Structural Capacity for Solar of Various Buildings on Campus

ENGR 327 Class, Fall 2024

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1 – Introduction

Calvin University is looking to reduce greenhouse gas emissions and upgrade the energy infrastructure. To do this, they have started to consider the impact of introducing solar photovoltaic panels to create a solar farm. Our class has been asked to analyze the structural capacity of roofs on campus to support solar panels. In order for each roof on these buildings to be considered, there must be a significant area facing south to maximize sunlight. Along with the correct direction these roofs must be without shade for most of the day, requiring little to no tree coverage.

Given these considerations, nine buildings on campus were selected for analysis. Our group selected some of the roofs, and others were added by request from the Mechanical Engineering class. Our class divided into three groups to conduct the structural analysis for nine roof structures across those nine buildings. Group A, consisting of MJ VanAntwerp, Reid Bentz, Catherine Grissom, and Josh Gage, analyzed the roof structures of the **Covenant Fine Arts Center (CFAC)** and the **Prince Conference Center**. Group B consisting of Annalise Holcomb, Daniel Oyer, Josh Lundberg, and Leah Huizenga analyzed the roof structures of **North Hall, Business Building,** and **Devos Communication Center**. A final group C consisted of David Bajwa, Garrett Schaaf, and Nate Van Dyke, and analyzed the roof structure of the **Aquatic Center, Van Noord Arena, Hekman Library**, and **Hiemenga Hall**.

All calculations for the structural capacity of buildings in this report were performed by students in the ENGR 327 class. Professor Leonard De Rooy served as an advisor for the project but was not involved in all of the calculations. If Calvin University decides to proceed with rooftop solar mounting, all work should be reviewed and approved by a licensed Professional Engineer in the state of Michigan.

The team has provided a file alongside this report with all of the extensive reference materials needed to support our findings. This zip file contains folders for each of the buildings that were analyzed. In those folders are all of the relevant structural plans, capacity calculations, load tables, and other reference materials used to compile this report.

2 – Executive Summary

The buildings in this analysis were assessed as potential candidates for the installation of photovoltaic panels according to the additional capacity supported by the existing structure. This report is limited in its application, as the load of the photovoltaic panel and its racking system differ depending on the type. Specified possible additional loading for each building is noted in each section of this report. A licensed Professional Engineer in the State of Michigan should review and approve the type according to the building and placement of the system.

The only building found to be structurally inadequate for solar panels in its current condition is the CFAC. The CFAC was divided into 5 roof sections in this analysis, and it was found that none can support the additional load in their current state. More detailed professional analysis and additional reinforcement in this building could make it a viable option.

The viable options for solar panel installation are as follows: Devos Communication Center, Business Building, Venema Aquatic Center, Van Noord Arena, Hekman Library, Hiemenga Hall, the circular area of North Hall, and part of the Prince Conference Center. Based on the calculations provided in this report, most of the buildings can support either type of ballasted or mechanically attached solar panels.

It is worth noting that while the Van Noord Arena is a feasible candidate, it is recommended that further analysis of the truss system is conducted with particular attention to the potential placement and load distribution of the photovoltaic system. Hiemenga Hall is another building in which further analysis is recommended, specifically with the type of photovoltaic system. The Prince Conference Center was divided into different roof sections, some which are viable and others which need further investigation due to lack of available documentation.

Overall, 8 out of 9 buildings in this report are able to support the possible additional load of a photovoltaic system. Once again, it is recommended that a licensed Professional Engineer conduct analyses on these systems and determine the appropriate type of photovoltaic system, along with the placement and structure of the solar panel racking on each roof.

Table 1. Summary of Findings of Structural Viability.

Covenant Fine Arts Center	Not viable for solar panel mounting
Prince Conference Center & Hotel	Conference center viable, hotel unknown
North Hall	Likely viable
Devos Communications Center	Viable for solar panels
Business Building	Viable for solar panels mounted with a ballast system
Venema Aquatic Center	Viable for solar panels
Van Noord Arena	Likely viable
Hekman Library	Viable for solar panels
Hiemenga Hall	

3 – Covenant Fine Arts Center

The Covenant Fine Arts Center was analyzed for its feasibility as a site for rooftop solar. The five roofs shown below in Figure 1 were selected for structural analysis. Roof 1 was selected as a west and slightly south facing roof option to boost solar production in the afternoon. Roof 1 is structurally the same as the roof adjoining Roofs 2 and 3, but analysis was only performed north of the black line drawn, due to the requirement that all roof structures be facing at least slightly south. Roof 2 was chosen as it is south facing with no shade from nearby trees or structures. Roof 3 was selected as it is east and slightly south facing to allow for extra energy production in the mornings. Roof 4 was selected due to its large, flat, and shade free area. Roof 5 was selected because of its large south facing area with relatively little shade. The rest of the roof structures were rejected either because of the direction they face or because of shading from nearby roofs and trees. Full calculations and load tables used to support those calculations can be found in the zip file that was sent alongside this report.

