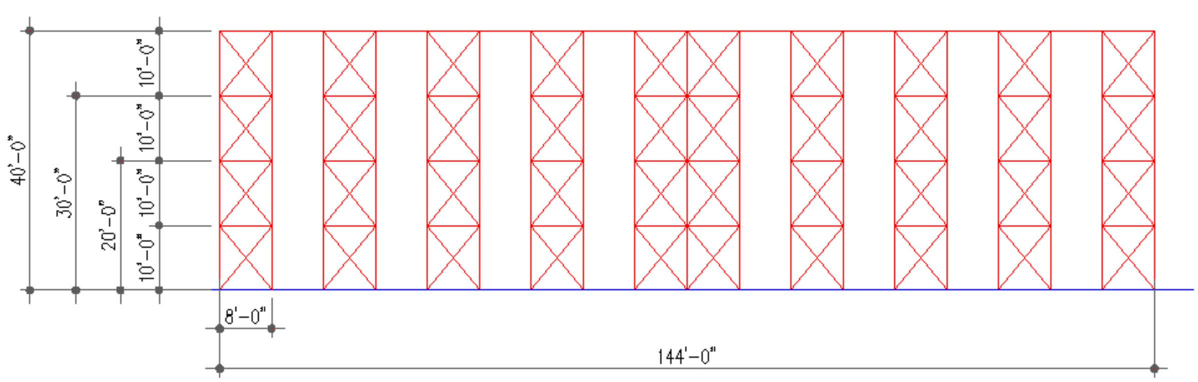


Figure 62 - Potential Column Line 1 Braced Frame Configuration with 4 X-Braces Vertically

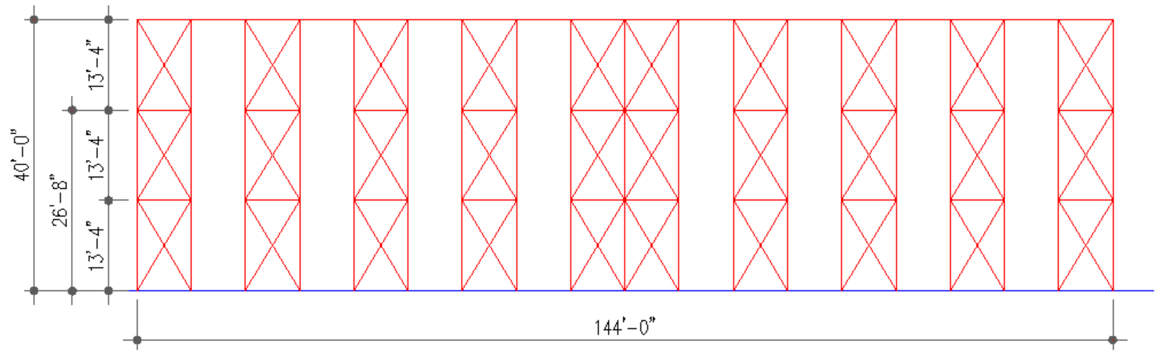


Figure 63 - Potential Column Line 1 Braced Frame Configuration with 3 X-Braces Vertically

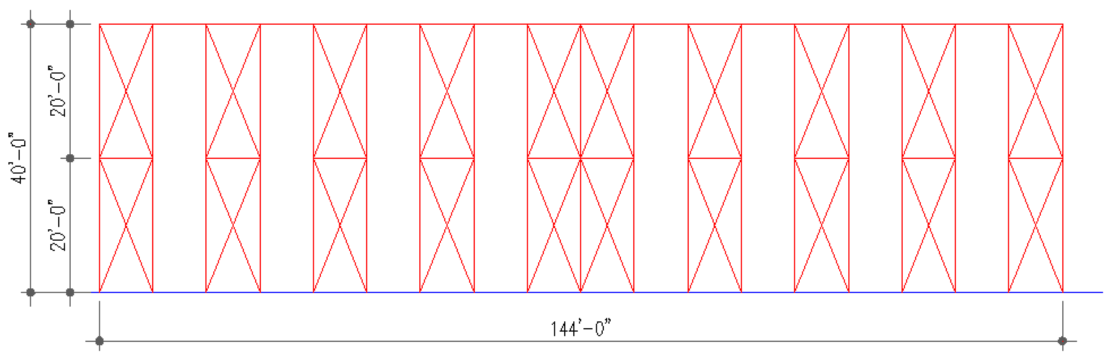


Figure 64 - Potential Column Line 1 Braced Frame Configuration with 3 X-Braces Vertically

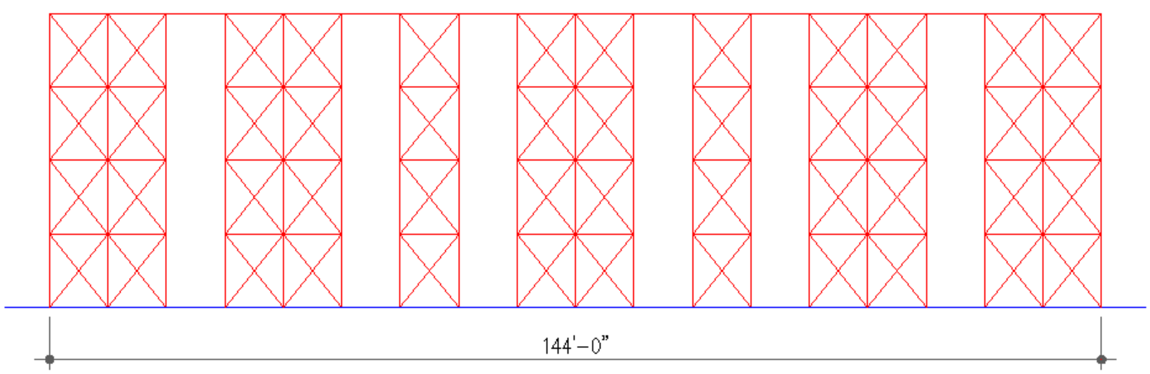


Figure 65 - Potential Column Line 1 Braced Frame Configuration with 4 X-Braces Vertically and Different Layout of Locations of Braced Frames

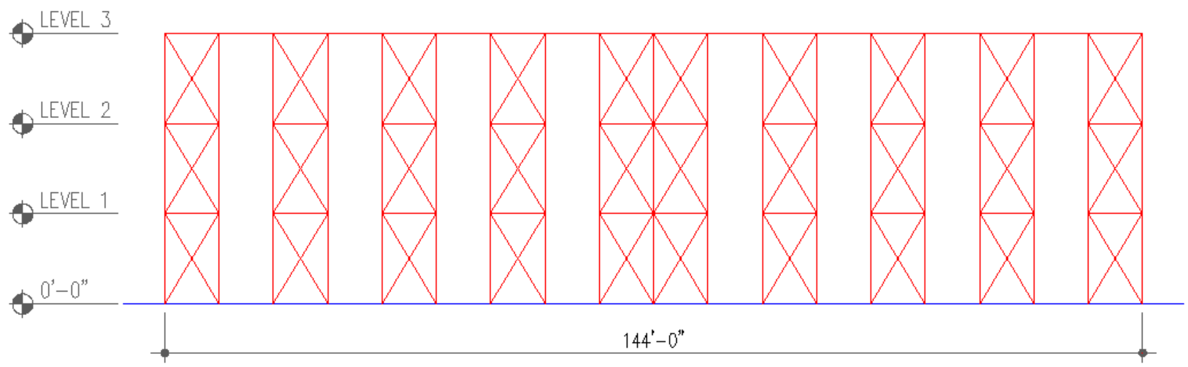


Figure 66 - Final Selected Braced Frame Configuration at Column Line 1

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 ½" 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.

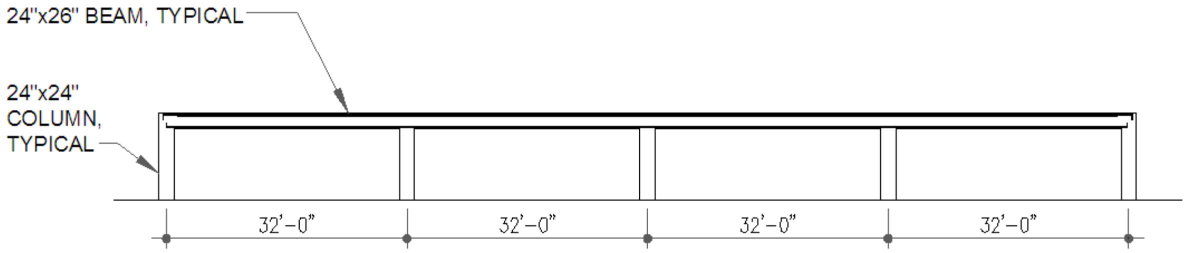


Figure 67 - Concrete Moment Frame at Column Line 1.8

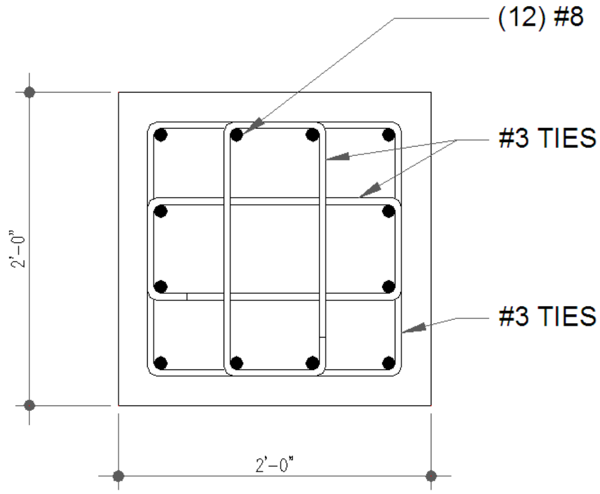


Figure 68 - Detail of Typical Reinforced Concrete Column at Column Line 1.8

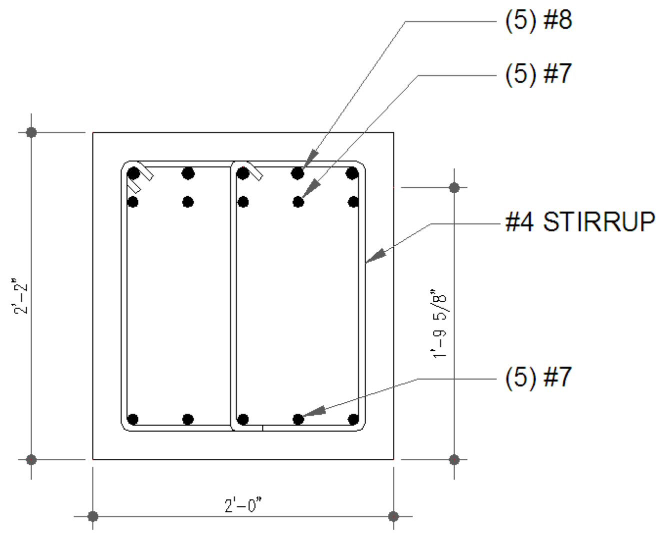


Figure 69 - Detail of Typical Reinforced Concrete Beam at Column Line 1.8

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.

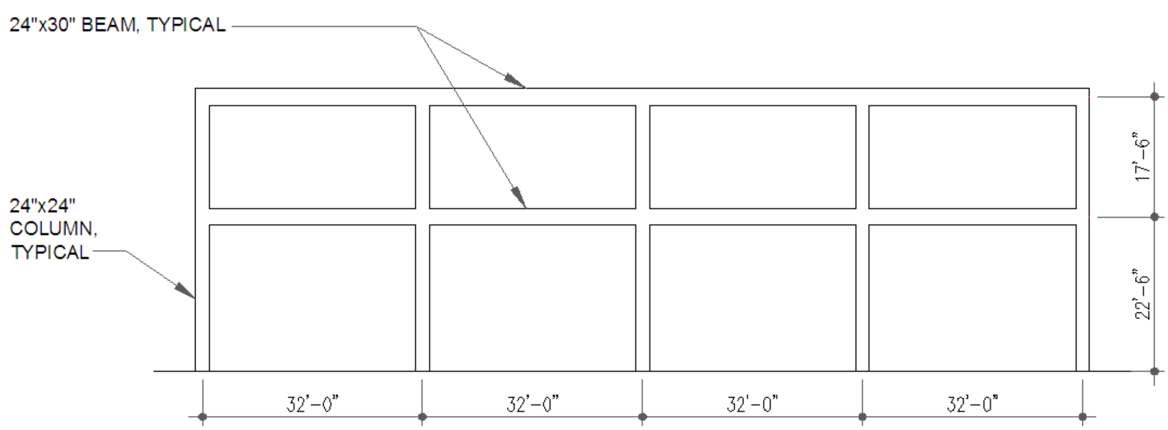


Figure 70 - Concrete Moment Frame at Column Line 2

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.

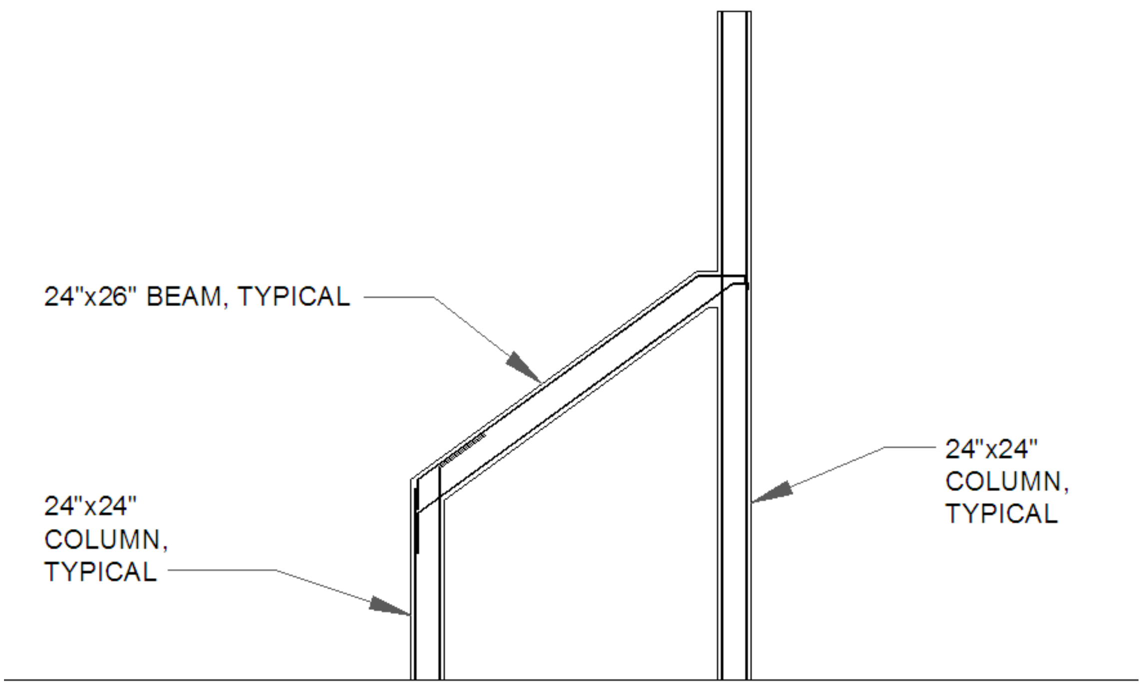


Figure 71 - Concrete Moment Frame in East/West Direction

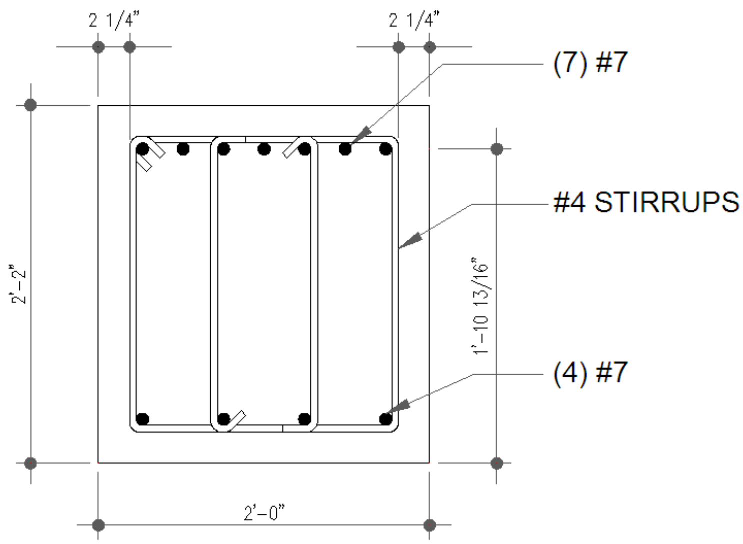


Figure 72 - Midspan Detail of Typical Sloped Reinforced Concrete Beam of Moment Frame in East/West Direction

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.

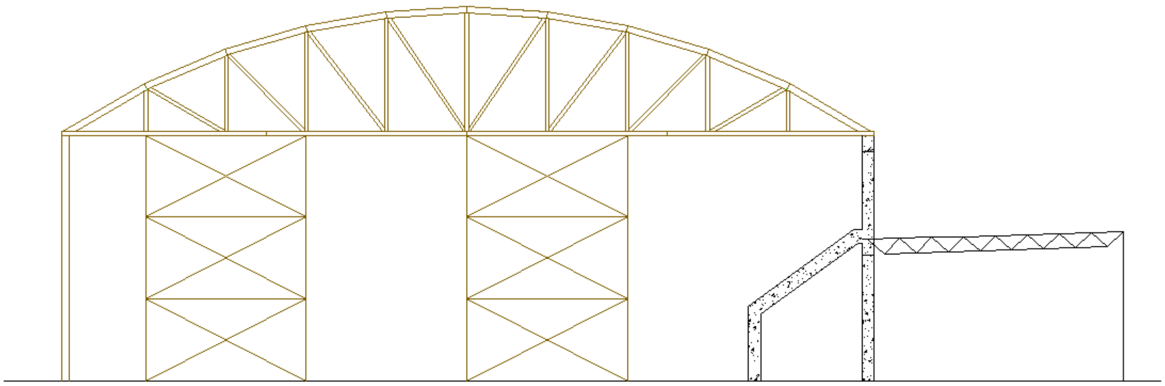


Figure 73 – Elevation Showing Wood Braced Frames in East/West Direction

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 ¾" 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.

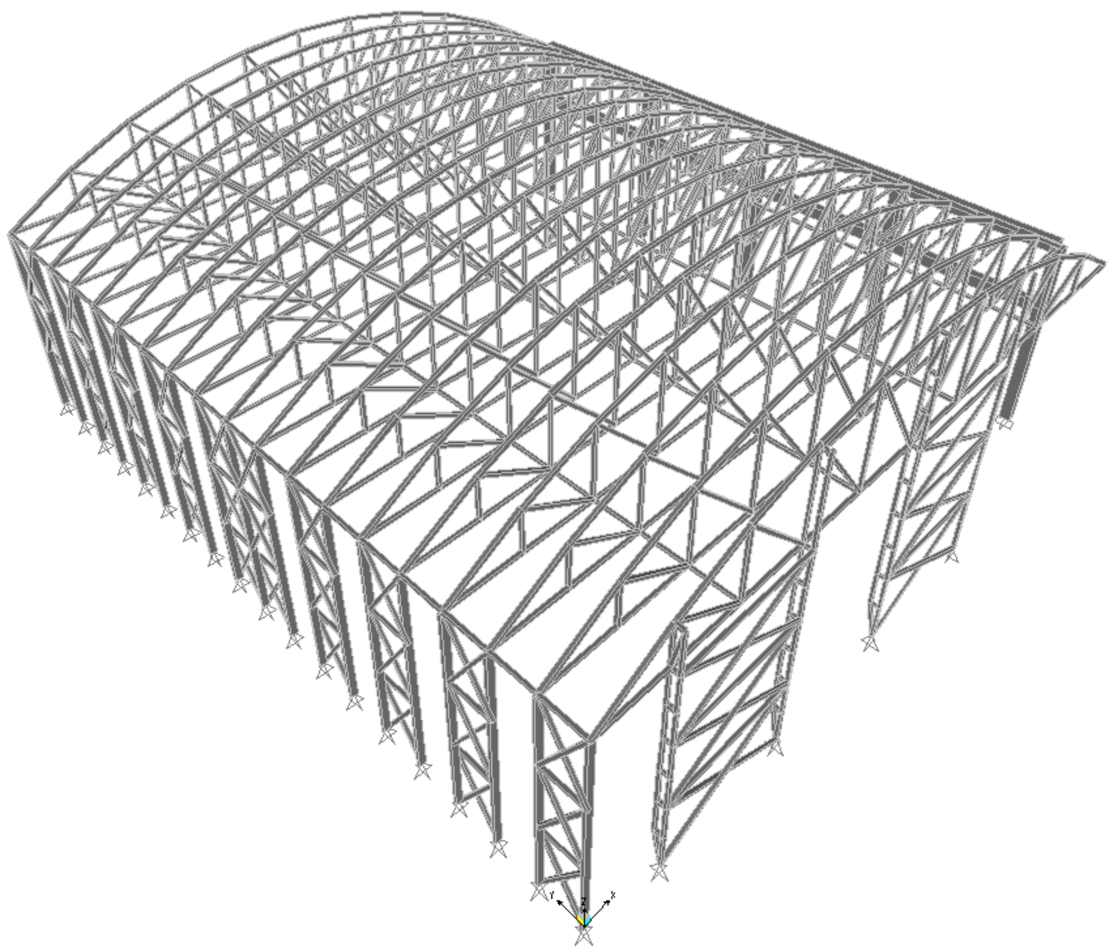
Overturning Check

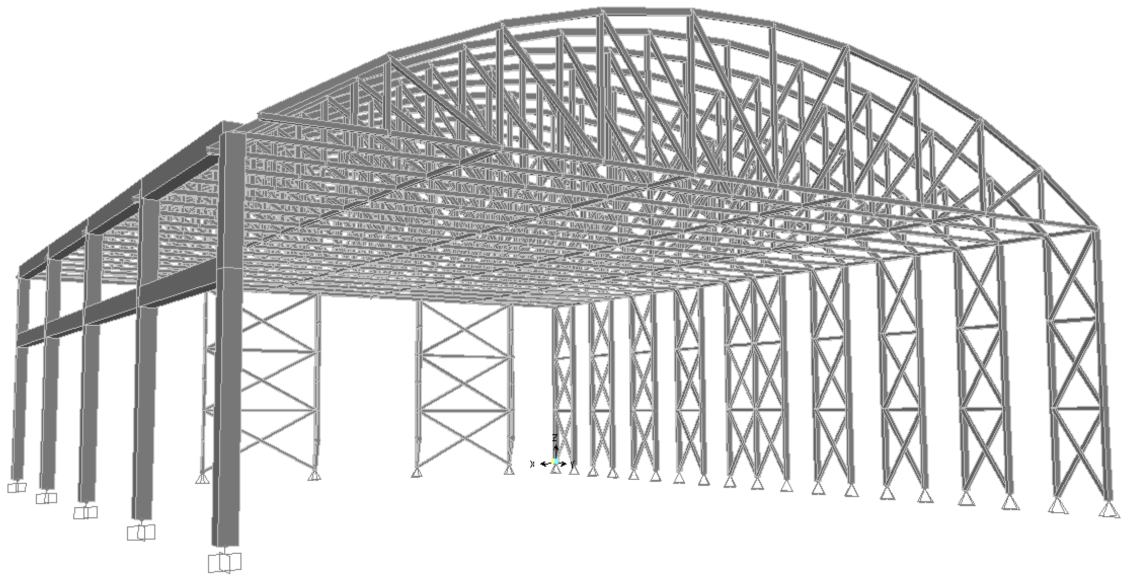
The lateral forces applied to the building cause overturning moments at the bases of the lateral force resisting frames. These overturning moments cause tensile, or uplift, forces in members of the lateral force resisting frames. Wind uplift forces also contribute to these forces. The dead weight of 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 it 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 room 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:

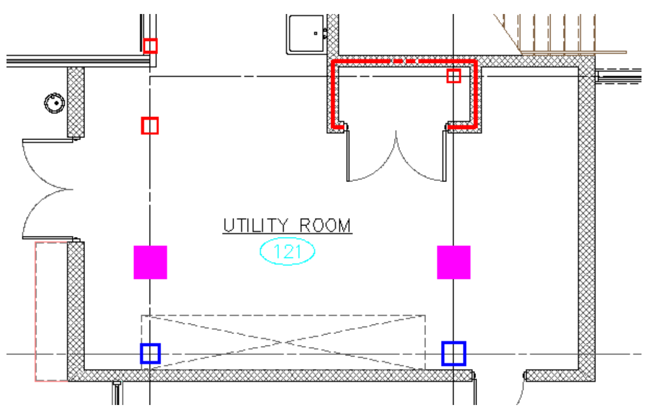


Figure 74 – Utility Room (Ground Level)

Solution:

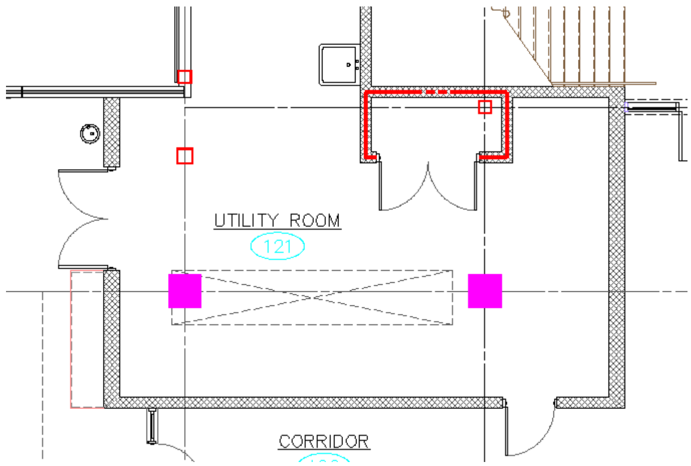


Figure 75 – Utility Room (Ground Level)

The new column locations in the utility room caused little or no problems. The original steel columns, shown in blue if 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:

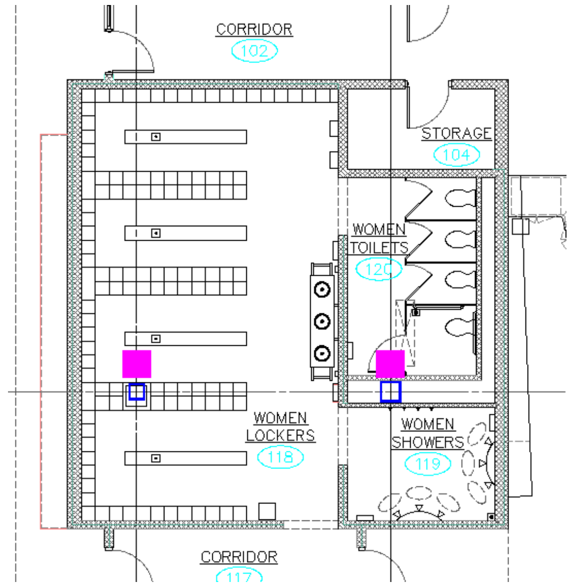


Figure 76 – Women’s Locker Room (Ground Level)

Solution:

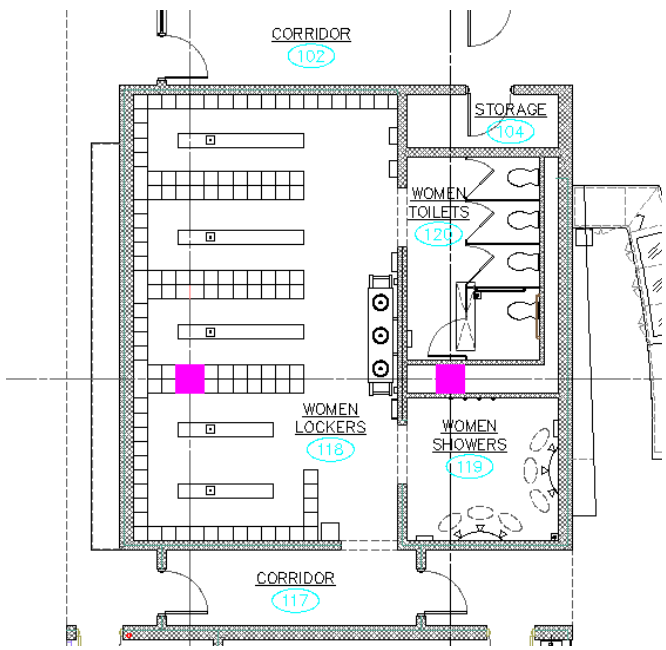


Figure 77 – Women’s Locker Room (Ground Level)

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:

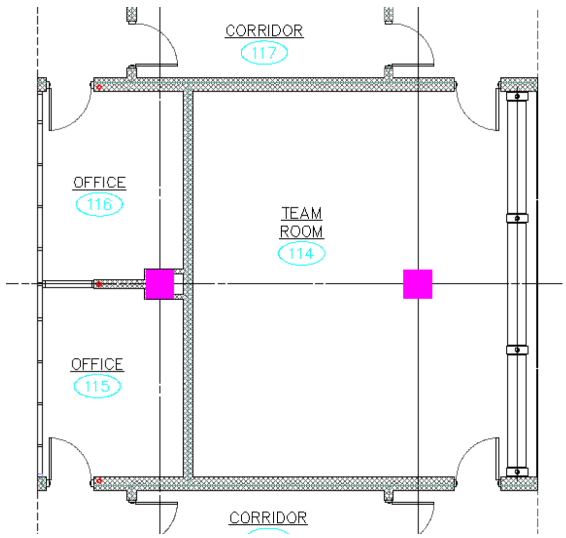


Figure 78 – Team Room and Offices Room (Ground Level)

Solution:

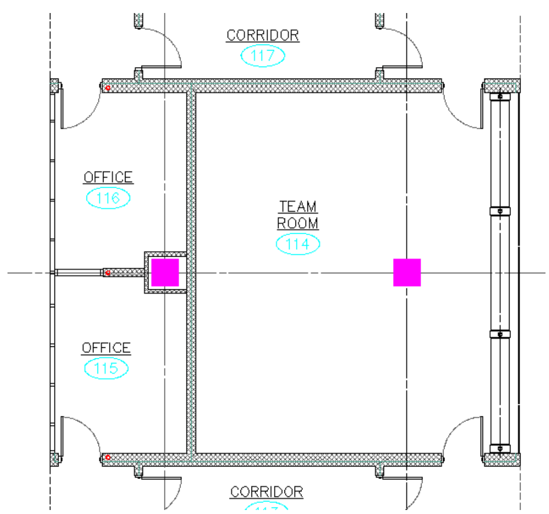


Figure 79 – Team Room and Offices Room (Ground Level)

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:

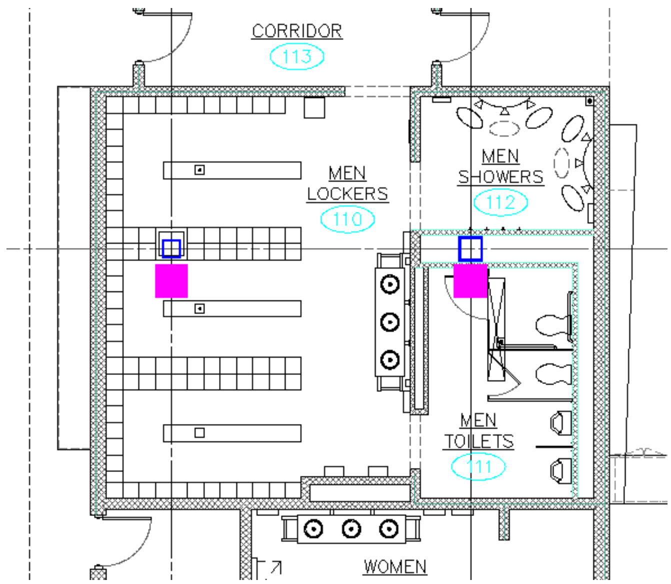


Figure 80 – Men's Locker Room (Ground Level)

Solution:

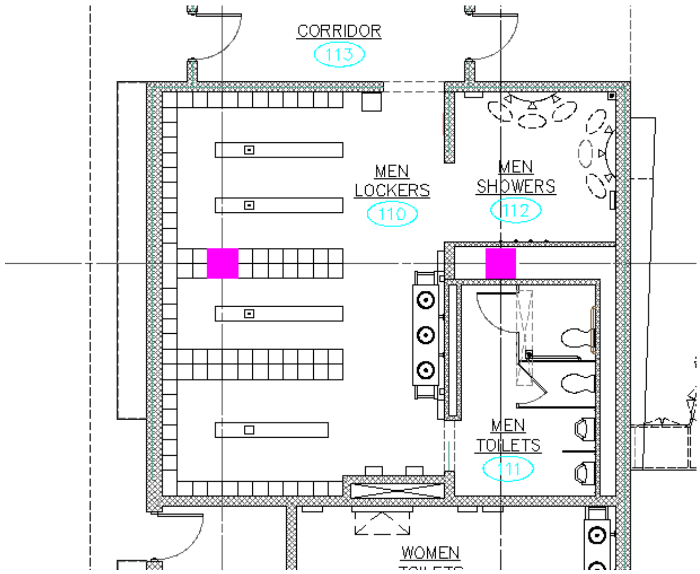


Figure 81 – Men's Locker Room (Ground Level)

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:

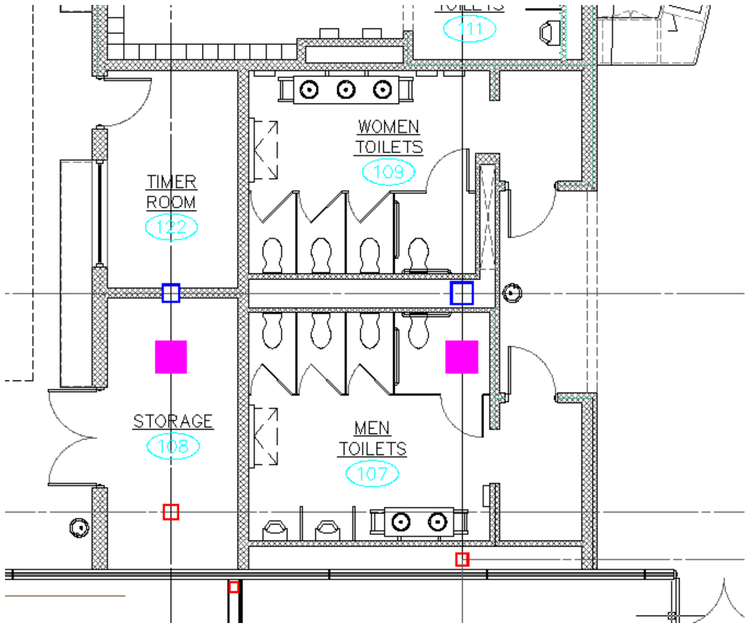


Figure 82 – Men's Restroom, Women's Restroom, Timer Room and Storage Room (Ground Level)

Solution:

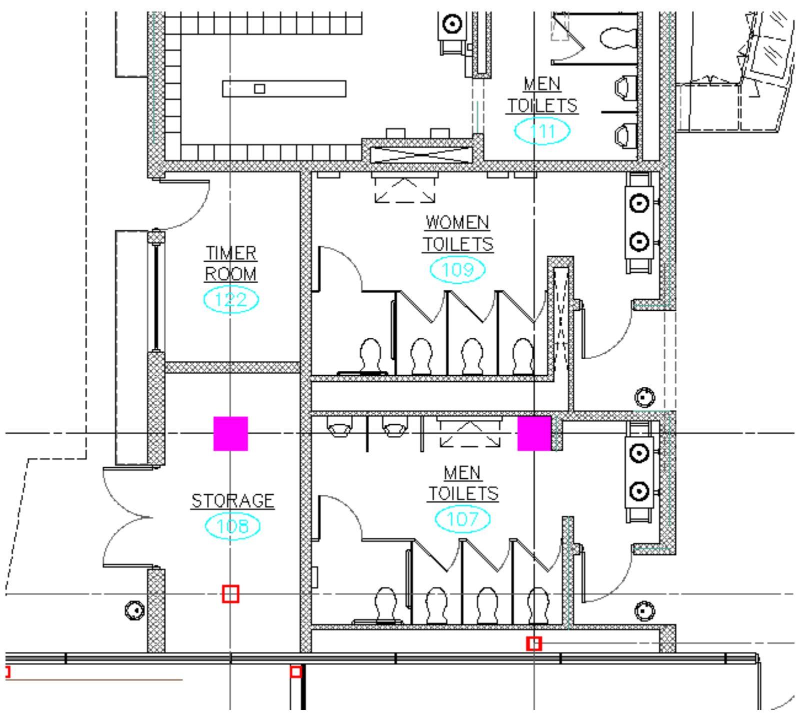


Figure 83 – Men's Restroom, Women's Restroom, Timer Room and Storage Room (Ground Level)

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:

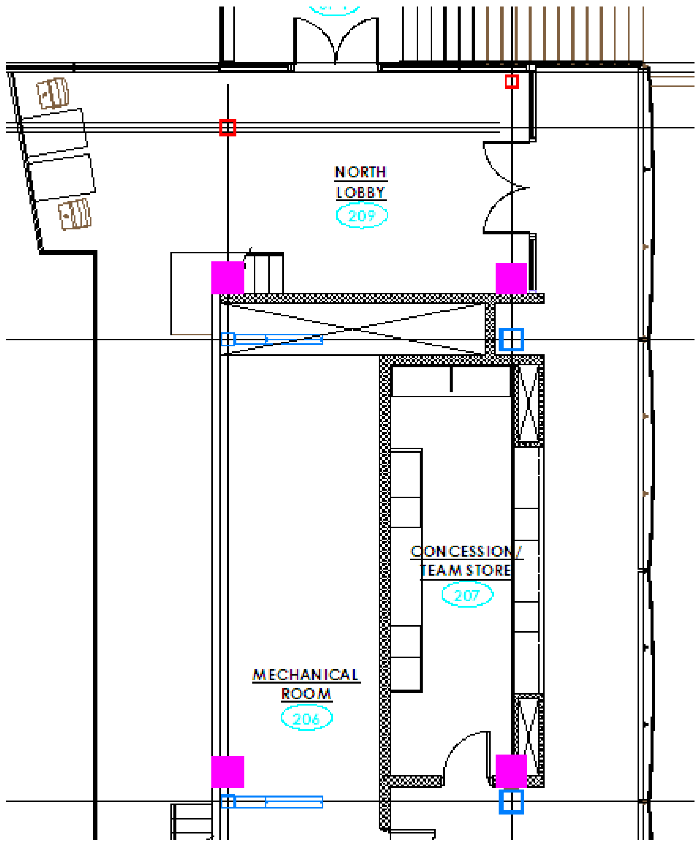


Figure 84 – Concession/Team Store and Mechanical Room (Concourse Level)

Solution:

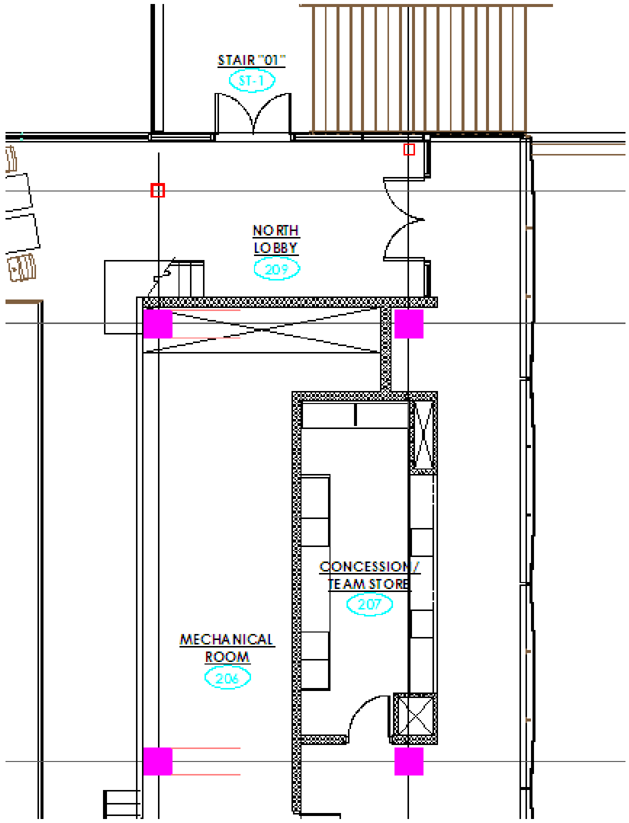


Figure 85 – Concession/Team Store and Mechanical Room (Concourse Level)

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:

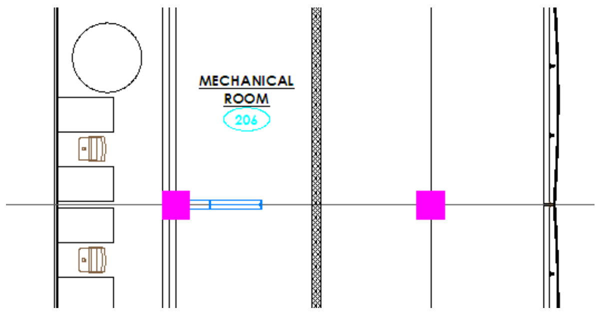


Figure 86 –Mechanical Room (Concourse Level)

Solution:

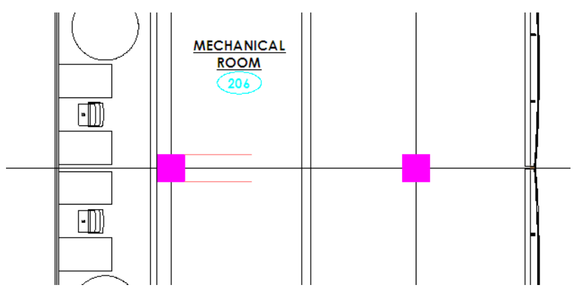


Figure 87 –Mechanical Room (Concourse Level)

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:

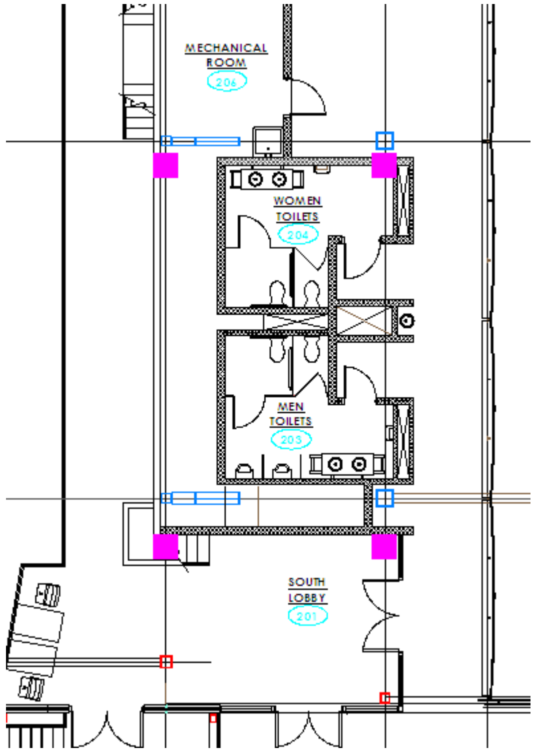


Figure 88 – Women's Restroom, Men's Restroom, Mechanical Room, and South Lobby (Concourse Level)

Solution:

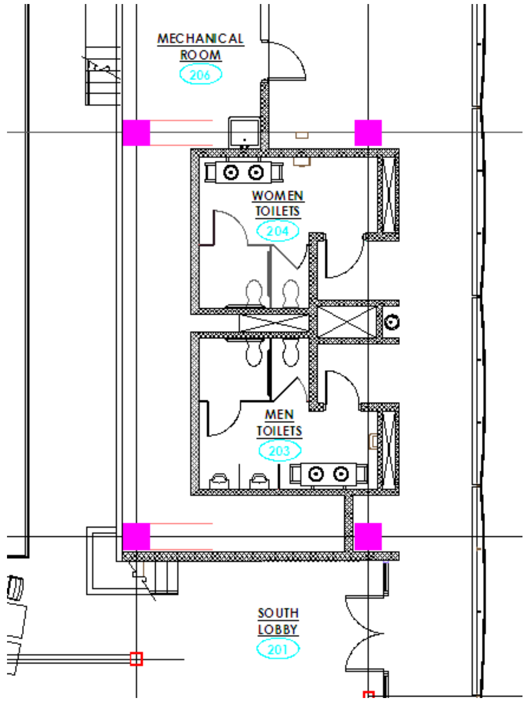


Figure 89 – Women's Restroom, Men's Restroom, Mechanical Room, and South Lobby (Concourse Level)

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 configuration 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):

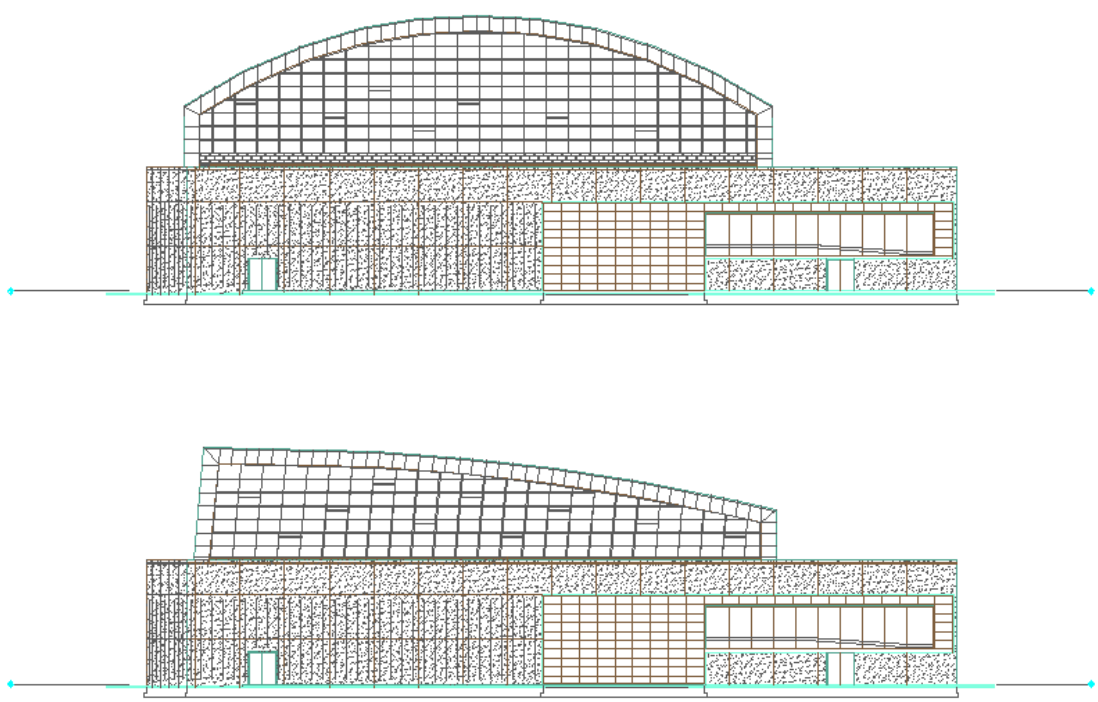


Figure 90 - Elevation with New Roof Shape and Facade (above); Elevation with Original Roof Shape and Façade (below)

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.

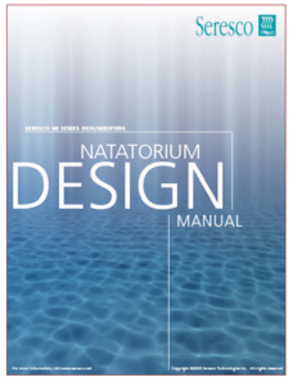


Figure 91 – The Natatorium Design Manual by Seresco Technologies, Inc.

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.

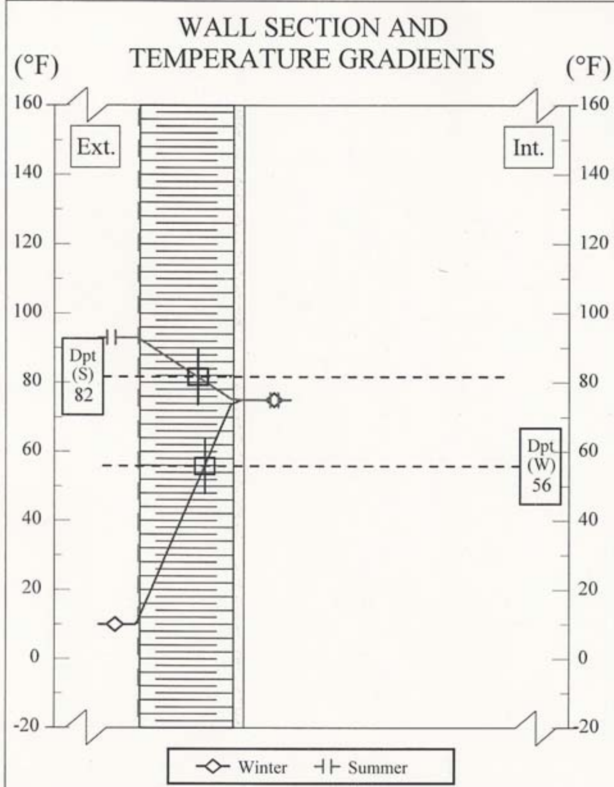
Roof System #4:

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.

R VALUE ANALYSIS

The Heat, Air and Moisture Building Science Toolbox - V.1B-E/U (11)



PROJECT

Name	Thesis Wall 1
Number	1
City	Philadelphia, PA
Date	3/4/2010
Analysis by:	Jason Kukorlo
Wall Type	
Option	

CLIMATIC CONDITIONS

	Winter		Summer	
	Int.	Ext.	Int.	Ext.
Temp (°F)	75	10	75	93
RH (%)	50	66	50	68
DPT (°F)	56	2	56	82

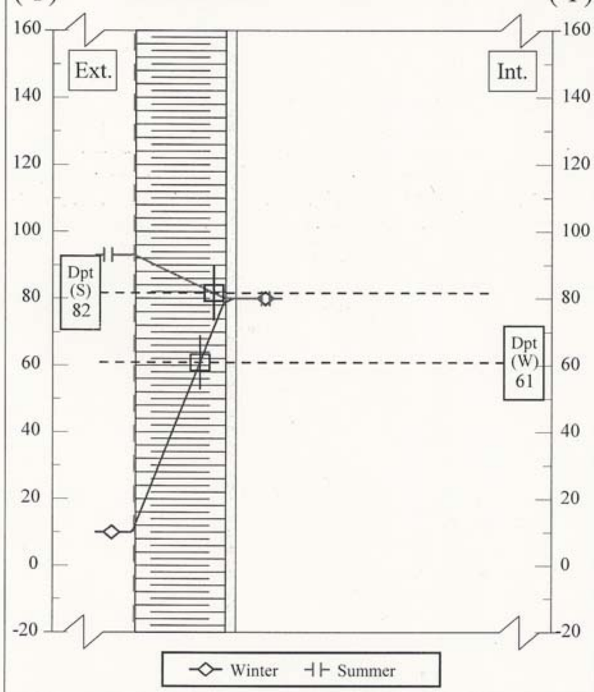
PENNSYLVANIA STATE UNIVERSITY
104 ENGINEERING, UNIT A
UNIVERSITY PARK, PA, USA, 16802

	Generic Material	Manufacturer	Model No.	Thick (in.)	RVal (R)	W.Temp. (°F)	S.Temp. (°F)
1	membrane (#1), .080 in.	No Recor...	Generic...	0.08	0.07	10.6	93.1
2	rigid ins.,(extru.), 4-1/2 in.	No Recor...	Generic...	4.50	23.12	73.7	75.4
3	gypsum bd., 1/2 in., (#1)	No Recor...	Generic...	0.50	0.46	75.0	75.0
				5.08	23.65	(10.4)	(93.2)

R VALUE ANALYSIS

The Heat, Air and Moisture Building Science Toolbox - V.1B-E/U (11)

WALL SECTION AND TEMPERATURE GRADIENTS



PROJECT

Name	Thesis Wall 1
Number	1
City	Philadelphia, PA
Date	3/4/2010
Analysis by:	Jason Kukorlo
Wall Type	
Option	

CLIMATIC CONDITIONS

	Winter		Summer	
	Int.	Ext.	Int.	Ext.
Temp (°F)	80	10	80	93
RH (%)	50	66	50	68
DPT (°F)	61	2	61	82

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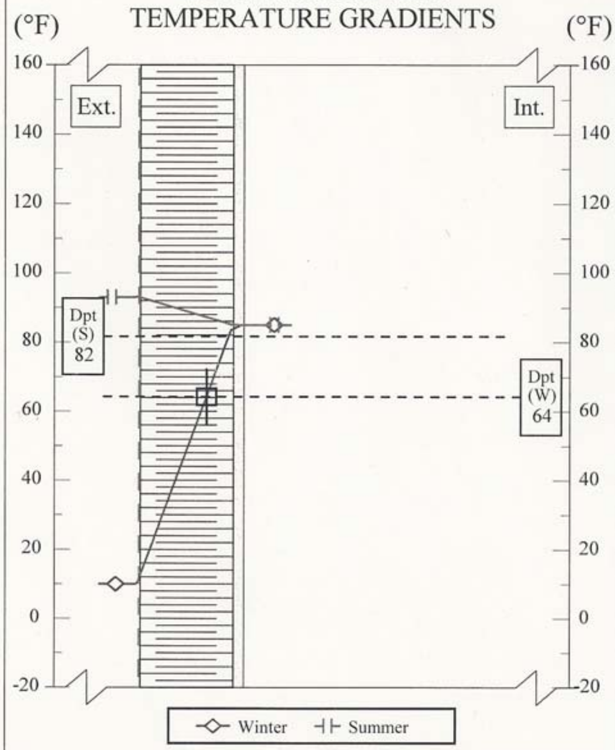
	Generic Material	Manufacturer	Model No.	Thick (in.)	RVal (R)	W.Temp. (°F)	S.Temp. (°F)
1	membrane (#1), .080 in.	No Recor...	Generic...	0.08	0.07	10.6	93.2
2	rigid ins.,(extru.), 4-1/2 in.	No Recor...	Generic...	4.50	23.12	78.6	80.3
3	gypsum bd., 1/2 in., (#1)	No Recor...	Generic...	0.50	0.46	80.0	80.0
				5.08	23.65	(10.4)	(93.2)

R VALUE ANALYSIS

The Heat, Air and Moisture Building Science Toolbox - V.1B-E/U (11)

WALL SECTION AND TEMPERATURE GRADIENTS



PROJECT

Name	Thesis Wall 1
Number	1
City	Philadelphia, PA
Date	3/4/2010
Analysis by:	Jason Kukorlo
Wall Type	
Option	

CLIMATIC CONDITIONS

	Winter		Summer	
	Int.	Ext.	Int.	Ext.
Temp (°F)	85	10	85	93
RH (%)	50	66	50	68
DPT (°F)	64	2	64	82

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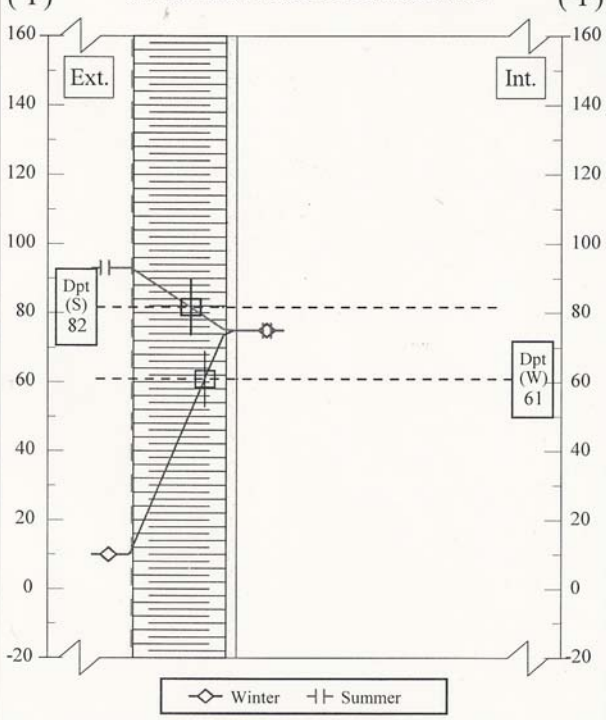
	Generic Material	Manufacturer	Model No.	Thick (in.)	RVal (R)	W.Temp. (°F)	S.Temp. (°F)
1	membrane (#1), .080 in.	No Recor...	Generic...	0.08	0.07	10.6	93.2
2	rigid ins.,(extru.), 4-1/2 in.	No Recor...	Generic...	4.50	23.12	83.5	85.2
3	gypsum bd., 1/2 in., (#1)	No Recor...	Generic...	0.50	0.46	85.0	85.0
				5.08	23.65	(10.4)	(93.2)

R VALUE ANALYSIS

The Heat, Air and Moisture Building Science Toolbox - V.1B-E/U (11)

WALL SECTION AND TEMPERATURE GRADIENTS



PROJECT

Name	Thesis Wall 1
Number	1
City	Philadelphia, PA
Date	3/4/2010
Analysis by:	Jason Kukorlo
Wall Type	
Option	

CLIMATIC CONDITIONS

	Winter		Summer	
	Int.	Ext.	Int.	Ext.
Temp (°F)	75	10	75	93
RH (%)	60	66	60	68
DPT (°F)	61	2	61	82

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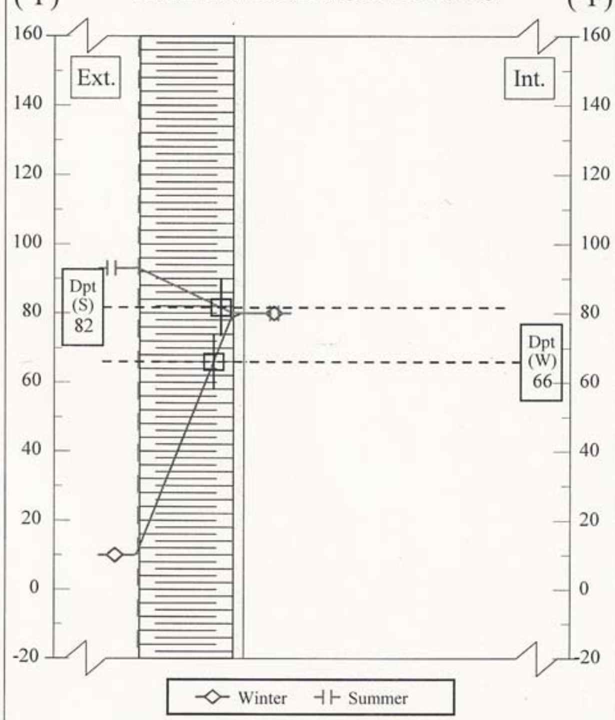
UNIVERSITY PARK, PA, USA, 16802

	Generic Material	Manufacturer	Model No.	Thick (in.)	RVal (R)	W.Temp. (°F)	S.Temp. (°F)
1	membrane (#1), .080 in.	No Recor...	Generic...	0.08	0.07	10.6	93.1
2	rigid ins.,(extru.), 4-1/2 in.	No Recor...	Generic...	4.50	23.12	73.7	75.4
3	gypsum bd., 1/2 in., (#1)	No Recor...	Generic...	0.50	0.46	75.0	75.0
				5.08	23.65	(10.4)	(93.2)

R VALUE ANALYSIS

The Heat, Air and Moisture Building Science Toolbox - V.1B-E/U (11)

WALL SECTION AND TEMPERATURE GRADIENTS



PROJECT

Name	Thesis Wall 1
Number	1
City	Philadelphia, PA
Date	3/4/2010
Analysis by:	Jason Kukorlo
Wall Type	
Option	

CLIMATIC CONDITIONS

	Winter		Summer	
	Int.	Ext.	Int.	Ext.
Temp (°F)	80	10	80	93
RH (%)	60	66	60	68
DPT (°F)	66	2	66	82

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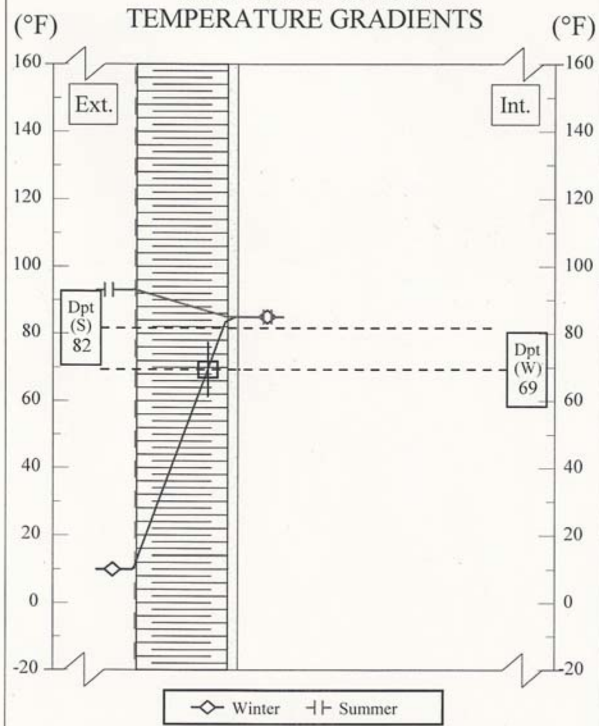
	Generic Material	Manufacturer	Model No.	Thick (in.)	RVal (R)	W.Temp. (°F)	S.Temp. (°F)
1	membrane (#1), .080 in.	No Recor...	Generic...	0.08	0.07	10.6	93.2
2	rigid ins.,(extru.), 4-1/2 in.	No Recor...	Generic...	4.50	23.12	78.6	80.3
3	gypsum bd., 1/2 in., (#1)	No Recor...	Generic...	0.50	0.46	80.0	80.0
				5.08	23.65	(10.4)	(93.2)

R VALUE ANALYSIS

The Heat, Air and Moisture Building Science Toolbox - V.1B-E/U (11)

WALL SECTION AND TEMPERATURE GRADIENTS



PROJECT

Name	Thesis Wall 1
Number	1
City	Philadelphia, PA
Date	3/4/2010
Analysis by:	Jason Kukorlo
Wall Type	
Option	

CLIMATIC CONDITIONS

	Winter		Summer	
	Int.	Ext.	Int.	Ext.
Temp (°F)	85	10	85	93
RH (%)	60	66	60	68
DPT (°F)	69	2	69	82

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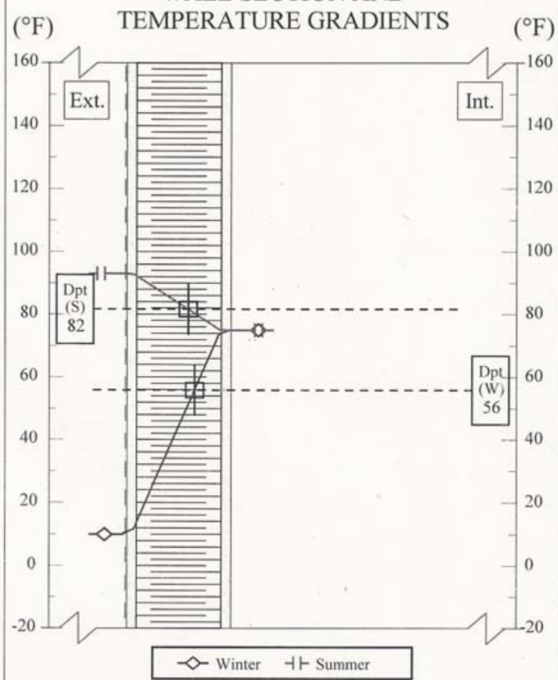
	Generic Material	Manufacturer	Model No.	Thick (in.)	RVal (R)	W.Temp. (°F)	S.Temp. (°F)
1	membrane (#1), .080 in.	No Recor...	Generic...	0.08	0.07	10.6	93.2
2	rigid ins.,(extru.), 4-1/2 in.	No Recor...	Generic...	4.50	23.12	83.5	85.2
3	gypsum bd., 1/2 in., (#1)	No Recor...	Generic...	0.50	0.46	85.0	85.0
				5.08	23.65	(10.4)	(93.2)

R VALUE ANALYSIS

The Heat, Air and Moisture Building Science Toolbox - V.1B-E/U (11)

WALL SECTION AND TEMPERATURE GRADIENTS



PROJECT

Name	Thesis Wall 2
Number	1
City	Philadelphia, PA
Date	3/4/2010
Analysis by:	Jason Kukorlo
Wall Type	
Option	

CLIMATIC CONDITIONS

	Winter		Summer	
	Int.	Ext.	Int.	Ext.
Temp (°F)	75	10	75	93
RH (%)	50	66	50	68
DPT (°F)	56	2	56	82

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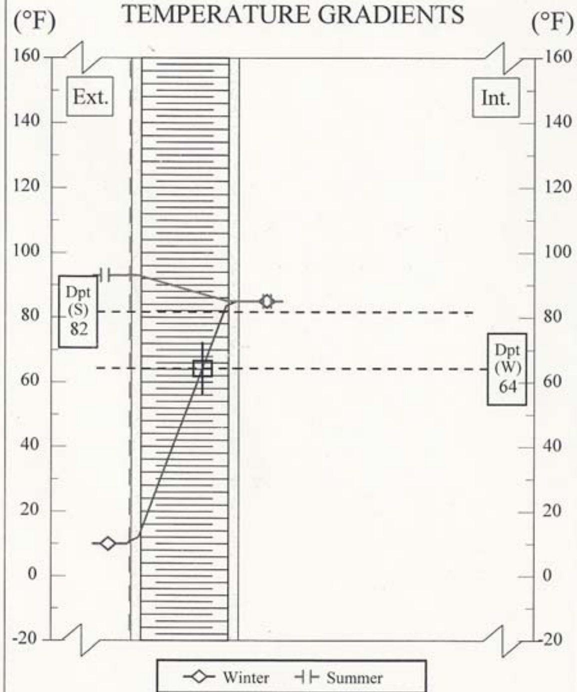
	Generic Material	Manufacturer	Model No.	Thick (in.)	RVal (R)	W.Temp. (°F)	S.Temp. (°F)
1	membrane (#1), .080 in.	No Recor...	Generic...	0.08	0.07	10.6	93.1
2	gypsum bd., 1/2 in., (#1)	No Recor...	Generic...	0.50	0.46	11.8	92.8
3	rigid ins.,(extru.), 4-1/2 in.	No Recor...	Generic...	4.50	23.12	73.8	75.3
4	gypsum bd., 1/2 in., (#1)	No Recor...	Generic...	0.50	0.46	75.0	75.0
	Total or (Layer 0)			5.58	24.11	(10.4)	(93.2)

R VALUE ANALYSIS

The Heat, Air and Moisture Building Science Toolbox - V.1B-E/U (11)

WALL SECTION AND TEMPERATURE GRADIENTS



PROJECT

Name	Thesis Wall 2
Number	1
City	Philadelphia, PA
Date	3/4/2010
Analysis by:	Jason Kukorlo
Wall Type	
Option	

CLIMATIC CONDITIONS

	Winter		Summer	
	Int.	Ext.	Int.	Ext.
Temp (°F)	85	10	85	93
RH (%)	50	66	50	68
DPT (°F)	64	2	64	82

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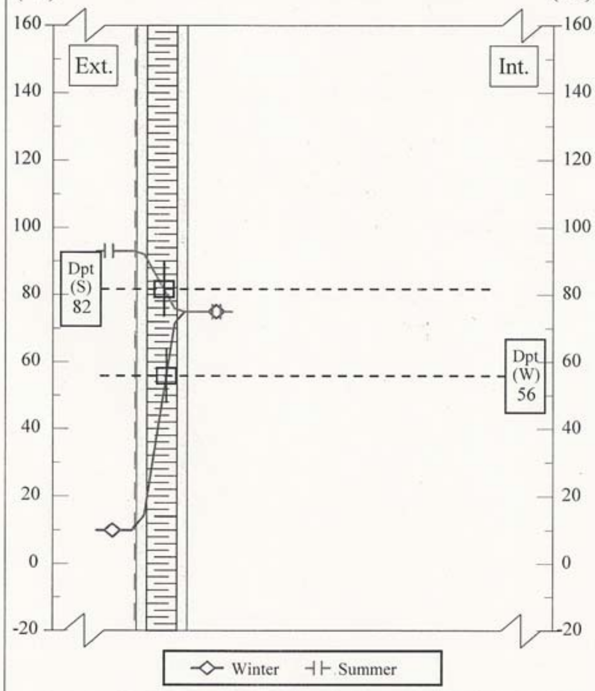
	Generic Material	Manufacturer	Model No.	Thick (in.)	RVal (R)	W.Temp. (°F)	S.Temp. (°F)
1	membrane (#1), .080 in.	No Recor...	Generic...	0.08	0.07	10.6	93.2
2	gypsum bd., 1/2 in., (#1)	No Recor...	Generic...	0.50	0.46	12.0	93.0
3	rigid ins.,(extru.), 4-1/2 in.	No Recor...	Generic...	4.50	23.12	83.6	85.2
4	gypsum bd., 1/2 in., (#1)	No Recor...	Generic...	0.50	0.46	85.0	85.0
	Total or (Layer 0)			5.58	24.11	(10.4)	(93.2)

R VALUE ANALYSIS

The Heat, Air and Moisture Building Science Toolbox - V.1B-E/U (11)

WALL SECTION AND TEMPERATURE GRADIENTS



PROJECT

Name	Thesis Wall 3
Number	1
City	Philadelphia, PA
Date	3/4/2010
Analysis by:	Jason Kukorlo
Wall Type	Option

CLIMATIC CONDITIONS

	Winter		Summer	
	Int.	Ext.	Int.	Ext.
Temp (°F)	75	10	75	93
RH (%)	50	66	50	68
DPT (°F)	56	2	56	82

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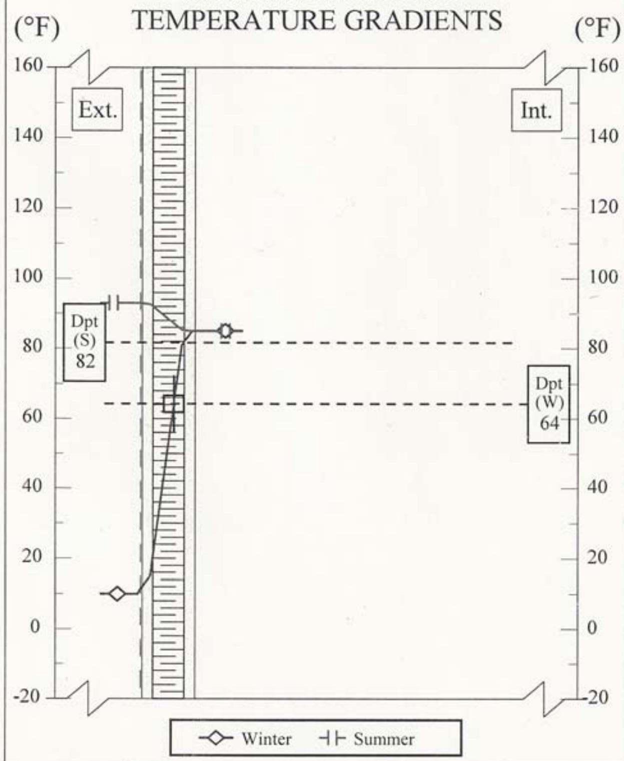
	Generic Material	Manufacturer	Model No.	Thick (in.)	RVal (R)	W.Temp. (°F)	S.Temp. (°F)
1	membrane (#1), .080 in.	No Recor...	Generic...	0.08	0.07	10.9	93.1
2	gypsum bd., 1/2 in., (#1)	No Recor...	Generic...	0.50	0.46	14.3	92.1
3	rigid ins.,(extru.), 1-1/2 in.	No Recor...	Generic...	1.50	7.71	71.6	76.0
4	gypsum bd., 1/2 in., (#1)	No Recor...	Generic...	0.50	0.46	75.0	75.0
				2.57	8.70	(10.4)	(93.2)

R VALUE ANALYSIS

The Heat, Air and Moisture Building Science Toolbox - V.1B-E/U (11)

WALL SECTION AND TEMPERATURE GRADIENTS



PROJECT

Name	Thesis Wall 3
Number	1
City	Philadelphia, PA
Date	3/4/2010
Analysis by:	Jason Kukorlo
Wall Type	
Option	

CLIMATIC CONDITIONS

	Winter		Summer	
	Int.	Ext.	Int.	Ext.
Temp (°F)	85	10	85	93
RH (%)	50	66	50	68
DPT (°F)	64	2	64	82

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	Generic Material	Manufacturer	Model No.	Thick (in.)	RVal (R)	W.Temp. (°F)	S.Temp. (°F)
1	membrane (#1), .080 in.	No Recor...	Generic...	0.08	0.07	11.0	93.1
2	gypsum bd., 1/2 in., (#1)	No Recor...	Generic...	0.50	0.46	15.0	92.7
3	rigid ins.,(extru.), 1-1/2 in.	No Recor...	Generic...	1.50	7.71	81.0	85.4
4	gypsum bd., 1/2 in., (#1)	No Recor...	Generic...	0.50	0.46	85.0	85.0
				2.57	8.70	(10.4)	(93.2)

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 and very durable and strong and typically have a 2-4 hour fire raing. 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 ¾" 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 Farquahar 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 ¼" fully tempered monolithic glass and an inner lite of ¼" 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 ¼" 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.

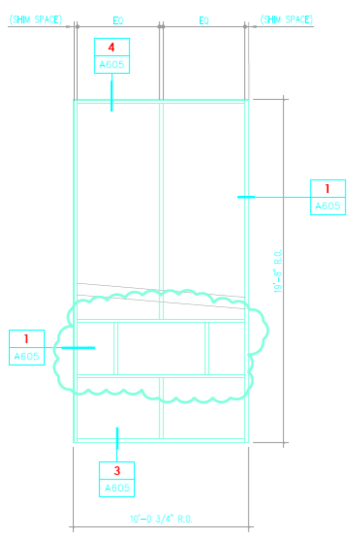


Figure 92 – Glass Panel Used for Glass Calculations

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.

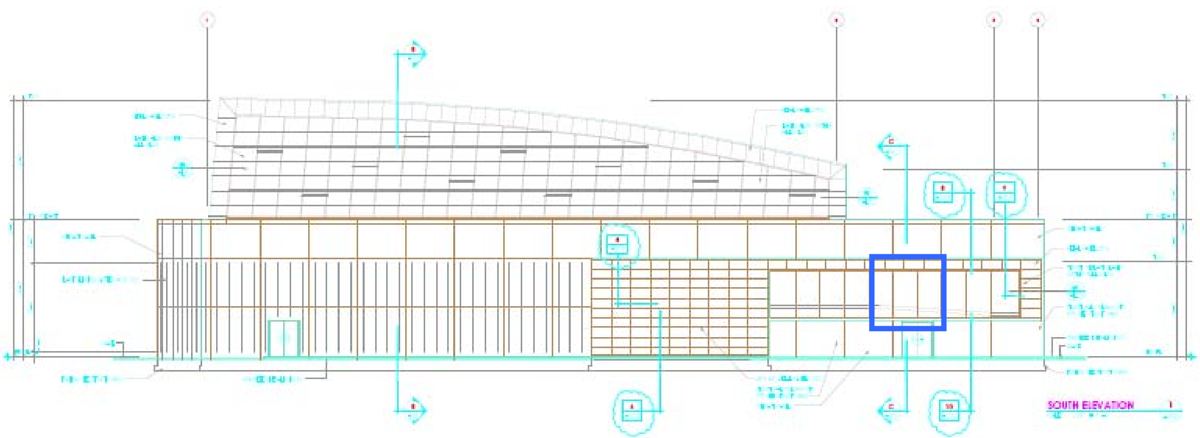


Figure 93 – Elevation of South Façade of Natatorium Showing Glass Panel Used for Second Glass Calculations

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 it 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 are 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.

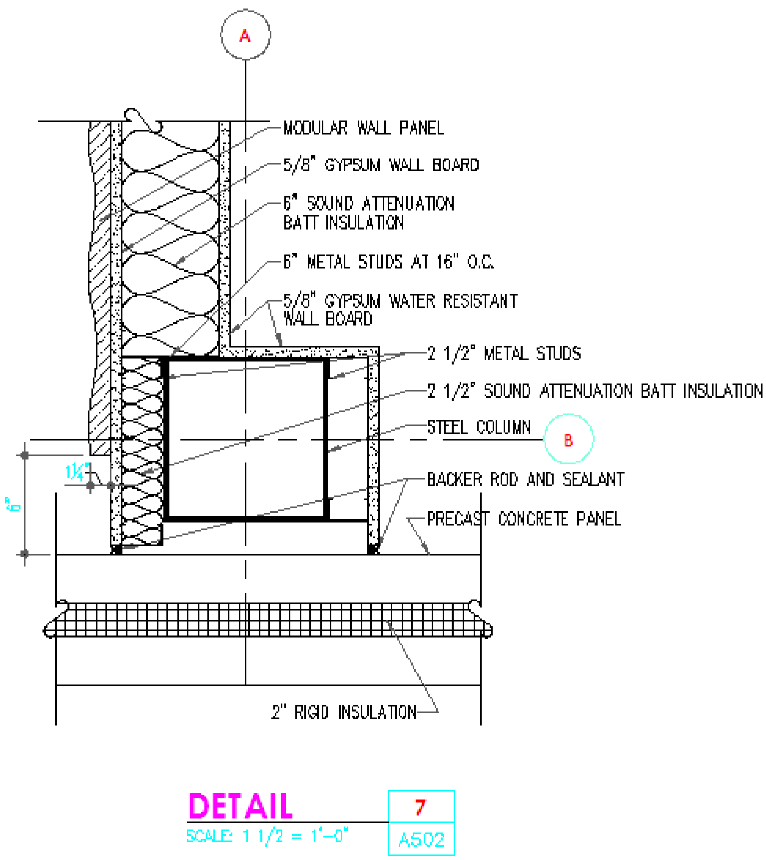
The design of the glulam trusses for the Farquhar Park Aquatic Center provides additional permanent bracing members connecting the bottom chords of the trusses. This lateral bracing spans the entire length of the roof in the North/South direction and is spaced at 26’-0” o.c. Temporary lateral bracing 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

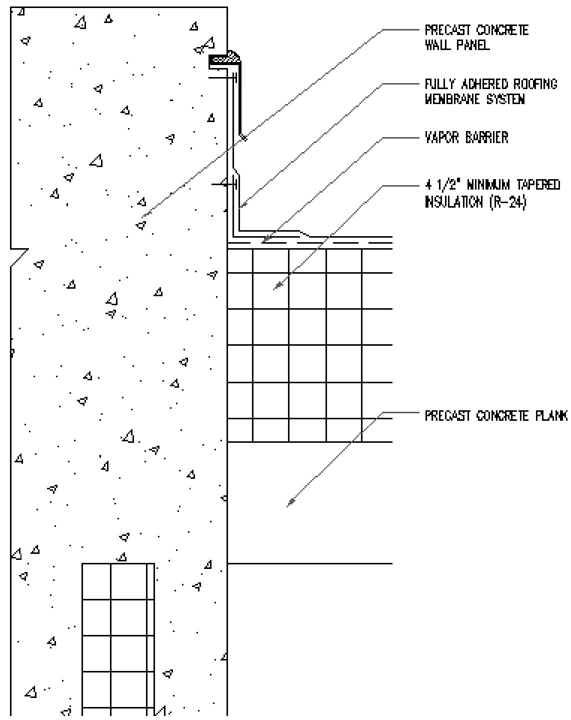
A large portion of building problems and construction claims occur at the roof and façade. Improper detailing, lack of detailing, and misunderstanding of the behavior of a wall system are some of the most common sources of problems. The building envelope is the most expensive part of the building, typically 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.

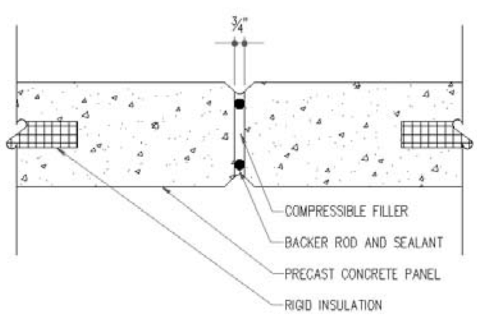


The parapet flashing detail pictured next shows the correct shape of the sealant at the top of the flashing. A convex sealant profile will typically fail, so a concave sealant profile is often suggested for optimum performance.



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

The ideal depth-to-width ratio of sealants is ½:1 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 ¼" with a ¾" 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 ½:1 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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Appendix A – Structural Depth: Gravity System Calculations

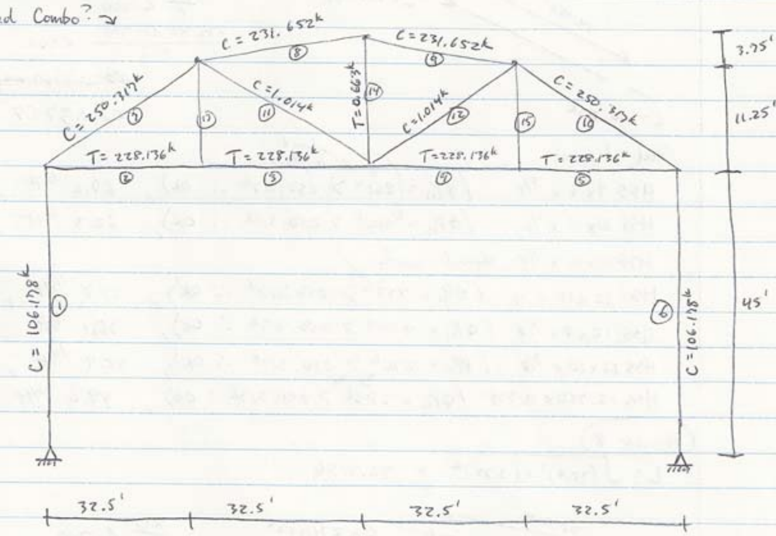
King Post Truss Members

2/5/10

Thesis

Truss 3f) → Steel

* Load Combo? ↘



* Try to avoid slenderness ratio of $\frac{L}{r} > 300$

(1) $P_u = 228.136^k$

$A_{g,min} = \frac{P_u}{\phi_t F_y} = \frac{228.136^k}{(0.9)(46\ ksi)} = 5.5105\ in^2$

HSS 4x4x 1/2 ($A_g = 6.02\ in^2$, 21.50 lb/ft) $\frac{L}{r} = \frac{(32.5')(12)}{1.41} = 276.60 < 300 \therefore$ OK

HSS 5x5x 3/8 ($A_g = 6.18\ in^2$, 22.30 lb/ft) $\frac{L}{r} = \frac{(32.5)(12)}{1.87} = 208.56 < 300 \therefore$ OK

HSS 5 1/2 x 5 1/2 x 5/16 ($A_g = 5.85\ in^2$, 21.16 lb/ft) $\frac{L}{r} = \frac{(32.5)(12)}{2.11} = 184.83 < 300 \therefore$ OK

$A_{e,min} = \frac{P_u}{\phi_t F_u} = \frac{228.136k}{(0.75)(58\ ksi)} = 5.2445\ in^2$

Assume $A_e = 0.75 A_g \rightarrow A_{g,min} = \frac{5.2445\ in^2}{0.75} = 6.993\ in^2$

HSS 5x5x1/2 ($A_g = 7.88\ in^2$, 28.30 lb/ft) $\frac{L}{r} = \frac{(32.5)(12)}{1.82} = 214.29 < 300 \therefore$ OK

HSS 6x6x 3/8 ($A_g = 7.58\ in^2$, 27.41 lb/ft) $\frac{L}{r} = \frac{(32.5')(12)}{2.28} = 171.05 < 300 \therefore$ OK

HSS 7x7x 5/16 ($A_g = 7.59\ in^2$, 27.54 lb/ft) $\frac{L}{r} = \frac{(32.5')(12)}{2.72''} = 143.38 < 300 \therefore$ OK

HSS 8x8x 1/4 ($A_g = 7.10\ in^2$, 25.79 lb/ft) $\frac{L}{r} = \frac{(32.5')(12\ in/ft)}{3.15\ in} = 123.81 < 300 \therefore$ OK

Steel Manual:

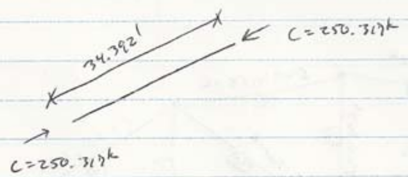
2L5x3x 1/2 ( ) 2L6x3 1/2 x 1/2 ( ) 2L7x4x 3/8 ( )

2L5x3 1/2 x 1/2 ( ) 2L6x4x 3/8 ( ) 2L8x4x 7/16 ( )

2L 5x5x 3/8 ( ) 2L6x6x 5/16 ( )

Member 7

$L = \sqrt{(32.5')^2 + (11.25')^2} = 34.39204123'$



$\frac{KL}{r} < 300$

$\frac{(34.392')(12 \text{ in/ft})}{r} < 300$

~~0.7269123642~~

$r > 1.3757''$

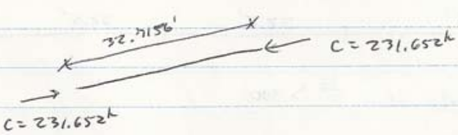
Steel Manual:

(36' unbraced length)

HSS 9x9 x 5/8 $(\phi P_n = 261^k > 250.317^k \therefore ok)$, 67.6 lb/ft, r = 3.40"
HSS 10x10 x 1/2 $(\phi P_n = 306^k > 250.317^k \therefore ok)$, 62.3 lb/ft, r = 3.86"
HSS 10x10 x 3/8 should work
HSS 12x12 x 1/4 $(\phi P_n = 255^k > 250.317^k \therefore ok)$, 39.4 lb/ft, r = 4.79"
HSS 12x8 x 5/8 $(\phi P_n = 254^k > 250.317^k \therefore ok)$, 76.1 lb/ft, r_y = 3.16"
HSS 12x10 x 3/8 $(\phi P_n = 276^k > 250.317^k \therefore ok)$, 52.9 lb/ft, r_y = 4.01"
HSS 12.750 x 0.375 $(\phi P_n = 283^k > 250.317^k \therefore ok)$, 49.6 lb/ft, r = 4.39"

Member 8

$L = \sqrt{(32.5)^2 + (3.75')^2} = 32.7156'$



$\frac{KL}{r} < 300$

$\frac{(32.7156')(12 \text{ in/ft})}{r} < 300$

$r > 1.3086''$

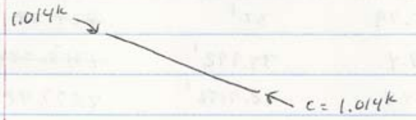
Steel Manual: (all: 34' unbraced length)

HSS 9x9x 1/2 $(\phi P_n = 248^k > 231.652^k \therefore ok)$, 55.5 lb/ft, r = 3.45"
HSS 10x10x 3/8 $(\phi P_n = 262^k > 231.652^k \therefore ok)$, 47.8 lb/ft, r = 3.92"
HSS 12x12x 1/4 $(\phi P_n = 267^k > 231.652^k \therefore ok)$, 39.4 lb/ft, r = 4.79"
HSS 10x8x 5/8 $(\phi P_n = 242^k > 231.652^k \therefore ok)$, 67.6 lb/ft, r = 3.09"
HSS 12x8x 1/2 $(\phi P_n = 241^k > 231.652^k \therefore ok)$, 62.3 lb/ft, r = 3.21"
HSS 12x10x 5/16 $(\phi P_n = 255^k > 231.652^k \therefore ok)$, 44.6 lb/ft, r = 4.04"
HSS 10.000 x ~~0.500~~ 0.625 $(\phi P_n =$ ~~238k~~ $259 > 231.652^k \therefore ok)$, ~~50.8~~ 62.6 lb/ft, r = ~~3.38"~~ 3.34"
HSS 10.750 x 0.500 $(\phi P_n =$ ~~237k~~ $263 > 231.652^k \therefore ok)$, 54.8 lb/ft, r = 3.64"
HSS 12.750 x 0.375 $(\phi P_n =$ ~~301k~~ $302^k > 231.652^k) \therefore ok$, 49.6 lb/ft, r = 4.39"

2/17/10

Member 11

$L = \sqrt{(32.5')^2 + (11.25')^2} = 34.39204'$



$\frac{KL}{r} < 300$

$\frac{(34.392')(12)}{r} < ~~300~~\ 200$

$r > ~~1.3757''~~\ 2.0635''$

Steel Manual:

HSS 5½ x 5½ x ⅛ should work, 9 lb/ft, r = 2.19"

HSS 6 x 6 x ⅛, 9.85 lb/ft, r = 2.39"

HSS 7x7x ⅛, 11.6 lb/ft, r = 2.80"

HSS 7x5 x ⅛, should work, 9.85 lb/ft, r = 2.07"

HSS 8x6 x 3/16, 17.1 lb/ft, r = 2.46"

HSS 6.000 x 0.125, 7.85 lb/ft, r = 2.08"

HSS 6.625x0.125, 8.69 lb/ft, r = 2.30"

Pipe 6 Std 19 lb/ft, r = 2.25"

Member 13

L = 11.25'

$\frac{KL}{r} < ~~300~~\ 200$

$\frac{(11.25')(12)}{r} < ~~300~~\ 200$

$r > 0.675''$

* → * Is it ok (does it seem right) that hardly any forces in the diagonal web members?

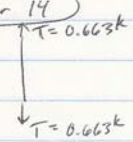
Pipe 4 Std (t = 0.221"), 10.8 lb/ft, r = 1.51"

HSS 4.000 x 0.125, 5.18 lb/ft, r = 1.37"

HSS 2x2 x ⅛, 3.04 lb/ft, r = 0.761"

HSS 4x2 x ⅛, 4.75 lb/ft, r = 0.830"

Member 14



$\frac{KL}{r} < 200$

$\frac{(15')(12)}{r} < 200$

$r > 0.90''$

HSS 2x2x ⅛ (3.04 lb/ft)

HSS 1.660 x 0.140 (2.27 lb/ft)

Using the lightest HSS members: (mostly lightest)

Member(s) #	Shape	lb/ft	Length	Weight (lb)
2, 3, 4, 5	HSS 8x8x$\frac{1}{4}$	25.79	32'	~~825.28~~ 3301.12
7, 10	HSS 12x12x$\frac{1}{4}$	39.4	34.392'	~~1355.0448~~ 2710.0896
8, 9	HSS 12x12x$\frac{1}{4}$	39.4	32.7156'	2577.98928
11, 12	HSS 5$\frac{1}{2}$x5$\frac{1}{2}$x$\frac{1}{8}$	9.0	34.392'	619.056
13, 15	HSS 2x2x$\frac{1}{8}$	3.04	11.25'	68.4
14	HSS 2x2x$\frac{1}{8}$	3.04	15'	45.6
				9322.25488 lb

(5 trusses)(9322.25488 lb) = **46,611.2744 lb**

* Not including bracing / diaphragm members and columns

* Do these trusses count as "king-post" trusses since they are arched?

Glulam Truss Members

Loads:

Dead Load:

Zinc Standing Seam Metal Roof Panels:	1.5 PSF
½" Moisture Resistant Gypsum Board:	2.5 PSF
4 ½" Rigid Insulation = (1.5 psf/in.)(4.5 in.):	6.75 PSF
Southern Pine 3 in. Decking:	7.6 PSF
TOTAL:	18.35 PSF
	Say = 20 PSF

D_{Total} = 20 PSF + 5 PSF (superimposed) + 5 PSF (self weight of trusses) = 30 PSF

*Applied to top chord of wood trusses (bottom of trusses is open to below; assuming superimposed loads are attached to top chord)

L_r = 20 PSF

S = 23.1 PSF

*C_s = 1.0 for roof slopes less than 30 degrees

Load Combinations (ASD):

D = 30 PSF

D + L = 20 + 0 = 20 PSF

D + (L_r or S or R) = D + S = 30 + 23.1 = **53.1 PSF**

D + 0.75L + 0.75(L_r or S or R) = D + 0.75L_r = 30 PSF + (0.75)(20 PSF) = 45 PSF

D +/- (W or 0.7E) = D = 30 PSF

D + 0.75(W or 0.7E) + 0.75L + 0.75(L_r or S or R) = D + 0.75L_r

= 30 PSF + (0.75)(20 PSF) = 45 PSF

0.6D +/- (W or 0.7E) = 0.6D = (0.6)(30 PSF) = 18 PSF

53.1 PSF controls for maximum load, but the load combination of D + S may not necessarily control. It is important to look at other load combinations as well because the duration factor (C_D) changes for other load combinations.

Load Combination: D + S

Members 13 and 22:

Load along roof slope:

$w_{TL} = w_D + w_S(L_2/L_1)$

w_{TL} = 30 PSF + (23.1 PSF)(13'/15.0833') = 49.9094 PSF

w_{TL} = (49.9094 PSF)(8') = 399.2751381 lb/ft = 0.3992751381 k/ft

Members 14 and 21:

Load along roof slope:

$w_{TL} = w_D + w_S(L_2/L_1)$

w_{TL} = 30 PSF + (23.1 PSF)(13'/14.1458') = 51.22886598 PSF

w_{TL} = (51.22886598 PSF)(8') = 409.8309278 lb/ft = 0.4098309278 k/ft

Members 15 and 20:

Load along roof slope:

$w_{TL} = w_D + w_S(L_2/L_1)$

w_{TL} = 30 PSF + (23.1 PSF)(13'/13.546875') = 52.16747405 PSF

w_{TL} = (52.16747405 PSF)(8') = 417.3397924 lb/ft = 0.4173397924 k/ft

Members 16 and 19:

Load along roof slope:

$w_{TL} = w_D + w_S(L_2/L_1)$

w_{TL} = 30 PSF + (23.1 PSF)(13'/13.1875') = 52.77156398 PSF

w_{TL} = (52.77156398 PSF)(8') = 422.1725118 lb/ft = 0.4221725118 k/ft

Members 17 and 18:

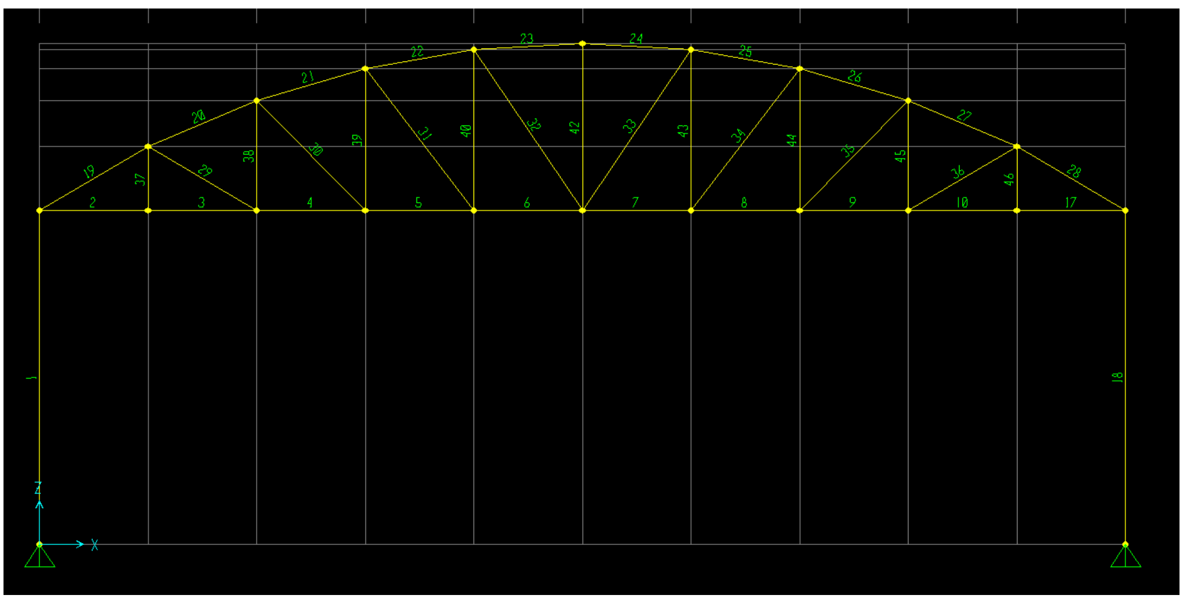
Load along roof slope:

$$w_{TL} = w_D + w_S(L_2/L_1)$$

$$w_{TL} = 30 \text{ PSF} + (23.1 \text{ PSF})(13'/13.0208') = 53.06304 \text{ PSF}$$

$$w_{TL} = (53.06304 \text{ PSF})(8') = 424.50432 \text{ lb/ft} = 0.42450432 \text{ k/ft}$$

These loads were applied to models of the glulam truss in SAP, and the results were recorded. Results for other load combinations were obtained by taking fractions of the results from the D + S load combination. For instance, since the dead load is (30 psf/53.1 psf), or 0.565 of the total load for the D + S load combination, results for just dead load were obtained by multiplying the results from the D + S load combination by 0.565. This same process was carried out to obtain results from the live roof load by itself. See Tables ____ - _____ below for a summary of the results for each load combination. In the tables, axial and shear forces are in kips and moments are in ft-kips.



Axial Load, Shear, and Moment (Unfactored) for Wood Trusses

	1 West Column	2 Bottom Chord	3 Bottom Chord	4 Bottom Chord	5 Bottom Chord	6 Bottom Chord	19 Top Chord	20 Top Chord	21 Top Chord	22 Top Chord	23 Top Chord
P_D	-16.14	24.62	24.62	25.18	25.55	25.73	-29.40	-28.03	-27.07	-26.38	-25.92
$P_{D,BOTTOM\ CHORD}$	-5.20	7.98	7.98	8.20	8.35	8.43	-9.25	-8.92	-8.70	-8.55	-8.46
P_{Lr}	-10.76	16.41	16.41	16.78	17.03	17.15	-19.60	-18.69	-18.05	-17.59	-17.28
P_S	-12.43	18.95	18.95	19.39	19.67	19.81	-22.64	-21.58	-20.84	-20.31	-19.95
$P_{W,LATERAL}$	0.00	-3.24	-3.24	-3.24	-3.24	-3.24	0.00	0.00	0.00	0.00	0.00
$P_{W,UPLIFT}$	8.90	-13.52	-13.52	-13.77	-13.94	-14.02	16.16	15.34	14.77	14.37	14.11
P_E	0.00	-4.27	-4.27	-4.27	-4.27	-4.27	0.00	0.00	0.00	0.00	0.00
V_D (Top or Left)	0.00	0.00	0.00	0.00	0.00	0.00	-1.47	-1.51	-1.53	-1.55	-1.56
V_D (Bottom or Right)	0.00	0.00	0.00	0.00	0.00	0.00	1.47	1.51	1.53	1.55	1.56
$V_{D,BOTTOM\ CHORD}$ (Top or Left)	0.00	-0.52	-0.52	-0.52	-0.52	-0.52	0.00	0.00	0.00	0.00	0.00
$V_{D,BOTTOM\ CHORD}$ (Bottom or Right)	0.00	0.52	0.52	0.52	0.52	0.52	0.00	0.00	0.00	0.00	0.00
V_{Lr} (Top or Left)	0.00	0.00	0.00	0.00	0.00	0.00	-0.98	-1.00	-1.02	-1.03	-1.04
V_{Lr} (Bottom or Right)	0.00	0.00	0.00	0.00	0.00	0.00	0.98	1.00	1.02	1.03	1.04
V_S (Top or Left)	0.00	0.00	0.00	0.00	0.00	0.00	-1.13	-1.16	-1.18	-1.19	-1.20
V_S (Bottom or Right)	0.00	0.00	0.00	0.00	0.00	0.00	1.13	1.16	1.18	1.19	1.20
$V_{W,LATERAL}$ (Top or Left)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
$V_{W,LATERAL}$ (Bottom or Right)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
$V_{W,UPLIFT}$ (Top or Left)	0.00	0.00	0.00	0.00	0.00	0.00	0.84	0.84	0.84	0.84	0.84
$V_{W,UPLIFT}$ (Bottom or Right)	0.00	0.00	0.00	0.00	0.00	0.00	-0.84	-0.84	-0.84	-0.84	-0.84
V_E (Top or Left)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
V_E (Bottom or Right)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
M_D (Max. Positive)	0.00	0.00	0.00	0.00	0.00	0.00	5.53	5.32	5.19	5.11	5.08
$M_{D,BOTTOM\ CHORD}$ (Max. Positive)	0.00	1.66	1.66	1.66	1.66	1.66	0.00	0.00	0.00	0.00	0.00
M_{Lr} (Max. Positive)	0.00	0.00	0.00	0.00	0.00	0.00	3.68	3.55	3.46	3.41	3.38
M_S (Max. Positive)	0.00	0.00	0.00	0.00	0.00	0.00	4.25	4.10	4.00	3.94	3.91
$M_{W,LATERAL}$ (Max. Positive)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
$M_{W,UPLIFT}$ (Max. Positive)	0.00	0.00	0.00	0.00	0.00	0.00	-3.16	-2.97	-2.84	-2.77	-2.73
M_E (Max. Positive)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

D

	1	2	3	4	5	6	19	20	21	22	23
Max $V_{TOP/LEFT}$ (kips)	0.00	-0.52	-0.52	-0.52	-0.52	-0.52	-1.47	-1.51	-1.53	-1.55	-1.56
Max $V_{BOTTOM/RIGHT}$ (kips)	0.00	0.52	0.52	0.52	0.52	0.52	1.47	1.51	1.53	1.55	1.56
Max $M_{MIDSPAN}$ (ft-kips)	0.00	1.66	1.66	1.66	1.66	1.66	5.53	5.32	5.19	5.11	5.08
Max P_u (kips)	-21.34	32.60	32.60	33.38	33.90	34.16	-38.65	-36.95	-35.77	-34.94	-34.38

D + Lr

	1	2	3	4	5	6	19	20	21	22	23
Max $V_{TOP/LEFT}$ (kips)	0.00	-0.52	-0.52	-0.52	-0.52	-0.52	-2.44	-2.51	-2.56	-2.58	-2.60
Max $V_{BOTTOM/RIGHT}$ (kips)	0.00	0.52	0.52	0.52	0.52	0.52	2.44	2.51	2.56	2.58	2.60
Max $M_{MIDSPAN}$ (ft-kips)	0.00	1.66	1.66	1.66	1.66	1.66	9.21	8.87	8.65	8.52	8.46
Max P_u (kips)	-32.10	49.01	49.01	50.16	50.93	51.31	-58.25	-55.63	-53.82	-52.52	-51.66

D + S

	1	2	3	4	5	6	19	20	21	22	23
Max $V_{TOP/LEFT}$ (kips)	0.00	-0.52	-0.52	-0.52	-0.52	-0.52	-2.59	-2.66	-2.71	-2.74	-2.76
Max $V_{BOTTOM/RIGHT}$ (kips)	0.00	0.52	0.52	0.52	0.52	0.52	2.59	2.66	2.71	2.74	2.76
Max $M_{MIDSPAN}$ (ft-kips)	0.00	1.66	1.66	1.66	1.66	1.66	9.78	9.42	9.19	9.05	8.98
Max P_u (kips)	-33.76	51.55	51.55	52.76	53.57	53.97	-61.28	-58.53	-56.62	-55.25	-54.33

D +/- W

	1	2	3	4	5	6	19	20	21	22	23
Max $V_{TOP/LEFT}$ (kips)	0.00	-0.52	-0.52	-0.52	-0.52	-0.52	-0.63	-0.67	-0.69	-0.71	-0.72
Max $V_{BOTTOM/RIGHT}$ (kips)	0.00	0.52	0.52	0.52	0.52	0.52	0.63	0.67	0.69	0.71	0.72
Max $M_{MIDSPAN}$ (ft-kips)	0.00	1.66	1.66	1.66	1.66	1.66	2.37	2.36	2.35	2.35	2.34
Max P_u (kips)	-12.44	15.84	15.84	16.36	16.72	16.90	-22.49	-21.61	-21.00	-20.56	-20.27

D +/- E											
Max $V_{TOP/LEFT}$ (kips)	0.00	-0.52	-0.52	-0.52	-0.52	-0.52	-1.47	-1.51	-1.53	-1.55	-1.56
Max $V_{BOTTOM/RIGHT}$ (kips)	0.00	0.52	0.52	0.52	0.52	0.52	1.47	1.51	1.53	1.55	1.56
Max $M_{MIDSPAN}$ (ft-kips)	0.00	1.66	1.66	1.66	1.66	1.66	5.53	5.32	5.19	5.11	5.08
Max P_u (kips)	-21.34	28.32	28.32	29.11	29.63	29.89	-38.65	-36.95	-35.77	-34.94	-34.38

D + 0.75W + 0.75Lr											
Max $V_{TOP/LEFT}$ (kips)	0.00	-0.52	-0.52	-0.52	-0.52	-0.52	-1.57	-1.63	-1.67	-1.70	-1.71
Max $V_{BOTTOM/RIGHT}$ (kips)	0.00	0.52	0.52	0.52	0.52	0.52	1.57	1.63	1.67	1.70	1.71
Max $M_{MIDSPAN}$ (ft-kips)	0.00	1.66	1.66	1.66	1.66	1.66	5.92	5.76	5.66	5.60	5.56
Max P_u (kips)	-22.73	32.33	32.33	33.20	33.78	34.08	-41.23	-39.46	-38.23	-37.35	-36.75

D + 0.75W + 0.75S											
Max $V_{TOP/LEFT}$ (kips)	0.00	-0.52	-0.52	-0.52	-0.52	-0.52	-1.68	-1.74	-1.79	-1.82	-1.83
Max $V_{BOTTOM/RIGHT}$ (kips)	0.00	0.52	0.52	0.52	0.52	0.52	1.68	1.74	1.79	1.82	1.83
Max $M_{MIDSPAN}$ (ft-kips)	0.00	1.66	1.66	1.66	1.66	1.66	6.35	6.17	6.06	5.99	5.96
Max P_u (kips)	-23.98	34.24	34.24	35.16	35.76	36.07	-43.50	-41.63	-40.33	-39.39	-38.76

0.6D + W											
Max $V_{TOP/LEFT}$ (kips)	0.00	-0.31	-0.31	-0.31	-0.31	-0.31	-0.04	-0.06	-0.08	-0.09	-0.10
Max $V_{BOTTOM/RIGHT}$ (kips)	0.00	0.31	0.31	0.31	0.31	0.31	0.04	0.06	0.08	0.09	0.10
Max $M_{MIDSPAN}$ (ft-kips)	0.00	0.99	0.99	0.99	0.99	0.99	0.16	0.23	0.27	0.30	0.31
Max P_u (kips)	-3.90	2.80	2.80	3.01	3.16	3.23	-7.03	-6.83	-6.69	-6.59	-6.52

Summary:

Summary of Maximum Forces, Moments, and Shears for West Column				
	Axial Force	Shear	Moment	C_D
D	-21.34	0.00	0.00	0.9
D + Lr	-32.10	0.00	0.00	1.0
D + S	-33.76	0.00	0.00	1.15
D +/- W	-12.44	0.00	0.00	1.6
D +/- E	-21.34	0.00	0.00	1.6
D + 0.75W + 0.75Lr	-22.73	0.00	0.00	1.6
D + 0.75W + 0.75S	-23.98	0.00	0.00	1.6
0.6D + W	-3.90	0.00	0.00	1.6

Summary of Maximum Forces, Moments, and Shears for Bottom Chord				
	Axial Force	Shear	Moment	C_D
D	34.16	0.52	1.66	0.9
D + Lr	51.31	0.52	1.66	1.0
D + S	53.97	0.52	1.66	1.15
D +/- W	16.90	0.52	1.66	1.6
D +/- E	29.89	0.52	1.66	1.6
D + 0.75W + 0.75Lr	34.08	0.52	1.66	1.6
D + 0.75W + 0.75S	36.07	0.52	1.66	1.6
0.6D + W	2.80	0.31	0.99	1.6

Summary of Maximum Forces, Moments, and Shears for Top Chord				
	Axial Force	Shear	Moment	C_D
D	-38.65	1.56	5.53	0.9
D + Lr	-58.25	2.60	9.21	1.0
D + S	-61.28	2.76	9.78	1.15
D +/- W	-22.49	0.72	2.37	1.6
D +/- E	-38.65	1.56	5.53	1.6
D + 0.75W + 0.75Lr	-41.23	1.71	5.92	1.6
D + 0.75W + 0.75S	-43.50	1.83	6.35	1.6
0.6D + W	-7.03	0.10	0.31	1.6

Units for Above Tables:
Axial Force: kips
Shear: kips
Moment: ft-kips

Wood Truss Member Design:

Top Chord: Combined Bending and Axial Forces (Member 6 is worst case)

Try 6 ¾" x 9 5/8"

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

F_b = 2100 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

A = 64.97 in^2

S = 104.2 in^3

E_{min} = 980,000 psi

LOAD COMBINATION: D + S

Axial Load: P = 61.284 kips (Compression) (from SAP2000)

Maximum Moment = 9.779 ft-kips = 117.342 in-kips (from SAP2000)

L = 15'-1" = 15.0833'

Axial Load:

f_c = P/A = 61,284 lb/64.97 in^2 = 943.266 psi

$(l_e/d)_x$ = [(15.0833')(12 in/ft)]/9.625" = 18.8052 < 50 ∴ OK

$(l_e/d)_y$ = 0 because of lateral support provided by roof diaphragm

$(l_e/d)_{max} = (l_e/d)_x$ = 18.8052

The larger slenderness ratio governs the adjusted design value. Therefore, the strong axis of the member is critical, and $(l_e/d)_x$ is used to determine F'_c.

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

E_{min} = 980,000 psi

C_D = 1.15 (for snow load; load combination D+S)

C_M = 0.73 for F_c (p. 64, NDS Supplement)

C_M = 0.833 for E and E_{min} (p. 64, NDS Supplement)

C_M = 0.8 for F_b (p. 64, NDS Supplement)

$C_t = 1.0$

$E'_{min} = (E_{min})(C_M)(C_t) = (980{,}000)(0.833)(1.0) = 816{,}340$ psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340 \text{ psi})]/[(18.8052)^2] = 1897.524$ psi

Here, l_e/d is based on $(l_e/d)_{max}$.

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.15)(0.73)(1.0) = 1930.85$ psi

$F_{cE}/F_c^* = 1897.529/1930.85 = 0.9827$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.9827]/[(2)(0.9)] = 1.1015$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{1.1015\} - \sqrt{\{[1.1015]^2 - [0.9827/0.9]\}}$

$= 1.1015 - 0.3485$

$= 0.7531$

$F'_c = F_c^*(C_P) = (1930.85 \text{ psi})(0.7531) = 1454.068$ psi

Axial stress ratio $= f_c/F'_c = (943.266 \text{ psi})/(1454.068 \text{ psi}) = 0.6487$

Net Section Check:

Assume connections will be made with (2) rows of ¾" diameter bolts.

Assume the hole diameter is 1/16" larger than the bolt (for stress calculations only).

$A_n = (6.75")[9.625" - (2)(0.8125")] = 54 \text{ in}^2$

(3/4" + 1/16" = 0.8125")

$f_c = P/A_n = 61{,}284 \text{ lb}/54 \text{ in}^2 = 1134.889$ psi

At braced location there is no reduction for stability.

$F'_c = F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.15)(0.73)(1.0) = 1930.85$ psi

1930.85 psi > 1134.889 psi ∴ OK

Bending:
Bending is about the strong axis of the cross section. The adjusted bending design value for a glulam is governed by the smaller of two criteria: volume effect or lateral stability. In this case,

the beam has full lateral support. Therefore, l_u and R_B are zero and the lateral stability factor is $C_L = 1.0$.

M = 117.342 in-kips = 117,342 in-lb

S = 104.2 in^3 (for 6 ¾" x 9 5/8")

f_b = M/S = 117,342 in-lb/104.2 in^3 = 1,126.123 psi

$F'_b = F_b(C_D)(C_M)(C_t)(C_L)$ or

$F'_b = F_b(C_D)(C_M)(C_t)(C_V)$

For Southern Pine glulam:

$C_V = (21'/L)^{1/20}(12''/d)^{1/20}(5.125''/b)^{1/20} \leq 1.0$

$C_V = (21'/15.0833')^{1/20}(12''/9.625'')^{1/20}(5.125''/6.75'')^{1/20} \leq 1.0$

$C_V = 1.0139 \leq 1.0$

$\therefore C_V = 1.0$

$F'_b = F_b(C_D)(C_M)(C_t)(C_L \text{ or } C_V)$ = (2100 psi)(1.15)(0.8)(1.0)(1.0) = 1932 psi

Bending stress ratio = f_b/F'_b = 1126.123 psi/1932 psi = 0.5829

Combined Stresses:

The bending moment is about the strong axis of the cross section, and the amplification for P-Δ is measured by the column slenderness ratio about the x axis.

$(l_e/d)_{\text{bending moment}} = (l_e/d)_x$ = 18.80519481

$F_{cEx} = [0.822E'_{min}]/[(l_e/d)_x]^2 = [(0.822)(816{,}340 \text{ psi})]/[(18.8052)^2]$ = 1897.524 psi

*Here, (l_e/d) is based on the axis about which the bending moment occurs.

Amplification factor = $1/[1 - (f_c/F_{cEx})]$ = 1/[1 – (943.266 psi/1897.524 psi)] = 1.9885

$(f_c/F'_c)^2 + \{1/[1 - (f_c/F_{cEx})]\}(f_b/F'_b) = (0.6487)^2 + (1.9885)(0.5829) = 1.5799 > 1.0$ ∴ **N.G.**

Try 6 ¾" x 11"

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

F_b = 2100 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

A = 74.25 in^2

$S = 136.1\ in^3$

$E_{min} = 980{,}000$ psi

LOAD COMBINATION: D + S

Axial Load: P = 61.284 kips (Compression) (from SAP2000)

Maximum Moment = 9.779 ft-kips = 117.342 in-kips (from SAP2000)

L = 15'-1" = 15.083333'

Axial Load:

$f_c = P/A = 61{,}284\ lb/74.25\ in^2 = 825.374$ psi

$(l_e/d)_x = [(15.0833')(12\ in/ft)]/11" = 16.4545 < 50\ \therefore$ OK

$(l_e/d)_y = 0$ because of lateral support provided by roof diaphragm

$(l_e/d)_{max} = (l_e/d)_x = 16.4545$

The larger slenderness ratio governs the adjust design value. Therefore, the strong axis of the member is critical, and $(l_e/d)_x$ is used to determine F'_c.

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340\ psi)]/[(16.4545)^2] = 2478.398$ psi

Here, l_e/d is based on $(l_e/d)_{max}$.

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300\ psi)(1.15)(0.73)(1.0) = 1930.85$ psi

$F_{cE}/F_c^* = 2478.398/1930.85 = 1.2836$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 1.2836]/[(2)(0.9)] = 1.2687$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{1.2687\} - \sqrt{\{[1.2687]^2 - [1.2836/0.9]\}}$

$= 1.2687 - 0.4281$

$= 0.8405$

$F'_c = F_c^*(C_P) = (1930.85\ psi)(0.8405) = 1622.947\ psi > 825.374\ psi\ \therefore$ OK

Axial stress ratio = $f_c/F'_c = 825.374/1622.9472 = 0.5086$

Net Section Check:

Assume connections will be made with (2) rows of ¾" diameter bolts.

Assume the hole diameter is 1/16" larger than the bolt (for stress calculations only).

A_n = (6.75")[11" – (2)(0.8125")] = 63.281 in^2

(3/4" + 1/16" = 0.8125")

$f_c = P/A_n$ = 61,284 lb/63.281 in^2 = 968.442 psi

At braced location there is no reduction for stability.

$F'_c = F_c^* = F_c(C_D)(C_M)(C_t)$ = (2300 psi)(1.15)(0.73)(1.0) = 1930.85 psi

1930.85 psi > 968.442 psi ∴ OK

Bending:

Bending is about the strong axis of the cross section. The adjusted bending design value for a glulam is governed by the smaller of two criteria: volume effect or lateral stability. In this case, the beam has full lateral support. Therefore, l_u and R_B are zero and the lateral stability factor is C_L = 1.0.

M = 117.342 in-kips = 117,342 in-lb

S = 136.1 in^3

$f_b = M/S$ = 117,342 in-lb/136.1 in^3 = 862.175 psi

$F'_b = F_b(C_D)(C_M)(C_t)(C_L)$ or

$F'_b = F_b(C_D)(C_M)(C_t)(C_V)$

For Southern Pine glulam:

$C_V = (21'/L)^{1/20}(12"/d)^{1/20}(5.125"/b)^{1/20} \leq 1.0$

$C_V = (21'/15.0833')^{1/20}(12"/11")^{1/20}(5.125"/6.75")^{1/20} \leq 1.0$

C_V = 1.0072 ≤ 1.0

∴ C_V = 1.0

$F'_b = F_b(C_D)(C_M)(C_t)(C_L \text{ or } C_V)$ = (2100 psi)(1.15)(0.8)(1.0)(1.0) = 1932 psi

Bending stress ratio = f_b/F'_b = 862.175/1932 = 0.4463

Combined Stresses:

The bending moment is about the strong axis of the cross section, and the amplification for P-Δ is measured by the column slenderness ratio about the x axis.

$(l_e/d)_{bending\ moment} = (l_e/d)_x =$ 16.4545

$F_{cEx} = [0.822E'_{min}]/[(l_e/d)_x]^2 = [(0.822)(816{,}340 \text{ psi})]/[(16.4545)^2] = 2478.398$ psi

*Here, (l_e/d) is based on the axis about which the bending moment occurs.
Amplification factor = $1/[1 - (f_c/F_{cEx})] = 1/[1 - (968.442/2478.398)] = 1.6414$

$(f_c/F'_c)^2 + \{1/[1 - (f_c/F_{cEx})]\}(f_b/F'_b) = (0.5086)^2 + (1.6414)(0.4463) = 0.9912 < 1.0 \therefore$ OK

To be a little more conservative, use a slightly larger member.

Check Shear:

$f_v = 1.5(V/A) = (1.5)[(2759 \text{ lb})/(74.25 \text{ in}^2) = 37.158$ psi

$F_v = 300$ psi

$F'_v = F_v(C_D)(C_M)(C_t) = (300 \text{ psi})(1.15)(0.875)(1.0) = 301.875 \text{ psi} > 37.158 \text{ psi} \therefore$ **OK**

Try 6 ¾” x 12 3/8”

$F_c = 2300$ psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

$F_b = 2100$ psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

$A = 83.53 \text{ in}^2$

$S = 172.3 \text{ in}^3$

$E_{min} = 980{,}000$ psi

Load Combination: D + S

Axial Load: P = 61.284 kips (Compression) (from SAP2000)

Maximum Moment = 9.779 ft-kips = 117.342 in-kips (from SAP2000)

L = 15’-1” = 15.083333’

Axial Load:

$f_c = P/A = 61{,}284 \text{ lb}/83.53 \text{ in}^2 = 733.677$ psi

$(l_e/d)_x = [(15.0833')(12 \text{ in/ft})]/12.375'' = 14.6263 < 50 \therefore$ OK

$(l_e/d)_y = 0$ because of lateral support provided by roof diaphragm

$(l_e/d)_{max} = (l_e/d)_x = 14.6263$

The larger slenderness ratio governs the adjust design value. Therefore, the strong axis of the member is critical, and $(l_e/d)_x$ is used to determine F'_c.

E'_{min} = 816,340 psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340\text{ psi})]/[(14.6263)^2] = 3136.723\text{ psi}$

Here, l_e/d is based on $(l_e/d)_{max}$.

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300\text{ psi})(1.15)(0.73)(1.0) = 1930.85\text{ psi}$

$F_{cE}/F_c^* = 3136.7229/1930.85 = 1.6245$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 1.6245]/[(2)(0.9)] = 1.4581$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{1.4581\} - \sqrt{\{[1.4581]^2 - [1.6245/0.9]\}}$

$= 1.4581 - 0.5665$

$= 0.8916$

$F'_c = F_c^*(C_P) = (1930.85\text{ psi})(0.8916) = 1721.460\text{ psi} > 733.677 \therefore$ OK

Axial stress ratio = $f_c/F'_c = 733.677/1721.460 = 0.4262$

Net Section Check:

Assume connections will be made with (2) rows of ¾" diameter bolts.

Assume the hole diameter is 1/16" larger than the bolt (for stress calculations only).

$A_n = (6.75")[12.375" - (2)(0.8125")] = 72.5625\text{ in}^2$

(3/4" + 1/16" = 0.8125")

$f_c = P/A_n = 61{,}284\text{ lb}/72.5625\text{ in}^2 = 844.568\text{ psi}$

At braced location there is no reduction for stability.

$F'_c = F_c^* = F_c(C_D)(C_M)(C_t) = (2300\text{ psi})(1.15)(0.73)(1.0) = 1930.85\text{ psi}$

1930.85 psi > 844.568 psi $\therefore$ OK

Bending:

Bending is about the strong axis of the cross section. The adjusted bending design value for a glulam is governed by the smaller of two criteria: volume effect or lateral stability. In this case,

the beam has full lateral support. Therefore, l_u and R_B are zero and the lateral stability factor is $C_L = 1.0$.

M = 117.342 in-kips = 117,342 in-lb

$S = 172.3\ in^3$

$f_b = M/S = 117{,}342\ in\text{-}lb/172.3\ in^3 = 681.033\ psi$

$F'_b = F_b(C_D)(C_M)(C_t)(C_L)$ or

$F'_b = F_b(C_D)(C_M)(C_t)(C_V)$

For Southern Pine glulam:

$C_V = (21'/L)^{1/20}(12''/d)^{1/20}(5.125''/b)^{1/20} \leq 1.0$

$C_V = (21'/15.0833')^{1/20}(12''/12.375'')^{1/20}(5.125''/6.75'')^{1/20} \leq 1.0$

$C_V = 1.0012 \leq 1.0$

$\therefore C_V = 1.0$

$F'_b = F_b(C_D)(C_M)(C_t)(C_L \text{ or } C_V) = (2100\ psi)(1.15)(0.8)(1.0)(1.0) = 1932\ psi$

$> 681.033\ psi \therefore$ OK

Bending stress ratio = $f_b/F'_b = 681.033/1932 = 0.3525$

Combined Stresses:

The bending moment is about the strong axis of the cross section, and the amplification for P-Δ is measured by the column slenderness ratio about the x axis.

$(l_e/d)_{bending\ moment} = (l_e/d)_x = 14.62626263$

$F_{cEx} = [0.822E'_{min}]/[(l_e/d)_x]^2 = [(0.822)(816{,}340\ psi)]/[(14.6262)^2] = 3136.723\ psi$

*Here, (l_e/d) is based on the axis about which the bending moment occurs.

Amplification factor = $1/[1 - (f_c/F_{cEx})] = 1/[1 - (733.677/3136.723)] = 1.3053$

$(f_c/F'_c)^2 + \{1/[1 - (f_c/F_{cEx})]\}(f_b/F'_b) = (0.4262)^2 + (1.3053)(0.3525) = 0.6418 < 1.0 \therefore$ OK

Check Shear:

$f_v = 1.5(V/A) = (1.5)[(2759\ lb)/(83.53\ in^2) = 49.545\ psi$

$F_v = 300\ psi$

$F'_v = F_v(C_D)(C_M)(C_t)$ = (300 psi)(1.15)(0.875)(1.0) = 301.875 psi > 49.545psi ∴ **OK**

USE 6 ¾" x 12 3/8"

LOAD COMBINATIOIN: D + L_r

<u>Try 6 ¾" x 12 3/8"</u>

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

F_b = 2100 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

A = 83.53 in^2

S = 172.3 in^3

E_{min} = 980,000 psi

Axial Load: P = 58.247 kips (Compression)

Maximum Moment = 9.208 ft-kips = 110.496 in-kips

L = 15'-1" = 15.083333'

Axial Load:

f_c = P/A = 58,247 lb/83.53 in^2 = 697.318 psi

$(l_e/d)_x$ = [(15.0833')(12 in/ft)]/12.375" = 14.6263 < 50 ∴ OK

$(l_e/d)_y$ = 0 because of lateral support provided by roof diaphragm

$(l_e/d)_{max} = (l_e/d)_x$ = 14.6263

The larger slenderness ratio governs the adjust design value. Therefore, the strong axis of the member is critical, and $(l_e/d)_x$ is used to determine F'_c.

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

E_{min} = 980,000 psi

C_D = 1.0 (for live load; load combination D + L_r)

C_M = 0.73 for F_c (p. 64, NDS Supplement)

C_M = 0.833 for E and E_{min} (p. 64, NDS Supplement)

C_M = 0.8 for F_b (p. 64, NDS Supplement)

C_t = 1.0

$E'_{min} = (E_{min})(C_M)(C_t) = (980{,}000)(0.833)(1.0) = 816{,}340$ psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340 \text{ psi})]/[(14.6263)^2] = 3136.723$ psi

Here, l_e/d is based on $(l_e/d)_{max}$.

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.0)(0.73)(1.0) = 1679$ psi

$F_{cE}/F_c^* = 3136.723/1679 = 1.8682$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 1.8682]/[(2)(0.9)] = 1.5934$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{1.5934\} - \sqrt{\{[1.5934]^2 - [1.8682/0.9]\}}$

$= 1.5934 - 0.6807$

$= 0.9128$

$F'_c = F_c^*(C_P) = (1679 \text{ psi})(0.9128) = 1532.579 \text{ psi} > 697.318 \text{ psi} \therefore$ OK

Axial stress ratio = $f_c/F'_c = 697.318/1532.579 = 0.4550$

Net Section Check:

Assume connections will be made with (2) rows of ¾" diameter bolts.

Assume the hole diameter is 1/16" larger than the bolt (for stress calculations only).

$A_n = (6.75")[12.375" - (2)(0.8125")] = 72.5625 \text{ in}^2$

(3/4" + 1/16" = 0.8125")

$f_c = P/A_n = 58{,}247 \text{ lb}/72.5625 \text{ in}^2 = 802.715$ psi

At braced location there is no reduction for stability.

$F'_c = F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.0)(0.73)(1.0) = 1679$ psi

$1679 \text{ psi} > 802.715 \text{ psi} \therefore$ OK

Bending:

Bending is about the strong axis of the cross section. The adjusted bending design value for a glulam is governed by the smaller of two criteria: volume effect or lateral stability. In this case, the beam has full lateral support. Therefore, l_u and R_B are zero and the lateral stability factor is $C_L = 1.0$.

M = 110.496 in-kips = 110,496 in-lb

S = 172.3 in^3

f_b = M/S = 110,496 in-lb/172.3 in^3 = 641.300 psi

$F'_b = F_b(C_D)(C_M)(C_t)(C_L)$ or

$F'_b = F_b(C_D)(C_M)(C_t)(C_V)$

For Southern Pine glulam:

$C_V = (21'/L)^{1/20}(12''/d)^{1/20}(5.125''/b)^{1/20} \leq 1.0$

$C_V = (21'/15.0833')^{1/20}(12''/12.375'')^{1/20}(5.125''/6.75'')^{1/20} \leq 1.0$

$C_V = 1.0012 \leq 1.0$

$\therefore C_V = 1.0$

$F'_b = F_b(C_D)(C_M)(C_t)(C_L \text{ or } C_V)$ = (2100 psi)(1.0)(0.8)(1.0)(1.0) = 1680 psi

> 641.300 psi $\therefore$ OK

Bending stress ratio = f_b/F'_b = 641.300/1680 = 0.3817

Combined Stresses:

The bending moment is about the strong axis of the cross section, and the amplification for P-Δ is measured by the column slenderness ratio about the x axis.

$(l_e/d)_{\text{bending moment}} = (l_e/d)_x = 14.6263$

$F_{cEx} = [0.822E'_{min}]/[(l_e/d)_x]^2 = [(0.822)(816{,}340 \text{ psi})]/[(14.6263)^2] = 3136.723$ psi

*Here, (l_e/d) is based on the axis about which the bending moment occurs.

Amplification factor = $1/[1 - (f_c/F_{cEx})] = 1/[1 - (697.318/3136.723)] =$

= 1.2859

$(f_c/F'_c)^2 + \{1/[1 - (f_c/F_{cEx})]\}(f_b/F'_b) = (0.4550)^2 + (1.2859)(0.3817) = 0.6978 < 1.0$ $\therefore$ **OK**

CONTROLS OVER "D + S"

Check Shear:

f_v = 1.5(V/A) = (1.5)[(2,598 lb)/(83.53 in^2) = 46.654 psi

F_v = 300 psi

$F'_v = F_v(C_D)(C_M)(C_t)$ = (300 psi)(1.0)(0.875)(1.0) = 262.5 psi > 46.654 psi ∴ OK

USE 6 ¾" x 12 3/8"

LOAD COMBINATION: D

Try 6 ¾" x 12 3/8"

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

F_b = 2100 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

A = 83.53 in^2

S = 172.3 in^3

E_{min} = 980,000 psi

Axial Load: P = 38.648 kips (Compression)

Maximum Moment = 5.525 ft-kips = 66.30 in-kips

L = 15'-1" = 15.083333'

Axial Load:

f_c = P/A = 38,648 lb/83.53 in^2 = 462.684 psi

$(l_e/d)_x$ = [(15.0833')(12 in/ft)]/12.375" = 14.6263 < 50 ∴ OK

$(l_e/d)_y$ = 0 because of lateral support provided by roof diaphragm

$(l_e/d)_{max} = (l_e/d)_x$ = 14.6263

The larger slenderness ratio governs the adjust design value. Therefore, the strong axis of the member is critical, and $(l_e/d)_x$ is used to determine F'_c.

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

E_{min} = 980,000 psi

C_D = 0.9 (for dead load; load combination D)

C_M = 0.73 for F_c (p. 64, NDS Supplement)

C_M = 0.833 for E and E_{min} (p. 64, NDS Supplement)

C_M = 0.8 for F_b (p. 64, NDS Supplement)

$C_t = 1.0$

$E'_{min} = (E_{min})(C_M)(C_t) = (980{,}000)(0.833)(1.0) = 816{,}340$ psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340 \text{ psi})]/[(14.6263)^2] = 3136.723$ psi

Here, l_e/d is based on $(l_e/d)_{max}$.

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(0.9)(0.73)(1.0) = 1511.1$ psi

$F_{cE}/F_c^* = 3136.723/1511.1 = 2.0758$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 2.0758]/[(2)(0.9)] = 1.7088$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{1.7088\} - \sqrt{\{[1.7088]^2 - [2.0758/0.9]\}}$

$= 1.7088 - 0.7832$

$= 0.9255$

$F'_c = F_c^*(C_P) = (1511.1 \text{ psi})(0.9255) = 1398.581 \text{ psi} > 462.684 \text{ psi} \therefore$ OK

Axial stress ratio = $f_c/F'_c = 462.684/1398.5805 = 0.3308$

Net Section Check:

Assume connections will be made with (2) rows of ¾" diameter bolts.

Assume the hole diameter is 1/16" larger than the bolt (for stress calculations only).

$A_n = (6.75")[12.375" - (2)(0.8125")] = 72.5625 \text{ in}^2$

(3/4" + 1/16" = 0.8125")

$f_c = P/A_n = 38{,}648 \text{ lb}/72.5625 \text{ in}^2 = 532.617$ psi

At braced location there is no reduction for stability.

$F'_c = F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(0.9)(0.73)(1.0) = 1511.1$ psi

$1511.1 \text{ psi} > 532.617 \text{ psi} \therefore$ OK

Bending:

Bending is about the strong axis of the cross section. The adjusted bending design value for a glulam is governed by the smaller of two criteria: volume effect or lateral stability. In this case,

the beam has full lateral support. Therefore, l_u and R_B are zero and the lateral stability factor is $C_L = 1.0$.

$M = 66.30$ in-kips $= 66{,}300$ in-lb

$S = 172.3\ in^3$

$f_b = M/S = 66{,}300\ in\text{-}lb/172.3\ in^3 = 384.794$ psi
$F'_b = F_b(C_D)(C_M)(C_t)(C_L)$ or

$F'_b = F_b(C_D)(C_M)(C_t)(C_V)$

For Southern Pine glulam:

$C_V = (21'/L)^{1/20}(12''/d)^{1/20}(5.125''/b)^{1/20} \leq 1.0$

$C_V = (21'/15.0833')^{1/20}(12''/12.375'')^{1/20}(5.125''/6.75'')^{1/20} \leq 1.0$

$C_V = 1.0012 \leq 1.0$

$\therefore\ C_V = 1.0$

$F'_b = F_b(C_D)(C_M)(C_t)(C_L \text{ or } C_V) = (2100\ psi)(0.9)(0.8)(1.0)(1.0) = 1512$ psi

> 384.794 psi $\therefore$ OK

Bending stress ratio $= f_b/F'_b = 384.794/1512 = 0.2545$

Combined Stresses:

The bending moment is about the strong axis of the cross section, and the amplification for P-Δ is measured by the column slenderness ratio about the x axis.

$(l_e/d)_{bending\ moment} = (l_e/d)_x = 14.6263$

$F_{cEx} = [0.822E'_{min}]/[(l_e/d)_x]^2 = [(0.822)(816{,}340\ psi)]/[(14.6263)^2] = 3136.723$ psi

*Here, (l_e/d) is based on the axis about which the bending moment occurs.

Amplification factor $= 1/[1 - (f_c/F_{cEx})] = 1/[1 - (462.684/3136.723)] = 1.1730$

$(f_c/F'_c)^2 + \{1/[1 - (f_c/F_{cEx})]\}(f_b/F'_b) = (0.3308)^2 + (1.1730)(0.2545) = 0.4080 < 1.0$ $\therefore$ **OK**

Check Shear:

$f_v = 1.5(V/A) = (1.5)[(1{,}559\ lb)/(83.53\ in^2)) = 27.996$ psi

$F_v = 300$ psi

$F'_v = F_v(C_D)(C_M)(C_t)$ = (300 psi)(0.9)(0.875)(1.0) = 236.25 psi > 27.996 psi ∴ OK

DOES NOT CONTROL

*Make Members 20, 21, 22, and 23 the same size cross section as Member 19 so that the entire top chord of the truss is the same size cross-section (the member size used for Member 19 will work for Members 20, 21, 22, and 23 since Members 20, 21, 22, and 23 are shorter in length and are required to carry less axial load than Member 19)

FINAL MEMBER SIZE = 6 ¾" x 12 3/8" Southern Pine Glulam I.D. #50

Bottom Chord: Combined Tension and Bending Forces (Members 3 and 4 are worst case)

LOAD COMBINATION: D + S

Axial Load: P = 53.974 kips (Tension)

Moment = 1.656 ft-kips = 19.872 in-kips = 19,872 in-lb (due to Dead Load)

Try d = 6 3/4" = 6.75" (same width as top chord members)

Axial Tension:

F_t = 1550 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

C_D = 1.15 (for snow load; load combination D+S)

C_M = 0.8 for F_t (p. 64, NDS Supplement)

C_t = 1.0

$F'_t = F_t(C_D)(C_M)(C_t)$ = (1550 psi)(1.15)(0.8)(1.0) = 1426 psi

$P = (F'_t)(A)$

Req'd $A_n = P/F'_t$ = 53,974 lb/1426 psi = 37.850 in^2

Assume (2) rows of ¾" diameter bolts.

Req'd $A_g = A_n + A_h$ = 37.850 in^2 + (6.75")[(2)(3/4" + 1/16")] = 48.819 in^2

Try 6 ¾" x 8 ¼" (A = 55.69 in^2 > 48.819 in^2 ∴OK)

A_n = 55.69 in^2 - (6.75")[(2)(3/4" + 1/16")] = 44.721 in^2

$f_t = T/A_n$ = (53,974 lb)/(44.721 in^2) = 1206.898 psi < 1426 psi ∴ OK

Determine tension stress at the point of maximum bending stress (midspan) for use in the interaction formula.

$f_t = T/A_g = 53{,}974$ lb/55.69 in^2 = 969.187 psi < 1426 psi $\therefore$ OK

Bending:

$S_x = 76.57$ in^3

$f_b = M/S$ = (19,872 in-lb)/(76.57 in^3) = 259.527 psi

$F'_b = F_b(C_D)(C_M)(C_t)(C_L)$ or

$F'_b = F_b(C_D)(C_M)(C_t)(C_V)$

For C_L: l_u/d = [(13.0')(12 in/ft)]/8.25" = 18.909 > 7

$\therefore$ $l_e = 1.63l_u + 3d$ = (1.63)[(13.0')(12 in/ft)] + (3)(8.25") = 279.03"

$R_B = \sqrt{l_e d/b^2}$ = √ [(279.03")(8.25")/(6.75")^2] = 7.1080

$F_{bE} = 1.20E'_{min}/R_B^2$ = [(1.20)(816,340 psi)]/(7.1080)^2 = 19,388.98 psi

$F^*_b = F_b(C_D)(C_M)(C_t)$ = (2100 psi)(1.15)(0.8)(1.0) = 1932 psi

F_{bE}/F^*_b = (19,388.98)/(1932) = 10.0357

$(1 + F_{bE}/F^*_b)/1.9$ = (1 + 10.0357)/1.9 = 5.8083

$C_L = [(1 + F_{bE}/F^*_b)/1.9] - \sqrt{\{[(1 + F_{bE}/F^*_b)/1.9]^2 - [F_{bE}/F^*_b/0.95]\}}$

= 5.8083 - √(5.8083)^2 – (10.0357/0.95)] = 0.9946

For Southern Pine glulam:

$C_V = (21'/L)^{1/20}(12"/d)^{1/20}(5.125"/b)^{1/20} \leq 1.0$

$C_V = (21'/13.0')^{1/20}(12"/8.25")^{1/20}(5.125"/6.75")^{1/20} \leq 1.0$

$C_V = 1.0294 \leq 1.0 \therefore C_V = 1.0$

C_L controls over C_V

$F^*_b = F'_b = F_b(C_D)(C_M)(C_t)(C_L)$ = (2100 psi)(1.15)(0.8)(1.0)(0.9946) = 1921.567 psi

> 259.527 psi $\therefore$ OK

Bending stress ratio = f_b/F'_b = (259.527 psi)/(1921.567 psi) = 0.1351

Combined Stresses:

$(f_t/F'_t) + (f_{bx}/F^*_{bx}) = (969.187/1426 \text{ psi}) + (259.527/1921.567) = 0.8147 < 1.0 \therefore$ **OK**

Check Shear:

$f_v = 1.5(V/A) = (1.5)[(520 \text{ lb})/(55.69 \text{ in}^2) = 14.006 \text{ psi}$

$F_v = 300 \text{ psi}$

$F'_v = F_v(C_D)(C_M)(C_t) = (300 \text{ psi})(1.15)(0.875)(1.0) = 301.875 \text{ psi} > 14.006 \text{ psi} \therefore$ OK

LOAD COMBINATION: D + L_r

Try 6 ¾" x 8 ¼"

Axial Load: P = 51.315 kips (Tension)

Moment = 1.656 ft-kips = 19.872 in-kips = 19,872 in-lb (due to Dead Load)

$A = 55.69 \text{ in}^2$

$S_x = 76.57 \text{ in}^3$

Axial Tension:

Assume (2) rows of ¾" diameter bolts.

$A_n = 55.69 \text{ in}^2 - (6.75")[(2)(3/4" + 1/16")] = 44.721 \text{ in}^2$

$f_t = T/A_n = (51{,}315 \text{ lb})/(44.721 \text{ in}^2) = 1147.448 \text{ psi}$

$F_t = 1550$ psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

$C_D = 1.0$ (for live load; load combination D + L_r)

$C_M = 0.8$ for F_t (p. 64, NDS Supplement)

$C_t = 1.0$

$F'_t = F_t(C_D)(C_M)(C_t) = (1550 \text{ psi})(1.0)(0.8)(1.0) = 1240 \text{ psi} > 1147.448 \text{ psi} \therefore$ OK

Determine tension stress at the point of maximum bending stress (midspan) for use in the interaction formula.

$f_t = T/A_g = 51{,}315 \text{ lb}/55.69 \text{ in}^2 = 921.440 \text{ psi} < 1240 \text{ psi} \therefore$ OK

Bending:

f_b = M/S = (19,872 in-lb)/(76.57 in^3) = 259.527 psi

$F'_b = F_b(C_D)(C_M)(C_t)(C_L)$ or

$F'_b = F_b(C_D)(C_M)(C_t)(C_V)$

For C_L: l_u/d = [(13.0')(12 in/ft)]/8.25" = 18.909 > 7

$\therefore$ $l_e = 1.63l_u + 3d$ = (1.63)[(13.0')(12 in/ft)] + (3)(8.25") = 279.03"

$R_B = \sqrt{l_e d/b^2}$ = √ [(279.03")(8.25")/(6.75")^2] = 7.1080

$F_{bE} = 1.20E'_{min}/R_B{}^2$ = [(1.20)(816,340 psi)]/(7.1080)^2 = 19,388.98 psi

$F^*_b = F_b(C_D)(C_M)(C_t)$ = (2100 psi)(1.15)(0.8)(1.0) = 1932 psi

F_{bE}/F^*_b = (19,388.98)/(1932) = 10.0357

$(1 + F_{bE}/F^*_b)/1.9$ = (1 + 10.0357)/1.9 = 5.8083

$C_L = [(1 + F_{bE}/F^*_b)/1.9] - \sqrt{\{[(1 + F_{bE}/F^*_b)/1.9]^2 - [F_{bE}/F^*_b/0.95]\}}$

= 5.8083 - √(5.8083)^2 – (10.0357/0.95)] = 0.9946

For Southern Pine glulam:

$C_V = (21'/L)^{1/20}(12"/d)^{1/20}(5.125"/b)^{1/20} \leq 1.0$

$C_V = (21'/13.0')^{1/20}(12"/8.25")^{1/20}(5.125"/6.75")^{1/20} \leq 1.0$

C_V = 1.0294 ≤ 1.0 $\therefore$ C_V = 1.0

C_L controls over C_V

$F^*_b = F'_b = F_b(C_D)(C_M)(C_t)(C_L)$ = (2100 psi)(1.0)(0.8)(1.0)(0.9946) = 1670.928 psi

> 259.527 psi $\therefore$ OK

Bending stress ratio = f_{bx}/F^*_{bx} = (259.527 psi)/(1670.928 psi) = 0.1553

Combined Stresses:

$(f_t/F'_t) + (f_{bx}/F^*_{bx})$ = (921.440/1240) + (259.527/1670.928) = 0.8984 < 1.0 $\therefore$ OK

CONTROLS OVER LOAD COMBINATION "D + S"

Check Shear:

f_v = 1.5(V/A) = (1.5)[(520 lb)/(55.69 in^2) = 14.006 psi

F_v = 300 psi

$F'_v = F_v(C_D)(C_M)(C_t)$ = (300 psi)(1.0)(0.875)(1.0) = 262.5 psi > 14.006 psi ∴ **OK**

LOAD COMBINATION: D

Try 6 ¾" x 8 ¼"

Axial Load: P = 34.160 kips (Tension)

Moment = 1.656 ft-kips = 19.872 in-kips = 19,872 in-lb (due to Dead Load)

A = 55.69 in^2

S_x = 76.57 in^3

Axial Tension:
Assume (2) rows of ¾" diameter bolts.

A_n = 55.69 in^2 - (6.75")[(2)(3/4" + 1/16")] = 44.721 in^2

$f_t = T/A_n$ = (34,160 lb)/(44.721 in^2) = 763.847 psi

F_t = 1550 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

C_D = 0.9 (for dead load; load combination D)

C_M = 0.8 for F_t (p. 64, NDS Supplement)

C_t = 1.0

$F'_t = F_t(C_D)(C_M)(C_t)$ = (1550 psi)(0.9)(0.8)(1.0) = 1116 psi > 763.847 psi ∴ OK
Determine tension stress at the point of maximum bending stress (midspan) for use in the interaction formula.

$f_t = T/A_g$ = 34,160 lb/55.69 in^2 = 613.396 psi < 1116 psi ∴ OK

Bending:

f_b = M/S = (19,872 in-lb)/(76.57 in^3) = 259.527 psi

$F'_b = F_b(C_D)(C_M)(C_t)(C_L)$ or

$F'_b = F_b(C_D)(C_M)(C_t)(C_V)$

C_L = 0.9946

For Southern Pine glulam: C_V = 1.0294 ≤ 1.0 ∴ C_V = 1.0

C_L controls over C_V

$F^*_b = F'_b = F_b(C_D)(C_M)(C_t)(C_L)$ = (2100 psi)(0.9)(0.8)(1.0)(0.9946) = 1503.835 psi

> 259.527 psi ∴ OK

Bending stress ratio = f_{bx}/F^*_{bx} = (259.527 psi)/(1503.835 psi) = 0.1726

Combined Stresses:

$(f_t/F'_t) + (f_{bx}/F^*_{bx})$ = (763.847/1116) + (259.527/1503.835) = 0.8570 < 1.0 ∴ **OK**

Check Shear:

$f_v = 1.5(V/A)$ = (1.5)[(520 lb)/(55.69 in^2) = 14.006 psi

F_v = 300 psi

$F'_v = F_v(C_D)(C_M)(C_t)$ = (300 psi)(0.9)(0.875)(1.0) = 236.25 psi > 14.006 psi ∴ OK

DOES NOT CONTROL

*Use same member size for all bottom chord members (for consistency); the member size used for Member 6 will work for the rest of the bottom chord members since the axial (tensile) force in each of these other bottom chord members is less than the axial tensile force in Member 6.

FINAL MEMBER SIZE = 6 ¾” x 8 ¼” Southern Pine Glulam ID #50

<u>*Member 24 in SAP2000:*</u>

Load Combination: D + S

Axial Load: P = 0.262 kips (Compression)

L = 20’-0” = 20.0’

$(l_e/d)_{max}$ = 50

$d \geq l_e/50$ = [(20’)(12 in/ft)]/50 = 4.8”

Try d = 6 3/4” = 6.75”

(l_e/d) = [(20.0’)(12 in/ft)]/6.75” = 35.556 < 50 ∴ OK

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66, NDS Supplement)

E_{min} = 980,000 psi

C_D = 1.15 (for snow load; load combination D+S)

C_M = 0.73 for F_c (p. 64, NDS Supplement)

C_M = 0.833 for E and E_{min} (p. 64, NDS Supplement)

C_t = 1.0

$E'_{min} = (E_{min})(C_M)(C_t) = (980,000)(0.833)(1.0) = 816,340$ psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816,340 \text{ psi})]/[(35.5556)^2] = 530.7963854$ psi

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.15)(0.73)(1.0) = 1930.85$ psi

$F_{cE}/F_c^* = 530.7964/1930.85 = 0.2749029626$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.2749]/[(2)(0.9)] = 0.7082794237$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.7082794237\} - \sqrt{\{[0.7082794237]^2 - [0.2749/0.9]\}}$

$= 0.7082794237 - 0.4429582438$

$= 0.2653211799$

$F'_c = F_c^*(C_P) = (1930.85 \text{ psi})(0.2653) = 512.2954001$ psi

$P = (F'_c)(A)$
$A_{req'd} = P/F'_c = 262 \text{ lb}/512.2954 \text{ psi} = 0.511424 \text{ in}^2$

Use 6 ¾" x 6 7/8" (A = 46.41 in² > 0.51 in² ∴ OK)

*Must use width of 6 ¾" to match that of the top and bottom chord members (need to keep consistent width of members for side plates (for connections for truss members))

*Other load combinations of "D" and "D + L_r" will not require a larger size member since load is so small; width of member must be ≥ 4.8" to meet $l_e/d \le 50$, which results in a members whose capacity is much greater than the required load it must carry

Member 32 in SAP2000:

Tension member

Very small axial force

Use 6 ¾" x 6 7/8" (minimum size with d = 6 ¾")

All web members forces are considerably small:

∴ **Use 6 ¾" x 6 7/8"** for all web members (minimum size to maintain same width as top and bottom chord members)

Member 1 (Member 1 in SAP2000 as well): Column

LOAD COMBINATION: D + S

Axial Load: P = 33.764 kips (Compression)

Analyze Column Buckling About x Axis:

$(l_e/d)_{max} = 50$

$(l_e/d)_x = [(1.0)(40.0')(12 \text{ in/ft})]/d \leq 50$

$d \geq l_e/50 = [(40.0')(12 \text{ in/ft})]/50 = 9.6"$

Analyze Column Bucking About y Axis:

Braced at the third-points (L = 40.0'/3 = 13.3333')

$(l_e/d)_{max} = 50$

$(l_e/d)_y = [(1.0)(13.333')(12 \text{ in/ft})]/d \leq 50$

$d \geq l_e/50 = [(13.3333')(12 \text{ in/ft})]/50 = 3.2"$

Try d = 6 3/4" = 6.75" (to match "d" of truss members)

$(l_e/d)_y = [(13.3333')(12 \text{ in/ft})]/6.75" = 23.7037037$

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

E_{min} = 980,000 psi

C_D = 1.15 (for snow load; load combination D+S)

C_M = 0.73 for F_c (p. 64, NDS Supplement)

C_M = 0.833 for E and E_{min} (p. 64, NDS Supplement)

C_t = 1.0

$E'_{min} = (E_{min})(C_M)(C_t)$ = (980,000)(0.833)(1.0) = 816,340 psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2]$ = [(0.822)(816,340 psi)]/[(23.7037037)^2] = 1194.291867 psi

$F_c^* = F_c(C_D)(C_M)(C_t)$ = (2300 psi)(1.15)(0.73)(1.0) = 1930.85 psi

F_{cE}/F_c^* = 1194.2919/1930.85 = 0.6185316661

$[1 + F_{cE}/F_c^*]/(2c)$ = [1 + 0.6185]/[(2)(0.9)] = 0.8991942589

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.8991942589\} - \sqrt{\{[0.8991942589]^2 - [0.6185/0.9]\}}$

= 0.8991942589 – 0.3482454949

= 0.550948764

$F'_c = F_c^*(C_P)$ = (1930.85 psi)(0.5509) = 1063.799421 psi

$P = (F'_c)(A)$

$A_{req'd} = P/F'_c$ = 33,764 lb/1063.7994 psi = 31.739 in^2

Use 6 ¾” x 8 ¼” (A = 55.69 in^2 > 31.74 in^2 ∴OK)

However, 8 ½” < 9.6” (required dimension to prevent buckling about x axis)

Try 6 ¾” x 9 5/8” (A = 64.97 in^2 > 31.74 in^2 ∴ OK)

Check Column Dimensions:

$(l_e/d)_x$ = [(1.0)(40.0’)(12 in/ft)]/9.625 = 49.8701 ≤ 50 ∴ OK [controls over $(l_e/d)_y$]

$(l_e/d)_y$ = [(1.0)(13.333’)(12 in/ft)]/6.75 = 23.7037 ≤ 50 ∴ OK

Analyze Column Buckling About x Axis:

$(l_e/d)_x$ = [(1.0)(40.0’)(12 in/ft)]/9.625 = 49.8701

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

E_{min} = 980,000 psi

C_D = 1.15 (for snow load; load combination D+S)

C_M = 0.73 for F_c (p. 64, NDS Supplement)

C_M = 0.833 for E and E_{min} (p. 64, NDS Supplement)

C_t = 1.0

$E'_{min} = (E_{min})(C_M)(C_t) = (980{,}000)(0.833)(1.0) = 816{,}340$ psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340\text{ psi})]/[(49.87012987)^2] = 269.812$ psi

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300\text{ psi})(1.15)(0.73)(1.0) = 1930.85$ psi
$F_{cE}/F_c^* = 269.812/1930.85 = 0.1397$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.1397]/[(2)(0.9)] = 0.6332$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.6332\} - \sqrt{\{[0.6332]^2 - [0.1397/0.9]\}}$

$= 0.6332 - 0.4956$

$= 0.1375$

$F'_c = F_c^*(C_P) = (1930.85\text{ psi})(0.1375) = 265.5770$ psi

$P = (F'_c)(A)$

$A_{req'd} = P/F'_c = 33{,}764\text{ lb}/265.5770\text{ psi} = 127.135\text{ in}^2$

$A = 64.97\text{ in}^2 < 127.135\text{ in}^2$ ∴ **NO GOOD**

Try 6 ¾" x 16 ½" (A = 111.4 in^2)

$(l_e/d)_x = [(1.0)(40.0')(12\text{ in/ft})]/16.5" = 29.0909$ [controls over $(l_e/d)_y$]

$(l_e/d)_y = [(1.0)(13.333')(12\text{ in/ft})]/6.75 = 23.7037$

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

E_{min} = 980,000 psi

C_D = 1.15 (for snow load; load combination D+S)

C_M = 0.73 for F_c (p. 64, NDS Supplement)

C_M = 0.833 for E and E_{min} (p. 64, NDS Supplement)

C_t = 1.0

$E'_{min} = (E_{min})(C_M)(C_t)$ = (980,000)(0.833)(1.0) = 816,340 psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2]$ = [(0.822)(816,340 psi)]/[(29.0909)^2] = 792.918 psi

$F_c^* = F_c(C_D)(C_M)(C_t)$ = (2300 psi)(1.15)(0.73)(1.0) = 1930.85 psi

F_{cE}/F_c^* = 792.918/1930.85 = 0.4107

$[1 + F_{cE}/F_c^*]/(2c)$ = [1 + 0.4107]/[(2)(0.9)] = 0.7837

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$
= $\{0.7837\} - \sqrt{\{[0.7837]^2 - [0.4107/0.9]\}}$

= 0.7837 – 0.3974

= 0.3863

$F'_c = F_c^*(C_P)$ = (1930.85 psi)(0.3863) = 745.956 psi

$P = (F'_c)(A)$

$A_{req'd} = P/F'_c$ = 33,764 lb/745.956 psi = 45.263 in^2

A = 111.4 in^2 > 45.263 in^2 $\therefore$ **OK**

Try 6 ¾" x 15 1/8" (A = 102.1 in^2)

$(l_e/d)_x$ = [(1.0)(40.0')(12 in/ft)]/15.125" = 31.7355 [controls over $(l_e/d)_y$]

$(l_e/d)_y$ = [(1.0)(13.333')(12 in/ft)]/6.75 = 23.7037

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

E_{min} = 980,000 psi

C_D = 1.15 (for snow load; load combination D+S)

C_M = 0.73 for F_c (p. 64, NDS Supplement)

C_M = 0.833 for E and E_{min} (p. 64, NDS Supplement)

C_t = 1.0

$E'_{min} = (E_{min})(C_M)(C_t)$ = (980,000)(0.833)(1.0) = 816,340 psi c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340 \text{ psi})]/[(31.7355)^2] = 666.2714 \text{ psi}$

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.15)(0.73)(1.0) = 1930.85 \text{ psi}$

$F_{cE}/F_c^* = 666.2714/1930.85 = 0.3451$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.3451]/[(2)(0.9)] = 0.7473$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.7473\} - \sqrt{\{[0.7473]^2 - [0.3451/0.9]\}}$

$= 0.7473 - 0.4183$

$= 0.3289$

$F'_c = F_c^*(C_P) = (1930.85 \text{ psi})(0.3289) = 635.138 \text{ psi}$

$P = (F'_c)(A)$

$A_{req'd} = P/F'_c = 33{,}764 \text{ lb}/635.138 \text{ psi} = 53.160 \text{ in}^2$

$A = 111.4 \text{ in}^2 > 53.16 \text{ in}^2$ ∴OK

Try 6 ¾" x 13 ¾" (A = 92.81 in²)

$(l_e/d)_x = [(1.0)(40.0')(12 \text{ in/ft})]/13.75" = 34.9091$ (controls over $(l_e/d)_y$)

$(l_e/d)_y = [(1.0)(13.333')(12 \text{ in/ft})]/6.75 = 23.7037$

$F_c = 2300$ psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

$E_{min} = 980{,}000$ psi

$C_D = 1.15$ (for snow load; load combination D+S)

$C_M = 0.73$ for F_c (p. 64, NDS Supplement)

$C_M = 0.833$ for E and E_{min} (p. 64, NDS Supplement)

$C_t = 1.0$

$E'_{min} = (E_{min})(C_M)(C_t) = (980{,}000)(0.833)(1.0) = 816{,}340 \text{ psi}$

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340 \text{ psi})]/[(34.9091)^2] = 550.6375 \text{ psi}$

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.15)(0.73)(1.0) = 1930.85 \text{ psi}$

$F_{cE}/F_c^* = 550.6375/1930.85 = 0.2852$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.2852]/[(2)(0.9)] = 0.7140$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.7140\} - \sqrt{\{[0.7140]^2 - [0.2852/0.9]\}}$

$= 0.7140 - 0.4392$

$= 0.2748$

$F'_c = F_c^*(C_P) = (1930.85 \text{ psi})(0.2748) = 530.5371 \text{ psi}$

$P = (F'_c)(A)$

$A_{req'd} = P/F'_c = 33{,}764 \text{ lb}/530.5371 \text{ psi} = 63.641 \text{ in}^2$
$A = 92.81 \text{ in}^2 > 63.64 \text{ in}^2$ ∴ **OK**

Try 6 ¾" x 12 3/8" (A = 83.53 in²)

$(l_e/d)_x = [(1.0)(40.0')(12 \text{ in/ft})]/12.375" = 38.7879$ (controls over $(l_e/d)_y$)

$(l_e/d)_y = [(1.0)(13.333')(12 \text{ in/ft})]/6.75 = 23.7037$

$F_c = 2300$ psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

$E_{min} = 980{,}000$ psi

$C_D = 1.15$ (for snow load; load combination D+S)

$C_M = 0.73$ for F_c (p. 64, NDS Supplement)

$C_M = 0.833$ for E and E_{min} (p. 64, NDS Supplement)

$C_t = 1.0$

$E'_{min} = (E_{min})(C_M)(C_t) = (980{,}000)(0.833)(1.0) = 816{,}340 \text{ psi}$

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340 \text{ psi})]/[(38.7879)^2] = 446.016 \text{ psi}$

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.15)(0.73)(1.0) = 1930.85 \text{ psi}$

$F_{cE}/F_c^* = 446.016/1930.85 = 0.2310$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.2310]/[(2)(0.9)] = 0.6839$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.6839\} - \sqrt{\{[0.6839]^2 - [0.2310/0.9]\}}$

$= 0.6839 - 0.4594$

$= 0.2245$

$F'_c = F_c^*(C_P) = (1930.85 \text{ psi})(0.2245) = 433.468 \text{ psi}$

$P = (F'_c)(A)$

$A_{req'd} = P/F'_c = 33{,}764 \text{ lb}/433.468 \text{ psi} = 77.893 \text{ in}^2$

$A = 83.53 \text{ in}^2 > 77.89 \text{ in}^2$ ∴ **OK**

Use 6 ¾" x 12 3/8"

Try 6 ¾" x 11" (A = 74.25 in^2)

$(l_e/d)_x = [(1.0)(40.0')(12 \text{ in/ft})]/11" = 43.6364$ [controls over $(l_e/d)_y$]

$(l_e/d)_y = [(1.0)(13.333')(12 \text{ in/ft})]/6.75 = 23.7037$

$F_c = 2300$ psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

$E_{min} = 980{,}000$ psi

$C_D = 1.15$ (for snow load; load combination D+S)

$C_M = 0.73$ for F_c (p. 64, NDS Supplement)

$C_M = 0.833$ for E and E_{min} (p. 64, NDS Supplement)

$C_t = 1.0$

$E'_{min} = (E_{min})(C_M)(C_t) = (980{,}000)(0.833)(1.0) = 816{,}340$ psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340 \text{ psi})]/[(43.6364)^2] = 352.408$ psi

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.15)(0.73)(1.0) = 1930.85$ psi

$F_{cE}/F_c^* = 352.408/1930.85 = 0.1825$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.1825]/[(2)(0.9)] = 0.6570$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.6570\} - \sqrt{\{[0.6570]^2 - [0.1825/0.9]\}}$

= 0.6570 – 0.4783

= 0.1786

$F'_c = F_c^*(C_P) = (1930.85\ psi)(0.1786) = 344.907\ psi$

$P = (F'_c)(A)$

$A_{req'd} = P/F'_c = 33{,}764\ lb/344.907\ psi = 97.893\ in^2$

$A = 74.25\ in^2 < 97.89\ in^2$ ∴ **NO GOOD**

Try 5 ½" x 13 ¾" (A = 75.63 in^2)

$(l_e/d)_x = [(1.0)(40.0')(12\ in/ft)]/13.75" = 34.9091$ (controls over $(l_e/d)_y$)

$(l_e/d)_y = [(1.0)(13.333')(12\ in/ft)]/5.5 = 29.0909$

$F_c = 2300\ psi$ (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

$E_{min} = 980{,}000\ psi$

$C_D = 1.15$ (for snow load; load combination D+S)

$C_M = 0.73$ for F_c (p. 64, NDS Supplement)

$C_M = 0.833$ for E and E_{min} (p. 64, NDS Supplement)

$C_t = 1.0$

$E'_{min} = (E_{min})(C_M)(C_t) = (980{,}000)(0.833)(1.0) = 816{,}340\ psi$

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340\ psi)]/[(34.9091)^2] = 550.6375\ psi$

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300\ psi)(1.15)(0.73)(1.0) = 1930.85\ psi$

$F_{cE}/F_c^* = 550.6375/1930.85 = 0.2852$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.2852]/[(2)(0.9)] = 0.7140$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.7140\} - \sqrt{\{[0.7140]^2 - [0.2852/0.9]\}}$

= 0.7140 – 0.4392

= 0.2748

$F'_c = F_c^*(C_P) = (1930.85 \text{ psi})(0.2748) = 530.537 \text{ psi}$

$P = (F'_c)(A)$

$A_{req'd} = P/F'_c = 33{,}764 \text{ lb}/530.537 \text{ psi} = 63.641 \text{ in}^2$

$A = 75.63 \text{ in}^2 > 63.64 \text{ in}^2 \therefore$ **OK**

Try 5 ½" x 12 3/8" (A = 68.06 in²)

$(l_e/d)_x = [(1.0)(40.0')(12 \text{ in/ft})]/12.375" = 38.7879$ (controls over $(l_e/d)_y$)

$(l_e/d)_y = [(1.0)(13.333')(12 \text{ in/ft})]/5.5 = 29.0909$

$F_c = 2300$ psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

$E_{min} = 980{,}000$ psi

$C_D = 1.15$ (for snow load; load combination D+S)
$C_M = 0.73$ for F_c (p. 64, NDS Supplement)

$C_M = 0.833$ for E and E_{min} (p. 64, NDS Supplement)

$C_t = 1.0$

$E'_{min} = (E_{min})(C_M)(C_t) = (980{,}000)(0.833)(1.0) = 816{,}340$ psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340 \text{ psi})]/[(38.7879)^2] = 446.016$ psi

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.15)(0.73)(1.0) = 1930.85$ psi

$F_{cE}/F_c^* = 446.016/1930.85 = 0.2310$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.2310]/[(2)(0.9)] = 0.6839$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.6839\} - \sqrt{\{[0.6839]^2 - [0.2310/0.9]\}}$

$= 0.6839 - 0.4594$

$= 0.2245$

$F'_c = F_c^*(C_P) = (1930.85 \text{ psi})(0.2245) = 433.468$ psi

$P = (F'_c)(A)$

$A_{req'd} = P/F'_c = 33{,}764 \text{ lb}/433.468 \text{ psi} = 77.893 \text{ in}^2$

A = 68.06 in^2 > 77.89 in^2 ∴ **N.G.**

LOAD COMBINATION: D+W (Combined Bending and Axial Forces)

<u>Try 6 ¾" x 16 ½"</u>

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

F_b = 2100 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

A = 111.4 in^2

S = 306.3 in^3

E_{min} = 980,000 psi

Axial Load: P = 12,438 lb (Compression)

Maximum Moment:

W = 26.85 k + 51.49 k + 44.89 k = 123.23 k

(123.23 k)/[(156')(40')] = 0.019748 ksf = 19.7484 psf

w = (19.7484 psf)(8') = 157.987 lb/ft = 0.157987 k/ft

M_{max} = $wL^2/8$ = (0.157987 k/ft)$(40')^2/8$ = 31.599 k-ft = 31,599 ft-lb = 379,188 in-lb

L = 40.0'

Axial Load:

f_c = P/A = 12,438 lb/111.4 in^2 = 111.652 psi

$(l_e/d)_x$ = [(40')(12 in/ft)]/16.5" = 29.0909 < 50 ∴ OK

$(l_e/d)_y$ = [(13.333')(12 in/ft)]/6.75" = 23.7037 < 50 ∴ OK

$(l_e/d)_{max}$ = $(l_e/d)_x$ = 29.0909

The larger slenderness ratio governs the adjusted design value. Therefore, the strong axis of the member is critical, and $(l_e/d)_x$ is used to determine F'_c.

C_D = 1.6 (for wind load; load combination D+W)

C_M = 0.73 for F_c (p. 64, NDS Supplement)

C_M = 0.833 for E and E_{min} (p. 64, NDS Supplement)

$C_M = 0.8$ for F_b (p. 64, NDS Supplement)

$C_t = 1.0$

$E'_{min} = (E_{min})(C_M)(C_t) = (980{,}000)(0.833)(1.0) = 816{,}340$ psi

$c = 0.9$ (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340 \text{ psi})]/[(29.0909)^2] = 792.919$ psi

Here, l_e/d is based on $(l_e/d)_{max}$.

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.6)(0.73)(1.0) = 2686.4$ psi

$F_{cE}/F_c^* = 792.919/2686.4 = 0.2952$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.2952]/[(2)(0.9)] = 0.7195$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.7195\} - \sqrt{\{[0.7195]^2 - [0.2952/0.9]\}}$

$= 0.2839$

$F'_c = F_c^*(C_P) = (2686.4 \text{ psi})(0.2839) = 762.727$ psi

Axial stress ratio $= f_c/F'_c = (111.652 \text{ psi})/(762.727 \text{ psi}) = 0.1464$

Net Section Check:

Assume connections will be made with (2) rows of ¾” diameter bolts.

Assume the hole diameter is 1/16” larger than the bolt (for stress calculations only).

$A_n = (6.75”)[16.5” – (2)(0.8125”)] = 97.03 \text{ in}^2$

(3/4” + 1/16” = 0.8125”)

$f_c = P/A_n = 12{,}438 \text{ lb}/97.03 \text{ in}^2 = 128.187$ psi

$F'_c = F_c^* = F_c(C_D)(C_M)(C_t)(C_P) = (2300 \text{ psi})(1.6)(0.73)(1.0)(0.2839) = 762.669$ psi

762.669 psi > 128.187 psi $\therefore$ OK

Bending:

Bending is about the strong axis of the cross section. The adjusted bending design value for a glulam is governed by the smaller of two criteria: volume effect or lateral stability.

M = 379,188 in-lb

S = 306.3 in^3

f_b = M/S = 379,188 in-lb/306.3 in^3 = 1237.963 psi

$F'_b = F_b(C_D)(C_M)(C_t)(C_L)$ or

$F'_b = F_b(C_D)(C_M)(C_t)(C_V)$

For C_L: l_u/d = [(13.333')(12 in/ft)]/16.5" = 9.697 > 7

∴ $l_e = 1.63l_u + 3d$ = (1.63)[(13.333')(12 in/ft)] + (3)(16.5") = 310.30"

$R_B = \sqrt{l_e d/b^2} = \sqrt{[(310.30")(16.5")/(6.75")^2]}$ = 10.601

$F_{bE} = 1.20E'_{min}/R_B^2 = [(1.20)(816,340 \text{ psi})]/(10.601)^2$ = 8717.544 psi

$F^*_b = F_b(C_D)(C_M)(C_t)$ = (2100 psi)(1.6)(0.8)(1.0) = 2688 psi

F_{bE}/F^*_b = (8717.544)/(2688) = 3.2431

$(1 + F_{bE}/F^*_b)/1.9$ = (1 + 3.2431)/1.9 = 2.233

$C_L = [(1 + F_{bE}/F^*_b)/1.9] - \sqrt{\{[(1 + F_{bE}/F^*_b)/1.9]^2 - [F_{bE}/F^*_b/0.95]\}}$

$= 2.233 - \sqrt{(2.233)^2 - (3.2431/0.95)]}$ = 0.9786

For Southern Pine glulam:

$C_V = (21'/L)^{1/20}(12"/d)^{1/20}(5.125"/b)^{1/20} \leq 1.0$

$C_V = (21'/40')^{1/20}(12"/16.5")^{1/20}(5.125"/6.75")^{1/20} \leq 1.0$

$C_V = 0.9400 \leq 1.0$

C_V governs of C_L

$F'_b = F^*_b(C_V)$ = (2688 psi)(0.9400) = 2526.72 psi

Bending stress ratio = f_b/F'_b = (1237.98 psi)/(2526.72 psi) = 0.4830

Combined Stresses:

The bending moment is about the strong axis of the cross section, and the amplification for P-Δ is measured by the column slenderness ratio about the x axis.

$(l_e/d)_{\text{bending moment}} = (l_e/d)_x$ = 29.0909

$F_{cEx} = [0.822E'_{min}]/[(l_e/d)_x]^2 = [(0.822)(816,340 \text{ psi})]/[(29.0909)^2]$ = 792.919 psi

*Here, (l_e/d) is based on the axis about which the bending moment occurs.

Amplification factor = $1/[1 - (f_c/F_{cEx})] = 1/[1 - (111.652 \text{ psi}/792.919 \text{ psi})] = 1.1639$

$(f_c/F'_c)^2 + \{1/[1 - (f_c/F_{cEx})]\}(f_b/F'_b) = (0.1464)^2 + (1.1639)(0.4830) = 0.5836 < 1.0 \therefore$ **OK**

Try 6 ¾" x 15 1/8"

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

F_b = 2100 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

$A = 102.1 \text{ in}^2$

$S = 257.4 \text{ in}^3$

E_{min} = 980,000 psi

Axial Load: P = 12,438 lb (Compression)

Maximum Moment: M_{max} = 379,188 in-lb

L = 40.0'

Axial Load:

$f_c = P/A = 12{,}438 \text{ lb}/102.1 \text{ in}^2 = 121.822 \text{ psi}$

$(l_e/d)_x = [(40')(12 \text{ in/ft})]/15.125" = 31.7355 < 50 \therefore$ OK

$(l_e/d)_y = [(13.333')(12 \text{ in/ft})]/6.75" = 23.7037 < 50 \therefore$ OK

$(l_e/d)_{max} = (l_e/d)_x = 31.7355$

The larger slenderness ratio governs the adjusted design value. Therefore, the strong axis of the member is critical, and $(l_e/d)_x$ is used to determine F'_c.

C_D = 1.6 (for wind load; load combination D+W)

C_M = 0.73 for F_c (p. 64, NDS Supplement)

C_M = 0.833 for E and E_{min} (p. 64, NDS Supplement)

C_M = 0.8 for F_b (p. 64, NDS Supplement)

$C_t = 1.0$

$E'_{min} = (E_{min})(C_M)(C_t) = (980{,}000)(0.833)(1.0) = 816{,}340$ psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340\ psi)]/[(31.7355)^2] = 666.271\ psi$

Here, l_e/d is based on $(l_e/d)_{max}$.

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300\ psi)(1.6)(0.73)(1.0) = 2686.4\ psi$

$F_{cE}/F_c^* = 666.271/2686.4 = 0.2480$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.2480]/[(2)(0.9)] = 0.6933$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.6933\} - \sqrt{\{[0.6933]^2 - [0.2480/0.9]\}}$

$= 0.2403$

$F'_c = F_c^*(C_P) = (2686.4\ psi)(0.2403) = 645.663\ psi$

Axial stress ratio = $f_c/F'_c = (121.822\ psi)/(645.663\ psi) = 0.1887$

Net Section Check:

Assume connections will be made with (2) rows of ¾" diameter bolts.

Assume the hole diameter is 1/16" larger than the bolt (for stress calculations only).

$A_n = (6.75")[15.125" - (2)(0.8125")] = 91.125\ in^2$

(3/4" + 1/16" = 0.8125")

$f_c = P/A_n = 12{,}438\ lb/91.125\ in^2 = 136.494\ psi$

$F'_c = F_c^* = F_c(C_D)(C_M)(C_t)(C_P) = (2300\ psi)(1.6)(0.73)(1.0)(0.2403) = 645.542\ psi$

$645.542\ psi > 136.494\ psi$ ∴ OK

Bending:

Bending is about the strong axis of the cross section. The adjusted bending design value for a glulam is governed by the smaller of two criteria: volume effect or lateral stability.

$M = 379{,}188$ in-lb

$S = 257.4\ in^3$

$f_b = M/S = 379{,}188\ in\text{-}lb/257.4\ in^3 = 1473.147\ psi$

$F'_b = F_b(C_D)(C_M)(C_t)(C_L)$ or

$F'_b = F_b(C_D)(C_M)(C_t)(C_V)$

For C_L: l_u/d = [(13.333')(12 in/ft)]/15.125" = 10.579 > 7

$\therefore$ $l_e = 1.63l_u + 3d$ = (1.63)[(13.333')(12 in/ft)] + (3)(15.125") = 306.17"

$R_B = \sqrt{l_e d/b^2} = \sqrt{[(306.17")(15.125")/(6.75")^2]}$ = 10.082

$F_{bE} = 1.20E'_{min}/R_B^2$ = [(1.20)(816,340 psi)]/(10.082)^2 = 9638.174 psi

$F^*_b = F_b(C_D)(C_M)(C_t)$ = (2100 psi)(1.6)(0.8)(1.0) = 2688 psi

F_{bE}/F^*_b = (9638.174)/(2688) = 3.5856

$(1 + F_{bE}/F^*_b)/1.9$ = (1 + 3.5856)/1.9 = 2.4135

$C_L = [(1 + F_{bE}/F^*_b)/1.9] - \sqrt{\{[(1 + F_{bE}/F^*_b)/1.9]^2 - [F_{bE}/F^*_b/0.95]\}}$

= 2.4135 - $\sqrt{(2.4135)^2 - (3.5856/0.95)]}$ = 0.9815

For Southern Pine glulam:

$C_V = (21'/L)^{1/20}(12"/d)^{1/20}(5.125"/b)^{1/20} \leq 1.0$

$C_V = (21'/40')^{1/20}(12"/15.125")^{1/20}(5.125"/6.75")^{1/20} \leq 1.0$

$C_V = 0.9441 \leq 1.0$

C_V governs of C_L

$F'_b = F^*_b(C_V)$ = (2688 psi)(0.9441) = 2537.741 psi

Bending stress ratio = f_b/F'_b = (1473.147 psi)/(2537.741 psi) = 0.5805

Combined Stresses:

The bending moment is about the strong axis of the cross section, and the amplification for P-Δ is measured by the column slenderness ratio about the x axis.

$(l_e/d)_{bending\ moment} = (l_e/d)_x$ = 31.7355

$F_{cEx} = [0.822E'_{min}]/[(l_e/d)_x]^2$ = [(0.822)(816,340 psi)]/[(31.7355)^2] = 666.271 psi

*Here, (l_e/d) is based on the axis about which the bending moment occurs.

Amplification factor = $1/[1 - (f_c/F_{cEx})]$ = 1/[1 – (121.822 psi/666.271 psi)] = 1.2238

$(f_c/F'_c)^2 + \{1/[1 - (f_c/F_{cEx})]\}(f_b/F'_b)$ = (0.1887)^2 + (1.2238)(0.5805) = 0.746 < 1.0 $\therefore$ **OK**

<u>Try 6 ¾" x 13 ¾"</u>

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

F_b = 2100 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

A = 92.81 in^2

S_x = 212.7 in^3

E_{min} = 980,000 psi

Axial Load: P = 12,438 lb (Compression)

Maximum Moment: M_{max} = 379,188 in-lb

L = 40.0'

Axial Load:

f_c = P/A = 12,438 lb/92.81 in^2 = 134.016 psi

$(l_e/d)_x$ = [(40')(12 in/ft)]/13.75" = 34.9091 < 50 ∴ OK

$(l_e/d)_y$ = [(13.333')(12 in/ft)]/6.75" = 23.7037 < 50 ∴ OK

$(l_e/d)_{max} = (l_e/d)_x$ = 34.9091

The larger slenderness ratio governs the adjusted design value. Therefore, the strong axis of the member is critical, and $(l_e/d)_x$ is used to determine F'_c.

C_D = 1.6 (for wind load; load combination D+W)

C_M = 0.73 for F_c (p. 64, NDS Supplement)

C_M = 0.833 for E and E_{min} (p. 64, NDS Supplement)

C_M = 0.8 for F_b (p. 64, NDS Supplement)

C_t = 1.0

$E'_{min} = (E_{min})(C_M)(C_t)$ = (980,000)(0.833)(1.0) = 816,340 psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2]$ = [(0.822)(816,340 psi)]/[$(34.9091)^2$] = 550.638 psi

Here, l_e/d is based on $(l_e/d)_{max}$.

$F_c^* = F_c(C_D)(C_M)(C_t)$ = (2300 psi)(1.6)(0.73)(1.0) = 2686.4 psi

F_{cE}/F_c^* = 550.638/2686.4 = 0.2050

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.2050]/[(2)(0.9)] = 0.6694$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.6694\} - \sqrt{\{[0.6694]^2 - [0.2050/0.9]\}}$

$= 0.2000$

$F'_c = F_c^*(C_P) = (2686.4 \text{ psi})(0.2000) = 537.220 \text{ psi}$

Axial stress ratio = $f_c/F'_c = (134.016 \text{ psi})/(537.220 \text{ psi}) = 0.2495$

Net Section Check:

Assume connections will be made with (2) rows of ¾" diameter bolts.

Assume the hole diameter is 1/16" larger than the bolt (for stress calculations only).

$A_n = (6.75")[13.75" - (2)(0.8125")] = 81.84 \text{ in}^2$

(3/4" + 1/16" = 0.8125")

$f_c = P/A_n = 12{,}438 \text{ lb}/81.84 \text{ in}^2 = 151.979 \text{ psi}$

$F'_c = F_c^* = F_c(C_D)(C_M)(C_t)(C_P) = (2300 \text{ psi})(1.6)(0.73)(1.0)(0.200) = 537.28 \text{ psi}$

537.28 psi > 151.979 psi $\therefore$ OK

Bending:

Bending is about the strong axis of the cross section. The adjusted bending design value for a glulam is governed by the smaller of two criteria: volume effect or lateral stability.

$M = 379{,}188 \text{ in-lb}$

$S = 257.4 \text{ in}^3$

$f_b = M/S = 379{,}188 \text{ in-lb}/212.7 \text{ in}^3 = 1782.736 \text{ psi}$

$F'_b = F_b(C_D)(C_M)(C_t)(C_L)$ or

$F'_b = F_b(C_D)(C_M)(C_t)(C_V)$

For C_L: $l_u/d = [(13.333')(12 \text{ in/ft})]/13.75" = 11.636 > 7$

$\therefore l_e = 1.63 l_u + 3d = (1.63)[(13.333')(12 \text{ in/ft})] + (3)(13.75") = 302.05"$

$R_B = \sqrt{l_e d/b^2} = \sqrt{[(302.05")(13.75")/(6.75")^2]} = 9.547$

$F_{bE} = 1.20E'_{min}/R_B^2 = [(1.20)(816{,}340 \text{ psi})]/(9.547)^2 = 10{,}746.782 \text{ psi}$

$F^*_b = F_b(C_D)(C_M)(C_t) = (2100 \text{ psi})(1.6)(0.8)(1.0) = 2688 \text{ psi}$

$F_{bE}/F^*_b = (10{,}176.782)/(2688) = 3.9981$

$(1 + F_{bE}/F^*_b)/1.9 = (1 + 3.9981)/1.9 = 2.6306$

$C_L = [(1 + F_{bE}/F^*_b)/1.9] - \sqrt{\{[(1 + F_{bE}/F^*_b)/1.9]^2 - [F_{bE}/F^*_b/0.95]\}}$

$= 2.6306 - \sqrt{(2.6306)^2 - (3.9981/0.95)]} = 0.9840$

For Southern Pine glulam:

$C_V = (21'/L)^{1/20}(12''/d)^{1/20}(5.125''/b)^{1/20} \leq 1.0$

$C_V = (21'/40')^{1/20}(12''/13.75'')^{1/20}(5.125''/6.75'')^{1/20} \leq 1.0$

$C_V = 0.9486 \leq 1.0$

C_V governs of C_L

$F'_b = F^*_b(C_V) = (2688 \text{ psi})(0.9486) = 2549.837 \text{ psi}$

Bending stress ratio = $f_b/F'_b = (1782.736 \text{ psi})/(2549.837 \text{ psi}) = 0.6992$

Combined Stresses:

The bending moment is about the strong axis of the cross section, and the amplification for P-Δ is measured by the column slenderness ratio about the x axis.

$(l_e/d)_{\text{bending moment}} = (l_e/d)_x = 34.9091$

$F_{cEx} = [0.822E'_{min}]/[(l_e/d)_x]^2 = [(0.822)(816{,}340 \text{ psi})]/[(34.9091)^2] = 550.637 \text{ psi}$

*Here, (l_e/d) is based on the axis about which the bending moment occurs.

Amplification factor = $1/[1 - (f_c/F_{cEx})] = 1/[1 - (134.016 \text{ psi}/550.637 \text{ psi})] = 1.3217$

$(f_c/F'_c)^2 + \{1/[1 - (f_c/F_{cEx})]\}(f_b/F'_b) = (0.2495)^2 + (1.3217)(0.6992) = 0.9864 < 1.0$ ∴ **OK**

LOAD COMBINATION: D + 0.75W + 0.75 S

<u>*Try 6 ¾” x 13 ¾”*</u>

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

F_b = 2100 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

$A = 92.81 \text{ in}^2$

$S_x = 212.7$ in^3

$E_{min} = 980,000$ psi

Axial Load: P = 23,983 lb (Compression)

Maximum Moment: M_{max} = 23.700 k-ft = 23,700 ft-lb = 284,400 in-lb

L = 40.0'

Axial Load:

$f_c = P/A$ = 23,983 lb/92.81 in^2 = 258.410 psi

$(l_e/d)_x$ = [(40')(12 in/ft)]/13.75" = 34.9091 < 50 ∴ OK

$(l_e/d)_y$ = [(13.333')(12 in/ft)]/6.75" = 23.7037 < 50 ∴ OK

$(l_e/d)_{max} = (l_e/d)_x = 34.9091$

The larger slenderness ratio governs the adjusted design value. Therefore, the strong axis of the member is critical, and $(l_e/d)_x$ is used to determine F'_c.

$C_D = 1.6$ (for wind load; load combination D+W)

$C_M = 0.73$ for F_c (p. 64, NDS Supplement)

$C_M = 0.833$ for E and E_{min} (p. 64, NDS Supplement)

$C_M = 0.8$ for F_b (p. 64, NDS Supplement)

$C_t = 1.0$

$E'_{min} = (E_{min})(C_M)(C_t)$ = (980,000)(0.833)(1.0) = 816,340 psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2]$ = [(0.822)(816,340 psi)]/[(34.9091)^2] = 550.638 psi

Here, l_e/d is based on $(l_e/d)_{max}$.

$F_c^* = F_c(C_D)(C_M)(C_t)$ = (2300 psi)(1.6)(0.73)(1.0) = 2686.4 psi

F_{cE}/F_c^* = 550.638/2686.4 = 0.2050

$[1 + F_{cE}/F_c^*]/(2c)$ = [1 + 0.2050]/[(2)(0.9)] = 0.6694

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.6694\} - \sqrt{\{[0.6694]^2 - [0.2050/0.9]\}}$

= 0.2000

$F'_c = F_c^*(C_P)$ = (2686.4 psi)(0.2000) = 537.220 psi

Axial stress ratio = f_c/F'_c = (258.410 psi)/(537.220 psi) = 0.4810

Net Section Check:

Assume connections will be made with (2) rows of ¾" diameter bolts.

Assume the hole diameter is 1/16" larger than the bolt (for stress calculations only).

A_n = (6.75")[13.75" – (2)(0.8125")] = 81.84 in^2

(3/4" + 1/16" = 0.8125")

$f_c = P/A_n$ = 23,983 lb/81.84 in^2 = 293.047 psi

$F'_c = F_c^* = F_c(C_D)(C_M)(C_t)(C_P)$ = (2300 psi)(1.6)(0.73)(1.0)(0.200) = 537.28 psi

537.28 psi > 293.047 psi ∴ OK

Bending:

Bending is about the strong axis of the cross section. The adjusted bending design value for a glulam is governed by the smaller of two criteria: volume effect or lateral stability.

M = 284,400 in-lb

S = 257.4 in^3

f_b = M/S = 284,400 in-lb/212.7 in^3 = 1337.094 psi

$F'_b = F_b(C_D)(C_M)(C_t)(C_L)$ or

$F'_b = F_b(C_D)(C_M)(C_t)(C_V)$

For C_L: l_u/d = [(13.333')(12 in/ft)]/13.75" = 11.636 > 7

∴ $l_e = 1.63l_u + 3d$ = (1.63)[(13.333')(12 in/ft)] + (3)(13.75") = 302.05"

$R_B = \sqrt{l_e d/b^2}$ = √ [(302.05")(13.75")/(6.75")^2] = 9.547

$F_{bE} = 1.20E'_{min}/R_B^2$ = [(1.20)(816,340 psi)]/(9.547)^2 = 10,746.782 psi

$F^*_b = F_b(C_D)(C_M)(C_t)$ = (2100 psi)(1.6)(0.8)(1.0) = 2688 psi

F_{bE}/F^*_b = (10,176.782)/(2688) = 3.9981

$(1 + F_{bE}/F^*_b)/1.9$ = (1 + 3.9981)/1.9 = 2.6306

$C_L = [(1 + F_{bE}/F^*_b)/1.9] - \sqrt{\{[(1 + F_{bE}/F^*_b)/1.9]^2 - [F_{bE}/F^*_b/0.95]\}}$

$= 2.6306 - \sqrt{(2.6306)^2 - (3.9981/0.95)]} = 0.9840$

For Southern Pine glulam:

$C_V = (21'/L)^{1/20}(12''/d)^{1/20}(5.125''/b)^{1/20} \leq 1.0$

$C_V = (21'/40')^{1/20}(12''/13.75'')^{1/20}(5.125''/6.75'')^{1/20} \leq 1.0$

$C_V = 0.9486 \leq 1.0$

C_V governs of C_L

$F'_b = F^*_b(C_V) = (2688 \text{ psi})(0.9486) = 2549.837 \text{ psi}$

Bending stress ratio = $f_b/F'_b = (1337.094 \text{ psi})/(2549.837 \text{ psi}) = 0.5244$

Combined Stresses:

The bending moment is about the strong axis of the cross section, and the amplification for P-Δ is measured by the column slenderness ratio about the x axis.

$(l_e/d)_{\text{bending moment}} = (l_e/d)_x = 34.9091$

$F_{cEx} = [0.822E'_{min}]/[(l_e/d)_x]^2 = [(0.822)(816{,}340 \text{ psi})]/[(34.9091)^2] = 550.637 \text{ psi}$

*Here, (l_e/d) is based on the axis about which the bending moment occurs.

Amplification factor = $1/[1 - (f_c/F_{cEx})] = 1/[1 - (258.410 \text{ psi}/550.637 \text{ psi})] = 1.8843$

$(f_c/F'_c)^2 + \{1/[1 - (f_c/F_{cEx})]\}(f_b/F'_b) = (0.4810)^2 + (1.8843)(0.5244) = 1.219 > 1.0 \therefore$ **N.G.**

Try 6 ¾" x 15 1/8"

$F_c = 2300$ psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

$F_b = 2100$ psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

$A = 102.1 \text{ in}^2$

$S = 257.4 \text{ in}^3$

$E_{min} = 980{,}000$ psi

Axial Load: P = 23,983 lb (Compression)

Maximum Moment: $M_{max} = 284{,}400$ in-lb

L = 40.0'

Axial Load:

f_c = P/A = 23,983 lb/102.1 in^2 = 234.897 psi

$(l_e/d)_x$ = [(40')(12 in/ft)]/15.125" = 31.7355 < 50 ∴ OK

$(l_e/d)_y$ = [(13.333')(12 in/ft)]/6.75" = 23.7037 < 50 ∴ OK

$(l_e/d)_{max} = (l_e/d)_x$ = 31.7355

The larger slenderness ratio governs the adjusted design value. Therefore, the strong axis of the member is critical, and $(l_e/d)_x$ is used to determine F'_c.

C_D = 1.6 (for wind load; load combination D+W)

C_M = 0.73 for F_c (p. 64, NDS Supplement)

C_M = 0.833 for E and E_{min} (p. 64, NDS Supplement)

C_M = 0.8 for F_b (p. 64, NDS Supplement)

C_t = 1.0

$E'_{min} = (E_{min})(C_M)(C_t)$ = (980,000)(0.833)(1.0) = 816,340 psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2]$ = [(0.822)(816,340 psi)]/[(31.7355)^2] = 666.271 psi

Here, l_e/d is based on $(l_e/d)_{max}$.

$F_c^* = F_c(C_D)(C_M)(C_t)$ = (2300 psi)(1.6)(0.73)(1.0) = 2686.4 psi

F_{cE}/F_c^* = 666.271/2686.4 = 0.2480

$[1 + F_{cE}/F_c^*]/(2c)$ = [1 + 0.2480]/[(2)(0.9)] = 0.6933

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.6933\} - \sqrt{\{[0.6933]^2 - [0.2480/0.9]\}}$

= 0.2403

$F'_c = F_c^*(C_P)$ = (2686.4 psi)(0.2403) = 645.663 psi

Axial stress ratio = f_c/F'_c = (234.897 psi)/(645.663 psi) = 0.3638

Net Section Check:

Assume connections will be made with (2) rows of ¾" diameter bolts.

Assume the hole diameter is 1/16" larger than the bolt (for stress calculations only).

A_n = (6.75")[15.125" – (2)(0.8125")] = 91.125 in^2

(3/4" + 1/16" = 0.8125")

$f_c = P/A_n$ = 23,983 lb/91.125 in^2 = 263.188 psi

$F'_c = F_c^* = F_c(C_D)(C_M)(C_t)(C_P)$ = (2300 psi)(1.6)(0.73)(1.0)(0.2403) = 645.542 psi

645.542 psi > 263.188 $\therefore$ OK

Bending:

Bending is about the strong axis of the cross section. The adjusted bending design value for a glulam is governed by the smaller of two criteria: volume effect or lateral stability.

M = 284,400 in-lb

S = 257.4 in^3

$f_b = M/S$ = 284,400 in-lb/257.4 in^3 = 1104.895 psi

$F'_b = F_b(C_D)(C_M)(C_t)(C_L)$ or

$F'_b = F_b(C_D)(C_M)(C_t)(C_V)$

For C_L: l_u/d = [(13.333')(12 in/ft)]/15.125" = 10.579 > 7

$\therefore l_e = 1.63l_u + 3d$ = (1.63)[(13.333')(12 in/ft)] + (3)(15.125") = 306.17"

$R_B = \sqrt{l_e d/b^2} = \sqrt{[(306.17")(15.125")/(6.75")^2]}$ = 10.082

$F_{bE} = 1.20E'_{min}/R_B^2$ = [(1.20)(816,340 psi)]/$(10.082)^2$ = 9638.174 psi

$F^*_b = F_b(C_D)(C_M)(C_t)$ = (2100 psi)(1.6)(0.8)(1.0) = 2688 psi

F_{bE}/F^*_b = (9638.174)/(2688) = 3.5856

$(1 + F_{bE}/F^*_b)/1.9$ = (1 + 3.5856)/1.9 = 2.4135

$C_L = [(1 + F_{bE}/F^*_b)/1.9] - \sqrt{\{[(1 + F_{bE}/F^*_b)/1.9]^2 - [F_{bE}/F^*_b/0.95]\}}$

$= 2.4135 - \sqrt{(2.4135)^2 - (3.5856/0.95)]}$ = 0.9815

For Southern Pine glulam:

$C_V = (21'/L)^{1/20}(12"/d)^{1/20}(5.125"/b)^{1/20} \leq 1.0$

$C_V = (21'/40')^{1/20}(12"/15.125")^{1/20}(5.125"/6.75")^{1/20} \leq 1.0$

$C_V = 0.9441 \leq 1.0$

C_V governs of C_L

$F'_b = F^*_b(C_V)$ = (2688 psi)(0.9441) = 2537.741 psi

Bending stress ratio = f_b/F'_b = (1104.895 psi)/(2537.741 psi) = 0.4354

Combined Stresses:

The bending moment is about the strong axis of the cross section, and the amplification for P-Δ is measured by the column slenderness ratio about the x axis.

$(l_e/d)_{bending\ moment} = (l_e/d)_x = 31.7355$

$F_{cEx} = [0.822E'_{min}]/[(l_e/d)_x]^2$ = [(0.822)(816,340 psi)]/[$(31.7355)^2$] = 666.271 psi

*Here, (l_e/d) is based on the axis about which the bending moment occurs.

Amplification factor = $1/[1 - (f_c/F_{cEx})]$ = 1/[1 – (234.897 psi/666.271 psi)] = 1.5445

$(f_c/F'_c)^2 + \{1/[1 - (f_c/F_{cEx})]\}(f_b/F'_b) = (0.3638)^2 + (1.5445)(0.4354) = 0.805 < 1.0$ ∴ **OK**

FINAL SECTION SIZE: 6 ¾" x 15 1/8" Southern Pine Glulam ID #50

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	

Deflection Check in SAP2000:

Member 1 (Column): 6 ¾" x 15 1/8"(Southern Pine, Glulam ID # 50)

$A = 102.1\ in^2$

$I_x = bh^3/12 = (6.75")(15.125")^3/12 = 1946\ in^4$

$I_y = bh^3/12 = (15.125")(6.75")^3/12 = 387.6\ in^4$

E = 1,900,000 psi

Member 13 (Top Chord): 6 ¾" x 9 5/8" (Southern Pine, Glulam ID #50)

$A = 64.97\ in^2$

$I_x = bh^3/12 = (6.75")(9.625")^3/12 = 501.6\ in^4$

$I_y = bh^3/12 = (9.625")(6.75")^3/12 = 246.7\ in^4$

E = 1,900,000 psi

Member 6 (Bottom Chord): 6 ¾" x 6 7/8" (Southern Pine, Glulam ID #50)

$A = 46.41\ in^2$

$I_x = bh^3/12 = (6.75")(6.875")^3/12 = 182.8\ in^4$

$I_y = bh^3/12 = (6.875")(6.75")^3/12 = 176.2\ in^4$

E = 1,900,000 psi

Total Load: D + S

Deflection at mid-span of truss (top chord) = 1.582" (from SAP2000 model)

1.582" < L/240 = [(130')(12 in/ft)]/240 = 6.5" ∴ **OK**

Deflection at mid span of truss (bottom chord) = 1.584" (from SAP2000 model)

1.584" < L/240 = [(130')(12 in/ft)]/240 = 6.5" ∴ **OK**
Deflections include distributed dead load of (10 PSF)(8') = 80 lb/ft = 0.080 k/ft to the bottom chord.

Live Load: L_r

Deflection at mid-span of truss (top chord) = 0.513"

0.513" < L/360 = [(130')(12 in/ft)]/360 = 4.333" ∴ **OK**

Deflection at mid-span of truss (bottom chord) = 0.512"

0.512" < < L/360 = [(130')(12 in/ft)]/360 = 4.333" ∴ **OK**

All Top Chord Members:

Load along roof slope:

w_{Lr} = (20 PSF)(8') = 160 lb/ft = 0.160 k/ft (due to roof live load)

Cost Comparison Using RS Means

From RS Means Building Construction Cost Data (2009)
(costs include material, labor, and equipment)

Wood Roof System:

Connector Plates, steel, with bolts, straight = ($34/plate)(22)(19 trusses) = $14,212

Laminated Roof Deck:
Cedar, 3" thick = ($5.61/SF)(20,280 SF) = $113,770.80
(values for Southern Pine were not given, so Cedar was conservatively assumed)

Sheathing, Plywood on Roofs:
3/8" thick = ($0.87/SF)(20,280 SF) = $17,643.60

Glued-Laminated Beams:
Bowstring trusses, 20' o.c., 120' clear span
= ($8.09/SF)(20280 SF) = $164,065.20
Although 8' o.c. is not listed in the tables, it is listed for other similar framing systems. On average, the total cost of various trusses @ 8' o.c. is only about $1/SF more than the same trusses @ 16' o.c. For this analysis, look at radial arches:
120' clear span, frames 8' o.c. = $13.86/SF
120' clear span, frames 16' o.c. = $12.34/SF
Increased by $13.86/$12.34 = 1.1232
So, for the bowstring trusses at 8' o.c., 120' clear span, assume:
(1.1232)($8.09/SF) = $9.09/SF
($9.09/SF)(20280 SF) = $184,274.20
For pressure treating, add 35" to material cost:
Material cost: (1.1232)($7.24/SF) = $8.14/SF
(1.35)($8.14/SF) = $10.99/SF
Total cost = $10.99/SF + (1.1232)($0.53/SF) + (1.1232)($0.31/SF) =
= $11.93/SF
($11.93/SF)(20280 SF) = $242,011.14

High-Strength Bolts:
¾" diameter x 8" long = ($9.26/bolt)(846 bolts/truss)(19 trusses) = $148,845.24

Original Steel Roof System:

Paints and Protective Coatings:
Galvanizing steel in shop:
Steel trusses: 1 ton to 20 tons = ($795/ton)(19.1865 tons) = $15,253.27
Long-span metal roof deck (galvanized and painted):
Galvanized steel, 18 ga, corrugated (2 ½" and 3") = 2.4 psf
For 7 ½", assume = (2)(2.4 psf) = 4.8 psf
(4.8 psf)(20280 SF) = 97.344 k = 48.672 tons
Over 20 tons: ($735/ton)(48.672 tons) = $35,773.92

Welded Rigid Frame:

Minimum: (\$3,475/ton)[(38.373 k + 45.595 k)/2] = \$145,894.40
Maximum: (\$5,055/ton)[(38.373 k + 45.595 k)/2] = \$212,229.12

Or use "roof trusses":

Minimum: (\$4,615/ton)[(38.373 k + 45.595 k)/2] = \$193,756.16
Maximum: (\$5,751/ton)[(38.373 k + 45.595 k)/2] = \$241,449.98

For projects 25 to 49 tons, add 30% to material costs:

Welded Rigid Frame:

Minimum: (1.30)(\$3,125/ton) = \$4,062.5/ton
Total = \$4062.5/ton + \$223/ton + \$127/ton = \$4,412.5/ton
(\$4,412.5/ton)(41.984 tons) = \$185,254.40
Maximum: (1.30)(\$4050/ton) = \$5,265/ton
Total = \$5,265/ton + \$640/ton + \$365/ton = \$6,270/ton
(\$6,270/ton)(41.984 tons) = \$263,239.68

Or use "roof trusses":

Minimum: (1.30)(\$4,200/ton) = \$5,460/ton
Total = \$5460/ton + \$271/ton + \$144/ton = \$5,875/ton
(\$5875/ton)(41.984 tons) = \$246,656.00
Maximum: (1.30)(\$5100/ton) = \$6,630/ton
Total = \$6,630/ton + \$425/ton + \$226/ton = \$7,281/ton
(\$7281/ton)(41.984 tons) = \$305,685.50

Average of all four = \$1,000,835.58/4 = \$250,208.90

Plus, the actual cost would probably be toward the maximum end anyway due to the complex truss configuration.

Steel Deck:
7 ½" deep, long span, 18 gauge: \$16.30/SF
For acoustical perforated, with fiberglass, add: \$1.91/SF
Total = \$16.30/SF + \$1.91/SF = \$18.21/SF
(\$18.21/SF)(20,280 SF) = \$369,298.80

Concrete Moment Frames:

Forms in place, beams and girders:

24" wide, 4 use = \$6.64/SFCA
Column line 2: SFCA = (8 beams)[(2*24")+(2*30")/12](32') = 2304 SFCA
Column line 1.8: SFCA = (4 beams)[(2*24")+(2*26")/12](32') = 1066.67 SFCA
East/West frame: SFCA = (5 beams)[(2*24")+(2*26")/12](32') = 1333.33 SFCA
Total = 4,704.00 SFCA

($6.64/SFCA)(4704.00 SFCA) = $22,381.23

Forms in place, columns:

24”x24” columns, 4 use = $5.91/SFCA
Column line 2: SFCA = (5 columns)[(4*24”)/12](40’) = 1,600 SFCA
Column line 1.8: SFCA = (5 columns)[(4*24”)/12](10.5’) = 420 SFCA
Total = 2020 SFCA
($5.91/SFCA)(2,020 SFCA) = $11,938.20

Concrete in place:

Columns, 24”x24”, average reinforcing = $1,068/CY
Column line 2: (5 columns)[(2’)(2’)(40’)/27] = 29.630 CY
Column line 1.8: (5 columns)[(2’)(2’)(10.5’)/27] = 7.778 CY
Total = 29.630 CY + 7.778 CY = 37.407 CY
($1,068/CY)(37.407 CY) = $39,951.08

Beams, 25’ span = $901/CY
Column line 2: (8 beams)[(2’)(2.5’)(32’)/27] = 47.407 CY
Column line 1.8: (4 beams)[(2’)(2.1667’)(32’)/27] = 20.543 CY
East/West frame: (5 beams)[(2’)(2.1667’)(23’)/27] = 18.457 CY
Total = 47.407 CY + 20.543 CY + 18.457= 86.407 CY
($901/CY)(86.407 CY) = $77,852.52

Reinforcing steel:

Beams and Girders: #3 to #7 = $2440/ton
Columns: #8 to #18 = $2170/ton

Beams: Use ρ_g = 0.015
Column line 2: (8 beams)[((24”*30”)/144)(32’)] = 1,280 ft^3
(0.015)(1280 ft^3) = 19.2 ft^3
(490 lb/ft^3)(19.2 ft^3) = 9,408 lb = 4.704 tons
($2,440/ton)(4.704 tons) = $11,477.76
Column line 1.8: (4 beams)[((24”*26”)/144)(32’)] = 554.667 ft^3
(0.015)(554.667 ft^3) = 8.32 ft^3
(490 lb/ft^3)(8.32 ft^3) = 4,076.80 lb = 2.038 tons
($2,440/ton)(2.038 tons) = $4,973.70
East/West frame: (5 beams)[((24”*26”)/144)(23’)] = 498.333 ft^3
(0.015)(498.333 ft^3) = 7.475 ft^3
(490 lb/ft^3)(7.475 ft^3) = 3,662.75 lb = 1.831 tons
($2,440/ton)(1.831 tons) = $4,468.56

Columns: Use ρ_g = 0.015
Column line 2: (5 columns)[((24”*24”)/144)(40’)] = 800 ft^3
(0.015)(800 ft^3) = 12.0 ft^3
(490 lb/ft^3)(12.0 ft^3) = 5,880 lb = 2.94 tons
($2440/ton)(2.94 tons) = $7173.60
Column line 1.8: (5 columns)[((24”*24”)/144)(10.5’)] = 210 ft^3

$(0.015)(210\ ft^3) = 3.15\ ft^3$
$(490\ lb/ft^3)(3.15\ ft^3) = 1{,}543.50\ lb = 0.772\ tons$
($2440/ton)(0.772 tons) = $1,883.07

Steel Moment Frame (Original Design):

Structural tubing, heavy sections = $1.63/lb
Column line 2:
Columns: (5) HSS18x18x5/8
(5)[(127 lb/ft)(37')] = 23,495 lb
($1.63/lb)(23,495 lb) = $38,296.85
Beams: (8) HSS12x12x3/8
(8)[(58.03 lb/ft)(30')] = 13,927.20 lb
($1.63/lb)(13,927.20 lb) = $22,701.34
Column line 1.8:
Columns: (5) HSS14x14x1/2
(5)[(89.55 lb/ft)(10.5')] = 4,701.375 lb
($1.63/lb)(4,701.375 lb) = $7,663.24
Beams: (4) W27x84
(4)(30') = 120'
($143.54/ft)(120') = $17,224.80
East/West frame:
Beams: (5) W27x84
(5)(23') = 115'
($143.54/ft)(115') = $16,507.10

Decking

From "AITC 112*-81: Standard for Tongue-and-Groove Heavy Timber Roof Decking"

1) Sizes (tongue-and-groove decking)
   Two-inch decking
   Three-inch decking
   Four-inch decking
   (nominal dimensions are given)

2) Patterns
   Controlled Random Layup
   Cantilever Spans with Controlled Random Layup
   Cantilevered Pieces Intermixed
   Combination Simple and Two-Span Continuous
   Two-Span Continuous

3) V-groove for architectural aspect since decking will be exposed from below.

4) Southern Pine
   Select Quality
      Bending Stress = 1650 psi
      Modulus of Elasticity = 1,600,000 psi
   Commercial Quality
      Bending Stress = 1650 psi
      Modulus of Elasticity = 1,600,000 psi

*"When decking is used where the moisture content will exceed 19% for an extended period of time, bending stress values should be multiplied by a factor of 0.86 and modules of elasticity by a factor of 0.97."

*These values include repetitive member factor

Adjusted Values for Southern Pine (moisture content exceeding 19% since natatorium):
   Select Quality
      Bending Stress = (0.86)(1650 psi) = **1419 psi**
      Modulus of Elasticity = (0.97)(1,600,000 psi) = **1,552,000 psi**

5) Table 4: "Two Inch Nominal Thickness, Allowable Roof Load Limited by Bending"
   Simple Span, 8 ft, Bending Stress = 1400 psi
      =66 psf
   Controlled Random Layup Span, 8 ft, Bending Stress = 1400 psi
      =55 psf

6) Table 5: "Two Inch Nominal Thickness, Allowable Roof Load Limited by Deflection"

Simple Span, 8 ft, Modulus of Elasticity = 1,500,000 psi
L/180……….29 psf
L/240……….22 psf
L/360……….(29 psf)(0.5) = 14.5 psf
Controlled Random Layup Span, 8 ft, Modulus of Elasticity = 1,500,000 psi
L/180……….38 psf
L/240……….29 psf
L/360……….(38 psf)(0.5) = 19 psf
Cantilevered Pieces Intermixed, 8 ft, Modulus of Elasticity = 1,500,000 psi
L/180……….(38 psf)(1.05) = 39.9 psf
L/240……….(29 psf)(1.05) = 30.45 psf
L/360……….(39.9 psf)(0.5) = 19.95 psf
Combination Simple Span and Two-Span Continuous, 8 ft, E = 1,500,000 psi
L/180……….(38 psf)(1.31) = 49.78 psf
L/240……….(29 psf)(1.31) = 37.99 psf
L/360……….(49.78 psf)(0.5) = 24.89 psf
Two-Span Continuous, 8 ft, E = 1,500,000 psi
L/180……….(38 psf)(1.85) = 70.3 psf
L/240……….(29 psf)(1.85) = 53.65 psf
L/360……….(70.3 psf)(0.5) = 35.15 psf

7) Table 6: "Three and Four Inch Nominal Thickness, Allowable Roof Load Limited by Bending, Simple Span and Controlled Random Layups (3 or more spans)"
3 in. Nominal Thickness, 8 ft, Bending Stress = 1400 psi
= 182 psf
4 in. Nominal Thickness, 8 ft, Bending Stress = 1400 psi
= 357 psi

8) Table 7: "Three and Four Inch Nominal Thickness, Allowable Roof Load Limited by Deflection, Simple Span Layup"
3 in. Nominal Thickness, 8 ft, E = 1,500,000 psi
L/180……….136 psf
L/240………..102 psf
L/360……….(136 psf)(0.5) = 68 psf
4 in. Nominal Thickness, 8 ft, E = 1,500,000 psi
L/180……….347 psf
L/240……….261 psf
L/360……….(347 psf)(0.5) = 173.5 psf

9) Table 8: "Three and Four Inch Nominal Thickness, Allowable Roof Load Limited by Deflection, Controlled Random Layup (3 or more spans)"
3 in. Nominal Thickness, 8 ft, E = 1,500,000 psi
L/180……….205 psf
L/240……….154 psf
L/360……….(205 psf)(0.5) = 102.5 psf
Cantilevered Pieces Intermixed, 3 in., 8 ft, E = 1,500,000 psi

L/180……….(205 psf)(0.90) = 184.5 psf
L/240……….(154 psf)(0.90) = 138.6 psf
L/360……….(184.5 psf)(0.5) = 92.25 psf

Combination Simple Spans and Two-Span Continuous, 3 in., 8 ft
L/180……….(205 psf)(1.13) = 231.65 psf
L/240……….(154 psf)(1.13) = 174.02 psf
L/360……….(231.65 psf)(0.5) = 115.825 psf

Two-Span Continuous, 3 in., 8 ft, E = 1,500,000 psi
L/180……….(205 psf)(1.59) = 325.95 psf
L/240……….(154 psf)(1.59) = 244.86 psf
L/360……….(325.95 psf)(0.5) = 162.975 psf

4 in. Nominal Thickness, 8 ft, E = 1,500,00 psi
L/180……….562 psf
L/240……….421 psf
L/360……….(562 psf)(0.5) = 281 psf

Cantilevered Pieces Intermixed, 4 in. 8 ft, E = 1,500,000 psi
L/180……….(562 psf)(0.90) = 505.8 psf
L/240……….(421 psf)(0.90) = 378.9 psf
L/360……….(505.8 psf)(0.5) = 252.9 psf

Combination Simple Spans and Two-Span Continuous, 4 in., 8 ft
L/180……….(562 psf)(1.13) = 635.06 psf
L/240……….(421 psf)(1.13) = 475.73 psf
L/360……….(635.06 psf)(1.13) = 717.6178 psf

Two-Span Continuous, 4 in., 8 ft, E = 1,500,000 psi
L/180……….(562 psf)(1.59) = 893.58 psf
L/240……….(421 psf)(1.59) = 669.39 psf
L/360……….(893.58 psf)(0.5) = 446.79 psf

Wood Diaphragm:

Support for gravity loads applied to the roof is provided by the 3-inch tongue-and-groove decking. Plywood will be nailed directly into the tongue-and-groove decking to ensure diaphragm action of the roof system.

From ANSI / AF&PA SDPWS-2005 "Special Design Provisions for Wind and Seismic":

Section 4.2.4: Diaphragm Aspect Ratios (p. 14)

Wood structural panel, blocked: Maximum L/W ratio = 3:1

Aspect ratio = (156'/130'):1 = 1.2:1 < 3:1 $\therefore$ OK

Section 4.2.3: Unit Shear Capacities

For ASD allowable unit shear capacity, divide table values (nominal unit shear capacity) by 2.0 (the ASD reduction factor).

Lateral Loads to Sheathing:

SEISMIC LOADS:

Will only see "Building 1" seismic loads

Total load = 8.96 k (level 1) + 31.43 k (level 2) + 40.79 k (level 3) = 81.16 k

(assuming that all lateral load is transferred to roof diaphragm: worst-case scenario)

Longitudinal Direction (North/South):

Assume load is evenly distributed: w_u = (81.16 k)/130' = 0.6243 k/ft

V_u = (0.6243 k/ft)(130')/2 = 40.58 k

$\nu_u = V_u/b$ = (40.58 k)/(156') = 0.26013 k/ft = 260.13 lb/ft

Transverse Direction (East/West):

Assume load is evenly distributed: w_u = (81.16 k)/156' = 0.5203 k/ft

V_u = (0.5203 k/ft)(156')/2 = 40.58 k

$\nu_u = V_u/b$ = (40.58 k)/(130') = 0.31215 k/ft = 312.15 lb/ft

Roof Unit Shears (ASD):

From load combinations: Use 0.7E

Longitudinal Direction: ν = 0.7E = (0.7)(260.13 lb/ft) = 182.09 lb/ft

Transverse Direction: $\nu = 0.7E = (0.7)(312.15 \text{ lb/ft}) = 218.51 \text{ lb/ft}$

Wood Structural Panel Sheathing and Nailing:

Assume load cases 2 and 4.

Transverse Direction (Case 4):

Need table value (from Table A.4.2A) of (218.51 lb/ft)(2) = 437.01 lb/ft

Use:

3/8" Structural I plywood
All edges supported and nailed into 3 in. minimum nominal framing
(blocking is provided by tongue-and-groove decking)
8d common nails at:
6-in. o.c. boundary and continuous panel edges
6-in. o.c. other panel edges (blocking is provided)
12-in. o.c. in field
Allowable $\nu = 600 \text{ lb/ft}/2 = 300 \text{ lb/ft} > 218.51 \text{ lb/ft} \therefore$ OK
$> 182.09 \text{ lb/ft} \therefore$ OK

WIND LOADS:

North/South Direction:

Total load = 66.68 k (level 1) + 46.46 k (level 2) + 37.63 k (level 3) = 150.77 k

Assume that half of total lateral load is transferred to roof diaphragm:

150.77 k/2 = 75.39 k

Longitudinal Direction (North/South):

Assume load is evenly distributed: $w_u = (75.385 \text{ k})/130' = 0.5799 \text{ k/ft}$

$V_u = (0.5799 \text{ k/ft})(130')/2 = 37.69 \text{ k}$

$\nu_u = V_u/b = (37.69 \text{ k})/(156') = 0.24162 \text{ k/ft} = 241.62 \text{ lb/ft}$

East/West Direction:

Total load = 44.89 k (level 1) + 51.49 k (level 2) + 26.85 k (level 3) = 123.23 k

Assume that half of total lateral load is transferred to roof diaphragm:

123.23 k/2 = 61.62 k

Transverse Direction (East/West):

Assume load is evenly distributed: $w_u = (61.62\ k)/156' = 0.3950\ k/ft$

$V_u = (0.3950\ k/ft)(156')/2 = 30.81\ k$

$\nu_u = V_u/b = (30.81\ k)/(130') = 0.2370\ k/ft = 236.98\ lb/ft$

Roof Unit Shears (ASD):

From load combinations: Use 1.0W

Longitudinal Direction: $\nu = 1.0W = (1.0)(241.62\ lb/ft) = 241.62\ lb/ft$

Transverse Direction: $\nu = 1.0W = (1.0)(236.98\ lb/ft) = 236.98\ lb/ft$

Wood Structural Panel Sheathing and Nailing:

Assume load cases 2 and 4.

Transverse Direction (Case 4):

Need table value (from Table A.4.2A) of (241.62 lb/ft)(2) = 483.24 lb/ft

Use:

5/16" Structural I plywood
All edges supported and nailed into 3 in. minimum nominal framing
(blocking is provided by tongue-and-groove decking)
6d common nails at:
6-in. o.c. boundary and continuous panel edges
6-in. o.c. other panel edges (blocking is provided)
12-in. o.c. in field
Allowable ν = 590 lb/ft/2 = 300 lb/ft > 241.62 lb/ft ∴ OK
> 236.98 lb/ft ∴ OK

Seismic load requirements control

∴ Use: 3/8" Structural I plywood
All edges supported and nailed into 3 in. minimum nominal framing
(blocking is provided by tongue-and-groove decking)
8d common nails at:
6-in. o.c. boundary and continuous panel edges
6-in. o.c. other panel edges (blocking is provided)
12-in. o.c. in field
Allowable ν = 600 lb/ft/2 = 300 lb/ft > 218.51 lb/ft ∴ OK
> 182.09 lb/ft ∴ OK

Design of Chords:

Longitudinal Direction:

SEISMIC LOADS:

$M_{u,max} = wL^2/8 = (0.6243\text{ k/ft})(130')^2/8 = 1318.83$ k-ft

$T_u = C_u = M_u/b = 1318.83$ k-ft/156' = 8.454 k

WIND LOADS:

$M_{u,max} = wL^2/8 = (0.5799\text{ k/ft})(130')^2/8 = 1225.039$ k-ft

$T_u = C_u = M_u/b = 1225.039$ k-ft/156' = 7.853 k

$\therefore$ Seismic controls

Check the 3 ½" x 5 ½" Southern Pine glulam ID #50 member already designed for the braced frames at column line 1.

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

F_b = 2100 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

A = 19.25 in^2

E_{min} = 980,000 psi

LOAD COMBINATION: E

Axial Compression:

P = 8.454 kips (Compression)

L = 8.0'

$f_c = P/A = 8{,}454\text{ lb}/19.25\text{ in}^2 = 439.169$ psi

$(l_e/d)_x = [(8.0')(12\text{ in/ft})]/5.5" = 17.4545 < 50 \therefore$ OK

$(l_e/d)_y = 0$ because of lateral support provided by roof diaphragm

$(l_e/d)_{max} = (l_e/d)_x = 17.4545$

The larger slenderness ratio governs the adjusted design value. Therefore, the strong axis of the member is critical, and $(l_e/d)_x$ is used to determine F'_c.

C_D = 1.6 (for seismic load; load combination E)

C_M = 0.73 for F_c (p. 64, NDS Supplement)

C_M = 0.833 for E and E_{min} (p. 64, NDS Supplement)

$C_t = 1.0$

$E'_{min} = (E_{min})(C_M)(C_t) = (980{,}000)(0.833)(1.0) = 816{,}340$ psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340 \text{ psi})]/[(17.4545)^2] = 2202.562$ psi

Here, l_e/d is based on $(l_e/d)_{max}$.

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.6)(0.73)(1.0) = 2686.4$ psi

$F_{cE}/F_c^* = 2202.562/2686.4 = 0.8199$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.8199]/[(2)(0.9)] = 1.0111$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{1.0111\} - \sqrt{\{[1.0111]^2 - [0.8199/0.9]\}}$

$= 0.6776$

$F'_c = F_c^*(C_P) = (2686.4 \text{ psi})(0.6776) = 1820.239 \text{ psi} > f_c = 439.169$ psi $\therefore$ **OK**

Axial Load: P = 8.454 kips (Tension)

Axial Tension:

P = 8.454 kips (Tension)

$F_t = 1550$ psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

$C_D = 1.6$ (for seismic load; load combination E)

$C_M = 0.8$ for F_t (p. 64, NDS Supplement)

$C_t = 1.0$

$F'_t = F_t(C_D)(C_M)(C_t) = (1550 \text{ psi})(1.6)(0.8)(1.0) = 1984$ psi

$P = (F'_t)(A)$

Req'd $A_n = P/F'_t = 8{,}454 \text{ lb}/1984 \text{ psi} = 4.261 \text{ in}^2$

Assume (2) rows of ¾" diameter bolts.

Req'd $A_g = A_n + A_h = 4.261 \text{ in}^2 + (3.5")[(2)(3/4" + 1/16")] = 9.949 \text{ in}^2$

Try 3 ½" x 5 ½" ($A = 19.25 \text{ in}^2 > 9.95 \text{ in}^2$ $\therefore$ OK)

A_n = 19.25 in^2 - (3.5")[(2)(3/4" + 1/16")] = 13.56 in^2

f_t = T/A_n = (8,454 lb)/(13.56 in^2) = 623.34 psi < F'_t = 1984 psi ∴ **OK**

Use 3 ½" x 5 ½" Southern Pine glulam ID #50

<u>Transverse Direction:</u>

SEISMIC LOADS:

$M_{u,max}$ = wL^2/8 = (0.5203 k/ft)(156')^2/8 = 1582.75 k-ft

T_u = C_u = M_u/b = 1582.75 k-ft/130' = 12.175 k

WIND LOADS:

$M_{u,max}$ = wL^2/8 = (0.3950 k/ft)(156')^2/8 = 1201.59 k-ft

T_u = C_u = M_u/b = 1201.59 k-ft/130' = 9.243 k

∴ Seismic controls

Check the 5" x 6 7/8" Southern Pine glulam ID #50 member already designed for the braced frames in the East/West direction.

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

F_b = 2100 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

A = 34.38 in^2

E_{min} = 980,000 psi

LOAD COMBINATION: W

Axial Compression:

P = 12.175 kips (Compression)

L = 26.0'

f_c = P/A = 12,175 lb/34.38 in^2 = 354.130 psi

$(l_e/d)_x$ = [(26.0')(12 in/ft)]/5.0" = 62.4 > 50 ∴ **N.G.**

Try 6 ¾" x 6 7/8"

A = 46.41 in^2

f_c = P/A = 12,175 lb/46.41 in^2 = 262.336 psi

$(l_e/d)_x$ = [(26.0')(12 in/ft)]/6.75" = 46.222 < 50 ∴ OK

$(l_e/d)_y$ = 0 because of lateral support provided by roof diaphragm

$(l_e/d)_{max}$ = $(l_e/d)_x$ = 46.222

The larger slenderness ratio governs the adjusted design value. Therefore, the strong axis of the member is critical, and $(l_e/d)_x$ is used to determine F'_c.

C_D = 1.6 (for seismic load; load combination E)

C_M = 0.73 for F_c (p. 64, NDS Supplement)

C_M = 0.833 for E and E_{min} (p. 64, NDS Supplement)

C_t = 1.0

$E'_{min} = (E_{min})(C_M)(C_t)$ = (980,000)(0.833)(1.0) = 816,340 psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2]$ = [(0.822)(816,340 psi)]/[(46.222)^2] = 314.081 psi

Here, l_e/d is based on $(l_e/d)_{max}$.

$F_c^* = F_c(C_D)(C_M)(C_t)$ = (2300 psi)(1.6)(0.73)(1.0) = 2686.4 psi

F_{cE}/F_c^* = 314.081/2686.4 = 0.1169

$[1 + F_{cE}/F_c^*]/(2c)$ = [1 + 0.1169]/[(2)(0.9)] = 0.6205

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.6205\} - \sqrt{\{[0.6205]^2 - [0.1169/0.9]\}}$

= 0.1154

$F'_c = F_c^*(C_P)$ = (2686.4 psi)(0.1154) = 309.969 psi < f_c = 354.130 psi ∴ **N.G.**

Try 6 ¾" x 8 ¼"

A = 55.69 in^2

f_c = P/A = 12,175 lb/55.69 in^2 = 218.621 psi

$(l_e/d)_x$ = [(26.0')(12 in/ft)]/8.25" = 37.818 < 50 ∴ OK

$(l_e/d)_y$ = 0 because of lateral support provided by roof diaphragm

$(l_e/d)_{max}$ = $(l_e/d)_x$ = 37.818

The larger slenderness ratio governs the adjusted design value. Therefore, the strong axis of the member is critical, and $(l_e/d)_x$ is used to determine F'_c.

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340 \text{ psi})]/[(37.818)^2] = 469.182$ psi

Here, l_e/d is based on $(l_e/d)_{max}$.

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.6)(0.73)(1.0) = 2686.4$ psi

$F_{cE}/F_c^* = 469.182/2686.4 = 0.1747$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.1747]/[(2)(0.9)] = 0.6526$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 – [F_{cE}/F_c^*]/c\}}$

$= \{0.6526\} - \sqrt{\{[0.6526]^2 – [0.1747/0.9]\}}$

$= 0.1712$

$F'_c = F_c^*(C_P) = (2686.4 \text{ psi})(0.1712) = 459.888 \text{ psi} > f_c = 218.621$ psi $\therefore$ **O.K.**

Axial Tension:

P = 12.175 kips (Tension)

F_t = 1550 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

C_D = 1.6 (for seismic load; load combination E)

C_M = 0.8 for F_t (p. 64, NDS Supplement)

C_t = 1.0

$F'_t = F_t(C_D)(C_M)(C_t) = (1550 \text{ psi})(1.6)(0.8)(1.0) = 1984$ psi

$P = (F'_t)(A)$

Req'd $A_n = P/F'_t = 12{,}175 \text{ lb}/1984 \text{ psi} = 6.137 \text{ in}^2$

Assume (2) rows of ¾" diameter bolts.

Req'd $A_g = A_n + A_h = 6.137 \text{ in}^2 + (6.75")[(2)(3/4" + 1/16")] = 17.106 \text{ in}^2$

Try 6 ¾" x 8 ¼" ($A = 55.69 \text{ in}^2 > 17.106 \text{ in}^2$ $\therefore$ OK)

$A_n = 55.69 \text{ in}^2 - (6.75")[(2)(3/4" + 1/16")] = 44.721 \text{ in}^2$

$f_t = T/A_n = (12{,}175 \text{ lb})/(44.72 \text{ in}^2) = 272.242 \text{ psi} < F'_t = 1984$ psi $\therefore$ **OK**

Use 6 ¾" x 8 ¼" Southern Pine glulam ID #50

Wood Truss Member Connections

Bolted Metal Side Plates

Bottom Chord Heel Connections

Maximum tension force at heel (from bottom chord):

D + S = (24.616 k + 7.979 k) + 18.954 k = 51.549 k

D + L_r = (24.616 k + 7.979 k) + 16.411 k = 49.006 k

Other load combinations will not control by inspection.

LOAD COMBINATION: D + S

For 6 ¾” thick southern pine glulam member, wit h ¼” steel side plates, load applied parallel to grain, the nominal design value “Z” of a ¾” bolt in double shear is:

Z = 3460 lb (Table 11I, p. 90, NDS)

The allowable bolt design value is:

$Z' = (Z)(C_D)(C_M)(C_t)(C_g)(C_\Delta)(C_{eg})(C_{di})(C_{tn})$

$C_D = 1.15$

$C_M = 0.7$ (for dowel-type fasteners with in-service moisture content > 19%)

$C_t = 1.0$

$C_{eg} = C_{di} = C_{tn} = 1.0$

$Z' = (3480 \text{ lb})(1.15)(0.7)(1.0)(C_g)(C_\Delta)(1.0)(1.0)(1.0) = (2801.4 \text{ lb})(C_g)(C_\Delta)$

Check bolt spacing and edge distances:

Bottom Chord: 6 ¾” x 8 ¼”

Table 11.5.1A: Edge Distance Requirements

Parallel to Grain:

l/D = minimum of [l_m/D or l_s/D]

l_m/D = 6.75”/0.75” = 9

l_s/D = (2)(1/4”)/0.75” = 0.667 (Governs)

$l/D = 0.667 < 6$ ∴ Min. Edge Distance = 1.5D = (1.5)(0.75") = 1.125"

Table 11.5.1B: End Distance Requirements

Direction of Loading is Parallel to Grain, Tension: (fastener bearing toward member end)

For softwoods: Minimum End Distance for $C_\Delta = 0.5$ is 3D = (3)(0.75") = 2.625"

Minimum End Distance for $C_\Delta = 1.0$ is 7D = (7)(0.75") = 5.25"

Table 11.5.1C: Spacing Requirements for Fasteners in a Row

Direction of Loading is Parallel to Grain:

Minimum Spacing = 3D = (3)(0.75") = 2.25"

Minimum Spacing for $C_\Delta = 1.0$ is 4D = (4)(0.75") = 3.0"

Table 11.5.1D: Spacing Requirements Between Rows

Direction of Loading is Parallel to Grain:

Minimum Spacing = 1.5D = (1.5)(0.75") = 1.125"

Spacing between outer rows of bolts ≤ 5"

Assuming that all bolt spacing, edge distances, and end distances meet the requirements for $C_\Delta = 1.0$

$Z' = (2801.4 \text{ lb})(C_g)(C_\Delta) = (2801.4 \text{ lb})(C_g)(1.0) = 2801.4 \text{ lb}(C_g)$

# of bolts required = (51,549 lb)/(2801.4 lb/bolt) = 18.4 bolts ∴ try 20 bolts

Try (20) ¾" bolts arranged in (2) rows of ten each.

Check bolt capacity with group action:

Area of main member: $A_m = (6.75")(8.25") = 55.69 \text{ in}^2$

Area of side plates, assuming ¼" x 6", is

$A_s = (2)[(0.25")(6")] = 3.0 \text{ in}^2$

$A_m/A_s = (55.69 \text{ in}^2)/(3.0 \text{ in}^2) = 18.5633$

Table 10.3.6C (NDS): Group Action Factors, C_g, for Bolt or Lag Screw Connections with Steel Side Plates

(Tabulated group action factors (C_g) are conservative for D < 1" or s < 4")

For A_m/A_s = 18:

A_m = 40 in^2……..(10) fasteners per row……..C_g = 0.80

A_m = 64 in^2……..(10) fasteners per row……..C_g = 0.86

Interpolate for A_m = 55.69 in^2: C_g = 0.8392

For A_m/A_s = 24:

A_m = 40 in^2……..(10) fasteners per row……..C_g = 0.79

A_m = 64 in^2……..(10) fasteners per row……..C_g = 0.85

Interpolate for A_m = 55.69 in^2: C_g = 0.8292

Interpolate for A_m/A_s = 18.5633: C_g = 0.8383

Connection Capacity = (20 bolts)(2801.4 lb)(0.8383) = 46,968 lb < 51,549 lb ∴ N.G.

Try (22) ¾" bolts arranged in (2) rows of eleven each.

Table 10.3.6C (NDS): Group Action Factors, C_g, for Bolt or Lag Screw Connections with Steel Side Plates

(Tabulated group action factors (C_g) are conservative for D < 1" or s < 4")

For A_m/A_s = 18:

A_m = 40 in^2……..(11) fasteners per row……..C_g = 0.77

A_m = 64 in^2……..(11) fasteners per row……..C_g = 0.83

Interpolate for A_m = 55.69 in^2: C_g = 0.8092

For A_m/A_s = 24:

A_m = 40 in^2……..(11) fasteners per row……..C_g = 0.76

A_m = 64 in^2……..(11) fasteners per row……..C_g = 0.83

Interpolate for A_m = 55.69 in^2: C_g = 0.8058

Interpolate for A_m/A_s = 18.5633: C_g = 0.8089

Connection Capacity = (22 bolts)(2801.4 lb)(0.8089) = 49,853 lb < 51,549 lb ∴ N.G.

Try (24) ¾" bolts arranged in (2) rows of twelve each.

Table 10.3.6C (NDS): Group Action Factors, C_g, for Bolt or Lag Screw Connections with Steel Side Plates

(Tabulated group action factors (C_g) are conservative for D < 1" or s < 4")

For $A_m/A_s = 18$:

$A_m = 40\ in^2$……..(11) fasteners per row……..$C_g = 0.73$

$A_m = 64\ in^2$……..(11) fasteners per row……..$C_g = 0.81$

Interpolate for $A_m = 55.69\ in^2$: $C_g = 0.7823$

For $A_m/A_s = 24$:

$A_m = 40\ in^2$……..(11) fasteners per row……..$C_g = 0.72$

$A_m = 64\ in^2$……..(11) fasteners per row……..$C_g = 0.80$

Interpolate for $A_m = 55.69\ in^2$: $C_g = 0.7723$

Interpolate for $A_m/A_s = 18.5633$: $C_g = 0.7814$

Connection Capacity = (24 bolts)(2801.4 lb)(0.7814) = 52,536 lb > 51,549 lb ∴ O.K.

LOAD COMBINATION: D + L_r

P = 49,006 lb

$C_D = 1.0$

The allowable bolt design value is:

$Z' = (Z)(C_D)(C_M)(C_t)(C_g)(C_\Delta)(C_{eg})(C_{di})(C_{tn})$

$Z' = (3480\ lb)(1.0)(0.7)(1.0)(C_g)(C_\Delta)(1.0)(1.0)(1.0) = (2436\ lb)(C_g)(C_\Delta)$

Assuming that all bolt spacing, edge distances, and end distances meet the requirements for $C_\Delta = 1.0$

$Z' = (2436\ lb)(C_g)(C_\Delta) = (2436\ lb)(C_g)(1.0) = 2436\ lb(C_g)$

# of bolts required = (49,006 lb)/(2436 lb/bolt) = 20.12 bolts ∴ try 22 bolts

Try (22) ¾" bolts arranged in (2) rows of eleven each.

$C_g = 0.8089$

Connection Capacity = (22 bolts)(2436 lb)(0.8089) = 43,351 lb < 49,006 lb ∴ N.G.

Try (24) ¾" bolts arranged in (2) rows of twelve each.

$C_g = 0.7814$

Connection Capacity = (24 bolts)(2436 lb)(0.7814) = 45,684 lb < 49,006 lb ∴ N.G.
Try (26) ¾" bolts arranged in (2) rows of thirteen each.

Group Action Factor, C_g

$C_g = \{[(m)(1-m^{2n})]/[(n)((1+R_{EA}m^n)(1+m) - 1 + m^{2n})]\}[(1+R_{EA})/(1-m)]$

n = number of fasteners in a row = 13

R_{EA} = lesser of $(E_sA_s)/(E_mA_m)$ or $(E_mA_m)/(E_sA_s)$

$E_s = 29{,}000{,}000$ psi

$A_s = 3.0\ in^2$

$E_m = 1{,}900{,}000$ psi

$A_m = 55.69\ in^2$

$(E_sA_s)/(E_mA_m) = [(29{,}000{,}000\ psi)(3.0\ in^2)]/[(1{,}900{,}000\ psi)(55.69\ in^2)]$

$= 0.8222$

$(E_mA_m)/(E_sA_s) = [(1{,}900{,}000\ psi)(55.69\ in^2)]/[(29{,}000{,}000\ psi)(3.0\ in^2)]$

$= 1.2162$

$\therefore R_{EA} = 0.8222$

s = 3"

$\gamma = (270{,}000)(D^{1.5}) = (270{,}000)(0.75)^{1.5} = 175{,}370.14$

$u = 1 + (\gamma)(s/2)[(1/(E_mA_m)) + (1/(E_sA_s))]$

$= 1 + (175{,}370.14)(3/2)[(1/(1{,}900{,}000)(55.69))+(1/(29{,}000{,}000)(3.0))]$

$= 1.005510$

$m = u - \sqrt{(u^2 - 1)} = 1.005510 - \sqrt{(1.005510^2 - 1)} = 0.90039$

$C_g = \{[(0.90039)(1 - (0.90039)^{2(13)})]/[(13)(1+(0.8222)(0.90039)^{13})(1+0.90039) - 1 +$

$+ (0.90039)^{2(13)})\}[(1+0.8222)/(1-0.90039)]$

$= 0.8675$

Connection Capacity = (26 bolts)(2436 lb)(0.8675) = 54,944 lb > 49,006 lb $\therefore$ **O.K.**

Try (24) ¾" bolts arranged in (2) rows of twelve each using calculated C_g from equation.

Group Action Factor, C_g

$C_g = \{[(m)(1\text{-}m^{2n})]/[(n)((1+R_{EA}m^n)(1+m) - 1 + m^{2n})]\}[(1+R_{EA})/(1\text{-}m)]$

n = number of fasteners in a row = 12

R_{EA} = 0.8222 (from previous)

s = 3"

γ = 175,370.14 (from previous)

u = 1.005510 (from previous)

m = 0.90039 (from previous)

$C_g = \{[(0.90039)(1 - (0.90039)^{2(12)})]/[(12)((1+(0.8222)(0.90039)^{12})(1+0.90039) - 1 +$

$+ (0.90039)^{2(12)})\}[(1+0.8222)/(1\text{-}0.90039)]$

$= 0.8858$

Connection Capacity = (24 bolts)(2436 lb)(0.8858) = 51,787 lb > 49,006 lb $\therefore$ **O.K.**

Try 4-in-diameter shear plates with ¾" bolts.

For Southern Pine, the specific gravity G = 0.55

Table 12A: Species Group B (for $0.49 \leq G < 0.60$)

The capacity of a 4-in shear plate with steel side plates, ¾" bolt, using species group B, loaded parallel to grain per NDS Table 12.2B:

P = 4320 lb

Table 12.3: Geometry Factors, C_Δ, for Split Ring and Shear Plate Connectors

Edge Distance: Parallel to Grain Loading

Minimum for C_Δ = 1.0 is 2 ¾"

End Distance: Parallel to Grain Loading, Tension Member

Minimum for C_Δ = 1.0 is 7"

Spacing: Parallel to Grain Loading

Spacing Parallel to Grain:

Minimum for C_Δ = 1.0 is 9”

Spacing Perpendicular to Grain:

Minimum for C_Δ = 1.0 is 5”

Assuming that all bolt spacing, edge distances, and end distances meet the requirements for C_Δ = 1.0

C_{st} = 1.11 (Table 12.2.4, Species Group B)

$P' = (P)(C_D)(C_M)(C_t)(C_g)(C_\Delta)(C_d)(C_{st})$

$= (4230 \text{ lb})(1.0)(0.7)(1.0)(C_g)(1.0)(1.0)(1.11)$

$= (3286.71 \text{ lb})(C_g)$

Number of shear plates required is:

(49,006 lb)/(3286.71 lb) = 14.91 = 15 shear plates

Due to excessive number of shear plates and required room for spacing of shear plates, use the (24) ¾” bolts for the connection.

Check Minimum End Distance for Steel Plates:

¾” bolts, ¼” steel plates (A36)

Assume end distance for steel plates = 1.5”

End bolts: L_c = 1.5” – (1/2)(3/4” + 1/16”) = 1.094” < 2d = (2)(0.75”) = 1.5”

∴ Tear-out Controls

$\phi r_n = \phi 1.2 F_u L_c t$ = (0.75)(1.2)(58 ksi)(1.094”)(0.25”) = 14.273 k

Bolt Shear Strength: ϕr_n = 15.9 k (for single ¾” A325N bolts)

Interior Bolts: L_c = 3 – (3/4” + 1/16”) = 2.188” > 2d = 1.5”

∴ Bearing Controls

$\phi r_n = \phi 2.4 d t F_u$ = (0.75)(2.4)(0.75”)(0.25”)(58 ksi) = 19.575 k

∴ Bolt shear strength controls for interior bolts.

ϕR_n = (2)(14.273 k) + (22)(15.9 k) = 378.346 k

P_u = 1.2D + 1.6S = (1.2)(24.616 k + 7.979 k) + (1.6)(18.954 k) = 69.440 k

P_u for each steel plate = (69.440 k)/2 = 34.720 k

ϕR_n = 378.346 k > P_u = 34.720 k ∴ **OK**

Block shear strength of steel plates is OK by inspection.

FINAL CONNECTION:

Use (24) ¾" bolts arranged in two rows of (12) each with ¼" steel side plates.

Bottom Chord Splice Connections

LOAD COMBINATION: D + L_r (controls)

Assume bottom chord is spliced at quarter points.

Maximum tension force at splice = 51,315 lb

Assume same steel side plates, spacing, and edge distances as used for the bottom chord heel connection.

(24) ¾" bolts arranged in (2) rows of twelve each will work (from previous calculations):

Connection Capacity = (24 bolts)(2436 lb)(0.8858) = 51,787 lb > 51,315 lb ∴ **O.K.**

Check Minimum End Distance for Steel Plates:

¾" bolts, ¼" steel plates (A36)

Assume end distance for steel plates = 1.5"

End bolts: L_c = 1.5" – (1/2)(3/4" + 1/16") = 1.094" < 2d = (2)(0.75") = 1.5"

∴ Tear-out Controls

$\phi r_n = \phi 1.2 F_u L_c t$ = (0.75)(1.2)(58 ksi)(1.094")(0.25") = 14.273 k

Bolt Shear Strength: ϕr_n = 15.9 k (for single ¾" A325N bolts)

Interior Bolts: L_c = 3 – (3/4" + 1/16") = 2.188" > 2d = 1.5"

∴ Bearing Controls

$\phi r_n = \phi 2.4 d t F_u$ = (0.75)(2.4)(0.75")(0.25")(58 ksi) = 19.575 k

∴ Bolt shear strength controls for interior bolts.

ϕR_n = (2)(14.273 k) + (22)(15.9 k) = 378.346 k

P_u = 1.2D + 1.6S = (1.2)(25.732 k + 8.428 k) + (1.6)(19.814 k) = 72.694 k

P_u for each steel plate = (72.694 k)/2 = 36.347 k

ϕR_n = 378.346 k > P_u = 36.347 k ∴ **OK**

Block shear strength of steel plates is OK by inspection.

FINAL CONNECTION:

Use (24) ¾" bolts arranged in two rows of (12) each with ¼" steel side plates.

Top Chord Member Connections

LOAD COMBINATON: D + L_r (controls)

P = 58,247 lb (compression)

C_D = 1.0

For 6 ¾" thick southern pine glulam member, wit h ¼" steel side plates, load applied parallel to grain, the nominal design value "Z" of a ¾" bolt in double shear is:

Z = 3460 lb (Table 11I, p. 90, NDS)

The allowable bolt design value is:

$Z' = (Z)(C_D)(C_M)(C_t)(C_g)(C_\Delta)(C_{eg})(C_{di})(C_{tn})$

$Z' = (3480\ lb)(1.0)(0.7)(1.0)(C_g)(C_\Delta)(1.0)(1.0)(1.0) = (2436\ lb)(C_g)(C_\Delta)$

Assuming that all bolt spacing, edge distances, and end distances meet the requirements for C_Δ = 1.0

$Z' = (2436\ lb)(C_g)(C_\Delta) = (2436\ lb)(C_g)(1.0) = 2436\ lb(C_g)$

\# of bolts required = (58,247 lb)/(2436 lb/bolt) = 23.91 bolts ∴ try 24 bolts

Try (24) ¾" bolts arranged in (2) rows of twelve each.

Group Action Factor, C_g

$C_g = \{[(m)(1-m^{2n})]/[(n)((1+R_{EA}m^n)(1+m) - 1 + m^{2n})]\}[(1+R_{EA})/(1-m)]$

n = number of fasteners in a row = 12

R_{EA} = lesser of $(E_sA_s)/(E_mA_m)$ or $(E_mA_m)/(E_sA_s)$

E_s = 29,000,000 psi

$A_s = (2)[(1/4")(8")] = 4.0\ in^2$

E_m = 1,900,000 psi

$A_m = 83.53\ in^2$

$(E_sA_s)/(E_mA_m) = [(29,000,000\ psi)(4.0\ in^2)]/[(1,900,000\ psi)(83.53\ in^2)]$

$= 0.7309$

$(E_mA_m)/(E_sA_s) = [(1,900,000\ psi)(83.53\ in^2)]/[(29,000,000\ psi)(4.0\ in^2)]$

$= 1.3682$

$\therefore R_{EA} = 0.7309$

s = 3"

$\gamma = (270,000)(D^{1.5}) = (270,000)(0.75)^{1.5} = 175,370.14$

$u = 1 + (\gamma)(s/2)[(1/(E_mA_m)) + (1/(E_sA_s))]$

$= 1 + (175,370.14)(3/2)[(1/(1,900,000)(83.53))+(1/(29,000,000)(4.0))]$

$= 1.003925$

$m = u - \sqrt{(u^2 - 1)} = 1.003925 - \sqrt{(1.003925^2 - 1)} = 0.91524$

$C_g = \{[(0.91524)(1 - (0.91524)^{2(12)})]/[(12)((1+(0.7309)(0.91524)^{12})(1+0.91524) - 1 +$

$+ (0.91524)^{2(12)})\}[(1+0.7309)/(1-0.91524)]$

$= 0.9034$

Connection Capacity = (24 bolts)(2436 lb)(0.9034) = 52,816 lb < 58,247 lb ∴ **N.G.**

Try (26) ¾" bolts arranged in (2) rows of thirteen each.

Group Action Factor, C_g

$C_g = \{[(m)(1-m^{2n})]/[(n)((1+R_{EA}m^n)(1+m) - 1 + m^{2n})]\}[(1+R_{EA})/(1-m)]$

n = number of fasteners in a row = 13

R_{EA} = 0.7309 (from previous)

s = 3”

γ = 175,370.14 (from previous)

u = 1.003925 (from previous)

m = 0.91524 (from previous)

$C_g = \{[(0.91524)(1 - (0.91524)^{2(13)})]/[(13)((1+(0.7309)(0.91524)^{13})(1+0.91524) - 1 +$

$+ (0.91524)^{2(13)})\}[(1+0.7309)/(1-0.91524)]$

$= 0.8876$

Connection Capacity = (26 bolts)(2436 lb)(0.8876) = 56,217 lb < 58,247 lb ∴ **N.G.**

Try (28) ¾” bolts arranged in (2) rows of fourteen each.

Group Action Factor, C_g

$C_g = \{[(m)(1-m^{2n})]/[(n)((1+R_{EA}m^n)(1+m) - 1 + m^{2n})]\}[(1+R_{EA})/(1-m)]$

n = number of fasteners in a row = 14

R_{EA} = 0.7309 (from previous)

s = 3”

γ = 175,370.14 (from previous)

u = 1.003925 (from previous)

m = 0.91524 (from previous)

$C_g = \{[(0.91524)(1 - (0.91524)^{2(14)})]/[(14)((1+(0.7309)(0.91524)^{14})(1+0.91524) - 1 +$

$+ (0.91524)^{2(14)})\}[(1+0.7309)/(1-0.91524)] = 0.8712$

Connection Capacity = (28 bolts)(2436 lb)(0.8712) = 59,423 lb > 58,247 lb ∴ **O.K.**

FINAL CONNECTION:

Use (28) ¾” bolts arranged in two rows of (14) each with ¼” steel side plates.

Appendix B – Structural Depth: Lateral System Calculations

Wind Calculations

Method 2 – Analytical Procedure

Building Natural Frequency = n_1

For concrete moment-resisting frames: $n_1 = 43.5/H^{0.9}$

H = building height = 60'

$n_1 = (43.5)/((60)^{0.9}) = 43.5/39.842 = 1.092 > 1$ Hz therefore $\therefore$ Structure is rigid

> *Building and Other Structure, Flexible: Slender buildings and other structures that have a fundamental natural frequency less than 1 Hz (p. 21).

$g_Q = g_v = 3.4$

$z = 0.6h = (0.6)(60') = 36' > z_{min} = 15'$ (Table 6-2, Exposure C)

> Use maximum roof height for "h" (most conservative) instead of trying to estimate mean roof height of curved roof.

$I_z = c[(33/z)^{1/6}] = (0.20)[(33/36)^{1/6}] = 0.1971$

> c = 0.20 (Table 6-2, Exposure C)

$L_z = l(z/33)^{\epsilon} = (500')(36/33)^{0.20} = 508.7773$

> l = 500' (Table 6-2, Exposure C)
>
> $\in$ = 1/5.0 = 0.20 (Table 6-2, Exposure C)

$Q = \sqrt{[1/(1 + 0.63((B+h)/L_z)^{0.63})]}$

North/South:

> B = 183'
>
> L = 156'
>
> $Q_{N/S} = \sqrt{[1/(1 + 0.63((183'+36')/508.777')^{0.63})]} = 0.9272$

East/West:

> B = 156'
>
> L = 183'

$Q_{E/W} = \sqrt{[1/(1 + 0.63((156'+36')/508.777')^{0.63})]} = 0.8636$

G = 0.85 or

$G = 0.925[(1 + 1.7g_Q I_z Q)/(1 + 1.7g_v I_z)]$

North/South:

$G_{N/S} = 0.925[(1 + 1.7g_Q I_z Q_{N/S})/(1 + 1.7g_v I_z)]$

$= 0.925[(1 + [(1.7)(3.4)(36)(0.9272)]/(1 + 1.7(3.4)(36))] = 0.8579848361$

$\therefore$ use $G_{N/S} = 0.8580$

East/West:

$G_{E/W} = 0.925[(1 + 1.7g_Q I_z Q_{E/W})/(1 + 1.7g_v I_z)]$

$= 0.925[(1 + [(1.7)(3.4)(36)(0.8636)]/(1 + 1.7(3.4)(36))] = 0.7994$

$\therefore$ use $G_{E/W} = 0.85$

Velocity Pressure:

V = 90 m.p.h. (Figure 6-1)

$K_d = 0.85$ (Table 6-4)

I = 1.15 (Table 6-1, Occupancy Category III)

Exposure Category = C

$K_{zt} = 1.0$ (ASCE 7-05, 6.5.7.2)

Level	Height	K_z
1	10.50'	0.85
2	24.67'	0.937
3	40.00'	1.04
4	60.00'	1.13

(Values of K_z from Table 6-2, Exposure C)

$K_h = 1.13$ (using maximum roof height to be conservative)

$q_z = 0.00256K_zK_{zt}K_dV^2I$

Level 1: $q_z = (0.00256)(0.85)(1.0)(0.85)(90^2)(1.15) = 17.2290$ psf

Level 2: $q_z = (0.00256)(0.937)(1.0)(0.85)(90^2)(1.15) = 18.9992$ psf

Level 3: $q_z = (0.00256)(1.04)(1.0)(0.85)(90^2)(1.15) = 21.0802$ psf

Level 4: $q_z = (0.00256)(1.13)(1.0)(0.85)(90^2)(1.15) = 22.9045$ psf
$= q_h = 22.9045$ psf

Pressure Coefficients, C_p, for the Walls and Roof (Figure 6-6):

Wall Pressure Coefficients, C_p

North/South:

Windward Wall: $C_p = 0.8$

Leeward Wall: C_p = L/B = 156'/183' = 0.852 ∴ $C_p = -0.5$

Side Wall: $C_p = -0.7$

East/West:

Windward Wall: $C_p = 0.8$

Leeward Wall: C_p = L/B = 183'/156' = 1.173 ∴ $C_p = -0.4654$

Side Wall: $C_p = -0.7$

Roof Pressure Coefficients, C_p, for use with q_h

Since roof slope, θ, for curved roof is less than 10˚ for most of the roof, use "Normal to ridge for <10 and Parallel to ridge for all θ."

North/South:

h/L = 60'/156' = 0.3846

Horizontal Distance from Windward Edge	C_p
0 to h/2	-0.9, -0.18
h/2 to	-0.9, -0.18
h to 2h	-0.5, -0.18
>2h	-0.3, -0.18

Use worst case scenario: $C_p = -0.9$ for entire roof

East/West:

h/L = 60'/183' = 0.3279

Same chart (above, for North/South) applies

Use worst case scenario: $C_p = -0.9$ for entire roof

Or use "Arched Roofs", Figure 6-8, ASCE 7-05

Rise-to-Span Ratio: r = 20'/130' = 0.1538 < 0.2

∴ C_p for Windward Quarter = -0.9

C_p for Center Half = -0.7 – r = -0.7 – 0.1538 = -0.8538

C_p for Leeward Quarter = -0.5

Conservatively use C_p = -0.9 for entire roof

Internal Pressure Coefficients (GC_{pi}) (Figure 6-5):

Enclosed Buildings: GC_{pi} = +0.18
= -0.18

Design Wind Pressures:

Windward Walls: $p_z = q_zGC_p - q_i(GC_{pi})$

However, internal pressures cancel on MLFRS

∴ $p_z = q_zGC_p$

Leeward Walls, Side Walls, and Roofs: $p_h = q_hGC_p - q_i(GC_{pi})$

However, internal pressures cancel on MLFRS

∴ $p_h = q_hGC_p$

North/South:

Windward Walls:

$p_z = q_zGC_p = (q_z)(0.858)(0.8) = 0.6864(q_z)$

(Varies by level, see Table)

Leeward Walls:

$p_h = q_hGC_p = (21.080)(0.858)(-0.5) = -9.0433$ psf

Side Walls:

$p_h = q_hGC_p = (21.080)(0.858)(-0.7) = -12.6606$ psf

Roof:

$p_h = q_hGC_p = (21.080)(0.858)(-0.9) = -16.2779$ psf

East/West:

Windward Walls:

$p_z = q_zGC_p = (q_z)(0.85)(0.8) = 0.68(q_z)$

(Varies by level, see Table)

Leeward Walls:

$p_h = q_hGC_p = (21.080)(0.85)(-0.4654) = -8.3391$ psf

Side Walls:

$p_h = q_hGC_p = (21.080)(0.85)(-0.7) = -12.5427$ psf

Roof:

$p_h = q_hGC_p = (21.080)(0.85)(-0.9) = -16.1264$ psf

*Forces, base shear, and moments are shown in spreadsheets

Wind Forces for Lateral Force Resisting System:

W = Wind Load

North/South: "Building 1"

Level 1:

W = (11.83 PSF + 9.04 PSF)(742.7109 SF) + (13.04 PSF + 9.04 PSF)(1002.0703 SF) =

= 37,626.09 lb = 37.626 kips

Level 2:

W = (13.04 PSF + 9.04 PSF)(1002.0703 SF) + (14.47 PSF + 9.04 PSF)(1034.8958 SF) =

= 46,456.11 lb = 46.456 kips

Level 3:

W = (14.47 PSF + 9.04 PSF)(996.6667 SF) + (15.72 PSF + 9.04 PSF)(1746.6029 SF) =

= 66,677.52 lb = 66.678 kips

OR if only looking at Level 2 and Level 3 for wind loads for "Building 1":

Level 2:

W = (13.04 PSF + 9.04 PSF)(1744.7813 SF) + (14.47 PSF + 9.04 PSF)(1034.8958 SF) =

= 62,855.17 lb = 62.855 kips

Level 3:

W = (14.47 PSF + 9.04 PSF)(996.6667 SF) + (15.72 PSF + 9.04 PSF)(1746.6029 SF) =

= 66,667.52 lb = 66.678 kips

North/South: "Building 4"

Level 2:

W = (13.04 PSF + 9.04 PSF)(499.8854 SF) + (14.47 PSF + 9.04 PSF)(135.1042 SF) =

= 14,213.77 lb = 14.214 kips

East/West:

Level 1:

W = (11.72 PSF + 8.34 PSF)(920.9375 SF) + (12.92 PSF + 8.34 PSF)(1242.5347 SF) =

= 44,890.29 lb = 44.890 kips

Level 2:

W = (12.92 PSF + 8.34 PSF)(1153.4239 SF) + (14.33 PSF + 8.34 PSF)(1189.5000 SF) =

= 51,487.76 lb = 51.488 kips

Level 3:

W = (14.33 PSF + 8.34 PSF)(1184.5000 SF) = 26.852 kips

Seismic Calculations

Equivalent Lateral Force Procedure

S_S = 0.20 (Figure 22-1, ASCE 7-05) (Also from www.seismicfactor.com)

S_1 = 0.054 (Figure 22-1, ASCE 7-05) (Also from www.seismicfactor.com)

Occupancy Category III, Site Class C

F_a = 1.2 (Table 11.4-1) ($S_S \leq 0.25$, Site Class C)

F_v = 1.7 (Table 11.4-2) ($S_1 \leq 0.1$, Site Class C)

$S_{MS} = F_aS_S = (1.2)(0.20) = 0.24$ (Eq. 11.4-1)

$S_{M1} = F_vS_1 = (1.7)(0.054) = 0.0918$ (Eq. 11.4-2)

$S_{DS} = (2/3)(S_{MS}) = (2/3)(0.24) = 0.16$ (Eq. 11.4-3)

$S_{D1} = (2/3)(S_{M1}) = (2/3)(0.0918) = 0.0612$ (Eq. 11.4-4)

Seismic Design Category based on S_{DS} (Table 11.6-1):

$S_{DS} = 0.16 < 0.167$, Occupancy Category III: SDC A

Seismic Design Category based on S_{D1}:

$S_{D1} = 0.0612 < 0.067$, Occupancy Category III: SDC A

Use most severe of the two Seismic Design Categories: (same in this case)

Seismic Design Category A

Could use methods of 11.7 "Design Requirements for Seismic Design Category A" (Lateral Forces: $F_x = 0.01w_x$) but continue to solve for C_s instead.

For Wood Braced Frames:

R = 4 (Table 12.2-1) (Light-framed wall systems using flat strap bracing)

I = 1.25 (Table 11.5-1) (Occupancy Category III)

$T_a = C_th_n^x$

C_t = 0.02 (Table 12.8-2)

h_n = 60'

x = 0.75 (Table 12.8-2)

$T_a = (0.02)(60')^{0.75} = 0.4312$

T_L = 6 seconds (Figure 22-15)

$T = T_a = 0.4312$ (this is allowed per Section 12.8.2, ASCE 7-05)

$< C_uT_a = (1.7)(0.4312) = 0.7330$

C_s = minimum of

$S_{DS}/(R/I) = 0.16/(4/1.25) = 0.05$

$S_{D1}/[(T)(R/I)] = 0.0612/[(0.4312)(4/1.25)] = 0.044353$

C_s = 0.044353

For Concrete Moment Frames:

R = 3 (Table 12.2-1) (Ordinary reinforced concrete moment frames)

I = 1.25 (Table 11.5-1) (Occupancy Category III)

$T_a = C_th_n^x$

C_t = 0.016 (Table 12.8-2)

h_n = 60'

x = 0.9 (Table 12.8-2)

$T_a = (0.016)(60')^{0.9} = 0.6375$

T_L = 6 seconds (Figure 22-15)

$T = T_a = 0.4312$ (this is allowed per Section 12.8.2, ASCE 7-05)

$< C_uT_a = (1.7)(0.6375) = 1.0837$

C_s = minimum of

$S_{DS}/(R/I) = 0.16/(3/1.25) = 0.066667$

$S_{D1}/[(T)(R/I)] = 0.0612/[(0.6375)(3/1.25)] = 0.040002$

C_s = 0.040002

Use **C_s = 0.044353** for entire building (worst case)

$V = C_sW$ (see spreadsheets for weights of building components, seismic forces, and story shears)

Stiffness Values

The stiffness of each frame at each applicable level was determined by applying a 1 kip load to the frame at that particular level and determining the displacement of the frame at that level. SAP was used to determine the displacements. The stiffness is equal to the 1 kip load divided by the displacement.

$$k = P/\Delta$$

Stiffness Values (k-values) - North/South Direction				
	Level	P (kips)	Deflection (in.)	k = P/Defl. (kip/in)
Braced Frame - Column Line 1	1	1	0.010448	95.712
Braced Frame - Column Line 1	2	1	0.032685	30.595
Braced Frame - Column Line 1	3	1	0.077295	12.937
Moment Frame - Column Line 1.8	1	1	0.002836	352.609
Moment Frame - Column Line 2	2	1	0.006298	158.781
Moment Frame - Column Line 2	3	1	0.014274	70.057
Moment Frame - Column Line 4	2	1	0.046756	21.388

Table ____ - Stiffness Values for Wood Braced Frames, Concrete Moment Frames, and Steel Moment Frame – North/South Direction

Stiffness Values (k-values) - East/West Direction				
	Level	P (kips)	Deflection (in.)	k = P/Defl. (kip/in)
Concrete Moment Frame	1	1	0.014789	67.618
Concrete Moment Frame	2	1	0.017769	56.278
Concrete Moment Frame	3	1	0.108563	9.211
Wood Braced Frame	1	1	0.002595	385.356
Wood Braced Frame	2	1	0.007476	133.761
Wood Braced Frame	3	1	0.015516	64.450

Table ____ - Stiffness Values for Concrete Moment Frames – East/West Direction

Center of Mass

The center of mass at each level was determined by hand. Tributary areas were used for building elements that did not exactly line up with a level or that crossed over several levels. The reference point used for the center of mass was the Southwest corner of the façade of the building. Center of mass values for each level are found in Tables ____ - ____ below. Calculations for the center of mass at each level are found in Appendix ____.

Center of Mass x = {∑[(weight)(x)]}/∑weight

Center of Mass y = {∑[(weight)(y)]}/∑weight

Center of Mass - Entire Building - Level 1			
	Weight (kips)	Center of Mass	
		x (ft)	y (ft)
Building 1 - Level 1	496.085	31.6634	80.7836
Building 2 - Level 1	404.340	112.6943	78.0000
Building 3 - Level 1	1089.540	125.7531	78.2569
TOTAL=	1989.965	99.6438	78.8346

Table ____ - Center of Mass of Entire Building at Level 1

Center of Mass - Entire Building - Level 2			
	Weight (kips)	Center of Mass	
		x (ft)	y (ft)
Building 1 - Level 2	740.563	55.8277	80.1876
Building 2 - Level 2	329.779	124.6779	75.2708
Building 4 - Level 2	760.650	151.5494	75.1941
TOTAL=	1830.992	107.9940	77.2276

Table ____ - Center of Mass of Entire Building at Level 2

Center of Mass - Entire Building - Level 3			
	Weight (kips)	Center of Mass	
		x (ft)	y (ft)
Building 1 - Level 3	593.006	52.7936	78.0000
TOTAL=	593.006	52.7936	78.0000

Table ____ - Center of Mass of Entire Building at Level 3

Center of Rigidity

The center of rigidity was calculated for each level using the stiffness values of the frames that contribute to that level. The reference point used for the center of rigidity was the Southwest corner of the façade of the building (the same as that used for the center of mass). The center of rigidity at each level for the North/South direction is found in Tables ____-____, and the center of rigidity for the East/West direction is found in Tables ____-____ below. Table ____ shows the overall center of rigidity at each level.

Center of Rigidity (x) = $[\text{sum}(k_{iy}x_i)]/[\text{sum}(k_{iy})]$

Center of Rigidity - North/South Direction - Entire Building - Level 1					
	k_{iy}	x_i (ft)	Quantity	$(k_{iy}x_i)$	Center of Rigidity x (ft)
Braced Frames - Column Line 1	95.712	1.1510	10	1101.6850	
Moment Frame - Column Line 1.8	352.609	111.9010	1	39457.3144	
TOTAL=	1309.729		TOTAL=	40558.9994	30.9675

Table ____ - Center of Rigidity for North/South Direction – Level 1

Center of Rigidity - North/South Direction - Entire Building - Level 2					
	k_{iy}	x_i (ft)	Quantity	$(k_{iy}x_i)$	Center of Rigidity x (ft)
Braced Frames - Column Line 1	30.595	1.1510	10	352.1612	
Moment Frame - Column Line 2	158.781	130.3177	1	20691.9760	
Moment Frame - Column Line 4	21.388	171.6510	1	3671.2089	
TOTAL=	486.119		TOTAL=	24715.3461	50.8422

Table ____ - Center of Rigidity for North/South Direction – Level 2

Center of Rigidity - North/South Direction - Entire Building - Level 3					
	k_{iy}	x_i (ft)	Quantity	$(k_{iy}x_i)$	Center of Rigidity x (ft)
Braced Frames - Column Line 1	12.937	1.1510	10	148.9103	
Moment Frame - Column Line 2	70.057	130.3177	1	9129.6677	
TOTAL=	199.427		TOTAL=	9278.5780	46.5262

Table ____ - Center of Rigidity for North/South Direction – Level 3

Center of Rigidity (y) = $[\text{sum}(k_{ix}y_i)]/[\text{sum}(k_{ix})]$

Center of Rigidity - East/West Direction - Entire Building - Level 1					
	k_{ix}	y_i (ft)	Quantity	(k_{ixyi})	Center of Rigidity y (ft)
Concrete Moment Frame	67.618	18.0000	1	1217.1208	
Concrete Moment Frame	67.618	48.0000	1	3245.6556	
Concrete Moment Frame	67.618	78.0000	1	5274.1903	
Concrete Moment Frame	67.618	108.0000	1	7302.7250	
Concrete Moment Frame	67.618	138.0000	1	9331.2597	
Wood Braced Frame	385.357	4.2500	2	3275.5303	
Wood Braced Frame	385.357	151.7500	2	116955.6978	
TOTAL=	1879.515		TOTAL=	146602.1794	78.0000

Table ____ - Center of Rigidity for East/Direction Direction – Level 1

Center of Rigidity - East/West Direction - Entire Building - Level 2					
	k_{ix}	y_i (ft)	Quantity	($k_{ix}y_i$)	Center of Rigidity y (ft)
Concrete Moment Frame	56.278	18.0000	1	1013.0002	
Concrete Moment Frame	56.278	48.0000	1	2701.3338	
Concrete Moment Frame	56.278	78.0000	1	4389.6674	
Concrete Moment Frame	56.278	108.0000	1	6078.0010	
Concrete Moment Frame	56.278	138.0000	1	7766.3346	
Wood Braced Frame	133.761	4.2500	2	1136.9719	
Wood Braced Frame	133.761	151.7500	2	40596.5849	
TOTAL=	816.435		TOTAL=	63681.8938	78.0000

Table ____ - Center of Rigidity for East/West Direction – Level 2

Center of Rigidity - East/West Direction - Entire Building - Level 3					
	k_{ix}	y_i (ft)	Quantity	($k_{ix}y_i$)	Center of Rigidity y (ft)
Concrete Moment Frame	9.211	18.0000	1	165.8023	
Concrete Moment Frame	9.211	48.0000	1	442.1396	
Concrete Moment Frame	9.211	78.0000	1	718.4768	
Concrete Moment Frame	9.211	108.0000	1	994.8141	
Concrete Moment Frame	9.211	138.0000	1	1271.1513	
Wood Braced Frame	64.450	4.2500	2	547.8216	
Wood Braced Frame	64.450	151.7500	2	19560.4536	
TOTAL=	303.855		TOTAL=	23700.6593	78.0000

Table ____ - Center of Rigidity for East/West Direction – Level 3

Center of Rigidity - Entire Building		
Level	Center of Rigidity x (ft)	Center of Rigidity y (ft)
1	30.9675	78.0000
2	50.8422	78.0000
3	46.5262	78.0000

Table ____ - Center of Rigidity for Entire Building at Each Level

Direct Shear

The direct shear values for each lateral force resisting frame and each level were calculated by hand and are found in Tables _____ - _____ below. Calculations for direct shear are found in Appendix _____. Direct shear values in the North/South direction for "Building 1" were based on tributary area since the wood roof diaphragm is considered to be a flexible diaphragm.

$$\text{Direct Load: } F_{iy} = [(k_{iy}/\sum k_{iy})](P_y)$$

Due to Seismic Loads:

1.2D + 1.0E + L + 0.2S

North/South Direction:

Direct Shear - North/South Direction - "Building 1"							
Load Combination = 1.2D+1.0E+L+0.2S	Force (k)	Factored Force (k)	Distributed Force (kips)				
			Braced Frame - Column Line 1 - Level 1	Braced Frame - Column Line 1 - Level 2	Braced Frame - Column Line 1 - Level 3	Moment Frame - Column Line 2 - Level 2	Moment Frame - Column Line 2 - Level 3
Level 1	8.96	8.96	0.90				
Level 2	31.41	31.41		1.57		15.71	
Level 3	40.79	40.79			2.04		20.40

Table _____ - Direct Shear Values due to Seismic Loads for "Building 1" (North/South)
*Assuming flexible diaphragm for "Building 1"
*Based on 10 braced frames at Column Line 1

Direct Shear - North/South Direction - "Building 2"				
Load Combination = 1.2D+1.0E+L+0.2S	Force (k)	Factored Force (k)	Distributed Force (kips)	
			Moment Frame - Column Line 1.8 - Level 1	Moment Frame - Column Line 2 - Level 2
Level 1	11.17	11.17	11.17	
Level 2	21.39	21.39		21.39

Table _____ - Direct Shear Values due to Seismic Loads for "Building 2" (North/South)

Direct Shear - North/South Direction - "Building 3"			
Load Combination = 1.2D+1.0E+L+0.2S	Force (k)	Factored Force (k)	Distributed Force (kips)
			Moment Frame - Column Line 1.8 - Level 1
Level 1	48.32	48.32	48.32

Table _____ - Direct Shear Values due to Seismic Loads for "Building 3" (North/South)

Direct Shear - North/South Direction - "Building 4"				
Load Combination = 1.2D+1.0E+L+0.2S	Force (k)	Factored Force (k)	Distributed Force (kips)	
			Moment Frame - Column Line 2 - Level 2	Moment Frame - Column Line 4 - Level 2
Level 2	33.74	33.74	29.73	4.01

Table _____ - Direct Shear Values due to Seismic Loads for "Building 4" (North/South)

Total Direct Shear - North/South Direction							
Load Combination = 1.2D+1.0E+L+0.2S	Distributed Force (kips)						
	Braced Frame - Column Line 1 - Level 1	Braced Frame - Column Line 1 - Level 2	Braced Frame - Column Line 1 - Level 3	Moment Frame - Column Line 1.8 - Level 1	Moment Frame - Column Line 2 - Level 2	Moment Frame - Column Line 2 - Level 3	Moment Frame - Column Line 4 - Level 2
Level 1	0.90			59.49			
Level 2		1.57			66.83		4.01
Level 3			2.04			20.40	

Table _____ - Total Direct Shear Values due to Seismic Loads (North/South)

East/West Direction:

Total Direct Shear - East/West Direction					
Load Combination = 1.2D+1.0E+L+0.2S	Force (k)	Factored Force (k)	Distributed Force (kips)		
			Inside Concrete Moment Frame (1 of 3)	Outer Concrete Moment Frame (1 of 2)	Wood Braced Frame (1 of 4)
Level 1	68.45	68.45	14.04	12.64	0.26
Level 2	86.54	86.54	17.75	14.81	0.92
Level 3	40.79	40.79	8.37	5.46	1.19

Table ____ - Total Direct Shear Values due to Seismic Loads (East/West)

Due to Wind Loads:

1.2D + 1.6W + L + 0.5(Lr or S or R)

North/South Direction:

Direct Shear - North/South Direction - "Building 1"							
Load Combination = 1.2D+1.6W+L+0.5 (Lr or S or R)	Force (k)	Factored Force (k)	Distributed Force (kips)				
			Braced Frame - Column Line 1 - Level 1	Braced Frame - Column Line 1 - Level 2	Braced Frame - Column Line 1 - Level 3	Moment Frame - Column Line 2 - Level 2	Moment Frame - Column Line 2 - Level 3
Level 1	37.63	60.21	6.02				
Level 2	46.46	74.34		3.72		37.17	
Level 3	66.68	106.69			5.33		53.34

Table ____ - Direct Shear Values due to Wind Loads for "Building 1" (North/South)

Direct Shear - North/South Direction - "Building 4"				
Load Combination = 1.2D+1.6W+L+0.5 (Lr or S or R)	Force (k)	Factored Force (k)	Distributed Force (kips)	
			Moment Frame - Column Line 2 - Level 2	Moment Frame - Column Line 4 - Level 2
Level 2	14.10	22.56	19.88	2.68

Table ____ - Direct Shear Values due to Wind Loads for "Building 4" (North/South)

Total Direct Shear - North/South Direction						
Load Combination = 1.2D+1.6W+L+0.5 (Lr or S or R)	Distributed Force (kips)					
	Braced Frame - Column Line 1 - Level 1	Braced Frame - Column Line 1 - Level 2	Braced Frame - Column Line 1 - Level 3	Moment Frame - Column Line 2 - Level 2	Moment Frame - Column Line 2 - Level 3	Moment Frame - Column Line 4 - Level 2
Level 1	6.02					
Level 2		3.72		57.05		2.68
Level 3			5.33		53.34	

Table ____ - Total Direct Shear Values due to Wind Loads (North/South)

East/West Direction:

Total Direct Shear - East/West Direction					
Load Combination = 1.2D+1.6W+L+0.5(Lr or S or R)	Force (k)	Factored Force (k)	Distributed Force (kips)		
			Inside Concrete Moment Frame (1 of 3)	Outer Concrete Moment Frame (1 of 2)	Wood Braced Frame (1 of 4)
Level 1	44.89	71.82	14.73	9.61	2.10
Level 2	51.49	82.38	16.90	11.02	2.41
Level 3	26.85	42.96	8.81	5.75	1.26

Table ____ - Total Direct Shear Values due to Wind Loads (East/West)

Direct Shear Calculations:

<u>Based on Seismic Load:</u>

"Building 1" seismic loads are distributed to the lateral force resisting frames based on tributary area. "Building 4" seismic loads are distributed to the lateral force resisting frames based on the relative stiffness of each frame.

Direct Shear – North/South Direction – "Building 4"

Moment Frame – Column Line 2 – Level 2

F = [158.781/(158.781+21.388)][33.74 k] = **29.7347 k**

Moment Frame – Column Line 4 – Level 2

F = [21.388/(158.781+21.388)][33.74 k] = **4.0053 k**

Direct Shear – East/West Direction

Tributary Width of Moment Frames:

Inside Frames: 32.0'

Outer Frames: 16.0' + 4.875' = 20.875'

Tributary Width of Wood Braced Frames (2 of 4) = 4.875 + 4.25' = 9.125'

Total Width = 156'

For Level 1: Assume that the 8.96 k load from "Building 1" is distributed to all lateral force resisting frames in the East/West direction. Assume that the 11.17 k load from "Building 2" and the 48.32 k from "Building 3" are taken only by the concrete moment frames.

Inside Moment Frame – Level 1

F_{BLDG1} = [32.0/156][8.96 k] = 1.8379 k

$F_{BLDG2,3}$ = [32.0/156][11.17 k + 48.32 k] = 12.2031 k

F_{TOTAL} = 1.8379 k + 12.2031 k = **14.0410 k**

Outer Moment Frame – Level 1

F_{BLDG1} = [20.875/156][8.96 k] = 1.1990 k

$F_{BLDG2,3}$ = [(11.17 k + 48.32 k) - (3)(12.2031 k)]/2 = 11.4404 k

F_{TOTAL} = 1.1990 k + 11.4404 k = **12.6394 k**

Wood Braced Frame (2 of 4) – Level 1

F = [9.125/156][8.96 k] = 0.5241 k

Each Wood Braced Frame: F = (0.5241 k)/2 = **0.2621 k**

For Level 2: Assume that the 31.41 k load from "Building 1" is distributed to all lateral force resisting frames in the East/West direction. Assume that the 21.39 k load from "Building 2" and the 33.74 k load from "Building 4" are taken only by the concrete moment frames.

Inside Moment Frame – Level 2

F_{BLDG1} = [32.0/156][31.41 k] = 6.4431 k

$F_{BLDG2,4}$ = [32.0/156][21.39 k + 33.74 k] = 11.3087 k

F_{TOTAL} = 6.4431 k + 11.3087 k = **17.7518 k**

Outer Moment Frame – Level 2

F_{BLDG1} = [20.875/156][31.41 k] = 4.2031 k

$F_{BLDG2,4}$ = [(21.39 k + 33.74 k) - (3)(11.3087 k)]/2 = 10.6020 k

F_{TOTAL} = 4.2031 k + 10.6020 k = **14.8051 k**

Wood Braced Frame (2 of 4) – Level 1

F = [9.125/156][31.41 k] = 1.8373 k

Each Wood Braced Frame: F = (1.8373 k)/2 = **0.9186 k**

For Level 3: Assume that the 40.79 k load from "Building 1" is distributed to all lateral force resisting frames in the East/West direction.

Inside Moment Frame – Level 3

F_{BLDG1} = [32.0/156][40.79 k] = **8.3672 k**

Outer Moment Frame – Level 3

F_{BLDG1} = [20.875/156][40.79 k] = **5.4583 k**

Wood Braced Frame (2 of 4) – Level 1

F = [9.125/156][40.79 k] = 2.3860 k

Each Wood Braced Frame: F = (2.3860 k)/2 = **1.1930 k**

<u>Based on Wind Load:</u>

Direct Shear – North/South Direction – "Building 4"(Factored Load)

Moment Frame – Column Line 2 – Level 2

F = [158.781/(158.781+21.388)][22.56 k] = **19.8819 k**

Moment Frame – Column Line 4 – Level 2

F = [21.388/(158.781+21.388)][22.56 k] = **2.6781 k**

Direct Shear – North/South Direction – "Building 4"(Unfactored Load)

Moment Frame – Column Line 2 – Level 2

F = [158.781/(158.781+21.388)][14.10 k] = **12.4262 k**

Moment Frame – Column Line 4 – Level 2

F = [21.388/(158.781+21.388)][14.10 k] = **1.6738 k**

Direct Shear – East/West Direction (Factored Load)

Tributary Width of Moment Frames:

Inside Frames: 32.0'

Outer Frames: 16.0' + 4.875' = 20.875'

Tributary Width of Wood Braced Frames (2 of 4) = 4.875 + 4.25' = 9.125'

Total Width = 156'

Inside Moment Frame – Level 1

F = [32.0/156][71.82 k] = **14.7323 k**

Outer Moment Frame – Level 1

F = [20.875/156][71.82 k] = **9.6105 k**

Wood Braced Frame (2 of 4) – Level 1

F = [9.125/156][71.82 k] = 4.2010 k

Each Wood Braced Frame: F = (4.2010 k)/2 = **2.1005 k**

Inside Moment Frame – Level 2

F = [32.0/156][82.38 k] = **16.8985 k**

Outer Moment Frame – Level 2

F = [20.875/156][82.38 k] = **11.0236 k**

Wood Braced Frame (2 of 4) – Level 2

F = [9.125/156][82.38 k] = 4.8187 k

Each Wood Braced Frame: F = (4.8187 k)/2 = **2.4094 k**

Inside Moment Frame – Level 3

F = [32.0/156][42.96 k] = **8.8123 k**

Outer Moment Frame – Level 3

F = [20.875/156][42.96 k] = **5.7487 k**

Wood Braced Frame (2 of 4) – Level 3

F = [9.125/156][42.96 k] = 2.5129 k

Each Wood Braced Frame: F = (2.5129 k)/2 = **1.2564 k**

Torsional Shear

The torsional shear values for each lateral force resisting frame and each level were calculated by hand and are found in Tables ____-____ below. Rather than breaking up the building into the four different "buildings" as was done when determining the direct shear values, torsional shear values due to loads in the North/South direction were calculated looking at the entire building at each level. Torsional shear values due to wind loads were determined for both Wind Load Cases 1 and 2. Wind Load Case 1 just looks at the total wind load in one direction. Wind Load Case 2 used (0.75)(wind load) but adds in an eccentricity of (0.15)(building width). Wind Load Case 1 was found to control over Wind Load Case 2. Torsional shear due to loads in the East/West direction were neglected since the center of mass and center of rigidity are located at the same point or within one foot of each other in that direction. Plus, the five concrete frames in the East/West direction are evenly spaced at 32'-0" apart and are centered on the center of the building in the East/West direction. Therefore, it was assumed that torsional shear values in this direction would be negligible. Torsional shear due to eccentricities from Wind Load Case 2 was also neglected and assumed not to control for the East/West direction. Calculations for torsional shear are found in Appendix ____.

$$\text{Torsional Shear: } F_{it} = [(k_i)(d_i)(P_y)(e_x)]/[\sum((k_j)(d_j)^2)]$$

Due to Seismic Loads:

1.2D + 1.0E + L + 0.2S

Torsional Shear - North/South Direction - Level 1							
			Distributed Force (kips)				
Load Combination = 1.2D+1.0E+L+0.2S	Force (k)	Factored Force (k)	Braced Frame - Column Line 1 - Level 1	Moment Frame - Column Line 1.8 - Level 1	Inside Concrete Moment Frame (1 of 3)	Outer Concrete Moment Frame (1 of 2)	Wood Braced Frame (1 of 4)
Level 1	68.45	68.45	1.10	10.96	0.83	1.66	10.92

Table ____ - Torsional Shear Values due to Seismic Loads for Level 1 (North/South)

Torsional Shear - North/South Direction - Level 2								
			Distributed Force (kips)					
Load Combination = 1.2D+1.0E+L+0.2S	Force (k)	Factored Force (k)	Braced Frame - Column Line 1 - Level 2	Moment Frame - Column Line 2 - Level 2	Moment Frame - Column Line 4 - Level 2	Inside Concrete Moment Frame (1 of 3)	Outer Concrete Moment Frame (1 of 2)	Wood Braced Frame (1 of 4)
Level 2	86.54	86.54	1.35	11.23	2.30	1.60	3.21	8.78

Table ____ - Torsional Shear Values due to Seismic Loads for Level 2 (North/South)

Torsional Shear - North/South Direction - Level 3							
			Distributed Force (kips)				
Load Combination = 1.2D+1.0E+L+0.2S	Force (k)	Factored Force (k)	Braced Frame - Column Line 1 - Level 3	Moment Frame - Column Line 2 - Level 3	Inside Concrete Moment Frame (1 of 3)	Outer Concrete Moment Frame (1 of 2)	Wood Braced Frame (1 of 4)
Level 3	40.79	40.79	0.07	0.67	0.03	0.07	0.54

Table ____ - Torsional Shear Values due to Seismic Loads for Level 3 (North/South)

Due to Wind Loads:

$1.2D + 1.6W + L + 0.5(L_r \text{ or } S \text{ or } R)$

<u>Load Case 1:</u>

Torsional Shear - North/South Direction - Level 1							
Load Combination = 1.2D+1.6W+L+0.5(Lr or S or R)	Force (k)	Factored Force (k)	Distributed Force (kips)				
			Braced Frame - Column Line 1 - Level 1	Moment Frame - Column Line 1.8 - Level 1	Inside Concrete Moment Frame (1 of 3)	Outer Concrete Moment Frame (1 of 2)	Wood Braced Frame (1 of 4)
Level 1	37.63	60.21	0.49	4.94	0.37	0.75	4.92

Table ____ - Torsional Shear Values due to Wind Load Case 1 for Level 1 (North/South)

Torsional Shear - North/South Direction - Level 2								
Load Combination = 1.2D+1.6W+L+0.5(Lr or S or R)	Force (k)	Factored Force (k)	Distributed Force (kips)					
			Braced Frame - Column Line 1 - Level 2	Moment Frame - Column Line 2 - Level 2	Moment Frame - Column Line 4 - Level 2	Inside Concrete Moment Frame (1 of 3)	Outer Concrete Moment Frame (1 of 2)	Wood Braced Frame (1 of 4)
Level 2	60.67	97.07	0.95	7.85	1.61	1.12	2.24	6.14

Table ____ - Torsional Shear Values due to Wind Load Case 1 for Level 2 (North/South)

Torsional Shear - North/South Direction - Level 3							
Load Combination = 1.2D+1.6W+L+0.5(Lr or S or R)	Force (k)	Factored Force (k)	Distributed Force (kips)				
			Braced Frame - Column Line 1 - Level 3	Moment Frame - Column Line 2 - Level 3	Inside Concrete Moment Frame (1 of 3)	Outer Concrete Moment Frame (1 of 2)	Wood Braced Frame (1 of 4)
Level 3	66.68	106.69	0.55	5.45	0.27	0.55	4.41

Table ____ - Torsional Shear Values due to Wind Load Case 1 for Level 3 (North/South)

<u>Load Case 2:</u>

Torsional Shear - North/South Direction - Level 1							
Load Combination = 1.2D+1.6W+L+0.5(Lr or S or R)	Force (k)	Factored Force (k)	Distributed Force (kips)				
			Braced Frame - Column Line 1 - Level 1	Moment Frame - Column Line 1.8 - Level 1	Inside Concrete Moment Frame (1 of 3)	Outer Concrete Moment Frame (1 of 2)	Wood Braced Frame (1 of 4)
Level 1	28.22	45.15	0.64	6.44	0.49	0.98	6.41

Table ____ - Torsional Shear Values due to Wind Load Case 2 for Level 1 (North/South)

Torsional Shear - North/South Direction - Level 2								
Load Combination = 1.2D+1.6W+L+0.5(Lr or S or R)	Force (k)	Factored Force (k)	Distributed Force (kips)					
			Braced Frame - Column Line 1 - Level 2	Moment Frame - Column Line 2 - Level 2	Moment Frame - Column Line 4 - Level 2	Inside Concrete Moment Frame (1 of 3)	Outer Concrete Moment Frame (1 of 2)	Wood Braced Frame (1 of 4)
Level 2	45.50	72.80	1.23	10.17	2.08	1.45	2.90	7.95

Table ____ - Torsional Shear Values due to Wind Load Case 2 for Level 2 (North/South)

Torsional Shear - North/South Direction - Level 3							
Load Combination = 1.2D+1.6W+L+0.5(Lr or S or R)	Force (k)	Factored Force (k)	Distributed Force (kips)				
			Braced Frame - Column Line 1 - Level 3	Moment Frame - Column Line 2 - Level 3	Inside Concrete Moment Frame (1 of 3)	Outer Concrete Moment Frame (1 of 2)	Wood Braced Frame (1 of 4)
Level 3	50.01	80.02	0.95	9.49	0.48	0.95	7.68

Table ____ - Torsional Shear Values due to Wind Load Case 2 for Level 3 (North/South)

Torsional Load Calculations

Torsional Load: $F_{it} = [(k_i)(d_i)(P_y)(e_x)]/[\sum((k_j)(d_j)^2)]$

For torsional loads, the entire building was analyzed per level instead of using “Buildings 1, 2, 3, and 4”. The results can be seen below.

North/South Direction:

Level 1: Seismic Load (unfactored)

e_x = 99.6438’ – 30.9675’ = 68.6763’

P_y = 8.96 k + 11.17 k + 48.32 k = 68.45 k

$\sum k_j d_j^2 = (10)(95.712)(29.8165')^2 + (352.609)(80.9335')^2 + (2)(67.618)(32')^2 + (2)(67.618)(64')^2 + (4)(385.357)(73.75')^2 = 12{,}236{,}893.56$

Braced Frame (column line 1):
F_{it} = (95.712 k/in)(29.8165’)(68.45 k)(68.6763’)/12,236,893.56 = **1.0963 k**

Moment Frame (column line 1.8):
F_{it} = (352.609 k/in)(80.9335’)(68.45 k)(68.6763’)/12,236,893.56 = **10.9630 k**

Inside Moment Frames (column lines D and F):
F_{it} = (67.618 k/in)(32’)(68.45 k)(68.6763’)/12,236,893.56 = **0.8312 k**

Outer Moment Frames (column lines C and G):
F_{it} = (67.618 k/in)(64’)(68.45 k)(68.6763’)/12,236,893.56 = **1.6625 k**

Braced Frames (East/West Direction):
F_{it} = (385.357 k/in)(73.75’)(68.45 k)(68.6763’)/12,236,893.56 = **10.9178 k**

Level 2: Seismic Load (unfactored)

e_x = 107.9940’ – 50.8422’ = 57.1518’

P_y = 31.41 k + 21.39 k + 33.74 k = 86.54 k

$\sum k_j d_j^2 = (10)(30.595)(49.6912')^2 + (158.781)(79.4755')^2 + (21.388)(120.8088')^2 + (2)(56.278)(32')^2 + (2)(56.278)(64')^2 + (4)(133.761)(73.75)^2 = 5{,}556{,}958.898$

Braced Frame (column line 1):

F_{it} = (30.595 k/in)(49.6912')(86.54 k)(57.1518')/5,556,958.898 = **1.3531 k**

Moment Frame (column line 2):
F_{it} = (158.781 k/in)(79.4755')(86.54 k)(57.1518')/5,556,958.898 = **11.2316 k**

Moment Frame (column line 4):
F_{it} = (21.388 k/in)(120.8088')(86.54 k)(57.1518')/5,556,958.898 = **2.2997 k**

Inside Moment Frames (column lines D and F):
F_{it} = (56.278 k/in)(32')(86.54 k)(57.1518')/5,556,958.898 = **1.6029 k**

Outer Moment Frames (column lines C and G):
F_{it} = (56.278 k/in)(64')(86.54 k)(57.1518')/5,556,958.898 = **3.2057 k**

Braced Frames (East/West Direction):
F_{it} = (133.761 k/in)(73.75')(86.54 k)(57.1518')/5,556,958.898 = **8.7802 k**

Level 3: Seismic Load (unfactored)

e_x = 52.7936' – 46.5262' = 6.2674'

P_y = 40.79 k

$\sum k_j d_j^2$ = (10)(12.937)(45.3752')^2 + (70.057)(83.7915')^2 + (2)(9.211)(32')^2 + (2)(9.211)(64')^2 + (4)(64.450)(73.75')^2 = 2,254,734.207

Braced Frame (column line 1):
F_{it} = (12.937 k/in)(45.3752')(40.79 k)(6.2674')/2,254,734.207 = **0.06656 k**

Moment Frame (column line 2):
F_{it} = (70.057 k/in)(83.7915')(40.79 k)(6.2674')/2,254,734.207 = **0.6656 k**

Inside Moment Frames (column lines D and F):
F_{it} = (9.211 k/in)(32')(40.79 k)(6.2674')/2,254,734.207 = **0.03342 k**

Outer Moment Frames (column lines C and G):
F_{it} = (9.211 k/in)(64')(40.79 k)(6.2674')/2,254,734.207 = **0.06684 k**

Braced Frames (East/West Direction):
F_{it} = (64.450 k/in)(73.75')(40.79 k)(6.2674')/2,254,734.207 = **0.5389 k**

Level 1: Wind Load (Unfactored) – Load Case 1

e_x = 66.1510' – 30.9675' = 35.1835'

P_y = 37.63 k

$\sum k_j d_j^2$ = (10)(95.712)(29.8165')^2 + (352.609)(80.9335')^2 + (2)(67.618)(32')^2 + (2)(67.618)(64')^2 + (4)(385.357)(73.75')^2 = 12,236,893.56

Braced Frame (column line 1):
F_{it} = (95.712 k/in)(29.8165')(37.63 k)(35.1835')/12,236,893.56 = **0.3088 k**

Moment Frame (column line 1.8):
F_{it} = (352.609 k/in)(80.9335')(37.63 k)(35.1835')/12,236,893.56 = **3.0876 k**

Inside Moment Frames (column lines D and F):
F_{it} = (67.618 k/in)(32')(37.63 k)(35.1835')/12,236,893.56 = **0.2341 k**

Outer Moment Frames (column lines C and G):
F_{it} = (67.618 k/in)(64')(37.63 k)(35.1835')/12,236,893.56 = **0.4682 k**

Braced Frames (East/West Direction):
F_{it} = (385.357 k/in)(73.75')(37.63 k)(35.1835')/12,236,893.56 = **3.0749 k**

Level 2: Wind Load (Unfactored) – Load Case 1

e_x = 86.4479' – 50.8422' = 35.6057'

P_y = 46.46 k + 14.21 k = 60.67 k

$\sum k_j d_j^2$ = (10)(30.595)(49.6912')^2 + (158.781)(79.4755')^2 + (21.388)(120.8088')^2 + (2)(56.278)(32')^2 + (2)(56.278)(64')^2 + (4)(133.761)(73.75)^2 = 5,556,958.898

Braced Frame (column line 1):
F_{it} = (30.595 k/in)(49.6912')(60.67 k)(35.6057')/5,556,958.898 = **0.5910 k**

Moment Frame (column line 2):
F_{it} = (158.781 k/in)(79.4755')(60.67 k)(35.6057')/5,556,958.898 = **4.9056 k**

Moment Frame (column line 4):
F_{it} = (21.388 k/in)(120.8088')(60.67 k)(35.6057')/5,556,958.898 = **1.0044 k**

Inside Moment Frames (column lines D and F):
F_{it} = (56.278 k/in)(32')(60.67 k)(35.6057')/5,556,958.898 = **0.7001 k**

Outer Moment Frames (column lines C and G):
F_{it} = (56.278 k/in)(64')(60.67 k)(35.6057')/5,556,958.898 = **1.4002 k**

Braced Frames (East/West Direction):
F_{it} = (133.761 k/in)(73.75')(60.67 k)(35.6057')/5,556,958.898 = **3.8349 k**

Level 3: Wind Load (Unfactored) – Load Case 1

e_x = 66.1510' – 46.5262' = 19.6248'

P_y = 66.68 k

$\sum k_j d_j^2$ = (10)(12.937)(45.3752')^2 + (70.057)(83.7915')^2 + (2)(9.211)(32')^2 + (2)(9.211)(64')^2 + (4)(64.450)(73.75')^2 = 2,254,734.207

Braced Frame (column line 1):
F_{it} = (12.937 k/in)(45.3752')(66.68 k)(19.6248')/2,254,734.207 = **0.3407 k**

Moment Frame (column line 2):
F_{it} = (70.057 k/in)(83.7915')(66.68 k)(19.6248')/2,254,734.207 = **3.4069 k**

Inside Moment Frames (column lines D and F):
F_{it} = (9.211 k/in)(32')(68.68 k)(19.6248')/2,254,734.207 = **0.1711 k**

Outer Moment Frames (column lines C and G):
F_{it} = (9.211 k/in)(64')(66.68 k)(19.6248')/2,254,734.207 = **0.3421 k**

Braced Frames (East/West Direction):
F_{it} = (64.450 k/in)(73.75')(66.68 k)(19.6248')/2,254,734.207 = **2.7586 k**

Load Case 2: Multiply loads by 0.75 and use an eccentricity of 0.15b_x

Level 1: Wind Load (Unfactored) – Load Case 2

e_x = 35.1835' + (0.15)(172.8958') = 61.1179'

P_y = (0.75)(37.63 k) = 28.22 k

$\sum k_j d_j^2$ = (10)(95.712)(29.8165')^2 + (352.609)(80.9335')^2 + (2)(67.618)(32')^2 + (2)(67.618)(64')^2 + (4)(385.357)(73.75')^2 = 12,236,893.56

Braced Frame (column line 1):
F_{it} = (95.712 k/in)(29.8165')(28.22 k)(61.1179')/12,236,893.56 = **0.4022 k**

Moment Frame (column line 1.8):
F_{it} = (352.609 k/in)(80.9335')(28.22 k)(61.1179')/12,236,893.56 = **4.0223 k**

Inside Moment Frames (column lines D and F):
F_{it} = (67.618 k/in)(32')(28.22 k)(61.1179')/12,236,893.56 = **0.3050 k**

Outer Moment Frames (column lines C and G):
F_{it} = (67.618 k/in)(64')(28.22 k)(61.1179')/12,236,893.56 = **0.6100 k**

Braced Frames (East/West Direction):
F_{it} = (385.357 k/in)(73.75')(28.22 k)(61.1179')/12,236,893.56 = **4.0057 k**

Level 2: Wind Load (Unfactored) – Load Case 2

e_x = 35.6057' + (0.15)(172.8958') = 61.5401'

P_y = (0.75)(60.67 k) = 45.50 k

$\sum k_j d_j^2$ = (10)(30.595)(49.6912')^2 + (158.781)(79.4755')^2 + (21.388)(120.8088')^2 + (2)(56.278)(32')^2 + (2)(56.278)(64')^2 + (4)(133.761)(73.75)^2 = 5,556,958.898

Braced Frame (column line 1):
F_{it} = (30.595 k/in)(49.6912')(45.50 k)(61.5401')/5,556,958.898 = **0.7661 k**

Moment Frame (column line 2):
F_{it} = (158.781 k/in)(79.4755')(45.50 k)(61.5401')/5,556,958.898 = **6.3586 k**

Moment Frame (column line 4):
F_{it} = (21.388 k/in)(120.8088')(45.50 k)(61.5401')/5,556,958.898 = **1.3020 k**

Inside Moment Frames (column lines D and F):
F_{it} = (56.278 k/in)(32')(45.50 k)(61.5401')/5,556,958.898 = **0.9074 k**

Outer Moment Frames (column lines C and G):
F_{it} = (56.278 k/in)(64')(45.50 k)(61.5401')/5,556,958.898 = **1.8149 k**

Braced Frames (East/West Direction):
F_{it} = (133.761 k/in)(73.75')(45.50 k)(61.5401')/5,556,958.898 = **4.9708 k**

Level 3: Wind Load (Unfactored) – Load Case 2

e_x = 19.6248' + (0.15)(172.8958') = 45.5592'

P_y = (0.75)(66.68 k) = 50.01 k

$\sum k_j d_j^2$ = (10)(12.937)(45.3752')^2 + (70.057)(83.7915')^2 + (2)(9.211)(32')^2 + (2)(9.211)(64')^2 + (4)(64.450)(73.75')^2 = 2,254,734.207

Braced Frame (column line 1):
F_{it} = (12.937 k/in)(45.3752')(50.01 k)(45.5592')/2,254,734.207 = **0.5932 k**

Moment Frame (column line 2):
F_{it} = (70.057 k/in)(83.7915')(50.01 k)(45.5592')/2,254,734.207 = **5.9318 k**

Inside Moment Frames (column lines D and F):
F_{it} = (9.211 k/in)(32')(50.01 k)(45.5592')/2,254,734.207 = **0.2978 k**

Outer Moment Frames (column lines C and G):
F_{it} = (9.211 k/in)(64')(50.01 k)(45.5592')/2,254,734.207 = **0.5957 k**

Braced Frames (East/West Direction):
F_{it} = (64.450 k/in)(73.75')(50.01 k)(45.5592')/2,254,734.207 = **4.8031 k**

East/West Direction:

Torsional effects were not accounted for in the East/West direction since the center of mass and center of rigidity either match up perfectly in the y-direction for each floor level or were only off by less than one foot. Hence, for seismic loads the eccentricity would be zero or very close to zero. Similarly, Wind Load Case 1 was not considered since the wind load would basically be applied at the center of the building in the East/West direction, which lines up with the center of rigidity in the East/West direction. Therefore, this case would also produce little or no eccentricity. Wind Load Case 2 was not considered for the East/West direction either because it was assumed that any small torsional effects would not control in this direction. The five moment frames and four braced frames in the East/West direction are centered on the building and spaced symmetrically on both sides of the building, so torsional effects should be minimal in this direction.

Total Shear

Total shear values were determined by combining the direct shear at each frame and level with the torsional shear at each frame and level. Torsional shear was either added or subtracted to the direct shear depending on which side of the center of rigidity the frames were located and which side of the center of rigidity the load was applied.

$$F_i = F_{i,direct} +/- F_{i,torsion}$$

Due to Seismic Loads:

North/South Direction:

Total Shear - North/South Direction - Braced Frame at Column Line 1			
Load Combination = 1.2D+1.0E+L+0.2S	Factored Direct Shear Force (k)	Factored Torsional Shear Force (k)	Total Factored Shear (k)
Level 1	0.90	-1.10	-0.20
Level 2	1.57	-1.35	0.22
Level 3	2.04	-0.07	1.97

Table ____ - Total Shear Values due to Seismic Loads for Braced Frame at Column Line 1 (North/South)

Total Shear - North/South Direction - Moment Frame at Column Line 2			
Load Combination = 1.2D+1.0E+L+0.2S	Factored Direct Shear Force (k)	Factored Torsional Shear Force (k)	Total Factored Shear (k)
Level 2	66.83	11.23	78.06
Level 3	20.40	0.67	21.07

Table ____ - Total Shear Values due to Seismic Loads for Moment Frame at Column Line 2 (North/South)

Total Shear - North/South Direction - Moment Frame at Column Line 1.8			
Load Combination = 1.2D+1.0E+L+0.2S	Factored Direct Shear Force (k)	Factored Torsional Shear Force (k)	Total Factored Shear (k)
Level 1	59.49	10.96	70.45

Table ____ - Total Shear Values due to Seismic Loads for Moment Frame at Column Line 1.8 (North/South)

Total Shear - North/South Direction - Moment Frame at Column Line 4			
Load Combination = 1.2D+1.0E+L+0.2S	Factored Direct Shear Force (k)	Factored Torsional Shear Force (k)	Total Factored Shear (k)
Level 2	4.01	2.30	6.31

Table ____ - Total Shear Values due to Seismic Loads for Moment Frame at Column Line 4 (North/South)

East/West Direction:

Total Shear - East/West Direction - Inside Concrete Moment Frame			
Load Combination = 1.2D+1.0E+L+0.2S	**Factored Direct Shear Force (k)**	**Factored Torsional Shear Force (k)**	**Total Factored Shear (k)**
Level 1	14.04	0.83	14.87
Level 2	17.75	1.60	19.35
Level 3	8.37	0.03	8.40

Table ____ - Total Shear Values due to Seismic Loads for Inside Concrete Moment Frame (East/West)

Total Shear - East/West Direction - Outer Concrete Moment Frame			
Load Combination = 1.2D+1.0E+L+0.2S	**Factored Direct Shear Force (k)**	**Factored Torsional Shear Force (k)**	**Total Factored Shear (k)**
Level 1	12.64	1.66	14.30
Level 2	14.81	3.21	18.02
Level 3	5.46	0.07	5.53

Table ____ - Total Shear Values due to Seismic Loads for Outer Concrete Moment Frame (East/West)

Total Shear - East/West Direction - Wood Braced Frame			
Load Combination = 1.2D+1.0E+L+0.2S	**Factored Direct Shear Force (k)**	**Factored Torsional Shear Force (k)**	**Total Factored Shear (k)**
Level 1	0.26	10.92	11.18
Level 2	0.92	8.78	9.70
Level 3	1.19	0.54	1.73

Table ____ - Total Shear Values due to Seismic Loads for Wood Braced Frame (East/West)

Due to Wind Loads:

Load Case 1:

North/South Direction

Total Shear - North/South Direction - Braced Frame at Column Line 1			
Load Combination = 1.2D+1.6W+L+0.5 (Lr or S or R)	**Factored Direct Shear Force (k)**	**Factored Torsional Shear Force (k)**	**Total Factored Shear (k)**
Level 1	6.02	-0.49	5.53
Level 2	3.72	-0.95	2.77
Level 3	5.33	-0.55	4.78

Table ____ - Total Shear Values due to Wind Load Case 1 for Braced Frame at Column Line 1 (North/South)

Total Shear - North/South Direction - Moment Frame at Column Line 2			
Load Combination = 1.2D+1.6W+L+0.5 (Lr or S or R)	Factored Direct Shear Force (k)	Factored Torsional Shear Force (k)	Total Factored Shear (k)
Level 2	57.05	7.85	64.90
Level 3	53.34	5.45	58.79

Table ____ - Total Shear Values due to Wind Load Case 1 for Moment Frame at Column Line 2 (North/South)

Total Shear - North/South Direction - Moment Frame at Column Line 4			
Load Combination = 1.2D+1.6W+L+0.5 (Lr or S or R)	Factored Direct Shear Force (k)	Factored Torsional Shear Force (k)	Total Factored Shear (k)
Level 2	2.68	1.61	4.29

Table ____ - Total Shear Values due to Wind Load Case 1 for Moment Frame at Column Line 4 (North/South)

East/West Direction:

Total Shear - East/West Direction - Inside Concrete Moment Frame			
Load Combination = 1.2D+1.6W+L+0.5 (Lr or S or R)	Factored Direct Shear Force (k)	Factored Torsional Shear Force (k)	Total Factored Shear (k)
Level 1	14.73	0.37	15.10
Level 2	16.90	1.12	18.02
Level 3	8.81	0.27	9.08

Table ____ - Total Shear Values due to Wind Load Case 1 for Inside Concrete Moment Frame (East/West)

Total Shear - East/West Direction - Outer Concrete Moment Frame			
Load Combination = 1.2D+1.6W+L+0.5 (Lr or S or R)	Factored Direct Shear Force (k)	Factored Torsional Shear Force (k)	Total Factored Shear (k)
Level 1	9.61	0.75	10.36
Level 2	11.02	2.24	13.26
Level 3	5.75	0.55	6.30

Table ____ - Total Shear Values due to Wind Load Case 1 for Outer Concrete Moment Frame (East/West)

Total Shear - East/West Direction - Wood Braced Frame			
Load Combination = 1.2D+1.6W+L+0.5 (Lr or S or R)	**Factored Direct Shear Force (k)**	**Factored Torsional Shear Force (k)**	**Total Factored Shear (k)**
Level 1	2.10	4.92	7.02
Level 2	2.41	6.14	8.55
Level 3	1.26	4.41	5.67

Table ____ - Total Shear Values due to Wind Load Case 1 for Wood Braced Frame (East/West)

<u>Load Case 2:</u>

North/South Direction:

Total Shear - North/South Direction - Braced Frame at Column Line 1			
Load Combination = 1.2D+1.6W+L+0.5 (Lr or S or R)	**Factored Direct Shear Force (k)**	**Factored Torsional Shear Force (k)**	**Total Factored Shear (k)**
Level 1	4.52	-0.64	3.88
Level 2	2.79	-1.23	1.56
Level 3	4.00	-0.95	3.05

Table ____ - Total Shear Values due to Wind Load Case 2 for Braced Frame at Column Line 1 (North/South)

Total Shear - North/South Direction - Moment Frame at Column Line 2			
Load Combination = 1.2D+1.6W+L+0.5 (Lr or S or R)	**Factored Direct Shear Force (k)**	**Factored Torsional Shear Force (k)**	**Total Factored Shear (k)**
Level 2	42.79	10.17	52.96
Level 3	40.01	9.49	49.50

Table ____ - Total Shear Values due to Wind Load Case 2 for Moment Frame at Column Line 2 (North/South)

Total Shear - North/South Direction - Moment Frame at Column Line 4			
Load Combination = 1.2D+1.6W+L+0.5 (Lr or S or R)	**Factored Direct Shear Force (k)**	**Factored Torsional Shear Force (k)**	**Total Factored Shear (k)**
Level 2	2.01	2.08	4.09

Table ____ - Total Shear Values due to Wind Load Case 2 for Moment Frame at Column Line 4 (North/South)

East/West Direction:

Total Shear - East/West Direction - Inside Concrete Moment Frame			
Load Combination = 1.2D+1.6W+L+0.5 (Lr or S or R)	**Factored Direct Shear Force (k)**	**Factored Torsional Shear Force (k)**	**Total Factored Shear (k)**
Level 1	11.05	0.49	11.54
Level 2	12.68	1.45	14.13
Level 3	6.61	0.48	7.09

Table ____ - Total Shear Values due to Wind Load Case 2 for Inside Concrete Moment Frame (East/West)

Total Shear - East/West Direction - Outer Concrete Moment Frame			
Load Combination = 1.2D+1.6W+L+0.5 (Lr or S or R)	**Factored Direct Shear Force (k)**	**Factored Torsional Shear Force (k)**	**Total Factored Shear (k)**
Level 1	7.21	0.98	8.19
Level 2	8.27	2.90	11.17
Level 3	4.31	0.95	5.26

Table ____ - Total Shear Values due to Wind Load Case 2 for Outer Concrete Moment Frame (East/West)

Total Shear - East/West Direction - Wood Braced Frame			
Load Combination = 1.2D+1.6W+L+0.5 (Lr or S or R)	**Factored Direct Shear Force (k)**	**Factored Torsional Shear Force (k)**	**Total Factored Shear (k)**
Level 1	1.58	6.41	7.99
Level 2	1.81	7.95	9.76
Level 3	0.95	4.80	5.75

Table ____ - Total Shear Values due to Wind Load Case 2 for Wood Braced Frame (East/West)

Drift and Displacement

Drift and displacement values were determined for each frame at each applicable level by applying the total forces due to direct loads and torsional loads to the SAP models of each frame. Drifts due to seismic loads were multiplied by a C_d factor of 3 ½ and divided by an importance factor of 1.25. Since two different seismic force-resisting systems were considered for the natatorium, the worst case C_d factor was used. For the wood braced frames, a C_d factor of 3 ½ applies to light-framed wall systems using flat strap bracing. For the concrete moment frames, a C_d factor of 2 ½ applies to ordinary reinforced concrete moment frames. Therefore, a C_d factor of 3 ½ was conservatively assumed to apply to all frames. This value was then compared to $0.015h_{sx}$ for each story, where h_{sx} is the story height below Level x. All frames met the seismic load drift limits.

For drift due to seismic loads:

$\Delta_x = (C_d)(\Delta_{xe})/I$

C_d = 3 ½ (Light-framed wall systems using flat strap bracing)

I = 1.25

Table 12.12.1 (ASCE 7-05):

Allowable Story Drift = $0.015h_{sx}$ (all other structures, Occupancy Category III)

Drifts due to unfactored wind loads were compared to an allowable limit of H/400, with H being the elevation height of the level, or with H being the story height.

North/South Direction:

Story Drifts - North/South Direction - Braced Frame at Column Line 1					
Unfactored Seismic	Deflection (in)	$Defl._x = (C_d*Defl._{xe})/I$	Story Height (ft)	Limit = $0.015h_{sx}$ (in)	
Level 1	0.0203	0.0569	13.33	2.4000	OK
Level 2	0.0053	0.0148	13.33	2.4000	OK
Level 3	0.0015	0.0042	13.33	2.4000	OK

Table ____ - Story Drifts due to Seismic Loads for Braced Frame at Column Line 1 (North/South)

Deflections - North/South Direction - Braced Frame at Column Line 1				
Unfactored Wind	**Deflection from SAP (in)**	**Elevation (ft)**	**Limit =H/400 (in)**	
Level 1	0.1270	13.33	0.4000	OK
Level 2	0.2764	26.67	0.8000	OK
Level 3	0.4236	40.00	1.2000	OK

Table ____ - Deflections due to Wind Loads for Braced Frame at Column Line 1 (North/South)

Story Drifts - North/South Direction - Braced Frame at Column Line 1				
Unfactored Wind	**Deflection (in)**	**Story Height (ft)**	**Limit =H/400 (in)**	
Level 1	0.1270	13.33	0.4000	OK
Level 2	0.1495	13.33	0.4000	OK
Level 3	0.1471	13.33	0.4000	OK

Table ____ - Story Drifts due to Wind Loads for Braced Frame at Column Line 1 (North/South)

Story Drifts - North/South Direction - Moment Frame at Column Line 2					
Unfactored Seismic	**Deflection from SAP (in)**	**Defl.$_x$ = (C_d*Defl.$_{xe}$)/I**	**Story Height (ft)**	**Limit = 0.015h$_{sx}$ (in)**	
Level 2	0.6591	1.8455	22.50	4.0500	OK
Level 3	0.2621	0.7339	17.50	3.1500	OK

Table ____ - Story Drifts due to Seismic Loads for Moment Frame at Column Line 2 (North/South)

Deflections - North/South Direction - Moment Frame at Column Line 2				
Unfactored Wind	**Deflection from SAP (in)**	**Elevation (ft)**	**Limit =H/400 (in)**	
Level 2	0.5475	22.50	0.6750	OK
Level 3	0.8469	40.00	1.2000	OK

Table ____ - Deflections due to Wind Loads for Moment Frame at Column Line 2 (North/South)

Story Drifts - North/South Direction - Moment Frame at Column Line 2				
Unfactored Wind	**Deflection from SAP (in)**	**Elevation (ft)**	**Limit =H/400 (in)**	
Level 2	0.5475	22.50	0.6750	OK
Level 3	0.2994	17.50	0.5250	OK

Table ____ - Story Drifts due to Wind Loads for Moment Frame at Column Line 2 (North/South)

Story Drifts - North/South Direction - Moment Frame at Column Line 1.8					
Unfactored Seismic	Deflection from SAP (in)	$Defl._x = (C_d{*}Defl._{xe})/I$	Elevation (ft)	Limit = $0.015h_{sx}$ (in)	
Level 1	0.0624	0.1748	10.50	1.8900	OK

Table ____ - Story Drifts due to Seismic Loads for Moment Frame at Column Line 1.8 (North/South)

Story Drifts - North/South Direction - Moment Frame at Column Line 4					
Unfactored Seismic	Deflection from SAP (in)	$Defl._x = (C_d{*}Defl._{xe})/I$	Elevation (ft)	Limit = $0.015h_{sx}$ (in)	
Level 2	0.2950	0.8261	24.67	4.4400	OK

Table ____ - Story Drifts due to Seismic Loads for Moment Frame at Column Line 4 (North/South)

Deflections - North/South Direction - Moment Frame at Column Line 4				
Unfactored Wind	Deflection from SAP (in)	Elevation (ft)	Limit =H/400 (in)	
Level 2	0.1253	24.67	0.7400	OK

Table ____ - Deflections due to Wind Loads for Moment Frame at Column Line 4 (North/South)

Story Drifts - North/South Direction - Moment Frame at Column Line 4				
Unfactored Wind	Deflection from SAP (in)	Elevation (ft)	Limit =H/400 (in)	
Level 2	0.1253	24.67	0.7400	OK

Table ____ - Story Drifts due to Wind Loads for Moment Frame at Column Line 4 (North/South)

East/West Direction:

Story Drifts - East/West Direction - Concrete Moment Frame					
Unfactored Seismic	Deflection (in)	$Defl._x = (C_d{*}Defl._{xe})/I$	Story Height (ft)	Limit = $0.015h_{sx}$ (in)	
Level 1	0.2298	0.6434	10.50	1.8900	OK
Level 2	-0.0011	-0.0030	12.00	2.1600	OK
Level 3	0.6772	1.8963	17.50	3.1500	OK

Table ____ - Story Drifts due to Seismic Loads for Moment Frame (East/West)

Deflections - East/West Direction - Concrete Moment Frame				
Unfactored Wind	**Deflection from SAP (in)**	**Elevation (ft)**	**Limit =L/400 (in)**	
Level 1	0.1434	10.50	0.3150	OK
Level 2	0.1420	22.50	0.6750	OK
Level 3	0.5964	40.00	1.2000	OK

Table ____ - Deflections due to Wind Loads for Moment Frame (East/West)

Story Drifts - East/West Direction - Concrete Moment Frame				
Unfactored Wind	**Deflection (in)**	**Story Height (ft)**	**Limit =L/400 (in)**	
Level 1	0.1434	10.50	0.3150	OK
Level 2	-0.0014	12.00	0.3600	OK
Level 3	0.4543	17.50	0.5250	OK

Table ____ - Story Drifts due to Wind Loads for Moment Frame (East/West)

Story Drifts - East/West Direction - Braced Frame					
Unfactored Seismic	**Deflection (in)**	**Defl.$_x$ = (C_d*Defl.$_{xe}$)/I**	**Story Height (ft)**	**Limit = 0.015h_{sx} (in)**	
Level 1	0.0733	0.2052	13.33	2.4000	OK
Level 2	0.0595	0.1666	13.33	2.4000	OK
Level 3	0.0367	0.1028	13.33	2.4000	OK

Table ____ - Story Drifts due to Seismic Loads for Braced Frame (East/West)

Deflections - East/West Direction - Braced Frame				
Unfactored Wind	**Deflection from SAP (in)**	**Elevation (ft)**	**Limit =H/400 (in)**	
Level 1	0.0875	13.33	0.4000	OK
Level 2	0.1719	26.67	0.8000	OK
Level 3	0.2325	40.00	1.2000	OK

Table ____ - Deflections due to Wind Loads for Braced Frame (East/West)

Story Drifts - East/West Direction - Braced Frame				
Unfactored Wind	**Deflection (in)**	**Story Height (ft)**	**Limit =H/400 (in)**	
Level 1	0.0875	13.33	0.4000	OK
Level 2	0.0844	13.33	0.4000	OK
Level 3	0.0606	13.33	0.4000	OK

Table ____ - Story Drifts due to Wind Loads for Braced Frame (East/West)

Wood Braced Frame – Column Line 1

Design of Diagonal Members:

Controlling Load Combination: D + 0.75W + 0.75S

D + 0.75W + 0.75S = 6.391 k + (0.75)(9.291 k) + (0.75)(5.015 k) = 17.121 k (compression)

Analyze Member Buckling About x Axis:

$(l_e/d)_{max} = 50$

$(l_e/d)_x = [(1.0)(15.5492')(12 \text{ in/ft})]/d \leq 50$

$d \geq l_e/50 = [(15.5492')(12 \text{ in/ft})]/50 = 3.73''$

Analyze Member Bucking About y Axis:

$(l_e/d)_{max} = 50$

$(l_e/d)_y = [(1.0)(7.7746')(12 \text{ in/ft})]/d \leq 50$

$d \geq l_e/50 = [(7.7746')(12 \text{ in/ft})]/50 = 1.87''$

Try 3 ½” x 5 ½”

$(l_e/d)_x = [(15.5492)(12 \text{ in/ft})]/5.5'' = 33.9255$

$(l_e/d)_y = [(7.7746')(12 \text{ in/ft})]/3.5'' = 26.6558$

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

E_{min} = 980,000 psi

C_D = 1.6 (for wind load))

C_M = 0.73 for F_c (p. 64, NDS Supplement)

C_M = 0.833 for E and E_{min} (p. 64, NDS Supplement)

C_t = 1.0

$E'_{min} = (E_{min})(C_M)(C_t) = (980{,}000)(0.833)(1.0) = 816{,}340$ psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340 \text{ psi})]/[(33.9255)^2] = 583.029$ psi

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.6)(0.73)(1.0) = 2686.4$ psi

$F_{cE}/F_c^* = 583.029/2686.4 = 0.2170$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.2170]/[(2)(0.9)] = 0.6761$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.6761\} - \sqrt{\{[0.6761]^2 - [0.2170/0.9]\}}$

$= 0.2113$

$F'_c = F_c^*(C_P) = (2686.4 \text{ psi})(0.2113) = 567.641 \text{ psi}$

$P = (F'_c)(A)$

$A_{req'd} = P/F'_c = 17{,}121 \text{ lb}/567.641 \text{ psi} = 30.16 \text{ in}^2 > A_{provided} = 19.25 \text{ in}^2$ ∴ **N.G.**

Try 3 ½" x 6 7/8"

$(l_e/d)_x = [(15.5492)(12 \text{ in/ft})]/6.875" = 27.1404$

$(l_e/d)_y = [(7.7746')(12 \text{ in/ft})]/3.5" = 26.6558$

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340 \text{ psi})]/[(27.1404)^2] = 910.982 \text{ psi}$

$F_{cE}/F_c^* = 910.982/2686.4 = 0.3391$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.3391]/[(2)(0.9)] = 0.7439$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.7439\} - \sqrt{\{[0.7439]^2 - [0.3391/0.9]\}}$

$= 0.3236$

$F'_c = F_c^*(C_P) = (2686.4 \text{ psi})(0.3236) = 869.221 \text{ psi}$

$P = (F'_c)(A)$

$A_{req'd} = P/F'_c = 17{,}121 \text{ lb}/869.221 \text{ psi} = 19.70 \text{ in}^2 < A_{provided} = 24.06 \text{ in}^2$ ∴ **OK**

Use 3 ½" x 6 7/8" for all diagonal members

Concrete Moment Frame – Column Line 1.8

Beams

*Use rebar cover of 1.5(1.5”) = 2.25” due to corrosive environment (natatorium) (see ACI 7.7.6.1)

Design beams as a continuous beam.
Design beams for worst case and make all four beams the same size.

Shear and Moment (Unfactored) for Column Line 1.8 (24x24 Columns and 24x26 Beams)

	Beam 2	Beam 4	Beam 6	Beam 8	Column 1 (Exterior Column)	Column 9 (Exterior Column)	Column 7 (Interior Column)
V_D (Top or Left)	-30.38	-31.95	-31.76	-33.31	-18.93	-19.28	1.71
V_D (Bottom or Right)	33.37	31.81	32.00	30.44	-18.93	-19.28	1.71
V_L (Top or Left)	-28.96	-30.45	-30.27	-31.75	-18.04	-18.38	1.62
V_L (Bottom or Right)	31.81	30.32	30.50	29.02	-18.04	-18.38	1.62
V_E (Top or Left)	2.25	1.83	1.75	1.94	13.25	-11.13	-14.78
V_E (Bottom or Right)	2.25	1.83	1.75	1.94	13.25	-11.13	-14.78
$V_{E,REVERSED}$ (Top or Left)	-1.94	-1.75	-1.83	-2.25	-11.13	13.25	16.26
$V_{E,REVERSED}$ (Bottom or Right)	-1.94	-1.75	-1.83	-2.25	-11.13	13.25	16.26
M_D (Top or Left)	-137.17	-171.67	-168.68	-184.05	137.17	-138.17	11.57
M_D (Bottom or Right)	-184.95	-169.40	-172.48	-138.17	-61.62	64.25	-6.37
M_L (Top or Left)	-130.71	-163.60	-160.66	-175.40	130.71	-131.72	10.99
M_L (Bottom or Right)	-176.31	-161.48	-164.41	-131.72	-58.61	61.30	-6.01
M_E (Top or Left)	38.11	29.42	28.31	29.75	-38.11	84.46	97.75
M_E (Bottom or Right)	-33.88	-29.16	-27.71	-32.40	101.00	-32.40	-57.47
$M_{E,REVERSED}$ (Top or Left)	-32.40	-27.71	-29.16	-33.88	32.40	-101.00	-107.38
$M_{E,REVERSED}$ (Bottom or Right)	29.75	28.31	29.42	38.11	-84.46	38.11	63.30
P_D					-30.38	-30.44	-65.32
P_L					-28.96	-29.02	-62.25
P_E					2.25	-1.94	0.19
$P_{E,REVERSED}$					-1.94	2.25	-0.42
M_D (Midspan)	93.96	84.49	84.49	93.91			
M_L (Midspan)	89.56	80.53	80.53	89.51			
M_E (Midspan)	2.12	0.13	0.30	-1.33			
$M_{E,REVERSED}$ (Midspan)	-1.33	0.30	0.13	2.12			
1.2D +/- 1.0E + 1.0L							
Max $V_{TOP/LEFT}$ (kips)	-67.36	-70.54	-70.21	-73.97	-51.89	-52.65	19.93
Max $V_{BOTTOM/RIGHT}$ (kips)	74.10	70.32	70.65	67.49	-51.89	-52.65	19.93
Max $M_{TOP/LEFT}$ (ft-kips)	-327.72	-397.32	-392.24	-430.14	327.72	**-398.52**	122.62
Max $M_{BOTTOM/RIGHT}$ (ft-kips)	-432.13	-393.92	-399.10	-329.93	-217.02	176.51	-71.12
Max $M_{MIDSPAN}$ (ft-kips)	204.43	182.21	182.21	204.32			
Max P_u (kips)					-67.36	-67.49	-141.05
1.2D + 1.6L							
Max $V_{TOP/LEFT}$ (kips)	-82.79	-87.06	-86.54	-90.77	-51.58	-52.54	4.64
Max $V_{BOTTOM/RIGHT}$ (kips)	**90.94**	86.68	87.20	82.96	-51.58	-52.54	4.64
Max $M_{TOP/LEFT}$ (ft-kips)	-373.74	-467.76	-459.47	-501.50	373.74	-376.56	31.47
Max $M_{BOTTOM/RIGHT}$ (ft-kips)	**-504.04**	-461.65	-470.03	-376.56	-167.72	175.18	-17.26
Max $M_{MIDSPAN}$ (ft-kips)	**256.05**	230.24	230.24	255.91			
Max P_u (kips)					-82.80	-82.96	**-177.98**

Tables Account for Torsional Effects

BEAM DESIGN:

$V_{u,max}$ = 90.94 kips (1.2D + 1.6L)
$M_{u,max}$ at Supports = 504.04 k-ft (1.2D + 1.6L)
$M_{u,max}$ at Midspan = 256.05 k-ft (1.2D + 1.6L)

Use normal-weight concrete with f'_c = 4000 psi
f_y = 60,000 psi for flexural reinforcement
f_{yt} = 60,000 psi for stirrups

1) Choose the actual size of the beam stem.

a) Calculate the minimum depth based on deflections.

Use worst case scenario (one-end continuous instead of both ends continuous).

ACI Table 9.5(a):

Minimum thickness, h = L/18.5 = [(32')(12 in/ft)]/18.5 = 20.76"

b) Determine the minimum depth based on the maximum negative moment.

$M_{u,max}$ at Supports = 504.04 k-ft

$\rho(\text{initial}) = [(\beta_1 f'_c)/(4f_y)] = [(0.85)(4\text{ ksi})/(4)(60\text{ ksi})] = 0.0142$

$\omega = \rho(f_y/f'_c) = (0.0142)(60\text{ ksi}/4\text{ ksi}) = 0.213$

$R = \omega f'c(1 - 0.59\omega) = (0.213)(4\text{ ksi})[1 - (0.59)(0.213)] = 0.745\text{ ksi}$

$bd^2 \geq M_u/\phi R = [(504.04\text{ ft-kips})(12\text{ in/ft})]/[(0.9)(0.745\text{ ksi})] = 9020.85\text{ in}^3$

Assuming b = 24 in.

$d \geq 19.39$ in.

h ≅ 19.39" + 3.25" = 22.64" (accounting for 2.25" clear cover due to corrosive environment; see ACI 7.7.6.1; (1.5)(1.5") = 2.25")

Try h = 26" > 20.76" ∴ Meets deflection criteria

d ≅ 26" – 3.25" = 22.75"

c) Check the shear capacity of the beam.

$V_u = \phi(V_c + V_s)$

$V_{u,max}$ = 90.94 kips

From ACI Code Section 11.2.1.1, the nominal V_c is

$$V_c = 2\lambda\sqrt{f'_c}b_wd = (2)(1.0)\sqrt{4000 \text{ psi}}(24")(22.75")/1000 = 69.06 \text{ kips}$$

ACI Code Section 11.4.7.9 sets the maximum nominal V_s as

$$V_s = 8\sqrt{f'_c}b_wd = (8)\sqrt{4000 \text{ psi}}(24")(22.75")/1000 = 276.26 \text{ kips}$$

Thus, the absolute maximum ϕV_n = 0.75(69.06 k + 276.26 k) = 258.99 kips

$\geq V_{u,max}$ = 90.94 kips $\therefore$ OK

d) Summary. Use:

b = 24"
h = 26"
d = 22.75"

2) Compute the dead load of the stem, and recompute the total moment.

Weight of 24"x26" concrete beam = [(24")(26")/144 in^2/ft^2][(150 lb/ft^3)/1000]

= 0.650 k/ft

Original dead load = 1.9923 k/ft

New dead load = 1.9923 k/ft + (0.650 k/ft – 0.375 k/ft) = 2.2673 k/ft

(2.2673 k/ft)/(1.9923 k/ft) = 1.1380

New $M_{u,max}$ at Supports $\cong$ (1.2)(-184.95 k-ft*1.1380) + (1.6)(-176.31 k-ft) = 534.66 k-ft

New $M_{u,max}$ at Midspan $\cong$ (1.2)(93.96 k-ft*1.1380) + (1.6)(89.56 k-ft) = 271.61 k-ft

New $V_{u,max} \cong$ (1.2)(33.37 k*1.1380) + (1.6)(31.81 k) = 96.47 k < ϕV_n = 258.99 kips

$\therefore$ Shear capacity is still OK.

3) Design the flexural reinforcement.

a) Compute the area of steel required at the point of maximum negative moment.

$$A_s \geq M_u/[\phi f_y(d - a/2)] \cong M_u/[\phi f_y(jd)]$$

Because there is negative moment at the support, the beams acts as a rectangular beam with compression in the web. Assume that j = 0.9 and ϕ = 0.90

$A_s \cong (534.66 \text{ k-ft})(12 \text{ in/ft})/[(0.9)(60 \text{ ksi})(0.9)(22.75")] = 5.80 \text{ in.}^2$

This value can be improved with one iteration to find the depth of the compression stress block, a:

$a = A_s f_y / 0.85 f'_c b = (5.80 \text{ in}^2)(60 \text{ ksi})/[(0.85)(4 \text{ ksi})(24")] = 4.267"$

and then recalculating the required A_s with this calculated value of a:

$A_s \geq M_u/[\phi f_y(d - a/2)] = (534.66 \text{ k-ft})(12 \text{ in/ft})/[(0.9)(60 \text{ ksi})(22.75" - 4.267"/2)]$

$= 5.76 \text{ in}^2$

Before proceeding, it must be confirmed that this is a tension-controlled section. This can be done by showing that the neutral axis, c, is less than 3/8 of d.

$a = A_s f_y / 0.85 f'_c b = (5.76 \text{ in}^2)(60 \text{ ksi})/[(0.85)(4 \text{ ksi})(24")] = 4.238"$

$c = a/\beta_1 = 4.238"/0.85 = 4.985" < (3/8)(d) = (3/8)(22.75") = 8.531"$

$\therefore$ Section is tension-controlled and can be designed using $\phi = 0.90$

b) Compute the area of steel required at the point of maximum positive moment.

$A_s \geq M_u/[\phi f_y(d - a/2)] \cong M_u/[\phi f_y(jd)]$

Assume that the compression zone is rectangular, and take j = 0.95 for the first calculation of A_s.

$A_s \cong (271.61 \text{ k-ft})(12 \text{ in/ft})/[(0.9)(60 \text{ ksi})(0.95)(22.75")] = 2.79 \text{ in.}^2$

This value can be improved with one iteration to find the depth of the compression stress block, a:

$a = A_s f_y / 0.85 f'_c b = (2.79 \text{ in}^2)(60 \text{ ksi})/[(0.85)(4 \text{ ksi})(24")] = 2.053"$

and then recalculating the required A_s with this calculated value of a:

$A_s \geq M_u/[\phi f_y(d - a/2)] = (271.61 \text{ k-ft})(12 \text{ in/ft})/[(0.9)(60 \text{ ksi})(22.75" - 2.053"/2)]$

$= 2.78 \text{ in}^2$

Before proceeding, it must be confirmed that this is a tension-controlled section. This can be done by showing that the neutral axis, c, is less than 3/8 of d.

$a = A_s f_y / 0.85 f'_c b = (2.78 \text{ in}^2)(60 \text{ ksi})/[(0.85)(4 \text{ ksi})(24")] = 2.043"$

$c = a/\beta_1 = 2.043"/0.85 = 2.404" < (3/8)(d) = (3/8)(22.75") = 8.531"$

∴ Section is tension-controlled and can be designed using $\phi = 0.90$

c) Calculate the minimum reinforcement (using ACI Code Section 10.5.1).

$A_{s,\,min}$ = max. of:

$[3\sqrt{f'_c}/f_y]b_w d = [3\sqrt{4000\text{ psi}}/60000\text{ psi}](24")(22.75") = 1.73\text{ in}^2$

$200 b_w d/f_y = (200)(24")(22.75")/60000\text{ psi} = 1.82\text{ in}^2$

$\therefore A_{s,min} = 1.82\text{ in}^2$

4) Calculate the area of steel and select the bars.

a) Negative-moment Region

$A_{s,req} = 5.76\text{ in}^2 > A_{s,min} = 1.82\text{ in}^2$ ∴ OK

Use (10) #7 bars [$A_s = (10)(0.60\text{ in}^2) = 6.00\text{ in}^2 > 5.76\text{ in}^2$ ∴ OK]

Small bars were selected at the supports because the bars have to be hooked into the exterior supports and there may not be enough room for a standard hook on larger bars.

b) Positive-moment Region

$A_{s,req} = 2.78\text{ in}^2 > A_{s,min} = 1.82\text{ in}^2$ ∴ OK

Use (5) #7 bars [$A_s = (5)(0.60\text{ in}^2) = 3.00\text{ in}^2 > 2.78\text{ in}^2$ ∴ OK]

5) Check the distribution of the reinforcement (spacing requirements).

a) Negative-moment Region

c_c = 2.25 in. cover + 0.5 in. stirrups = 2.75"

The maximum bar spacing is

$s = 15(40{,}000/f_s) - 2.5c_c$

$f_s = (2/3)(f_y) = (2/3)(60{,}000\text{ ksi}) = 40{,}000\text{ ksi}$

$s = 15(40{,}000/40{,}000) - (2.5)(2.75") = 8.125"$

Spacing of bars is less than 8.125" by inspection.

Minimum bar spacing:

s_c = max of [1", d_b, $(4/3)s_a$]; Assume s_a = 1" aggregate

s_c = max of [1", 0.875", (4/3)(1") = 1.333"]; Assume s_a = 1" aggregate

s_c = 1.333"

Side spacing and cover:

$b > (n)(d_b) + (n-1)(s_c) + 2d_{tr} + 2c_c$

24" > (10)(0.875") + (10-1)(1.333") + (2)(0.5") + (2)(2.25")

24" < 26.25" ∴ Need two rows of reinforcing in negative-moment regions

Minimum vertical spacing between layers of reinforcement

= max. of: (4/3)(s_a) or 1"

= max. of (4/3)(1") = 1.333", or 1"

=1.333"

New d_{eff} = 26" – 2.25" – 0.5" – 0.875" – (1/2)(1.333") = 21.708"

1) Re-check the shear capacity of the beam with d = 21.708".

$V_u = \phi(V_c + V_s)$

$V_{u,max}$ = 96.47 kips

From ACI Code Section 11.2.1.1, the nominal V_c is

$V_c = 2\lambda\sqrt{f'_c}b_wd$ = (2)(1.0) √4000 psi (24")(21.708")/1000 = 65.90 kips

ACI Code Section 11.4.7.9 sets the maximum nominal V_s as

$V_s = 8\sqrt{f'_c}b_wd$ = (8) √4000 psi (24")(21.708")/1000 = 263.60 kips

Thus, the absolute maximum ϕV_n = 0.75(65.90 k + 263.60 k) = 247.13 kips

≥ $V_{u,max}$ = 96.47 kips ∴ OK

Shear capacity is OK when accounting for weight of 24"x26" beam.

2) Re-design the flexural reinforcement with d = 21.708".

a) Compute the area of steel required at the point of maximum negative moment.

$A_s \geq M_u/[\phi f_y(d - a/2)] \cong M_u/[\phi f_y(jd)]$

Because there is negative moment at the support, the beams acts as a rectangular beam with compression in the web. Assume that j = 0.9 and $\phi = 0.90$

$A_s \cong (534.66\text{ k-ft})(12\text{ in/ft})/[(0.9)(60\text{ ksi})(0.9)(21.708")] = 6.08\text{ in.}^2$

This value can be improved with one iteration to find the depth of the compression stress block, a:

$a = A_s f_y/0.85f'_c b = (6.08\text{ in}^2)(60\text{ ksi})/[(0.85)(4\text{ ksi})(24")] = 4.472"$

and then recalculating the required A_s with this calculated value of a:

$A_s \geq M_u/[\phi f_y(d - a/2)] = (534.66\text{ k-ft})(12\text{ in/ft})/[(0.9)(60\text{ ksi})(21.708" - 4.472"/2)]$

$= 6.10\text{ in}^2$

Before proceeding, it must be confirmed that this is a tension-controlled section. This can be done by showing that the neutral axis, c, is less than 3/8 of d.

$a = A_s f_y/0.85f'_c b = (6.10\text{ in}^2)(60\text{ ksi})/[(0.85)(4\text{ ksi})(24")] = 4.487"$

$c = a/\beta_1 = 4.487"/0.85 = 5.278" < (3/8)(d) = (3/8)(21.708") = 8.141"$

$\therefore$ Section is tension-controlled and can be designed using $\phi = 0.90$

b) Compute the area of steel required at the point of maximum positive moment.

$A_s \geq M_u/[\phi f_y(d - a/2)] \cong M_u/[\phi f_y(jd)]$

Assume that the compression zone is rectangular, and take j = 0.95 for the first calculation of A_s.

$A_s \cong (271.61\text{ k-ft})(12\text{ in/ft})/[(0.9)(60\text{ ksi})(0.95)(21.708")] = 2.93\text{ in.}^2$

This value can be improved with one iteration to find the depth of the compression stress block, a:

$a = A_s f_y/0.85f'_c b = (2.93\text{ in}^2)(60\text{ ksi})/[(0.85)(4\text{ ksi})(24")] = 2.154"$

and then recalculating the required A_s with this calculated value of a:

$A_s \geq M_u/[\phi f_y(d - a/2)] = (271.61\text{ k-ft})(12\text{ in/ft})/[(0.9)(60\text{ ksi})(21.708" - 2.154"/2)]$

$= 2.93\text{ in}^2$

Before proceeding, it must be confirmed that this is a tension-controlled section. This can be done by showing that the neutral axis, c, is less than 3/8 of d.

$a = A_s f_y/0.85f'_c b = (2.93\text{ in}^2)(60\text{ ksi})/[(0.85)(4\text{ ksi})(24")] = 2.151"$

$c = a/\beta_1 = 2.151''/0.85 = 2.531'' < (3/8)(d) = (3/8)(21.708'') = 8.141''$

$\therefore$ Section is tension-controlled and can be designed using $\phi = 0.90$

c) Calculate the minimum reinforcement (using ACI Code Section 10.5.1).

$A_{s,\,min}$ = max. of:

$[3\sqrt{f'_c}/f_y]b_wd = [3\sqrt{4000\text{ psi}}/60000\text{ psi}](24'')(21.708'') = 1.65\text{ in}^2$

$200b_wd/f_y = (200)(24'')(21.708'')/60000\text{ psi} = 1.74\text{ in}^2$

$\therefore A_{s,min} = 1.74\text{ in}^2$

3) Re-calculate the area of steel and select the bars.

a) Negative-moment Region

$A_{s,req} = 6.10\text{ in}^2 > A_{s,min} = 1.74\text{ in}^2 \therefore$ OK

Use (5) #8 bars and (5) #7 bars in two rows.

$[A_s = (5)(0.79\text{ in}^2) + (5)(0.60\text{ in}^2) = 6.95\text{ in}^2 > 6.10\text{ in}^2 \therefore$ OK]

$a = A_sf_y/0.85f'_cb = (6.95\text{ in2})(60\text{ ksi})/[(0.85)(4\text{ ksi})(24'')] = 5.110''$

$a = \beta_1c =$ where $\beta = 0.85$ for $f'_c = 4{,}000$ psi

$c = a/\beta1 = 5.110''/0.85 = 6.012''$

$d_{actual} = 26'' - 2.25'' - 0.5'' - 1.0'' - (1/2)(1.333'') = 21.583''$

$\varepsilon_s = (d-c)(\varepsilon_u)/c = (21.583'' - 6.012'')(0.003)/6.012'' = 0.00777 > \varepsilon_y = 0.00207$

$\varepsilon_t \cong \varepsilon_s = 0.00777 > 0.005 \therefore$ Tension-controlled Section $\therefore \phi = 0.9$

$\phi M_n = \phi A_sf_y(d - a/2) = (0.9)(6.95\text{ in}^2)(60\text{ ksi})(21.583'' - 5.110''/2)/(12\text{ in/ft}) =$

$= 595.10\text{ k-ft} > 534.66\text{ k-ft} \therefore$ OK

Small bars were selected at the supports because the bars have to be hooked into the exterior supports and there may not be enough room for a standard hook on larger bars.

b) Positive-moment Region

$A_{s,req} = 2.93\text{ in}^2 > A_{s,min} = 1.74\text{ in}^2 \therefore$ OK

Use (5) #7 bars in one row $[A_s = (5)(0.60\text{ in}^2) = 3.00\text{ in}^2 > 2.93\text{ in}^2 \therefore$ OK]

*Using d = 21.708" for positive-moment region was conservative since using only one row of rebar in this region (actual "d" for this region will be greater than 21.708")

$a = A_s f_y / 0.85 f'_c b = (3.00 \text{ in}^2)(60 \text{ ksi})/[(0.85)(4 \text{ ksi})(24")] = 2.206"$

$a = \beta_1 c$ = where $\beta = 0.85$ for $f'_c = 4{,}000$ psi

$c = a/\beta 1 = 2.206"/0.85 = 2.595"$

$\varepsilon_s \cong (d-c)(\varepsilon_u)/c = (21.708" - 2.595")(0.003)/2.595" = 0.02210 > \varepsilon_y = 0.00207$

(actual "d" for positive-moment region is larger since only have one row of reinforcement)

$\varepsilon_t \cong \varepsilon_s = 0.02210 > 0.005$ $\therefore$ Tension-controlled Section $\therefore$ $\phi = 0.9$

$\phi M_n = \phi A_s f_y (d - a/2) = (0.9)(3.00 \text{ in}^2)(60 \text{ ksi})(21.708" - 2.206"/2)/(12 \text{ in/ft}) =$

$= 278.17$ k-ft > 271.61 k-ft $\therefore$ OK

5) Check the distribution of the reinforcement (spacing requirements).

a) Negative-moment Region

c_c = 2.25 in. cover + 0.5 in. stirrups = 2.75"

The maximum bar spacing is:

$s = 15(40{,}000/f_s) - 2.5c_c$

$f_s = (2/3)(f_y) = (2/3)(60{,}000 \text{ ksi}) = 40{,}000 \text{ ksi}$

$s = 15(40{,}000/40{,}000) - (2.5)(2.75") = 8.125"$

Spacing of bars is less than 8.125" by inspection.

Minimum bar spacing:

s_c = max of [1", d_b, $(4/3)s_a$]; Assume s_a = 1" aggregate

s_c = max of [1", 0.875", (4/3)(1") = 1.333"]; Assume s_a = 1" aggregate

$s_c = 1.333"$

Side spacing and cover:

$b > (n)(d_b) + (n-1)(s_c) + 2d_{tr} + 2c_c$

18” > (5)(1.00”) + (5-1)(1.333”) + (2)(0.5”) + (2)(2.25”)

24” > 15.83” ∴ OK

b) Positive-moment Region

The maximum bar spacing is 8.125”. Spacing of bars is less than 8.125” by inspection.

Minimum bar spacing = 1.333”

Side spacing and cover:

$b > (n)(d_b) + (n-1)(s_c) + 2d_{tr} + 2c_c$

24” > (5)(0.875”) + (5-1)(1.333”) + (2)(0.5”) + (2)(2.25”)

24” > 15.21” ∴ OK

6) Design the shear reinforcement.

a) The critical section for shear is located at the support. ACI Code Section 11.4.6.1 requires stirrups if $V_u \geq \phi V_c/2$

$V_c = 2\lambda\sqrt{f'_c}b_wd$ = (2)(1.0) √4000 psi (24”)(21.708”)/1000 = 65.90 kips

$V_c/2$ = 65.90 kips/2 = 32.95 kips

V_u/ϕ = (96.47 kips)/(0.75) = 128.63 kips > $V_c/2$ = 32.95 kips

∴ Stirrups are required.

b) Determine shear strength required by shear reinforcing.

$V_s = V_u/\phi - V_c$ = [(96.47 kips)/(0.75)] – 65.90 kips = 62.73 kips

$V_s \leq 8\sqrt{f'_c}b_wd$ = 8√4000 psi (24”)(21.708”)/1000 = 263.60 kips ∴ OK

c) Determine maximum spacing of shear reinforcing (ACI 318-08 Sections 11.4.5.1 and 11.4.5.3).

For $V_s \leq 8\sqrt{f'_c}b_wd$: s_{max} = min of {d/2, 24”}

d/2 = 21.708”/2 = 10.854”

s_{max} = 10”

d) Determine minimum shear reinforcement (ACI 318-08 Section 11.4.6.3).

$A_{v,min}$ = max of {$0.75\sqrt{f'_c} b_w s/f_{yt}$, $50 b_w s/f_{yt}$}

$0.75\sqrt{f'_c} b_w s/f_{yt} = 0.75\sqrt{4000 \text{ psi}} (24")(10")/60{,}000 \text{ psi} = 0.190 \text{ in}^2$

$50 b_w s/f_{yt} = 50(24")(10")/60{,}000 \text{ psi} = 0.200 \text{ in}^2$

$\therefore A_{v,min} = 0.200 \text{ in}^2$

Use #3 stirrups @ 10" as minimum shear reinforcement.

($A_v = 2 \text{ legs} \times 0.11 \text{ in}^2/\text{leg} = 0.22 \text{ in}^2 > 0.200 \text{ in}^2 \therefore$ OK)

e) Design the shear reinforcement.

$V_s = A_v f_{yt} d/s$

Rearranging: $s = A_v f_{yt} d/V_s = (0.22 \text{ in}^2)(60 \text{ ksi})(21.708")/62.73 \text{ kips} = 4.57"$

Usually absolute minimum "s" is 4".

Use (2) #3 stirrups @ 4", starting 2" from face of support.

Or use #4 stirrups instead of #3 stirrups.

For #4 stirrups: ($A_v = 2 \text{ legs} \times 0.20 \text{ in}^2/\text{leg} = 0.40 \text{ in}^2 > 0.200 \text{ in}^2 \therefore$ OK)

$s = A_v f_{yt} d/V_s = (0.40 \text{ in}^2)(60 \text{ ksi})(21.708")/62.73 \text{ kips} = 8.305"$

Use (2) #4 stirrups @ 8", starting 2" from face of support.

Use this stirrup layout throughout the entire length of the beam since lateral loads can change the shear forces (shear diagram) throughout the beam length (since the beam is part of a concrete moment frame).

FINAL DESIGN: Use 24" x 26" beam with (5) #8 and (5) #7 bars for negative moment reinforcement (at the supports) and (5) #7 bars for positive moment reinforcement. Use (2) #4 stirrups @ 8" throughout length of beam.

COLUMN DESIGN:

Load Case 1: 1.2D + 1.6L (Gravity Load Case)

Exterior Column:

P_u = 177.98 kips

M_2 = 31.47 k-ft

M_1 = -17.26 k-ft

1) Preliminary column size

$A_{g(trial)} \geq P_u/[0.40(f'_c + f_y\rho_g)$

$A_{g(trial)} \geq$ 177.98 kips/[0.40(4 ksi + (60 ksi)(0.015))] = 90.81 in^2

$\cong$ (9.53 in.)^2

Try 18"x18" column

2) Is the story being designed sway or nonsway?

$Q = [\sum P_u \times \Delta_o]/[V_{us} \times l_c]$

$\sum P_u \cong$ (5)(177.98 k) = 889.90 k

V_{us} = 1 kip

Δ_o = 0.017769"

l_c = 10.5' = 126"

Q = [(889.90 kips)(0.017769")]/[(1 kip)(126")] = 0.02002 < 0.05

$\therefore$ Nonsway (but assume sway story because $\sum P_u$ will actually be higher due to loads at other columns around the building at that level)

3) Are the columns slender?

r = 0.3h = (0.3)(18") = 5.4"

kl_u/r = (1.2)(126")/5.4" = 28 > 22 $\therefore$ Column is slender

4) Find δ_{ns} for the column.

$\delta_{ns} = C_m/[1 - (P_u/(0.75P_c))] \geq 1.0$

$C_m = 0.6 + 0.4(M_1/M_2) = 0.6 + 0.4(-17.26 \text{ k-ft}/31.47 \text{ k-ft}) = 0.3806$
$P_c = \pi^2EI/(kl_u)^2$

a) Calculation of EI values

$EI = [0.2E_cI_g + E_sI_{se}]/[1 + \beta_{dns}]$

$I_g = bh^3/12 = (18")(18")^3/12 = 8748 \text{ in}^4$

$E_c = 57{,}000\sqrt{f'_c} = 57{,}000\sqrt{4000} \text{ psi} = 3{,}605{,}000 \text{ psi} = 3605 \text{ ksi}$

$E_s = 29{,}000 \text{ ksi}$

$I_{se} \cong 2.2\rho_g\gamma^2 \times I_g$ (Table 12-1 in textbook "Reinforced Concrete Mechanics and Design by Wight and MacGregor)

Assume total steel ratio $\rho_g = 0.015$

For an 18"x18" column: $\gamma = [18" – (2)(2.5")]/18" = 0.7222$

$I_{se} \cong 2.2(0.015)(0.7222)^2 \times 8748 \text{ in}^4 = 150.58 \text{ in}^4$

Assuming that only the dead load is considered to cause a sustained axial load on the columns:

β_{dns} = (maximum factored sustained axial load)/(total factored axial load)

$\beta_{dns} = (1.2)(65.32 \text{ kips})/177.98 \text{ kips} = 0.6644$

$EI = [(0.2)(3605 \text{ ksi})(8748 \text{ in}^4) + (29{,}000 \text{ ksi})(150.58 \text{ in}^4)]/[1 + 0.6644]$

$= 6{,}413{,}198.75 \text{ kip-in}^2 = 6.4132 \times 10^6 \text{ kip-in}^2$

b) Calculation of P_c

$P_c = \pi^2EI/(kl_u)^2 = \pi^2(6{,}413{,}198.75 \text{ kip-in}^2)/[(1 \times 126")^2] = 3986.88 \text{ kips}$

c) Calculation of δ_{ns}

$\delta_{ns} = C_m/[1 – (P_u/(0.75P_c))] = 0.3806/[1 – (177.98 \text{ kips}/(0.75)(3986.88 \text{ kips}))]$

$= 0.4047 \therefore$ Use $\delta_{ns} = 1.0$

Thus, the moments do not need to be magnified for this loading case.

5) Check initial column sections for gravity-load case.

$e = M_c/P_u = (31.47 \text{ k-ft})(12 \text{ in/ft})/(177.98 \text{ kips}) = 2.12"$

$e/h = 2.12"/18" = 0.1179$

Fig. A-9b (from textbook "Reinforced Concrete Mechanics and Design by Wight and MacGregor):

Using $\gamma = 0.722 \cong 0.75$, $e/h = 0.1179$, and $\rho_g = 0.015$

$\phi P_n/A_g = 2.20$ ksi

$A_g \geq P_u/2.20$ ksi = 177.98 kips/2.20 ksi = 80.90 in^2

$A_g = (18")(18") = 324\ in^2 > 80.90\ in^2 \therefore$ OK

6) Select the longitudinal bars for this column.

$A_{st} = \rho_g A_g = (0.015)(324\ in^2) = 4.86\ in^2$

Select (12) #6 bars [$A_s = (12)(0.44\ in^2) = 5.28\ in^2 > 4.86\ in^2 \therefore$ OK]

It is OK to be a little conservative due to the corrosive natatorium environment.

$\phi P_n(max) = \phi \times 0.80[0.85f'_c(A_g - A_{st}) + f_y A_{st}]$

$= (0.65)(0.80)[(0.85)(4\ ksi)(324\ in^2 - 5.28\ in^2) + (60\ ksi)(5.28\ in^2)]$

$= 728.23$ kips > 177.98 kips $\therefore$ OK

*Could reduce reinforcement ratio and go back to graph, obtain new value, and use less reinforcement as long as the column still works

Load Case 2: Gravity Plus Lateral (Earthquake) Loads

Exterior Column:

$P_u = 67.49$ kips

$M_2 = -398.52$ k-ft

$M_1 = 176.51$ k-ft

1) Preliminary column size

$A_{g(trial)} \geq P_u/[0.40(f'_c + f_y\rho_g)$

$A_{g(trial)} \geq 67.49$ kips/[0.40(4 ksi + (60 ksi)(0.015))] = 34.43 in^2

$\cong (5.87\ in.)^2$

Try 18"x18" column (due to the large moments)

2) Is the story being designed sway or nonsway?

$Q = [\sum P_u \times \Delta_o]/[V_{us} \times l_c]$

$\sum P_u \cong (5)(177.98 \text{ k}) = 889.90 \text{ k}$

$V_{us} = 1$ kip

$\Delta_o = 0.002836''$

$l_c = 10.5' = 126''$

$Q = [(889.90 \text{ kips})(0.002836'')]/[(1 \text{ kips})(126'')] = 0.02002 < 0.05$

$\therefore$ Nonsway (but assume sway story because $\sum P_u$ will actually be higher due to loads at other columns around the building at that level)

3) Are the columns slender?

$r = 0.3h = (0.3)(18'') = 5.4''$

$kl_u/r = (1.2)(126'')/5.4'' = 28 > 22$ $\therefore$ Column is slender

4) Find δ_{ns} for the column.

$\delta_{ns} = C_m/[1 - (P_u/(0.75P_c))] \geq 1.0$

$C_m = 0.6 + 0.4(M_1/M_2) = 0.6 + 0.4(176.51 \text{ k-ft}/-398.52 \text{ k-ft}) = 0.4228$

$P_c = \pi^2 EI/(kl_u)^2$

a) Calculation of EI values

$EI = [0.2E_cI_g + E_sI_{se}]/[1 + \beta_{dns}]$

$I_g = bh^3/12 = (18'')(18'')^3/12 = 8748 \text{ in}^4$

$E_c = 57{,}000\sqrt{f'_c} = 57{,}000\sqrt{4000 \text{ psi}} = 3{,}605{,}000 \text{ psi} = 3605 \text{ ksi}$

$E_s = 29{,}000$ ksi

$I_{se} \cong 2.2\rho_g\gamma^2 \times I_g$ (Table 12-1 in textbook "Reinforced Concrete Mechanics and Design by Wight and MacGregor)

Assume total steel ratio $\rho_g = 0.015$

For an 18"x18" column: $\gamma = [18'' - (2)(2.5'')]/18'' = 0.7222$

$I_{se} \cong 2.2(0.015)(0.7222)^2 \times 8748 \text{ in}^4 = 150.58 \text{ in}^4$

Assuming that only the dead load is considered to cause a sustained axial load on the columns:

β_{dns} = (maximum factored sustained axial load)/(total factored axial load)

β_{dns} = (1.2)(30.44 kips)/67.49 kips = 0.5412

EI = [(0.2)(3605 ksi)(8748 in^4) + (29,000 ksi)(150.58 in^4)]/[1 + 0.5412]

= 6,925,855.18 kip-in^2 = 6.9259 x 10^6 kip-in^2

b) Calculation of P_c

$P_c = \pi^2 EI/(kl_u)^2 = \pi^2$(6,925,855.18 kip-in^2)/[(1 x 126")^2] = 4305.58 kips

c) Calculation of δ_{ns}

$\delta_{ns} = C_m/[1 - (P_u/(0.75P_c))]$ = 0.4228/[1 – (67.49 kips/(0.75)(4305.58 kips))]

= 0.4318 ∴ Use δ_{ns} = 1.0

Thus, the moments do not need to be magnified for this loading case.

5) Check initial column sections for gravity-load case.

e = M_c/P_u = (398.52 k-ft)(12 in/ft)/(67.49 kips) = 70.86"

e/h = 70.86"/18" = 3.94

Exceeds moment capacity of column.

Use interaction diagrams (Fig. A-9b) to determine required ρ_g:

The interaction diagrams are entered with:

$\phi P_n/A_g = P_u/A_g$ = (67.49 k)/(18"x18") = 0.208

$\phi M_n/A_g h = M_u/A_g h$ = (398.52 k-ft)(12 in/ft)/[(18"x18")(18")] = 0.820

Required ρ_g = 0.04 (which is too high)

∴ Must increase column size.

Try a 24"x24" column.

1) Use interaction diagrams (Fig. A-9b) to determine required ρ_g:

The interaction diagrams are entered with:

$\phi P_n/A_g = P_u/A_g$ = (67.49 k)/(24"x24") = 0.117

$\phi M_n/A_g h = M_u/A_g h$ = (398.52 k-ft)(12 in/ft)/[(24"x24")(24")] = 0.346

Required $\rho_g \cong 0.014$ ∴ OK to use 24"x24" column

2) Select the reinforcement

$A_{st} = \rho_g A_g$ = (0.014)(24"x24") = 8.064 in^2

Use (12) #8 bars [A_{st} = (12)(0.79 in^2) = 9.48 in^2 > 8.064 in^2 ∴ OK]

It is ok to be a little conservative due to the corrosive natatorium environment.

FINAL DESIGN: Use 24"x24" columns with (12) #8 bars.

Concrete Moment Frame – Column Line 2

Beams

*Use rebar cover of 1.5(1.5") = 2.25" due to corrosive environment (natatorium) (see ACI 7.7.6.1)

Design beams as a continuous beam.
Design beams for worst case and make all four beams the same size.

Axial Load and Moment (Unfactored) for Column Line 2 (24x24 Columns and 24x30 Beams)

	Beam 20	Beam 21	Beam 24	Beam 25	Column 10 Bottom, Exterior	Column 12 Bottom, Interior	Column 11 Top, Exterior	Column 13 Top, Interior
P_D					-130.28	-190.87	-67.61	-104.26
P_L					-29.47	-29.47	0.00	0.00
P_{Lr}					-59.92	-113.03	-24.93	-42.76
P_S					-35.71	-64.91	-28.28	-50.04
P_W					11.43	-1.55	3.55	-0.44
$P_{W,REVERSED}$					-11.39	1.52	-3.58	0.47
P_E					10.91	-1.31	2.51	-0.10
$P_{E,REVERSED}$					-11.13	1.48	-2.76	0.30
V_D (Top or Left)	-22.13	-22.72	-28.30	-31.31	-1.59	-0.11	-12.42	1.39
V_D (Bottom or Right)	23.37	22.78	33.63	30.62	-1.59	-0.11	-12.42	1.39
V_{Lr} (Top or Left)	-30.82	-32.38	-14.53	-15.69	-3.51	0.21	-9.46	0.89
V_{Lr} (Bottom or Right)	33.72	32.16	16.67	15.51	-3.51	0.21	-9.46	0.89
V_S (Top or Left)	-7.43	-7.39	-16.27	-18.26	-0.12	-0.15	-6.69	0.80
V_S (Bottom or Right)	7.48	7.51	19.77	17.77	-0.12	-0.15	-6.69	0.80
V_W (Top or Left)	7.88	6.77	3.55	3.11	14.23	16.60	3.91	9.72
V_W (Bottom or Right)	7.88	6.77	3.55	3.11	14.23	16.60	3.91	9.72
$V_{W,REVERSED}$ (Top or Left)	-7.81	-6.76	-3.58	-3.11	-13.85	-16.39	-4.08	-9.80
$V_{W,REVERSED}$ (Bottom or Right)	-7.81	-6.76	-3.58	-3.11	-13.85	-16.39	-4.08	-9.80
V_E (Top or Left)	8.40	7.19	2.51	2.41	18.74	21.20	0.63	6.21
V_E (Bottom or Right)	8.40	7.19	2.51	2.41	18.74	21.20	0.63	6.21
$V_{E,REVERSED}$ (Top or Left)	-8.37	-7.19	-2.76	-2.46	-17.84	-20.72	-1.43	-6.73
$V_{E,REVERSED}$ (Bottom or Right)	-8.37	-7.19	-2.76	-2.46	-17.84	-20.72	-1.43	-6.73
M_D (Top or Left)	-107.72	-120.68	-134.03	-203.29	23.39	1.41	134.03	-16.04
M_D (Bottom or Right)	-127.58	-121.71	-219.33	-192.13	-12.27	-1.03	-84.33	8.31
M_{Lr} (Top or Left)	-152.38	-188.86	-80.19	-118.16	**59.37**	**-62.90**	**80.19**	**-24.01**
M_{Lr} (Bottom or Right)	-190.66	-189.38	-124.66	-113.20	**-29.47**	**33.23**	**-93.02**	**70.78**
M_S (Top or Left)	-39.61	-38.48	-79.55	-125.40	1.46	2.14	79.55	-10.15
M_S (Bottom or Right)	-40.29	-40.42	-135.55	-117.56	-1.15	-1.26	-38.15	3.96
M_W (Top or Left)	132.76	107.83	59.58	49.47	-123.63	-159.89	-59.58	-103.48
M_W (Bottom or Right)	-119.47	-108.93	-54.01	-49.92	195.36	212.20	9.13	67.40
$M_{W,REVERSED}$ (Top or Left)	-131.34	-107.46	-60.23	-49.56	119.83	157.64	60.23	103.96
$M_{W,REVERSED}$ (Bottom or Right)	118.49	108.74	54.40	49.96	-190.65	-209.68	-11.51	-68.31
M_E (Top or Left)	141.68	114.25	41.03	38.46	-171.63	-209.94	-41.03	-77.86
M_E (Bottom or Right)	-126.96	-115.73	-39.40	-38.68	248.42	265.22	-29.94	31.27
$M_{E,REVERSED}$ (Top or Left)	-141.13	-114.26	-45.80	-39.47	161.78	204.63	45.80	81.99
$M_{E,REVERSED}$ (Bottom or Right)	126.69	115.73	42.51	39.19	-238.16	-259.92	20.65	-36.32
$M_{D,MIDSPAN}$	64.33	60.79	132.28	111.25				
$M_{Lr,MIDSPAN}$	94.47	85.42	69.86	61.87				
$M_{S,MIDSPAN}$	19.68	20.18	84.64	70.71				
$M_{W,MIDSPAN}$	6.64	-0.55	2.79	-0.23				
$M_{W,REVERSED,MIDSPAN}$	-6.43	0.64	-2.92	0.20				
$M_{E,MIDSPAN}$	7.36	-0.74	0.82	-0.11				
$M_{E,REVERSED,MIDSPAN}$	-7.22	0.74	-1.64	-0.14				

Torsional Effects are Included in Table

1.2D +/- 1.0E + 0.2S								
Max $V_{TOP/LEFT}$ (kips)	-36.411	-35.929	-39.974	-43.682	-19.773	-20.885	-17.672	8.034
Max $V_{BOTTOM/RIGHT}$ (kips)	37.935	36.025	46.823	42.709	-19.773	-20.885	-17.672	8.034
Max $M_{TOP/LEFT}$ (ft-kips)	-278.3137	-266.7756	-222.5419	-308.5008	190.1374	206.753	222.5419	-99.1366
Max $M_{BOTTOM/RIGHT}$ (ft-kips)	-288.1182	-269.8611	-329.7016	-292.7521	-253.1161	-261.4039	-138.7701	42.032
Max $M_{MIDSPAN}$ (ft-kips)	88.4919	77.7193	176.4816	147.5001				
Max P_u (kips)					-174.6034	-243.3334	-89.548	-135.222

1.2D + 1.6(Lr or S) + 0.8W								
Max $V_{TOP/LEFT}$ (kips)	-82.11	-84.48	-62.86	-69.28	-18.60	-13.48	-33.30	10.87
Max $V_{BOTTOM/RIGHT}$ (kips)	**88.30**	84.21	74.83	67.66	-18.60	-13.48	-33.30	10.87
Max $M_{TOP/LEFT}$ (ft-kips)	-478.15	-532.95	-336.30	-484.23	218.92	131.23	**336.30**	-90.13
Max $M_{BOTTOM/RIGHT}$ (ft-kips)	**-553.73**	-536.20	-523.28	-458.59	-214.40	-170.99	-259.23	177.14
Max $M_{MIDSPAN}$ (ft-kips)	233.66	210.13	**272.74**	232.65				
Max P_u (kips)					-261.32	**-411.13**	-129.25	-205.53

1.2D + 1.6W + 0.5(Lr or S)								
Max $V_{TOP/LEFT}$ (kips)	-54.457	-54.264	-47.827	-51.678	-25.824	-26.424	-26.162	17.662
Max $V_{BOTTOM/RIGHT}$ (kips)	57.515	54.254	55.921	50.599	-25.824	-26.424	-26.162	17.662
Max $M_{TOP/LEFT}$ (ft-kips)	-415.59795	-411.17805	-296.9865	-385.9405	249.48145	254.9868	296.9865	-189.89
Max $M_{BOTTOM/RIGHT}$ (ft-kips)	-439.5792	-415.0268	-417.3857	-369.2095	-334.50205	**-337.3473**	-166.1165	153.20445
Max $M_{MIDSPAN}$ (ft-kips)	135.061	116.6864	205.5139	169.1805				
Max P_u (kips)					-204.5154	-288.0394	-101.004	-150.842

1.2D + 1.6L + 0.5(L_r or S)								
Max P_u (kips)					-233.44	-332.70	-93.60	-146.49

1.4D								
Max P_u (kips)					-182.39	-267.21	-94.65	-145.96

Torsional Effects are Included in Tables

BEAM DESIGN

$V_{u,max}$ = 88.30 kips (1.2D + 1.6L_r + 0.8W)
$M_{u,max}$ at Supports = - 553.73 k-ft (1.2D + 1.6L_r + 0.8W)
$M_{u,max}$ at Midspan = 272.74 k-ft (1.2D + 1.6L_r + 0.8W)

Use normal-weight concrete with f'_c = 4000 psi
f_y = 60,000 psi for flexural reinforcement
f_{yt} = 60,000 psi for stirrups

1) Choose the actual size of the beam stem.

a) Calculate the minimum depth based on deflections.

Use worst case scenario (one-end continuous instead of both ends continuous).

ACI Table 9.5(a):

Minimum thickness, h = L/18.5 = [(32')(12 in/ft)]/18.5 = 20.76"

b) Determine the minimum depth based on the maximum negative moment.

$M_{u,max}$ at Supports = 553.73 k-ft

$\rho(\text{initial}) = [(\beta_1 f'_c)/(4f_y)] = [(0.85)(4\text{ ksi})/(4)(60\text{ ksi})] = 0.0142$

$\omega = \rho(f_y/f'_c) = (0.0142)(60\text{ ksi}/4\text{ ksi}) = 0.213$

$R = \omega f'c(1 - 0.59\omega) = (0.213)(4\text{ ksi})[1 - (0.59)(0.213)] = 0.745\text{ ksi}$

$bd^2 \geq M_u/\phi R = [(553.73\text{ ft-kips})(12\text{ in/ft})]/[(0.9)(0.745\text{ ksi})] = 9910.16\text{ in}^3$

Assuming b = 24 in. (for 24" x 24" column)

$d \geq 20.32$ in.

h ≅ 20.32" + 3.25" = 23.57" (accounting for 2.25" clear cover due to corrosive environment and assuming #4 stirrups and #8 bars; see ACI 7.7.6.1)

[(1.5)(1.5") = 2.25"; 2.25" + 0.5" + (1/2)(1.00") = 3.25"]

Try h = 30"

h = 30" > 20.76" ∴ Meets deflection criteria

d ≅ 30" – 3.25" = 26.75"

c) Check the shear capacity of the beam.

$V_u = \phi(V_c + V_s)$

$V_{u,max} = 88.30$ kips

From ACI Code Section 11.2.1.1, the nominal V_c is

$V_c = 2\lambda\sqrt{f'_c}b_wd = (2)(1.0)\sqrt{4000 \text{ psi}}\,(24")(26.75")/1000 = 81.21$ kips

ACI Code Section 11.4.7.9 sets the maximum nominal V_s as

$V_s = 8\sqrt{f'_c}b_wd = (8)\sqrt{4000 \text{ psi}}\,(24")(26.75")/1000 = 324.83$ kips

Thus, the absolute maximum $\phi V_n = 0.75(81.21 \text{ k} + 324.83 \text{ k}) = 304.53$ kips

$\geq V_{u,max} = 88.30$ kips $\therefore$ OK

d) Summary. Use:

b = 24”
h = 30”
d = 26.75”

2) Compute the dead load of the stem, and recompute the total moment.

Weight of 24”x30” concrete beam = $[(24")(30")/144 \text{ in}^2/\text{ft}^2][(150 \text{ lb/ft}^3)/1000]$

= 0.720 k/ft

Original dead load = 1.42 k/ft

New dead load = 1.42 k/ft + 0.720 k/ft = 2.14 k/ft

(2.14 k/ft)/(1.42 k/ft) = 1.507

New $M_{u,max}$ at Supports $\cong$

Beam 20: $1.2D + 1.6L_r + 0.8W$

= (1.2)(-127.58 k-ft*1.507) + (1.6)(-190.66 k-ft) + (0.8)(-119.47 k-ft) =

= -631.35 k-ft

New $M_{u,max}$ at Midspan $\cong$

Beam 24: $1.2D + 1.6S + 0.8W$

= (1.2)(132.28 k-ft*1.507) + (1.6)(84.64 k-ft) + (0.8)(2.79 k-ft)

= 376.87 k-ft

New $V_{u,max} \cong$

Beam 20: $1.2D + 1.6L_r + 0.8W$

$= (1.2)(23.37\text{ k}*1.507) + (1.6)(33.72\text{ k}) + (0.8)(7.88\text{ k}) =$

$= 102.52\text{ k} < \phi V_n = 304.53\text{ kips}$

$\therefore$ Shear capacity is still OK.

3) Design the flexural reinforcement.

a) Compute the area of steel required at the point of maximum negative moment.

$A_s \geq M_u/[\phi f_y(d - a/2)] \cong M_u/[\phi f_y(jd)]$

Because there is negative moment at the support, the beams acts as a rectangular beam with compression in the web. Assume that j = 0.9 and $\phi = 0.90$

$A_s \cong (631.35\text{ k-ft})(12\text{ in/ft})/[(0.9)(60\text{ ksi})(0.9)(26.75")] = 5.83\text{ in.}^2$

This value can be improved with one iteration to find the depth of the compression stress block, a:

$a = A_s f_y/0.85f'_c b = (5.83\text{ in}^2)(60\text{ ksi})/[(0.85)(4\text{ ksi})(24")] = 4.285"$

and then recalculating the required A_s with this calculated value of a:

$A_s \geq M_u/[\phi f_y(d - a/2)] = (631.35\text{ k-ft})(12\text{ in/ft})/[(0.9)(60\text{ ksi})(26.75" - 4.285"/2)]$

$= 5.70\text{ in}^2$

Before proceeding, it must be confirmed that this is a tension-controlled section. This can be done by showing that the neutral axis, c, is less than 3/8 of d.

$a = A_s f_y/0.85f'_c b = (5.70\text{ in}^2)(60\text{ ksi})/[(0.85)(4\text{ ksi})(24")] = 4.192"$

$c = a/\beta_1 = 4.192"/0.85 = 4.932" < (3/8)(d) = (3/8)(26.75") = 10.031"$

$\therefore$ Section is tension-controlled and can be designed using $\phi = 0.90$

b) Compute the area of steel required at the point of maximum positive moment.

$A_s \geq M_u/[\phi f_y(d - a/2)] \cong M_u/[\phi f_y(jd)]$

Assume that the compression zone is rectangular, and take j = 0.95 for the first calculation of A_s.

$A_s \cong (376.87\text{ k-ft})(12\text{ in/ft})/[(0.9)(60\text{ ksi})(0.95)(26.75")] = 3.30\text{ in.}^2$

This value can be improved with one iteration to find the depth of the compression stress block, a:

$a = A_s f_y / 0.85 f'_c b = (3.30\ in^2)(60\ ksi)/[(0.85)(4\ ksi)(24")] = 2.423"$

and then recalculating the required A_s with this calculated value of a:

$A_s \geq M_u/[\phi f_y(d - a/2)] = (376.87\ k\text{-}ft)(12\ in/ft)/[(0.9)(60\ ksi)(26.75" - 2.423"/2)]$

$= 3.28\ in^2$

Before proceeding, it must be confirmed that this is a tension-controlled section. This can be done by showing that the neutral axis, c, is less than 3/8 of d.

$a = A_s f_y / 0.85 f'_c b = (3.28\ in^2)(60\ ksi)/[(0.85)(4\ ksi)(24")] = 2.411"$

$c = a/\beta_1 = 2.411"/0.85 = 2.837" < (3/8)(d) = (3/8)(26.75") = 10.031"$

$\therefore$ Section is tension-controlled and can be designed using $\phi = 0.90$

c) Calculate the minimum reinforcement (using ACI Code Section 10.5.1).

$A_{s,\ min}$ = max. of:

$[3\sqrt{f'_c}/f_y]b_w d = [3\sqrt{4000\ psi}/60000\ psi](24")(26.75") = 2.03\ in^2$

$200 b_w d/f_y = (200)(24")(26.75")/60000\ psi = 2.14\ in^2$

$\therefore A_{s,min} = 2.14\ in^2$

4) Calculate the area of steel and select the bars.

a) Negative-moment Region

$A_{s,req} = 5.70\ in^2 > A_{s,min} = 2.14\ in^2\ \therefore$ OK

Use (10) #7 bars [$A_s = (10)(0.60\ in^2) = 6.00\ in^2 > 5.70\ in^2\ \therefore$ OK]

Small bars were selected at the supports because the bars have to be hooked into the exterior supports and there may not be enough room for a standard hook on larger bars.

b) Positive-moment Region

$A_{s,req} = 3.28\ in^2 > A_{s,min} = 2.14\ in^2\ \therefore$ OK

Use (6) #7 bars [$A_s = (6)(0.60\ in^2) = 3.60\ in^2 > 3.28\ in^2\ \therefore$ OK]

5) Check the distribution of the reinforcement (spacing requirements).

a) Negative-moment Region

c_c = 2.25 in. cover + 0.5 in. stirrups = 2.75”

The maximum bar spacing is

$s = 15(40{,}000/f_s) - 2.5c_c$

$f_s = (2/3)(f_y) = (2/3)(60{,}000\text{ ksi}) = 40{,}000\text{ ksi}$

$s = 15(40{,}000/40{,}000) - (2.5)(2.75”) = 8.125”$

Spacing of bars is less than 8.125” by inspection.

Minimum bar spacing:

s_c = max of [1”, d_b, $(4/3)s_a$]; Assume s_a = 1” aggregate

s_c = max of [1”, 0.875”, (4/3)(1”) = 1.333”]; Assume s_a = 1” aggregate

s_c = 1.333”

Side spacing and cover:

$b > (n)(d_b) + (n-1)(s_c) + 2d_{tr} + 2c_c$

24” > (10)(0.875”) + (10-1)(1.333”) + (2)(0.5”) + (2)(2.25”)

24” < 26.25” ∴ Need two rows of reinforcing in negative-moment region

Minimum vertical spacing between layers of reinforcement

= max. of: $(4/3)(s_a)$ or 1”

= max. of (4/3)(1”) = 1.333”, or 1”

=1.333”

New d_{eff} = 30” – 2.25” – 0.5” – 0.875” – (1/2)(1.333”) = 25.708”

1) Re-check the shear capacity of the beam with d = 25.708”.

$V_u = \phi(V_c + V_s)$

$V_{u,max}$ = 102.52 kips

From ACI Code Section 11.2.1.1, the nominal V_c is

$V_c = 2\lambda\sqrt{f'_c}b_wd$ = (2)(1.0) √4000 psi (24”)(25.708”)/1000 = 78.04 kips

ACI Code Section 11.4.7.9 sets the maximum nominal V_s as

$$V_s = 8\sqrt{f'_c}b_wd = (8)\ \sqrt{4000\text{ psi}}\ (24")(25.708")/1000 = 312.18\text{ kips}$$

Thus, the absolute maximum $\phi V_n = 0.75(78.04\text{ k} + 312.18\text{ k}) = 292.67$ kips

$\geq V_{u,max} = 102.52$ kips $\therefore$ OK

Shear capacity is OK when accounting for weight of 24x30 beam.

2) Re-design the flexural reinforcement with d = 25.708".

a) Compute the area of steel required at the point of maximum negative moment.

$$A_s \geq M_u/[\phi f_y(d - a/2)] \cong M_u/[\phi f_y(jd)]$$

Because there is negative moment at the support, the beams acts as a rectangular beam with compression in the web. Assume that j = 0.9 and $\phi = 0.90$

$$A_s \cong (631.35\text{ k-ft})(12\text{ in/ft})/[(0.9)(60\text{ ksi})(0.9)(25.708")] = 6.06\text{ in.}^2$$

This value can be improved with one iteration to find the depth of the compression stress block, a:

$$a = A_sf_y/0.85f'_cb = (6.06\text{ in}^2)(60\text{ ksi})/[(0.85)(4\text{ ksi})(24")] = 4.459"$$

and then recalculating the required A_s with this calculated value of a:

$$A_s \geq M_u/[\phi f_y(d - a/2)] = (631.35\text{ k-ft})(12\text{ in/ft})/[(0.9)(60\text{ ksi})(25.708" - 4.459"/2)]$$

$$= 5.98\text{ in}^2$$

Before proceeding, it must be confirmed that this is a tension-controlled section. This can be done by showing that the neutral axis, c, is less than 3/8 of d.

$$a = A_sf_y/0.85f'_cb = (5.98\text{ in}^2)(60\text{ ksi})/[(0.85)(4\text{ ksi})(24")] = 4.394"$$

$$c = a/\beta_1 = 4.394"/0.85 = 5.169" < (3/8)(d) = (3/8)(25.708") = 9.641"$$

$\therefore$ Section is tension-controlled and can be designed using $\phi = 0.90$

b) Compute the area of steel required at the point of maximum positive moment.

$$A_s \geq M_u/[\phi f_y(d - a/2)] \cong M_u/[\phi f_y(jd)]$$

Assume that the compression zone is rectangular, and take j = 0.95 for the first calculation of A_s.

$$A_s \cong (376.87\text{ k-ft})(12\text{ in/ft})/[(0.9)(60\text{ ksi})(0.95)(25.708")] = 3.43\text{ in.}^2$$

This value can be improved with one iteration to find the depth of the compression stress block, a:

$a = A_s f_y / 0.85 f'_c b = (3.43\ in^2)(60\ ksi)/[(0.85)(4\ ksi)(24")] = 2.521"$

and then recalculating the required A_s with this calculated value of a:

$A_s \geq M_u/[\phi f_y(d - a/2)] = (376.87\ k\text{-}ft)(12\ in/ft)/[(0.9)(60\ ksi)(25.708" - 2.521"/2)]$

$= 3.43\ in^2$

Before proceeding, it must be confirmed that this is a tension-controlled section. This can be done by showing that the neutral axis, c, is less than 3/8 of d.

$a = A_s f_y / 0.85 f'_c b = (3.43\ in^2)(60\ ksi)/[(0.85)(4\ ksi)(24")] = 2.522"$

$c = a/\beta_1 = 2.522"/0.85 = 2.967" < (3/8)(d) = (3/8)(25.708") = 9.641"$

$\therefore$ Section is tension-controlled and can be designed using $\phi = 0.90$

3) Re-calculate the area of steel and select the bars.

a) Negative-moment Region

$A_{s,req} = 5.98\ in^2 > A_{s,min} = 2.14\ in^2 \therefore$ OK

Use (10) #7 bars in two rows.

$[A_s = (10)(0.60\ in^2) = 6.00\ in^2 > 5.98\ in^2 \therefore$ OK]

$a = A_s f_y / 0.85 f'_c b = (6.00\ in^2)(60\ ksi)/[(0.85)(4\ ksi)(24")] = 4.4118"$

$a = \beta_1 c =$ where $\beta = 0.85$ for $f'_c = 4{,}000$ psi

$c = a/\beta 1 = 4.4118"/0.85 = 5.1903"$

$\varepsilon_s = (d\text{-}c)(\varepsilon_u)/c = (25.708" - 5.1903")(0.003)/5.1903" = 0.01186 > \varepsilon_y = 0.00207$

$\varepsilon_t \cong \varepsilon_s = 0.01186 > 0.005 \therefore$ Tension-controlled Section $\therefore \phi = 0.9$

$\phi M_n = \phi A_s f_y(d - a/2) = [(0.9)(6.00\ in^2)(60\ ksi)(25.708" - 4.4118"/2)]/(12\ in/ft) =$

$= 634.56\ k\text{-}ft > 631.35\ k\text{-}ft \therefore$ OK

Small bars were selected at the supports because the bars have to be hooked into the exterior supports and there may not be enough room for a standard hook on larger bars.

b) Positive-moment Region

$A_{s,req} = 3.43\ in^2 > A_{s,min} = 2.14\ in^2$ ∴ OK

Use (8) #6 bars in two rows [$A_s = (8)(0.44\ in^2) = 3.52\ in^2 > 3.43\ in^2$ ∴ OK]

$a = A_s f_y/0.85f'_c b = (3.52\ in^2)(60\ ksi)/[(0.85)(4\ ksi)(24")] = 2.5882"$

$a = \beta_1 c$ = where $\beta = 0.85$ for $f'_c = 4{,}000$ psi

$c = a/\beta 1 = 2.5882"/0.85 = 3.0450"$

$\varepsilon_s \cong (d-c)(\varepsilon_u)/c = (25.708" - 3.0450")(0.003)/3.0450" = 0.02233 > \varepsilon_y = 0.00207$

$\varepsilon_t \cong \varepsilon_s = 0.02233 > 0.005$ ∴ Tension-controlled Section ∴ $\phi = 0.9$

$\phi M_n = \phi A_s f_y(d - a/2) = (0.9)(3.52\ in^2)(60\ ksi)(25.708" - 2.5882"/2)/(12\ in/ft) =$

$= 386.72$ k-ft > 376.87 k-ft ∴ OK

5) Check the distribution of the reinforcement (spacing requirements).

a) Negative-moment Region

c_c = 2.25 in. cover + 0.5 in. stirrups = 2.75"

The maximum bar spacing is:

$s = 15(40{,}000/f_s) - 2.5c_c$

$f_s = (2/3)(f_y) = (2/3)(60{,}000\ ksi) = 40{,}000\ ksi$

$s = 15(40{,}000/40{,}000) - (2.5)(2.75") = 8.125"$

Spacing of bars is less than 8.125" by inspection.

Minimum bar spacing:

s_c = max of [1", d_b, $(4/3)s_a$]; Assume s_a = 1" aggregate

s_c = max of [1", 0.875", (4/3)(1") = 1.333"]; Assume s_a = 1" aggregate

$s_c = 1.333"$

Side spacing and cover:

$b > (n)(d_b) + (n-1)(s_c) + 2d_{tr} + 2c_c$

$24" > (5)(0.875") + (5-1)(1.333") + (2)(0.5") + (2)(2.25")$

$24" > 15.21"$ ∴ OK

b) Positive-moment Region

The maximum bar spacing is 8.125”. Spacing of bars is less than 8.125” by inspection.

Minimum bar spacing = 1.333”

Side spacing and cover:

$b > (n)(d_b) + (n-1)(s_c) + 2d_{tr} + 2c_c$

24” > (4)(0.75”) + (4-1)(1.333”) + (2)(0.5”) + (2)(2.75”)

24” > 12.50” $\therefore$ OK

6) Design the shear reinforcement.

a) The critical section for shear is located at the support. ACI Code Section 11.4.6.1 requires stirrups if $V_u \geq \phi V_c/2$

$V_c = 2\lambda\sqrt{f'_c}b_w d$ = (2)(1.0) √4000 psi (24”)(25.708”)/1000 = 78.04 kips

$V_c/2$ = 78.04 kips/2 = 39.02 kips

V_u/ϕ = (102.52 kips)/(0.75) = 136.69 kips > $V_c/2$ = 39.02 kips

$\therefore$ Stirrups are required.

b) Determine shear strength required by shear reinforcing.

$V_s = V_u/\phi - V_c$ = [(102.52 kips)/(0.75)] – 78.04 kips = 58.65 kips

$V_s \leq 8\sqrt{f'_c}b_w d$ = 8√4000 psi (24”)(25.708”)/1000 = 312.18 kips $\therefore$ OK

c) Determine maximum spacing of shear reinforcing (ACI 318-08 Sections 11.4.5.1 and 11.4.5.3).

For $V_s \leq 8\sqrt{f'_c}b_w d$: s_{max} = min of {d/2, 24”}

d/2 = 25.708”/2 = 12.854”

s_{max} = 12”

d) Determine minimum shear reinforcement (ACI 318-08 Section 11.4.6.3).

$A_{v,min}$ = max of $\{0.75\sqrt{f'_c}b_w s/f_{yt}, 50b_w s/f_{yt}\}$

$0.75\sqrt{f'_c}b_w s/f_{yt}$ = 0.75√4000 psi (24”)(12”)/60,000 psi = 0.23 in^2

$50b_w s/f_{yt}$ = 50(24")(12")/60,000 psi = 0.24 in^2

$\therefore$ $A_{v,min}$ = 0.24 in^2

s = $A_v f_{yt} d/V_s$ = (0.24 in^2)(60 ksi)(25.708")/58.65 kips = 6.312"

Use #4 stirrups @ 6" as minimum shear reinforcement.

e) Design the shear reinforcement.

$V_s = A_v f_{yt} d/s$

Rearranging: s = $A_v f_{yt} d/V_s$ = (0.24 in^2)(60 ksi)(25.708")/58.65 kips = 6.312"

Use #4 stirrups.

For #4 stirrups: (A_v = 2 legs x 0.20 in^2/leg = 0.40 in^2 > 0.24 in^2 $\therefore$ OK)

s = $A_v f_{yt} d/V_s$ = (0.40 in^2)(60 ksi)(25.708")/58.65 kips = 10.52"

Use (2) #4 stirrups @ 10", starting 2" from face of support.

Use this stirrup layout throughout the entire length of the beam since lateral loads can change the shear forces (shear diagram) throughout the beam length (since the beam is part of a concrete moment frame).

FINAL DESIGN: Use 24" x 30" beam with (10) #7 bars for negative moment reinforcement (at the supports) and (8) #6 bars for positive moment reinforcement.

COLUMN DESIGN

Load Case 1: 1.2D + 1.6W + 0.5L_r

Interior Column (worse case): Column 12 (bottom, interior)

P_u = 288.04 kips (compression)

M_2 = -337.35 k-ft

M_1 = 254.99 k-ft

1) Preliminary column size

$A_{g(trial)} \geq P_u/[0.40(f'_c + f_y \rho_g)$

$A_{g(trial)} \geq$ 288.04 kips/[0.40(4 ksi + (60 ksi)(0.015))] = 146.96 in^2

$\cong$ (12.12 in.)^2

Try 24"x24" column (due to large moments on column)

2) Is the story being designed sway or nonsway?

$Q = [\sum P_u \text{ x } \Delta_o]/[V_{us} \text{ x } l_c]$

$\sum P_u \cong (2)(204.52)$ kips + (3)(288.04) kips = 1273.16

$V_{us} = 1$ kip

$\Delta_o = 0.006298"$

$l_c = 22.5' = 270"$

Q = [(1273.16 kips)(0.006298")]/[(1 kips)(270")] = 0.02970 < 0.05

∴ Nonsway (but assume sway story because $\sum P_u$ will actually be higher due to loads at other columns around the building at that level)

3) Are the columns slender?

r = 0.3h = (0.3)(24") = 7.2"

kl_u/r = (1.2)(270")/7.2" = 45 > 22 ∴ Column is slender

4) Find δ_{ns} for the column.

$\delta_{ns} = C_m/[1 - (P_u/(0.75P_c))] \geq 1.0$

$C_m = 0.6 + 0.4(M_1/M_2)$ = 0.6 + 0.4(254.99 k-ft/-337.35 k-ft) = 0.2977

$P_c = \pi^2 EI/(kl_u)^2$

a) Calculation of EI values

$EI = [0.2E_cI_g + E_sI_{se}]/[1 + \beta_{dns}]$

$I_g = bh^3/12 = (24")(24")^3/12 = 27{,}648 \text{ in}^4$

$E_c = 57{,}000\sqrt{f'_c} = 57{,}000\sqrt{4000}$ psi = 3,605,000 psi = 3605 ksi

E_s = 29,000 ksi

$I_{se} \cong 2.2\rho_g\gamma^2 \text{ x } I_g$ (Table 12-1 in textbook "Reinforced Concrete Mechanics and Design by Wight and MacGregor)

Assume total steel ratio $\rho_g = 0.015$

For a 24"x24" column: γ = [24" – (2)(2.5")]/24" = 0.7917

$I_{se} \cong 2.2(0.015)(0.7917)^2 \times 27{,}648 \text{ in}^4 = 571.82 \text{ in}^4$

Assuming that only the dead load is considered to cause a sustained axial load on the columns:

β_{dns} = (maximum factored sustained axial load)/(total factored axial load)

$\beta_{dns} = (1.2)(190.87 \text{ kips})/288.04 \text{ kips} = 0.7952$

$EI = [(0.2)(3605 \text{ ksi})(27{,}648 \text{ in}^4) + (29{,}000 \text{ ksi})(571.82 \text{ in}^4)]/[1 + 0.7952]$

$= 20{,}341{,}459 \text{ kip-in}^2 = 20.3415 \times 10^6 \text{ kip-in}^2$

b) Calculation of P_c

$P_c = \pi^2 EI/(kl_u)^2 = \pi^2(20{,}341{,}459 \text{ kip-in}^2)/[(1 \times 270")^2] = 2753.94 \text{ kips}$

c) Calculation of δ_{ns}

$\delta_{ns} = C_m/[1 - (P_u/(0.75P_c))] = 0.2977/[1 - (288.04 \text{ kips}/(0.75)(2753.94 \text{ kips}))]$

$= 0.3459 \therefore \text{ Use } \delta_{ns} = 1.0$

Thus, the moments do not need to be magnified for this loading case.

5) Check initial column sections.

$e = M_c/P_u = [(337.35 \text{ k-ft})(12 \text{ in/ft})]/(288.04 \text{ kips}) = 14.054"$

$e/h = 14.054"/24" = 0.5856$

Fig. A-9b (from textbook "Reinforced Concrete Mechanics and Design by Wight and MacGregor):

Using $\gamma = 0.7917 \cong 0.75$, $e/h = 0.5856$, and $\rho_g = 0.015$

$\phi P_n/A_g = 0.85 \text{ ksi}$

$A_g \geq P_u/0.45 \text{ ksi} = 288.04 \text{ kips}/0.85 \text{ ksi} = 338.87 \text{ in}^2$

$A_g = (24")(24") = 576 \text{ in}^2 > 338.87 \text{ in}^2 \therefore \text{ OK}$

$\phi M_n/bh^2 = 0.47 \text{ ksi}$

$bh^2 \geq [(337.35 \text{ k-ft})(12 \text{ in/ft})]/0.47 \text{ ksi} = 8{,}613.19 \text{ in}^3$

$h \geq \sqrt{[(8{,}613.19 \text{ in}^3)/(b)]} = \sqrt{[(13{,}042 \text{ in}^3)/(24")]} = 18.94"$
$h = 24" > 18.94" \therefore \text{OK}$

6) Select the longitudinal bars for this column.

$A_{st} = \rho_g A_g = (0.015)(576\ in^2) = 8.64\ in^2$

Select (12) #8 bars [$A_s = (12)(0.79\ in^2) = 9.48\ in^2 > 8.64\ in^2 \therefore$ OK]

It is OK to be a little conservative due to the corrosive natatorium environment.

$\phi P_n(max) = \phi \times 0.80[0.85f'_c(A_g - A_{st}) + f_y A_{st}]$

$= (0.65)(0.80)[(0.85)(4\ ksi)(576\ in^2 - 9.48\ in^2) + (60\ ksi)(9.48\ in^2)]$

$= 1297.38$ kips > 288.04 kips $\therefore$ OK

Load Case 2: 1.2D + 1.6L$_r$ + 0.8W

Interior Column (worst case): Column 12 (bottom, interior)

$P_u = 411.13$ kips (compression)

$M_2 = -170.99$ k-ft

$M_1 = 131.23$ k-ft

1) Preliminary column size

$A_{g(trial)} \geq P_u/[0.40(f'_c + f_y\rho_g)$

$A_{g(trial)} \geq 411.13\ kips/[0.40(4\ ksi + (60\ ksi)(0.015))] = 209.76\ in^2$

$\cong (14.48\ in.)^2$

Try 24"x24" column (due to large moments on column)

2) Is the story being designed sway or nonsway?

$Q = [\sum P_u \times \Delta_o]/[V_{us} \times l_c]$

$\sum P_u \cong (2)(261.32)$ kips $+ (3)(411.12)$ kips $= 1756$ kips

$V_{us} = 1$ kip

$\Delta_o = 0.006298"$

$l_c = 22.5' = 270"$

$Q = [(1756\ kips)(0.006298")]/[(1\ kips)(270")] = 0.04096 < 0.05$
$\therefore$ Nonsway (but assume sway story because $\sum P_u$ will actually be higher due to

loads at other columns around the building at that level)

3) Are the columns slender?

$r = 0.3h = (0.3)(24") = 7.2"$

$kl_u/r = (1.2)(270")/7.2" = 45 > 22$ ∴ Column is slender

4) Find δ_{ns} for the column.

$\delta_{ns} = C_m/[1 - (P_u/(0.75P_c))] \geq 1.0$

$C_m = 0.6 + 0.4(M_1/M_2) = 0.6 + 0.4(131.23 \text{ k-ft}/-170.99 \text{ k-ft}) = 0.2930$

$P_c = \pi^2 EI/(kl_u)^2$

a) Calculation of EI values

$EI = [0.2E_cI_g + E_sI_{se}]/[1 + \beta_{dns}]$

$I_g = bh^3/12 = (24")(24")^3/12 = 27{,}648 \text{ in}^4$

$E_c = 57{,}000\sqrt{f'_c} = 57{,}000\sqrt{4000} \text{ psi} = 3{,}605{,}000 \text{ psi} = 3605 \text{ ksi}$

$E_s = 29{,}000 \text{ ksi}$

$I_{se} \cong 2.2\rho_g\gamma^2 \times I_g$ (Table 12-1 in textbook "Reinforced Concrete Mechanics and Design by Wight and MacGregor)

Assume total steel ratio $\rho_g = 0.015$

For a 24"x24" column: $\gamma = [24" - (2)(2.5")]/24" = 0.7917$

$I_{se} \cong 2.2(0.015)(0.7917)^2 \times 27{,}648 \text{ in}^4 = 571.82 \text{ in}^4$

Assuming that only the dead load is considered to cause a sustained axial load on the columns:

β_{dns} = (maximum factored sustained axial load)/(total factored axial load)

$\beta_{dns} = (1.2)(190.87 \text{ kips})/411.13 \text{ kips} = 0.5571$

$EI = [(0.2)(3605 \text{ ksi})(27{,}648 \text{ in}^4) + (29{,}000 \text{ ksi})(571.82 \text{ in}^4)]/[1 + 0.5571]$

$= 23{,}451{,}922.16 \text{ kip-in}^2 = 23.4519 \times 10^6 \text{ kip-in}^2$

b) Calculation of P_c

$P_c = \pi^2 EI/(kl_u)^2 = \pi^2(23{,}451{,}922.16 \text{ kip-in}^2)/[(1 \times 270")^2] = 3175.05 \text{ kips}$

c) Calculation of δ_{ns}

$\delta_{ns} = C_m/[1 - (P_u/(0.75P_c))] = 0.2930/[1 - (411.13 \text{ kips}/(0.75)(3175.05 \text{ kips}))]$

$= 0.3541 \therefore$ Use $\delta_{ns} = 1.0$

Thus, the moments do not need to be magnified for this loading case.

5) Check initial column sections.

$e = M_c/P_u = [(170.99 \text{ k-ft})(12 \text{ in/ft})]/(411.13 \text{ kips}) = 4.9908”$

$e/h = 4.9908”/24” = 0.2080$

Fig. A-9b (from textbook “Reinforced Concrete Mechanics and Design by Wight and MacGregor):

Using $\gamma = 0.7917 \cong 0.75$, $e/h = 0.2080$, and $\rho_g = 0.015$

$\phi P_n/A_g = 1.70$ ksi

$A_g \geq P_u/0.45 \text{ ksi} = 411.13 \text{ kips}/1.70 \text{ ksi} = 241.84 \text{ in}^2$

$A_g = (24”)(24”) = 576 \text{ in}^2 > 241.84 \text{ in}^2 \therefore$ OK

$\phi M_n/bh^2 = 0.34$ ksi

$bh^2 \geq [(170.99 \text{ k-ft})(12 \text{ in/ft})]/0.34 \text{ ksi} = 6{,}034.94 \text{ in}^3$

$h \geq \sqrt{[(6{,}034.94 \text{ in}^3)/(b)]} = \sqrt{[(13{,}042 \text{ in}^3)/(24”)]} = 15.86”$

$h = 24” > 15.86” \therefore$ OK

6) Select the longitudinal bars for this column.

$A_{st} = \rho_g A_g = (0.015)(576 \text{ in}^2) = 8.64 \text{ in}^2$

Select (12) #8 bars [$A_s = (12)(0.79 \text{ in}^2) = 9.48 \text{ in}^2 > 8.64 \text{ in}^2 \therefore$ OK]

It is OK to be a little conservative due to the corrosive natatorium environment.

$\phi P_n(\max) = \phi \times 0.80[0.85f'_c(A_g - A_{st}) + f_y A_{st}]$

$= (0.65)(0.80)[(0.85)(4 \text{ ksi})(576 \text{ in}^2 - 9.48 \text{ in}^2) + (60 \text{ ksi})(9.48 \text{ in}^2)]$

$= 1297.38 \text{ kips} > 288.04 \text{ kips} \therefore$ OK

FINAL DESIGN: Use 24” x 24” column with (12) #8 bars.

Concrete Moment Frame – East/West Direction

Beams

*Use rebar cover of 1.5(1.5”) = 2.25” due to corrosive environment (natatorium) (see ACI 7.7.6.1)

Shear and Moment (Unfactored) for Columns and Sloped Concrete Beams (E/W Direction)

	Beam 13/14	**West Column (C.L. 1.8)**	**East Column (C.L. 2) - Bottom**	**East Column (C.L. 2) - Top**
V_D (Top or Left)	-22.29	-4.08	-4.08	0.00
V_D (Bottom or Right)	28.65	-4.08	-4.08	0.00
V_L (Top or Left)	-6.89	-4.92	-4.92	0.00
V_L (Bottom or Right)	34.57	-4.92	-4.92	0.00
V_E (Top or Left)	11.43	36.62	6.00	8.40
V_E (Bottom or Right)	30.06	36.62	6.00	8.40
$V_{E,REVERSED}$ (Top or Left)	-11.43	-36.62	-6.00	-8.40
$V_{E,REVERSED}$ (Bottom or Right)	-30.06	-36.62	-6.00	-8.40
V_W (Top or Left)	7.26	23.01	3.37	5.68
V_W (Bottom or Right)	18.91	23.01	3.37	5.68
$V_{W,REVERSED}$ (Top or Left)	-7.26	-23.01	-3.37	-5.68
$V_{W,REVERSED}$ (Bottom or Right)	-18.91	-23.01	-3.37	-5.68
M_D (Top or Left)	-50.27	50.27	0.00	0.00
M_D (Bottom or Right)	-91.83	7.41	-91.83	0.00
M_L (Top or Left)	-60.64	60.64	0.00	0.00
M_L (Bottom or Right)	-110.79	8.94	-110.79	0.00
M_E (Top or Left)	136.74	-136.74	-8.88	0.00
M_E (Bottom or Right)	-155.88	247.73	126.21	147.00
$M_{E,REVERSED}$ (Top or Left)	-136.74	136.74	8.88	0.00
$M_{E,REVERSED}$ (Bottom or Right)	155.88	-247.73	-126.21	-147.00
M_W (Top or Left)	86.20	-86.20	0.16	0.00
M_W (Bottom or Right)	-99.16	155.61	75.97	99.31
$M_{W,REVERSED}$ (Top or Left)	-86.20	86.20	-0.16	0.00
$M_{W,REVERSED}$ (Bottom or Right)	99.16	-155.36	-75.97	-99.31
P_D	-21.35	-30.59	-28.65	0.00
P_L	-25.75	-36.90	-34.57	0.00
P_E	35.29	30.06	-30.06	0.00
$P_{E,REVERSED}$	-35.29	-30.06	30.06	0.00
P_W	22.11	18.91	-18.91	0.00
$P_{W,REVERSED}$	-22.11	-18.91	18.91	0.00
M_D (Midspan)	65.63	28.84	-45.92	0.00
M_L (Midspan)	79.19	34.79	-55.40	0.00
M_E (Midspan)	**20.49**	**55.49**	58.66	73.50
$M_{E,REVERSED}$ (Midspan)	**-20.49**	**-55.49**	-58.66	-73.50
M_W (Midspan)	**12.42**	**34.58**	38.06	49.66
$M_{W,REVERSED}$ (Midspan)	**-12.42**	**-34.58**	-38.06	-49.66

Table Accounts for Torsional Effects

1.2D +/- 1.0E + 1.0L				
Max $V_{TOP/LEFT}$ (kips)	-45.07	-46.44	-15.83	-8.40
Max $V_{BOTTOM/RIGHT}$ (kips)	99.01	26.79	-15.83	8.40
Max $M_{TOP/LEFT}$ (ft-kips)	-257.71	**257.71**	-8.88	0.00
Max $M_{BOTTOM/RIGHT}$ (ft-kips)	-376.87	**265.56**	**-347.20**	147.00
Max $M_{MIDSPAN}$ (ft-kips)	178.44	124.89	-169.16	73.50
Max P_u (kips)	-86.66	-103.67	-99.01	0.00

1.2D + 1.6L				
Max $V_{TOP/LEFT}$ (kips)	-37.77	-12.78	-12.78	0.00
Max $V_{BOTTOM/RIGHT}$ (kips)	89.70	-12.78	-12.78	0.00
Max $M_{TOP/LEFT}$ (ft-kips)	-157.35	**157.35**	0.00	0.00
Max $M_{BOTTOM/RIGHT}$ (ft-kips)	-287.46	**23.20**	-287.46	0.00
Max $M_{MIDSPAN}$ (ft-kips)	205.46	90.27	-143.73	0.00
Max P_u (kips)	-66.82	-95.75	-89.70	0.00

1.2D + 1.6W + 1.0L				
Max $V_{TOP/LEFT}$ (kips)	-45.24	-46.63	-15.21	-9.08
Max $V_{BOTTOM/RIGHT}$ (kips)	**99.20**	-46.63	-15.21	-9.08
Max $M_{TOP/LEFT}$ (ft-kips)	**-258.88**	**258.88**	-0.25	-158.90
Max $M_{BOTTOM/RIGHT}$ (ft-kips)	**-379.64**	**266.81**	-342.54	158.90
Max $M_{MIDSPAN}$ (ft-kips)	177.83	124.73	-171.39	79.45
Max P_u (kips)	-86.74	-103.86	-99.20	0.00

Table Accounts for Torsional Effects

BEAM DESIGN:

$V_{u,max}$ = 99.20 kips (1.2D + 1.6W + 1.0L)
$M_{u,max}$ at Supports = -379.64 k-ft (1.2D + 1.6W + 1.0L)
$M_{u,max}$ at Midspan = 205.46 k-ft (1.2D + 1.6L)

Use normal-weight concrete with f'_c = 4000 psi
f_y = 60,000 psi for flexural reinforcement
f_{yt} = 60,000 psi for stirrups

1) Choose the actual size of the beam stem.

a) Calculate the minimum depth based on deflections.

Use worst case scenario (use "simply supported" criteria).

ACI Table 9.5(a):

Minimum thickness, h = L/16 = [(23')(12 in/ft)]/16 = 17.25"

b) Determine the minimum depth based on the maximum negative moment.

$M_{u,max}$ at Supports = 379.64 k-ft

$\rho(\text{initial}) = [(\beta_1 f'_c)/(4f_y)] = [(0.85)(4\text{ ksi})/(4)(60\text{ ksi})] = 0.0142$

$\omega = \rho(f_y/f'_c) = (0.0142)(60\text{ ksi}/4\text{ ksi}) = 0.213$

$R = \omega f'c(1 - 0.59\omega) = (0.213)(4\text{ ksi})[1 - (0.59)(0.213)] = 0.745\text{ ksi}$

$bd^2 \geq M_u/\phi R = [(379.64\text{ ft-kips})(12\text{ in/ft})]/[(0.9)(0.745\text{ ksi})] = 6794.45\text{ in}^3$

Assuming b = 24 in.

$d \geq 16.83$ in.

h ≅ 16.83" + 3.25" = 20.08" (accounting for 2.25" clear cover due to corrosive environment; see ACI 7.7.6.1; (1.5)(1.5") = 2.25")

Try h = 26" > 20.76" ∴ Meets deflection criteria

d ≅ 26" – 3.25" = 22.75"

c) Check the shear capacity of the beam.

$V_u = \phi(V_c + V_s)$

$V_{u,max}$ = 99.20 kips

From ACI Code Section 11.2.1.1, the nominal V_c is

$$V_c = 2\lambda\sqrt{f'_c}b_w d = (2)(1.0)\sqrt{4000\text{ psi}}(24")(22.75")/1000 = 69.06\text{ kips}$$

ACI Code Section 11.4.7.9 sets the maximum nominal V_s as

$$V_s = 8\sqrt{f'_c}b_w d = (8)\sqrt{4000\text{ psi}}(24")(22.75")/1000 = 276.26\text{ kips}$$

Thus, the absolute maximum $\phi V_n = 0.75(69.06\text{ k} + 276.26\text{ k}) = 258.99$ kips

$\geq V_{u,max} = 99.20$ kips $\therefore$ OK

d) Summary. Use:

b = 24”
h = 26”
d = 22.75”

2) Compute the dead load of the stem, and recompute the total moment.

Weight of 24”x26” concrete beam = $[(24")(26")/144\text{ in}^2/\text{ft}^2][(150\text{ lb/ft}^3)/1000]$

= 0.650 k/ft

Original dead load = 2.6524 k/ft

New dead load = 2.6524 k/ft + (0.650 k/ft – 0.375 k/ft) = 2.9274 k/ft

(2.9274 k/ft)/(2.6524 k/ft) = 1.1037

New $M_{u,max}$ at Supports $\cong$ (1.2)(-91.83 k-ft*1.1037) + (1.6)(-99.16 k-ft) – 100.79 =

= 381.07 k-ft

New $M_{u,max}$ at Midspan $\cong$ (1.2)(65.63 k-ft*1.1037) + (1.6)(79.19 k-ft) = 213.63 k-ft

New $V_{u,max} \cong$ (1.2)(28.65 k*1.1037) + (1.6)(18.91 k) + 34.57 k = 102.77 k

$< \phi V_n = 258.99$ kips $\therefore$ Shear capacity is still OK.

3) Design the flexural reinforcement.

a) Compute the area of steel required at the point of maximum negative moment.

$$A_s \geq M_u/[\phi f_y(d - a/2)] \cong M_u/[\phi f_y(jd)]$$

Because there is negative moment at the support, the beams acts as a rectangular beam with compression in the web. Assume that j = 0.9 and $\phi = 0.90$

$A_s \cong$ (381.07 k-ft)(12 in/ft)/[(0.9)(60 ksi)(0.9)(22.75")] = 4.14 in.^{2}

This value can be improved with one iteration to find the depth of the compression stress block, a:

$a = A_s f_y / 0.85 f'_c b$ = (4.14 in^2)(60 ksi)/[(0.85)(4 ksi)(24")] = 3.041"

and then recalculating the required A_s with this calculated value of a:

$A_s \geq M_u/[\phi f_y(d - a/2)]$ = (381.07 k-ft)(12 in/ft)/[(0.9)(60 ksi)(22.75" – 3.041"/2)]

= 3.99 in^2

Before proceeding, it must be confirmed that this is a tension-controlled section. This can be done by showing that the neutral axis, c, is less than 3/8 of d.

$a = A_s f_y / 0.85 f'_c b$ = (3.99 in^2)(60 ksi)/[(0.85)(4 ksi)(24")] = 2.933"

$c = a/\beta_1$ = 2.933"/0.85 = 3.451" < (3/8)(d) = (3/8)(22.75") = 8.531"

$\therefore$ Section is tension-controlled and can be designed using $\phi = 0.90$

b) Compute the area of steel required at the point of maximum positive moment.

$A_s \geq M_u/[\phi f_y(d - a/2)] \cong M_u/[\phi f_y(jd)]$

Assume that the compression zone is rectangular, and take j = 0.95 for the first calculation of A_s.

$A_s \cong$ (213.63 k-ft)(12 in/ft)/[(0.9)(60 ksi)(0.95)(22.75")] = 2.20 in.^{2}

This value can be improved with one iteration to find the depth of the compression stress block, a:

$a = A_s f_y / 0.85 f'_c b$ = (2.20 in^2)(60 ksi)/[(0.85)(4 ksi)(24")] = 1.618"

and then recalculating the required A_s with this calculated value of a:

$A_s \geq M_u/[\phi f_y(d - a/2)]$ = (213.63 k-ft)(12 in/ft)/[(0.9)(60 ksi)(22.75" – 1.618"/2)]

= 2.16 in^2

Before proceeding, it must be confirmed that this is a tension-controlled section. This can be done by showing that the neutral axis, c, is less than 3/8 of d.

$a = A_s f_y / 0.85 f'_c b$ = (2.16 in^2)(60 ksi)/[(0.85)(4 ksi)(24")] = 1.591"

$c = a/\beta_1$ = 1.591"/0.85 = 1.872" < (3/8)(d) = (3/8)(22.75") = 8.531"

$\therefore$ Section is tension-controlled and can be designed using $\phi = 0.90$

c) Calculate the minimum reinforcement (using ACI Code Section 10.5.1).

$A_{s,\,min}$ = max. of:

$[3\sqrt{f'_c}/f_y]b_w d = [3\sqrt{4000 \text{ psi}}/60000 \text{ psi}](24")(22.75") = 1.73 \text{ in}^2$

$200b_w d/f_y = (200)(24")(22.75")/60000 \text{ psi} = 1.82 \text{ in}^2$

$\therefore A_{s,min} = 1.82 \text{ in}^2$

4) Calculate the area of steel and select the bars.

a) Negative-moment Region

$A_{s,req} = 3.99 \text{ in}^2 > A_{s,min} = 1.82 \text{ in}^2 \therefore$ OK

Use (7) #7 bars [$A_s = (7)(0.60 \text{ in}^2) = 4.20 \text{ in}^2 > 3.99 \text{ in}^2 \therefore$ OK]

$a = A_s f_y/0.85f'_c b = (4.20 \text{ in}^2)(60 \text{ ksi})/[(0.85)(4 \text{ ksi})(24")] = 3.088"$

$a = \beta_1 c$ = where $\beta = 0.85$ for $f'_c = 4{,}000$ psi

$c = a/\beta 1 = 3.088"/0.85 = 3.633"$

$d_{actual} = 26" - 2.25" - 0.5" - (1/2)(0.875") = 22.8125$

$\varepsilon_s = (d-c)(\varepsilon_u)/c = (22.8125" - 3.633")(0.003)/3.633" = 0.01584 > \varepsilon_y = 0.00207$

$\varepsilon_t \cong \varepsilon_s = 0.01584 > 0.005 \therefore$ Tension-controlled Section $\therefore \phi = 0.9$

$\phi M_n = \phi A_s f_y(d - a/2) = (0.9)(4.20 \text{ in}^2)(60 \text{ ksi})(22.8125" - 3.088"/2)/(12 \text{ in/ft}) =$

$= 401.97 \text{ k-ft} > 381.07 \text{ k-ft} \therefore$ OK

Small bars were selected at the supports because the bars have to be hooked into the exterior supports and there may not be enough room for a standard hook on larger bars.

b) Positive-moment Region

$A_{s,req} = 2.16 \text{ in}^2 > A_{s,min} = 1.82 \text{ in}^2 \therefore$ OK

Use (4) #7 bars [$A_s = (4)(0.60 \text{ in}^2) = 2.40 \text{ in}^2 > 2.16 \text{ in}^2 \therefore$ OK]

$a = A_s f_y/0.85f'_c b = (2.40 \text{ in}^2)(60 \text{ ksi})/[(0.85)(4 \text{ ksi})(24")] = 1.765"$

$a = \beta_1 c$ = where $\beta = 0.85$ for $f'_c = 4{,}000$ psi

$c = a/\beta 1 = 1.765''/0.85 = 2.076''$

$\varepsilon_s \cong (d\text{-}c)(\varepsilon_u)/c = (22.8125'' - 2.076'')(0.003)/2.076'' = 0.02997 > \varepsilon_y = 0.00207$

$\varepsilon_t \cong \varepsilon_s = 0.02997 > 0.005 \therefore$ Tension-controlled Section $\therefore \phi = 0.9$

$\phi M_n = \phi A_s f_y (d - a/2) = (0.9)(2.40 \text{ in}^2)(60 \text{ ksi})(22.8125'' - 1.765''/2)/(12 \text{ in/ft}) =$

$= 236.84$ k-ft > 213.63 k-ft $\therefore$ OK

5) Check the distribution of the reinforcement (spacing requirements).

a) Negative-moment Region

$c_c = 2.25$ in. cover $+ 0.5$ in. stirrups $= 2.75''$

The maximum bar spacing is

$s = 15(40{,}000/f_s) - 2.5c_c$

$f_s = (2/3)(f_y) = (2/3)(60{,}000 \text{ ksi}) = 40{,}000$ ksi

$s = 15(40{,}000/40{,}000) - (2.5)(2.75'') = 8.125''$

Spacing of bars is less than 8.125" by inspection.

Minimum bar spacing:

s_c = max of [1", d_b, $(4/3)s_a$]; Assume s_a = 1" aggregate

s_c = max of [1", 0.875", (4/3)(1") = 1.333"]; Assume s_a = 1" aggregate

$s_c = 1.333''$

Side spacing and cover:

$b > (n)(d_b) + (n\text{-}1)(s_c) + 2d_{tr} + 2c_c$

$24'' > (7)(0.875'') + (7\text{-}1)(1.333'') + (2)(0.5'') + (2)(2.25'')$

$24'' > 19.62'' \therefore$ OK

b) Positive-moment Region

The maximum bar spacing is 8.125". Spacing of bars is less than 8.125" by inspection.

Minimum bar spacing = 1.333”

Side spacing and cover:

$b > (n)(d_b) + (n\text{-}1)(s_c) + 2d_{tr} + 2c_c$

24” > (4)(0.875”) + (4-1)(1.333”) + (2)(0.5”) + (2)(2.25”)

24” > 14.00” ∴ OK

6) Design the shear reinforcement.

a) The critical section for shear is located at the support. ACI Code Section 11.4.6.1 requires stirrups if $V_u \geq \phi V_c/2$

$V_c = 2\lambda\sqrt{f'_c}b_wd$ = (2)(1.0) √4000 psi (24”)(22.8175”)/1000 = 69.27 kips

$V_c/2$ = 69.27 kips/2 = 34.63 kips

V_u/ϕ = (102.77 kips)/(0.75) = 137.03 kips > $V_c/2$ = 34.63 kips

∴ Stirrups are required.

b) Determine shear strength required by shear reinforcing.

$V_s = V_u/\phi - V_c$ = [(102.77 kips)/(0.75)] – 69.27 kips = 67.76 kips

$V_s \leq 8\sqrt{f'_c}b_wd$ = 8√4000 psi (24”)(22.8125”)/1000 = 277.02 kips ∴ OK

c) Determine maximum spacing of shear reinforcing (ACI 318-08 Sections 11.4.5.1 and 11.4.5.3).

For $V_s \leq 8\sqrt{f'_c}b_wd$: s_{max} = min of {d/2, 24”}

d/2 = 22.8125”/2 = 11.41”

s_{max} = 11”

d) Determine minimum shear reinforcement (ACI 318-08 Section 11.4.6.3).

$A_{v,min}$ = max of $\{0.75\sqrt{f'_c}b_ws/f_{yt}, 50b_ws/f_{yt}\}$

$0.75\sqrt{f'_c}b_ws/f_{yt}$ = 0.75√4000 psi (24”)(11”)/60,000 psi = 0.209 in^2

$50b_ws/f_{yt}$ = 50(24”)(11”)/60,000 psi = 0.220 in^2

∴ $A_{v,min}$ = 0.220 in^2

Use #3 stirrups @ 11” as minimum shear reinforcement.

(A_v = 2 legs x 0.11 in^2/leg = 0.22 $in^2 \geq 0.220$ in^2 $\therefore$ OK)

e) Design the shear reinforcement.

$V_s = A_v f_{yt} d/s$

Rearranging: $s = A_v f_{yt} d/V_s$ = (0.22 in^2)(60 ksi)(22.8125")/67.76 kips = 4.44"

Usually absolute minimum "s" is 4".

Use (2) #3 stirrups @ 4", starting 2" from face of support.

Or use #4 stirrups instead of #3 stirrups.

For #4 stirrups: (A_v = 2 legs x 0.20 in^2/leg = 0.40 $in^2 > 0.200$ in^2 $\therefore$ OK)

$s = A_v f_{yt} d/V_s$ = (0.40 in^2)(60 ksi)(22.8125")/67.76 kips = 8.08"

Use (2) #4 stirrups @ 8", starting 2" from face of support.

Use this stirrup layout throughout the entire length of the beam since lateral loads can change the shear forces (shear diagram) throughout the beam length (since the beam is part of a concrete moment frame).

FINAL DESIGN: Use 24" x 26" beam with (7) #7 bars in a single layer for negative moment reinforcement (at the supports) and (4) #7 bars for positive moment reinforcement. Use (2) #4 stirrups @ 8" throughout length of beam.

COLUMN DESIGN:

Columns at Column Line 1.8:

These columns were already designed for gravity forces and lateral forces in the North/South direction. The design resulted in 24"x24" concrete columns with (12) #8 bars.

Check this column size and reinforcement for gravity loads and lateral loads in the East/West direction. The total P_u will be the same (may vary depending on load cases), but the moments (M_1 and M_2) at the top and bottom of the column will change. The P_u used for the North/South design already been calculated and that value for P_u will thus be used for this column check.

Controlling Load Case: 1.2D + 1.6L

P_u = 177.98 kips (same as the design for the North/South direction)

M_2 = 266.81 k-ft

M_1 = 258.88 k-ft

1) Preliminary column size

$A_{g(trial)} \geq P_u/[0.40(f'_c + f_y\rho_g)$

$A_{g(trial)} \geq$ 177.98 kips/[0.40(4 ksi + (60 ksi)(0.015))] = 90.81 in^2

$\cong$ (9.53 in.)^2

Try 24"x24" column (already designed for North/South direction)

2) Is the story being designed sway or nonsway?

$Q = [\sum P_u \times \Delta_o]/[V_{us} \times l_c]$

$\sum P_u \cong$ (5)(177.98 k) = 889.90 k

V_{us} = 1 kip

Δ_o = 0.014789"

l_c = 10.5' = 126"

Q = [(889.90 kips)(0.014789")]/[(1 kip)(126")] = 0.01045 < 0.05

$\therefore$ Nonsway (but assume sway story because $\sum P_u$ will actually be higher due to loads at other columns around the building at that level)

3) Are the columns slender?

$r = 0.3h = (0.3)(24") = 7.2"$
$kl_u/r = (1.2)(126")/7.2" = 21 < 22$ (for a sway frame) ∴ Column is not slender

2) Compute γ

For a 24"x24" column: $\gamma = [24" - (2)(2.5")]/24" = 0.7917$

3) Use interaction diagrams to determine ρ_g

$\phi P_n/A_g = P_u/A_g = 177.98 \text{ k}/[(24")(24")] = 0.3099$

$\phi M_n/A_g h = M_u/A_g h = (379.64 \text{ k-ft})(12 \text{ in/ft})/[(24"\text{x}24")(24")] = 0.3295$

From Fig. A-9b (from "Reinforced Concrete Mechanics and Design" by White and MacGregor):

$\rho_g = 0.010 < 0.016$ (provided) ∴ OK

$\rho_{g,provided} = (12)(0.79 \text{ in}^2)/[(24")(24")] = 0.016$

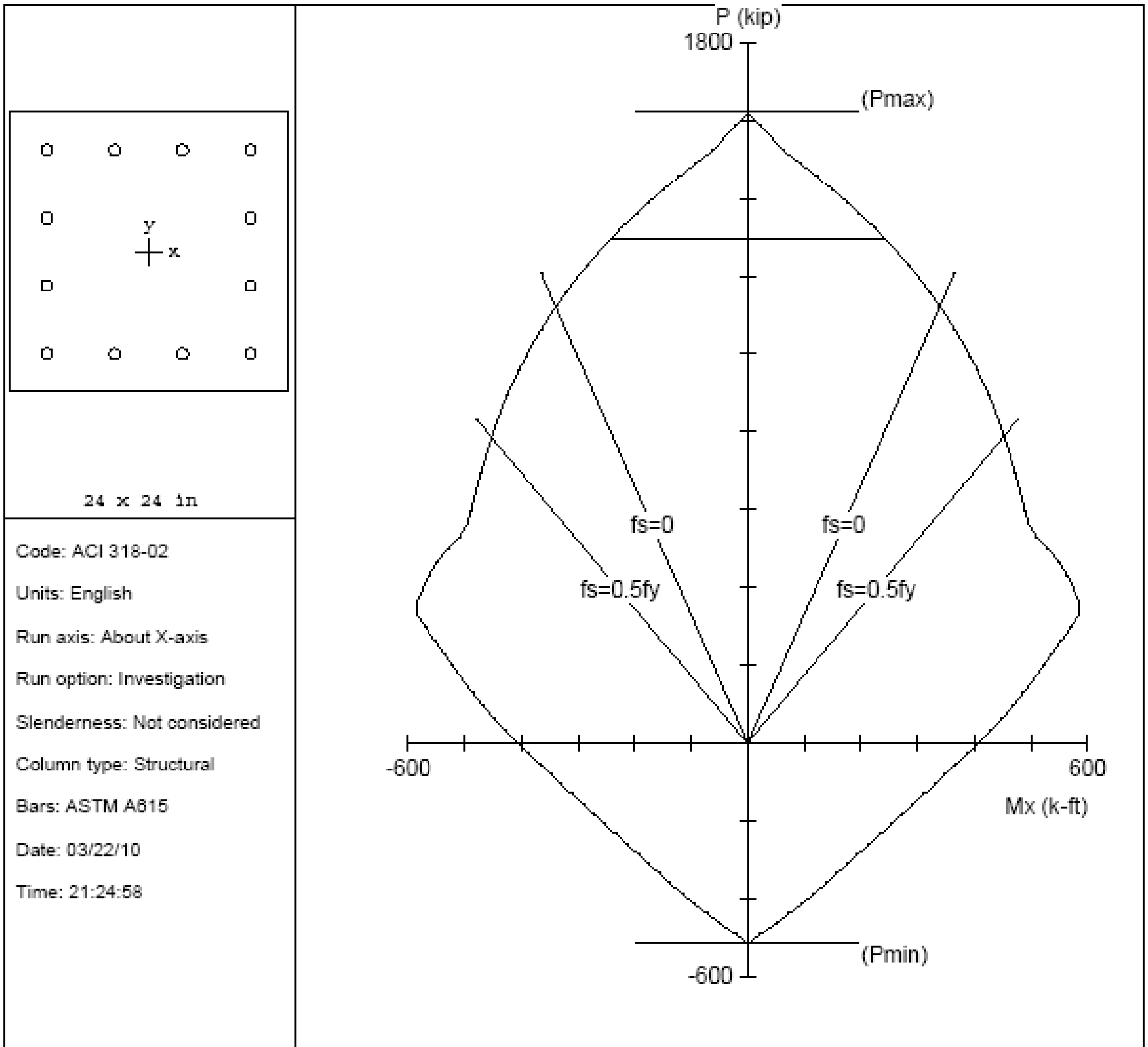
The 24"x24"column with (12) #8 bars is OK

PCA Column was also used to check the 24"x24" column with (12) #8 bars

(Pu,Mu) = (177.98 k, 266.81 k-ft)

This point lies within the boundaries on the interaction diagram from PCA column (see diagram below).

∴ Column is OK



pcaColumn v3.64. Licensed to: Penn State University. License ID: 52411-1010265-4-22545-28F4D

File: F:\AE 897G\3-22-10\18x18 with (8) #8 bars.col

Project: Kukorlo - Thesis - 18x18

Column:		Engineer:	
f'c = 4 ksi	fy = 60 ksi	Ag = 576 in^2	12 #8 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 9.48 in^2	Rho = 1.65%
fc = 3.4 ksi	fc = 3.4 ksi	Xo = 0.00 in	Ix = 27648 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 27648 in^4
Beta1 = 0.85		Clear spacing = 4.83 in	Clear cover = 2.75 in
Confinement: Tied	phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65		

Wood Braced Frame – East/West Direction

Design of Diagonal Members:

P_u = 13.72 k (compression)

Analyze Member Buckling About x Axis:

$(l_e/d)_{max} = 50$

$(l_e/d)_x = [(1.0)(26.2552')(12 \text{ in/ft})]/d \leq 50$

$d \geq l_e/50 = [(26.2552')(12 \text{ in/ft})]/50 = 6.30"$

Analyze Member Bucking About y Axis:

$(l_e/d)_{max} = 50$

$(l_e/d)_y = [(1.0)(13.1276')(12 \text{ in/ft})]/d \leq 50$

$d \geq l_e/50 = [(13.1276')(12 \text{ in/ft})]/50 = 3.15"$

Try 5”” x 6 7/8”

$(l_e/d)_x = [(26.2662')(12 \text{ in/ft})]/6.875" = 45.846$

$(l_e/d)_y = [(13.1276')(12 \text{ in/ft})]/5" = 31.5062$

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

E_{min} = 980,000 psi

C_D = 1.6 (for wind load))

C_M = 0.73 for F_c (p. 64, NDS Supplement)

C_M = 0.833 for E and E_{min} (p. 64, NDS Supplement)

$C_t = 1.0$

$E'_{min} = (E_{min})(C_M)(C_t) = (980{,}000)(0.833)(1.0) = 816{,}340$ psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340 \text{ psi})]/[(45.846)^2] = 319.257$ psi

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.6)(0.73)(1.0) = 2686.4$ psi

$F_{cE}/F_c^* = 319.257/2686.4 = 0.1188$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.1188]/[(2)(0.9)] = 0.6216$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.6216\} - \sqrt{\{[0.6216]^2 - [0.1188/0.9]\}}$

$= 0.1173$

$F'_c = F_c^*(C_P) = (2686.4 \text{ psi})(0.1173) = 315.004 \text{ psi}$

$P = (F'_c)(A)$

$A_{req'd} = P/F'_c = 13{,}720 \text{ lb}/315.004 \text{ psi} = 43.56 \text{ in}^2 > A_{provided} = 34.38 \text{ in}^2$ ∴ **N.G.**

Try 6 ¾” x 6 7/8”

$(l_e/d)_x = [(26.2662')(12 \text{ in/ft})]/6.875" = 45.846$

$(l_e/d)_y = [(13.1276')(12 \text{ in/ft})]/6.75" = 23.338$

Same C_P and Areq’d

$A_{req'd} = P/F'_c = 13{,}720 \text{ lb}/315.004 \text{ psi} = 43.56 \text{ in}^2 < A_{provided} = 46.41 \text{ in}^2$ ∴ **OK**

Use 6 ¾” x 6 7/8” Southern Pine glulam ID #50

Wind Columns

Try truss design with 3'-0" depth:

LOAD COMBINATION: D+W (Combined Bending and Axial Forces) (Controls)

"Top Chord"

P_{max} = 22.238 k + (30 psf/53.1 psf)(5.5522 k) = 25.375 k (Compression)

M_{max} = 4.1695 ft-k = 4169.5 ft-lb = 50,034 in-lb

Try 6 ¾" x 11"

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

F_b = 2100 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

A = 74.25 in^2

S = 136.1 in^3

E_{min} = 980,000 psi

Axial Load: P_{max} = 25,375 lb (Compression)

Maximum Moment: M_{max} = 50,034 in-lb

L = 6.667'

Axial Load:

f_c = P/A = 25,375 lb/74.25 in^2 = 341.751 psi

$(l_e/d)_x$ = [(6.667')(12 in/ft)]/11" = 7.2727 < 50 ∴ OK

$(l_e/d)_y$ = [(13.333')(12 in/ft)]/6.75" = 23.7037 < 50 ∴ OK

$(l_e/d)_{max} = (l_e/d)_x$ = 23.7037

The larger slenderness ratio governs the adjusted design value. Therefore, the weak axis of the member is critical, and $(l_e/d)_y$ is used to determine F'_c.

C_D = 1.6 (for wind load; load combination D+W)

C_M = 0.73 for F_c (p. 64, NDS Supplement)

C_M = 0.833 for E and E_{min} (p. 64, NDS Supplement)

C_M = 0.8 for F_b (p. 64, NDS Supplement)
C_t = 1.0

$E'_{min} = (E_{min})(C_M)(C_t) = (980{,}000)(0.833)(1.0) = 816{,}340$ psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340 \text{ psi})]/[(27.7037)^2] = 874.314$ psi

Here, l_e/d is based on $(l_e/d)_{max}$.

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.6)(0.73)(1.0) = 2686.4$ psi

$F_{cE}/F_c^* = 874.314/2686.4 = 0.3255$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.3255]/[(2)(0.9)] = 0.7364$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.7364\} - \sqrt{\{[0.7364]^2 - [0.3255/0.9]\}}$

$= 0.3115$

$F'_c = F_c^*(C_P) = (2686.4 \text{ psi})(0.3115) = 836.723$ psi

Axial stress ratio = f_c/F'_c = (341.751 psi)/(836.723 psi) = 0.4084

Net Section Check:

Assume connections will be made with (2) rows of ¾" diameter bolts.

Assume the hole diameter is 1/16" larger than the bolt (for stress calculations only).

A_n = (6.75")[11" – (2)(0.8125")] = 63.28 in^2

(3/4" + 1/16" = 0.8125")

$f_c = P/A_n$ = 25,375 lb/63.28 in^2 = 400.988 psi

$F'_c = F_c^* = F_c(C_D)(C_M)(C_t)(C_P) = (2300 \text{ psi})(1.6)(0.73)(1.0)(0.3115) = 836.814$ psi

836.814 psi > 400.988 psi ∴ OK

Bending:

Bending is about the strong axis of the cross section. The adjusted bending design value for a glulam is governed by the smaller of two criteria: volume effect or lateral stability.

M = 50,034 in-lb

$S = 136.1\ in^3$

$f_b = M/S = 50{,}034$ in-lb/136.1 in^3 = 367.627 psi

$F'_b = F_b(C_D)(C_M)(C_t)(C_L)$ or

$F'_b = F_b(C_D)(C_M)(C_t)(C_V)$

For C_L: l_u/d = [(13.333')(12 in/ft)]/11" = 14.545 > 7

$\therefore$ $l_e = 1.63l_u + 3d$ = (1.63)[(13.333')(12 in/ft)] + (3)(11") = 293.799"

$R_B = \sqrt{l_e d/b^2} = \sqrt{[(293.799")(11")/(6.75")^2]} = 8.422$

$F_{bE} = 1.20E'_{min}/R_B^2$ = [(1.20)(816,340 psi)]/$(8.422)^2$ = 13,810.721 psi

$F^*_b = F_b(C_D)(C_M)(C_t)$ = (2100 psi)(1.6)(0.8)(1.0) = 2688 psi

F_{bE}/F^*_b = (13810.721)/(2688) = 5.1379

$(1 + F_{bE}/F^*_b)/1.9$ = (1 + 5.1379)/1.9 = 3.2305

$C_L = [(1 + F_{bE}/F^*_b)/1.9] - \sqrt{\{[(1 + F_{bE}/F^*_b)/1.9]^2 - [F_{bE}/F^*_b/0.95]\}}$

$= 3.2305 - \sqrt{(3.2305)^2 - (5.1379/0.95)]} = 0.9882$

For Southern Pine glulam:

$C_V = (21'/L)^{1/20}(12"/d)^{1/20}(5.125"/b)^{1/20} \leq 1.0$

$C_V = (21'/60')^{1/20}(12"/11")^{1/20}(5.125"/6.75")^{1/20} \leq 1.0$

$C_V = 0.9400 \leq 1.0$

C_V governs of C_L

$F'_b = F^*_b(C_V)$ = (2688 psi)(0.9400) = 2526.72 psi

Bending stress ratio = f_b/F'_b = (367.627 psi)/(2526.72 psi) = 0.1455

Combined Stresses:

The bending moment is about the strong axis of the cross section, and the amplification for P-Δ is measured by the column slenderness ratio about the x axis.

$(l_e/d)_{\text{bending moment}} = (l_e/d)_x = 7.2727$

$F_{cEx} = [0.822E'_{min}]/[(l_e/d)_x]^2$ = [(0.822)(816,340 psi)]/[$(7.2727)^2$] = 12686.784 psi

*Here, (l_e/d) is based on the axis about which the bending moment occurs.

Amplification factor = $1/[1 – (f_c/F_{cEx})] = 1/[1 – (341.751 \text{ psi}/12686.784 \text{ psi})] = 1.0277$

$(f_c/F'_c)^2 + \{1/[1 – (f_c/F_{cEx})]\}(f_b/F'_b) = (0.4084)^2 + (1.0277)(0.1455) = 0.3163 < 1.0 \therefore$ **OK**

Try 6 ¾” x 6 7/8”

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

F_b = 2100 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

$A = 46.41 \text{ in}^2$

$S = 53.17 \text{ in}^3$

E_{min} = 980,000 psi

Axial Load: P_{max} = 25,375 lb (Compression)

Maximum Moment: M_{max} = 50,034 in-lb

L = 6.667’

Axial Load:

$f_c = P/A = 25{,}375 \text{ lb}/46.41 \text{ in}^2 = 546.757 \text{ psi}$

$(l_e/d)_x = [(6.667')(12 \text{ in/ft})]/6.875" = 11.6364 < 50 \therefore$ OK

$(l_e/d)_y = [(13.333')(12 \text{ in/ft})]/6.75" = 23.7037 < 50 \therefore$ OK

$(l_e/d)_{max} = (l_e/d)_x = 23.7037$

The larger slenderness ratio governs the adjusted design value. Therefore, the weak axis of the member is critical, and $(l_e/d)_y$ is used to determine F'_c.

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340 \text{ psi})]/[(27.7037)^2] = 874.314 \text{ psi}$

Here, l_e/d is based on $(l_e/d)_{max}$.

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.6)(0.73)(1.0) = 2686.4 \text{ psi}$

$F_{cE}/F_c^* = 874.314/2686.4 = 0.3255$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.3255]/[(2)(0.9)] = 0.7364$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 – [F_{cE}/F_c^*]/c\}}$

$= \{0.7364\} - \sqrt{\{[0.7364]^2 – [0.3255/0.9]\}}$

= 0.3115

$F'_c = F_c^*(C_P)$ = (2686.4 psi)(0.3115) = 836.723 psi

Axial stress ratio = f_c/F'_c = (546.757 psi)/(836.723 psi) = 0.6535

Net Section Check:

Assume connections will be made with (2) rows of ¾" diameter bolts.

Assume the hole diameter is 1/16" larger than the bolt (for stress calculations only).

A_n = (6.75")[6.875" – (2)(0.8125")] = 35.44 in^2

(3/4" + 1/16" = 0.8125")

$f_c = P/A_n$ = 25,375 lb/35.44 in^2 = 715.999 psi

$F'_c = F_c^* = F_c(C_D)(C_M)(C_t)(C_P)$ = (2300 psi)(1.6)(0.73)(1.0)(0.3115) = 836.814 psi

836.814 psi > 715.999 psi ∴ OK

Bending:

Bending is about the strong axis of the cross section. The adjusted bending design value for a glulam is governed by the smaller of two criteria: volume effect or lateral stability.

M = 50,034 in-lb

S = 53.17 in^3

$f_b = M/S$ = 50,034 in-lb/53.17 in^3 = 941.019 psi

$F'_b = F_b(C_D)(C_M)(C_t)(C_L)$ or

$F'_b = F_b(C_D)(C_M)(C_t)(C_V)$

For C_L: l_u/d = [(13.333')(12 in/ft)]/6.875" = 23.272 > 7

∴ $l_e = 1.63l_u + 3d$ = (1.63)[(13.333')(12 in/ft)] + (3)(6.875") = 281.425"

$R_B = \sqrt{l_e d/b^2} = \sqrt{[(281.425")(6.875")/(6.75")^2]}$ = 6.516

$F_{bE} = 1.20E'_{min}/R_B^2$ = [(1.20)(816,340 psi)]/$(6.516)^2$ = 23,068.884 psi

$F^*_b = F_b(C_D)(C_M)(C_t)$ = (2100 psi)(1.6)(0.8)(1.0) = 2688 psi

F_{bE}/F^*_b = (23068.884)/(2688) = 8.5821

$(1 + F_{bE}/F^*_b)/1.9 = (1 + 8.5821)/1.9 = 5.0432$

$C_L = [(1 + F_{bE}/F^*_b)/1.9] - \sqrt{\{[(1 + F_{bE}/F^*_b)/1.9]^2 - [F_{bE}/F^*_b/0.95]\}}$

$= 5.0432 - \sqrt{(5.0432)^2 - (8.5821/0.95)]} = 0.9935$

For Southern Pine glulam:

$C_V = (21'/L)^{1/20}(12''/d)^{1/20}(5.125''/b)^{1/20} \leq 1.0$

$C_V = (21'/60')^{1/20}(12''/6.875'')^{1/20}(5.125''/6.75'')^{1/20} \leq 1.0$

$C_V = 0.9623 \leq 1.0$

C_V governs of C_L

$F'_b = F^*_b(C_V) = (2688 \text{ psi})(0.9623) = 2586.662 \text{ psi}$

Bending stress ratio = $f_b/F'_b = (941.019 \text{ psi})/(2586.662 \text{ psi}) = 0.3638$

Combined Stresses:

The bending moment is about the strong axis of the cross section, and the amplification for P-Δ is measured by the column slenderness ratio about the x axis.

$(l_e/d)_{\text{bending moment}} = (l_e/d)_x = 11.6364$

$F_{cEx} = [0.822E'_{min}]/[(l_e/d)_x]^2 = [(0.822)(816{,}340 \text{ psi})]/[(11.6364)^2] = 4955.707 \text{ psi}$

*Here, (l_e/d) is based on the axis about which the bending moment occurs.

Amplification factor = $1/[1 - (f_c/F_{cEx})] = 1/[1 - (546.757 \text{ psi}/4955.707 \text{ psi})] = 1.1240$

$(f_c/F'_c)^2 + \{1/[1 - (f_c/F_{cEx})]\}(f_b/F'_b) = (0.6535)^2 + (1.1240)(0.3638) = 0.8360 < 1.0$ ∴ **OK**

FINAL MEMBER SIZE = 6 ¾” x 6 7/8” Southern Pine Glulam ID #50

Overturning Check

Wood Braced Frame at Column Line 1:

Look at load combination: 0.9D + 1.6W (controlling load combination)

Tributary area for each frame = (8')(130'/2) = 520 SF

Wind uplift = 16.28 PSF

Upward/overturning force due to 1.6W (applied lateral force)

= 36.71 k (from SAP model)

Upward/overturning force due to wind uplift = (1.6)(16.28 PSF)(520 SF)/1000 =

= 13.54 k

Total upward force at base = 36.71 k + 13.54 k = 50.25 k

Resistance is provided by applied dead load plus dead load of concrete footing and concrete pier.

Dead load applied to column = 21.34 k (from SAP model)

Footing: [(19')(19')(2')](150 PCF)/1000 = 108.3 k

Pier: [(9.667')(8.333')(10')](150 PCF)/1000 = 106.3 k

These footing and pier sizes are from the original building, which had columns spaced at 30'-0" o.c. at column line 1. Since the design with the wood trusses has columns spaced at 8' o.c., it will be assumed that the dead load of the footing and pier will be about one-quarter of that from the original design.

Footing ≅ (1/4)(108.3 k) = 27.035 k

Pier ≅ (1/4)(106.3 k) = 26.575 k

Total resistance due to dead load = (0.9)(21.34 k + 27.035 k + 26.575 k) = 67.46 k

67.46 k > 50.25 k ∴ OK

The dead weight of the roof load plus the estimated self weight of the concrete footings and piers at this location was able to resist the upward forces caused by the overturning moments due to the wind loads. However, since the weight of the footings and piers is only an estimate, overturning will need to be investigated more closely using the final concrete footing and piers sizes. The applied live roof load was conservatively omitted from this check and would help resist overturning as well.

Concrete Moment Frame at Column Line 2 (North/South Direction):

Look at load combination: 0.9D + 1.6W

Tributary area for each frame = (32')(130'/2) = 2080 SF

Wind uplift = 16.28 PSF

Upward/overturning force due to 1.6W (applied lateral force)

= (1.6)(11.43 k) = 18.29 k (from SAP model)

Upward/overturning force due to wind uplift = (1.6)(16.28 PSF)(2080 SF)/1000 =

= 54.18 k

Total upward force at base = 18.29 k + 54.18 k = 72.47 k

Resistance is provided by applied dead load plus dead load of concrete column, concrete footing, and concrete pier.

Dead load applied to column = 130.28 k (from SAP model)

Resistance due to dead load = (0.9)(130.28 k) = 117.25 k

117.25 k > 72.47 k ∴ OK

The dead weight applied to the exterior column of the concrete moment frame at column line 2 was able to resist the overturning forces by itself. Therefore, there was no need to consider the self weight of the concrete column, concrete footing, and pier, which also help to resist the overturning moment. Hence, overturning is not a concern at the moment frame at column line 2.

Concrete Moment Frame in East/West Direction:

Look at load combination: 0.9D + 1.6W (controlling load combination)

Tributary area for each frame = (32')(130'/2) = 2080 SF

Wind uplift = 16.28 PSF

Upward/overturning force due to 1.6W (applied lateral force)

= (1.6)(18.91 k) = 30.26 (from SAP model)

Upward/overturning force due to wind uplift = (1.6)(16.28 PSF)(2080 SF)/1000 =

= 54.18 k

Total upward force at base = 30.26 k + 54.18 k = 84.44 k

Resistance is provided by applied dead load plus the self weight of the concrete footing and the concrete column.

Dead load applied to column = 30.59 k (from SAP model)

Footing: [(13.5')(13.5')(2.75')](150 PCF)/1000 = 75.18 k

Total resistance due to dead load = (0.9)(30.59 k + 75.18 k) = 95.19 k

95.19 k > 84.44 k ∴ OK

The applied dead load and self weight of the concrete footing can resist the overturning moment due to wind. The self weight of the column was conservatively not considered, but would assist in resisting overturning as well.

<u>Wood Braced Frame in East/West Direction:</u>

Look at load combination: 0.9D + 1.6W (controlling load combination)

Tributary area for each frame = (26')(9.125') = 237.25 SF

Wind uplift = 16.28 PSF

Upward/overturning force due to 1.6W (applied lateral force)

= (1.6)(17.55 k) = 28.08 k (from SAP model)

Upward/overturning force due to wind uplift = (1.6)(16.28 PSF)(237.25 SF)/1000 =

= 6.18 k

Total upward force at base = 28.08 k + 6.18 k = 34.26 k

Resistance is provided by applied dead load plus the self weight of the concrete footing.

Dead load applied to column = 5.10 k

Footing: [(5')(5')(1')](150 PCF)/1000 = 3.75 k

Total resistance due to dead load = (0.9)(5.10 k + 3.75 k) = 8.00 k

8.00 k < 34.26 k ∴ N.G.

The applied dead load and self weight of the concrete footing cannot resist the overturning moment due to wind. Therefore, connections at the base of the column need to be investigated further (connections must be able to resist the uplift forces and hence prevent overturning).

Foundation Check

Concrete Moment Frame – Column Line 2

P_D = 190.87 k

P_{Lr} = 113.03 k

P_W = 1.55 k

P_u = 411.13 k (1.2D + 1.6L_r + 0.8W) + Weight of Concrete Column

[(24”)(24”)]/(144 in^2/ft^2) = 4 SF

(4 SF)(40’) = 160 ft^3

Weight of Concrete Column = (160 ft^3)(150 lb/ft^3)/1000 = 24 k

P_u = 411.13 k + (1.2)(24 k) = 439.93 k

M_D = 1.03 k-ft

M_{Lr} = 1.26 k-ft

M_W = 209.68 k-ft

M_u = 170.99 k-ft (1.2D + 1.6L_r + 0.8W)

Foundation Size: 15’-0” x 15’-0” x 2’-9” with (17) #7 bars each way, top and bottom

q_a = 2500 psf

f'_c = 4,000 psi

P = P_D + P_L + P_W = 190.87 k + 113.03 k + 1.55 k = 305.45 k

M = M_D + M_{Lr} + M_W = 1.03 k-ft + 1.26 k-ft + 209.68 k-ft = 211.97 k-ft

M = (P)(e)

211.97 k-ft = (305.45 k)(e)

e = 0.694’ = 8.328”

$q_a \geq P/A + M/S$

$S = bh^2/6$

2.5 ≥ = (305.45 k)/[(15’)(15’)] + (211.97 k-ft)/[(15’)(15’)^2/6] = 1.358 ksf + 0.377 ksf = 1.734 ksf

∴ OK

B/6 = 15'/6 = 2.5' > e = 0.694' ∴ In the kern (do not need to worry about overturning)

L' = L – 2e = 15' – (2)(0.694') = 13.612'

A' = (B)(L') = (15')(13.612') = 204.18 ft^2

P/A' = (305.45 k)/(204.18 ft^2) = 1.496 ksf < 2.5 ksf = q_a ∴ OK

∑M = [(305.45 k)(15'/2) – 211.97 k-ft] = +2078.91 k-ft (∴ Stable since positive)

$M_{resisting}$ = (305.45)(15'/2) = 2290.88 k-ft

$M_{overturning}$ = 211.97 k-ft

P_u = 439.93 k

M_u = 170.99 k-ft

e = M_u/P_u = (170.99 k-ft)/(439.93 k) = 0.389' = 4.664"

L' = L – 2e = 15' – (2)(0.346') = 14.308'

A' = (B)(L') = (15')(14.31') = 214.65 ft^2

q = P_u/A' = (439.93 k)/(214.65 ft^2) = 2.050 ksf

Wide Beam Shear:

V_u = (2.050 ksf)[[(15'-2')/2] – d/12](1') = (0.75)(2)√4000(12")(d)/1000

13.325 – 0.1708d = 1.138d

d ≥ 10.178"

$d_{provided}$ > 10.178" ∴ OK

Punching Shear:

v_c = P_u/{[2d(b+d) + 2d(c+d)]}

$4d^2$ + 2d(b+c) = P_u/v_c

$v_c = \phi v_c = \phi(2 + 4/\beta)\sqrt{f'_c} = \phi(2 + 4/1)\sqrt{f'_c} = \phi 6\sqrt{f'_c}$

$= \phi 4\sqrt{f'c} = (0.75)(4)\sqrt{4000}$ = 189.737 psi

$4d^2 + 2d(24" + 24") = (439,930 \text{ lb})/(189.737 \text{ psi})$

$4d^2 + 96d - 2318.63 = 0$

$d \geq 14.90"$

With #7 bars: $h = 14.90" + 3" + 0.875" = 18.78" > h = 33"$ ∴ OK

Assume $d = 33" - 3" - (1/2)(0.875") = 20.563"$

Flexure:

$l = (15'-2')/2 = 6.5'$

$M = ql^2/2 = (2.050 \text{ ksf})(6.5')^2/2 = 43.31 \text{ k-ft}$

$a = A_s f_y / 0.85 f'_c b = (A_s)(60 \text{ ksi})/[(0.85)(4 \text{ ksi})(12")] = 1.471A_s$

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

$(43.31 \text{ k-ft})(12 \text{ in/ft}) = (0.9)(A_s)(60 \text{ ksi})(29.563" - 1.471A_s/2)$

$519.72 = 1596.40A_s - 39.717A_s^2$

$39.717A_s^2 - 1596.40A_s + 519.72 = 0$

$A_s \geq 0.328 \text{ in}^2/\text{ft}$

$A_{s,provided} = (17)(0.60 \text{ in}^2)/15' = 0.680 \text{ in}^2/\text{ft} > 0.328 \text{ in}^2/\text{ft}$ ∴ **OK**

Appendix C – Glass Strength Calculations

1) <u>Determination of the Load Resistance of a Solar-Control Low-E Insulating-Glass Unit</u>

Location: South Façade, Enclosing Lobby Area

Outer Lite: ¼" Fully Tempered (FT) Clear Float Glass, Monolithic

Inner Lite: ¼" Annealed Clear Float Glass, Monolithic

Air Space: ½"

Dimensions: 5'-0" x 9'-2" = 60" x 110"

Maximum Wind Pressure = 13.04 psf

NFL = Non-Factored Load, GTF = Glass Type Factor, LS = Load Share Factor

LR = Load Resistance

Assume an 8 in 1,000 breakage probability

Outer Lite (for Short Duration Load):

NFL = 1.18 kPa (Fig. A1.6, p. 12, E 1300) = (1.8 kPa)(20.9 psf/kPa) = 24.662 psf

Plate Length = 110", Plate Width = 60", Four Sides Simply Supported

GTF = 3.8 (Table 2, p. 2, E 1300, Fully Tempered, Short Duration Load)

LS = 2.00 (Table 5, p. 5, E 1300)

LR = (NFL)(GTF)(LS) = (24.662 psf)(3.8)(2.00) = 187.43 psf

Inner Lite (for Short Duration Load):

NFL = 1.18 kPa (Fig. A1.6, p. 12, E 1300) = (1.8 kPa)(20.9 psf/kPa) = 24.662 psf

Plate Length = 110", Plate Width = 60", Four Sides Simply Supported

GTF = 1.0 (Table 2, p. 2, E 1300, Annealed, Short Duration Load)

LS = 2.00 (Table 5, p. 5, E 1300)

LR = (NFL)(GTF)(LS) = (24.662 psf)(1.0)(2.00) = 49.32 psf

Outer Lite (for Long Duration Load):

NFL = 1.18 kPa (Fig. A1.6, p. 12, E 1300) = (1.8 kPa)(20.9 psf/kPa) = 24.662 psf

Plate Length = 110”, Plate Width = 60”, Four Sides Simply Supported

GTF = 2.85 (Table 3, p. 2, E 1300, Fully Tempered, Long Duration Load)

LS = 2.00 (Table 5, p. 5, E 1300)

LR = (NFL)(GTF)(LS) = (24.662 psf)(2.85)(2.00) = 140.57 psf

Inner Lite (for Long Duration Load):

NFL = 1.18 kPa (Fig. A1.6, p. 12, E 1300) = (1.8 kPa)(20.9 psf/kPa) = 24.662 psf

Plate Length = 110”, Plate Width = 60”, Four Sides Simply Supported

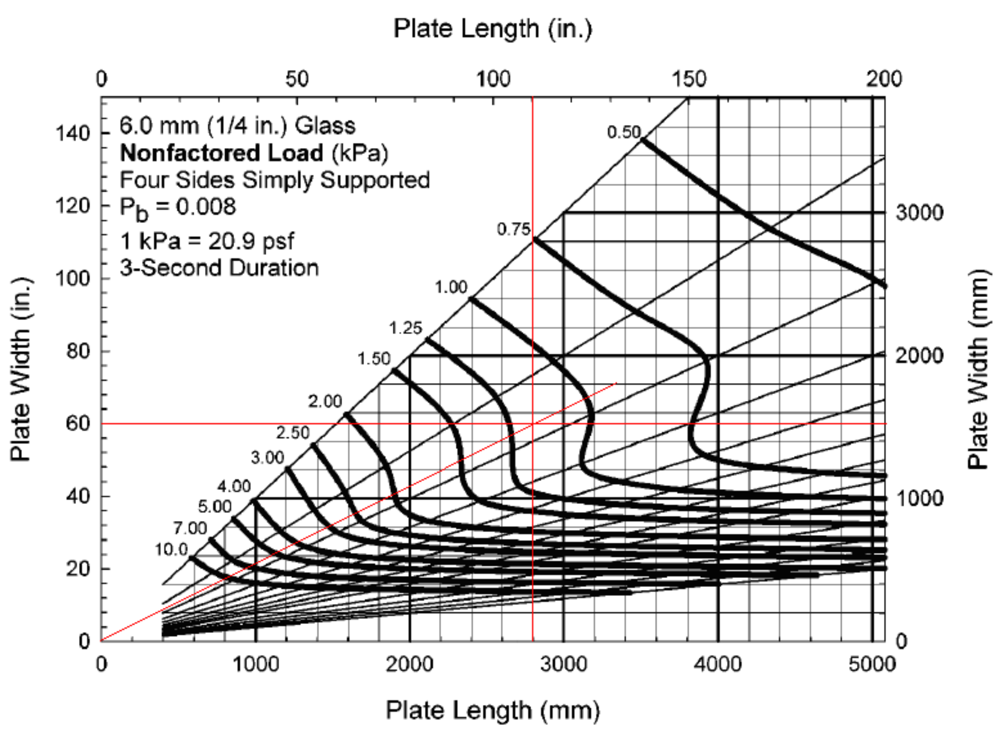
GTF = 0.5 (Table 3, p. 2, E 1300, Annealed, Long Duration Load)

LS = 2.00 (Table 5, p. 5, E 1300)

LR = (NFL)(GTF)(LS) = (24.662 psf)(0.5)(2.00) = 24.66 psf (Controls)

The load resistance of the IGU is 24.66 psf, being the least of the four values: 187.43, 49.32, 140.57, or 24.66 psf

LR = 24.66 psf > 13.04 psf ∴ **OK**



ASTM E-1300 Fig. A1.6

TABLE 2 Glass Type Factors (GTF) for Insulating Glass (IG), Short Duration Load

Lite No. 1 Monolithic Glass or Laminated Glass Type	Lite No. 2 Monolithic Glass or Laminated Glass Type					
	AN		HS		FT	
	GTF1	GTF2	GTF1	GTF2	GTF1	GTF2
AN	0.9	0.9	1.0	1.9	1.0	3.8
HS	1.9	1.0	1.8	1.8	1.9	3.8
FT	3.8	1.0	3.8	1.9	3.6	3.6

ASTM E 1300 – Table 2 – Glass Type Factors for Insulating Glass, Short Duration Load

E 1300 – 04ε1

TABLE 5 Load Share (LS) Factors for Insulating Glass (IG) Units

NOTE 1—Lite No. 1 Monolithic glass, Lite No. 2 Monolithic glass, short or long duration load, or Lite No. 1 Monolithic glass, Lite No. 2 Laminated glass, short duration load only, or Lite No. 1 Laminated Glass, Lite No. 2 Laminated Glass, short or long duration load.

Lite No. 1		Lite No. 2																					
Monolithic Glass		Monolithic Glass, Short or Long Duration Load or Laminated Glass, Short Duration Load Only																					
Nominal Thickness		2.5 (3⁄32)		2.7 (lami)		3 (1⁄8)		4 (5⁄32)		5 (3⁄16)		6 (1⁄4)		8 (5⁄16)		10 (3⁄8)		12 (1⁄2)		16 (5⁄8)		19 (3⁄4)	
mm	(in.)	LS1	LS2	LS1	LS2	LS1	LS2	LS1	LS2	LS1	LS2	LS1	LS2	LS1	LS2	LS1	LS2	LS1	LS2	LS1	LS2	LS1	LS2
2.5	(3⁄32)	2.00	2.00	2.73	1.58	3.48	1.40	6.39	1.19	10.5	1.11	18.1	1.06	41.5	1.02	73.8	1.01	169.	1.01	344.	1.00	606.	1.00
2.7	(lami)	1.58	2.73	2.00	2.00	2.43	1.70	4.12	1.32	6.50	1.18	10.9	1.10	24.5	1.04	43.2	1.02	98.2	1.01	199.	1.01	351.	1.00
3	(1⁄8)	1.40	3.48	1.70	2.43	2.00	2.00	3.18	1.46	4.83	1.26	7.91	1.14	17.4	1.06	30.4	1.03	68.8	1.01	140.	1.01	245.	1.00
4	(5⁄32)	1.19	6.39	1.32	4.12	1.46	3.18	2.00	2.00	2.76	1.57	4.18	1.31	8.53	1.13	14.5	1.07	32.2	1.03	64.7	1.02	113.	1.01
5	(3⁄16)	1.11	10.5	1.18	6.50	1.26	4.83	1.57	2.76	2.00	2.00	2.80	1.56	5.27	1.23	8.67	1.13	18.7	1.06	37.1	1.03	64.7	1.02
6	(1⁄4)	1.06	18.1	1.10	10.9	1.14	7.91	1.31	4.18	1.56	2.80	2.00	2.00	3.37	1.42	5.26	1.23	10.8	1.10	21.1	1.05	36.4	1.03
8	(5⁄16)	1.02	41.5	1.04	24.5	1.06	17.4	1.13	8.53	1.23	5.27	1.42	3.37	2.00	2.00	2.80	1.56	5.14	1.24	9.46	1.12	15.9	1.07
10	(3⁄8)	1.01	73.8	1.02	43.2	1.03	30.4	1.07	14.5	1.13	8.67	1.23	5.26	1.56	2.80	2.00	2.00	3.31	1.43	5.71	1.21	9.31	1.12
12	(1⁄2)	1.01	169.	1.01	98.2	1.01	68.8	1.03	32.2	1.06	18.7	1.10	10.8	1.24	5.14	1.43	3.31	2.00	2.00	3.04	1.49	4.60	1.28
16	(5⁄8)	1.00	344.	1.01	199.	1.01	140.	1.02	64.7	1.03	37.1	1.05	21.1	1.12	9.46	1.21	5.71	1.49	3.04	2.00	2.00	2.76	1.57
19	(3⁄4)	1.00	606.	1.00	351.	1.00	245.	1.01	113.	1.02	64.7	1.03	36.4	1.07	15.9	1.12	9.31	1.28	4.60	1.57	2.76	2.00	2.00

ASTM E 1300 – Table 5 – Load Share Factors for Insulating Glass Units

TABLE 3 Glass Type Factors (GTF) for Insulating Glass (IG), Long Duration Load

Lite No. 1 Monolithic Glass or Laminated Glass Type	Lite No. 2 Monolithic Glass or Laminated Glass Type					
	AN		HS		FT	
	GTF1	GTF2	GTF1	GTF2	GTF1	GTF2
AN	0.45	0.45	0.5	1.25	0.5	2.85
HS	1.25	0.5	1.25	1.25	1.25	2.85
FT	2.85	0.5	2.85	1.25	2.85	2.85

ASTM E 1300 – Table 3 – Glass Type Factors for Insulating Glass, Long Duration Load

2) Determination of the Load Resistance of a Solar-Control Low-E Insulating-Glass Unit

Location: East Façade, Enclosing Concessions Area

Outer Lite: ¼" Fully Tempered (FT) Clear Float Glass, Monolithic

Inner Lite: ¼" Annealed Clear Float Glass, Monolithic

Air Space: ½"

Dimensions: 5'-0" x 12'-6" = 60" x 150"

Maximum Wind Pressure = 12.92 psf

NFL = Non-Factored Load, GTF = Glass Type Factor, LS = Load Share Factor

LR = Load Resistance

Assume an 8 in 1,000 breakage probability

Outer Lite (for Short Duration Load):

NFL = 0.75 kPa (Fig. A1.6, p. 12, E 1300) = (0.75 kPa)(20.9 psf/kPa) = 15.675 psf

Plate Length = 150", Plate Width = 60", Four Sides Simply Supported

GTF = 3.8 (Table 2, p. 2, E 1300, Fully Tempered, Short Duration Load)

LS = 2.00 (Table 5, p. 5, E 1300)

LR = (NFL)(GTF)(LS) = (15.675 psf)(3.8)(2.00) = 119.13 psf

Inner Lite (for Short Duration Load):

NFL = 0.75 kPa (Fig. A1.6, p. 12, E 1300) = (0.75 kPa)(20.9 psf/kPa) = 15.675 psf

Plate Length = 150", Plate Width = 60", Four Sides Simply Supported

GTF = 1.0 (Table 2, p. 2, E 1300, Annealed, Short Duration Load)

LS = 2.00 (Table 5, p. 5, E 1300)

LR = (NFL)(GTF)(LS) = (15.675 psf)(1.0)(2.00) = 31.35 psf

Outer Lite (for Long Duration Load):

NFL = 0.75 kPa (Fig. A1.6, p. 12, E 1300) = (0.75 kPa)(20.9 psf/kPa) = 15.675 psf

Plate Length = 150", Plate Width = 60", Four Sides Simply Supported

GTF = 2.85 (Table 3, p. 2, E 1300, Fully Tempered, Short Duration Load)

LS = 2.00 (Table 5, p. 5, E 1300)

LR = (NFL)(GTF)(LS) = (15.675 psf)(2.85)(2.00) = 89.35 psf

Inner Lite (for Long Duration Load):

NFL = 0.75 kPa (Fig. A1.6, p. 12, E 1300) = (0.75 kPa)(20.9 psf/kPa) = 15.675 psf

Plate Length = 150", Plate Width = 60", Four Sides Simply Supported

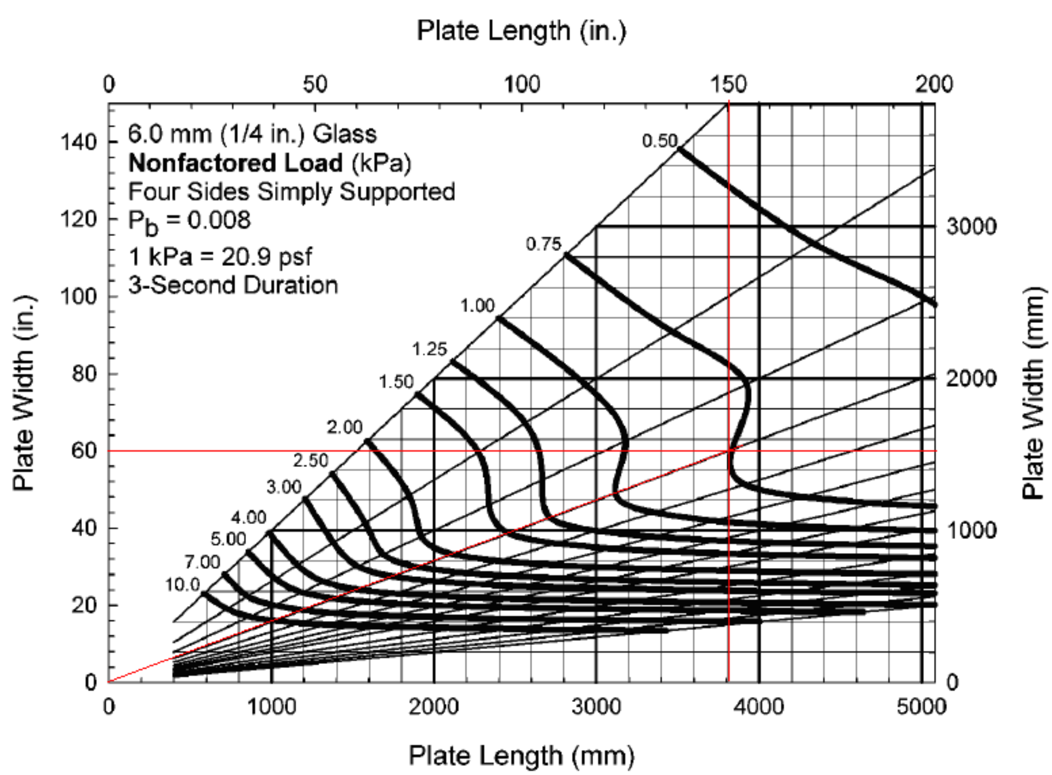
GTF = 0.5 (Table 3, p. 2, E 1300, Annealed, Short Duration Load)

LS = 2.00 (Table 5, p. 5, E 1300)

LR = (NFL)(GTF)(LS) = (15.675 psf)(0.5)(2.00) = 15.675 psf

The load resistance of the IGU is 15.675 psf, being the least of the four values: 119.13, 31.35, 89.35, or 15.675 psf

LR = 15.675 psf > 12.92 psf ∴ **OK**



See ASTM E-1300 Tables 2, 3, and 5 from #1 (above)