Figure 1. CFAC Roof Labels for Analysis.

3.1 – CFAC Roof 1

The east side of the Covenant Fine Arts Center has a large roof that spans from the north to the south end of the building. For our purposes, only the north half of this roof was analyzed, as the north half is slightly south facing. This roof structure consists of a mix of VS and K series joists, W beams, and custom trusses. Figure 2 below shows the roof section that was analyzed. The yellow lines show the roof area that was analyzed, the red lines show the VS series joists, the blue lines show the K series joists, the green lines show the custom trusses, the pink lines show the W beams, and the purple outlines a mechanical room.

Figure 2. Roof 1 of the CFAC with Specified Areas and Supports Highlighted.

The dead load of the roof over the mechanical room is higher than the typical roof deadload, due to extra equipment typically being mounted to the roof structure. The structural plans for the building call out a roof dead load of 35 psf (pounds per square foot) for mechanical rooms and 23 psf for other areas. Because of this higher dead load, the section of the roof over the mechanical room is not able to support solar panels. Additionally, the custom trusses (shown in green in Figure 2) were designed to support just the weight of the roof structure and snow load, and do not have extra capacity for solar panels according to the structural plans. The W beams, shown in pink, do have plenty of extra capacity, but that area is small and mostly not facing south at all, making it a poor location for solar panels.

It is possible that the custom trusses were built to have extra capacity outside of the standard safety factor, but in order to determine that, we would have to go measure the beams to get more information than is provided in the plans and inspect the welds. Based on the structural plans as shown and our calculations, we would **<u>NOT</u>** recommend this roof as an option for roof top solar. The full and unannotated structural plans and truss details for this roof, and our calculations can be found in the Appendix A and the attached zip files.

3.2 – CFAC Roof 2

Roof 2 in the CFAC is a south facing triangular shaped roof. It is symmetric with VS and KCS series joists spanning from the outside wall to the ridgeline. The ridgeline is formed by a custom truss, Truss M (shown in Appendix A). The joists extend past the outside wall, forming a cantilever section. This cantilever section was not analyzed, as the solar layout provided did not show the panels that close to the edge of the roof. The full structural plans for this section can be found in Appendix A and the attached files. For this section of roof, it was found that the joists could support solar panels. However, the joists are supported by Truss M. The structural plans show Truss M being designed to only hold the weight of the roof as it is. In order to determine if the truss was built with enough safety factor to hold solar panels, a much more extensive physical assessment of the truss and the roof structure would be required. That physical assessment is highly time consuming and would require an outside consultant to take detailed measurements and assess the welds. With those detailed measurements, the truss could be modeled in STAAD Pro. However, given the small size of the roof, the plethora of other options, and the resources required to conduct the analysis, our team decided to not pursue that avenue. Roof 2 of the CFAC is **NOT** recommended as a viable option to support solar panels.

3.3 – CFAC Roof 3

Roof 3 of the CFAC consists of VS and KCS series joists spanning from the outside wall to the diagonal ridgeline on the south portion, and custom trusses spanning the whole roof on the north portion. The ridgeline on the south portion is formed by a custom truss, Truss M. The trusses on the north portion are comprised of Truss L and Truss K. Full structural plans and truss details can be found in Appendix A. For this section of roof, it was found that the joists and Trusses L and K could support the weight of solar panels. However, the joists on the southern portion of the roof distribute their loads onto Truss M, which, as discussed in the Roof 2 section above, cannot support the load of solar panels. The section of roof comprised by Truss L and Truss K is small and faces east rather than south, making it an inefficient location on its own. Because of this, our team does **NOT** recommend Roof 3 as a viable option for solar panels.

3.4 – CFAC Roof 4

The fourth roof analyzed was the central section of the building with the white roof. This section of the roof is the oldest, built in 1964. The roof consists of five (5) trusses. The roof is a built-up roof, with a gypsum and bulb tees roof deck. Figure 3 below shows the structural drawings of Roof 4 and has the trusses that were analyzed highlighted. The structural drawings for Roof 4 display a total load on the trusses in the roof. Using the architectural drawings and the ASCE 7 code book, the deadload on the trusses was determined. Based on the LRFD (Load and Resistance Factor Design) method, the ultimate load was calculated to determine the extra capacity for each truss in Roof 4. On average, the trusses had an extra capacity of 4 psf, which is not enough to hold solar panels. Due to this, it is **NOT** recommended that solar panels be placed on Roof 4 of the CFAC. Full structural plans and calculations can be found in the attached files.

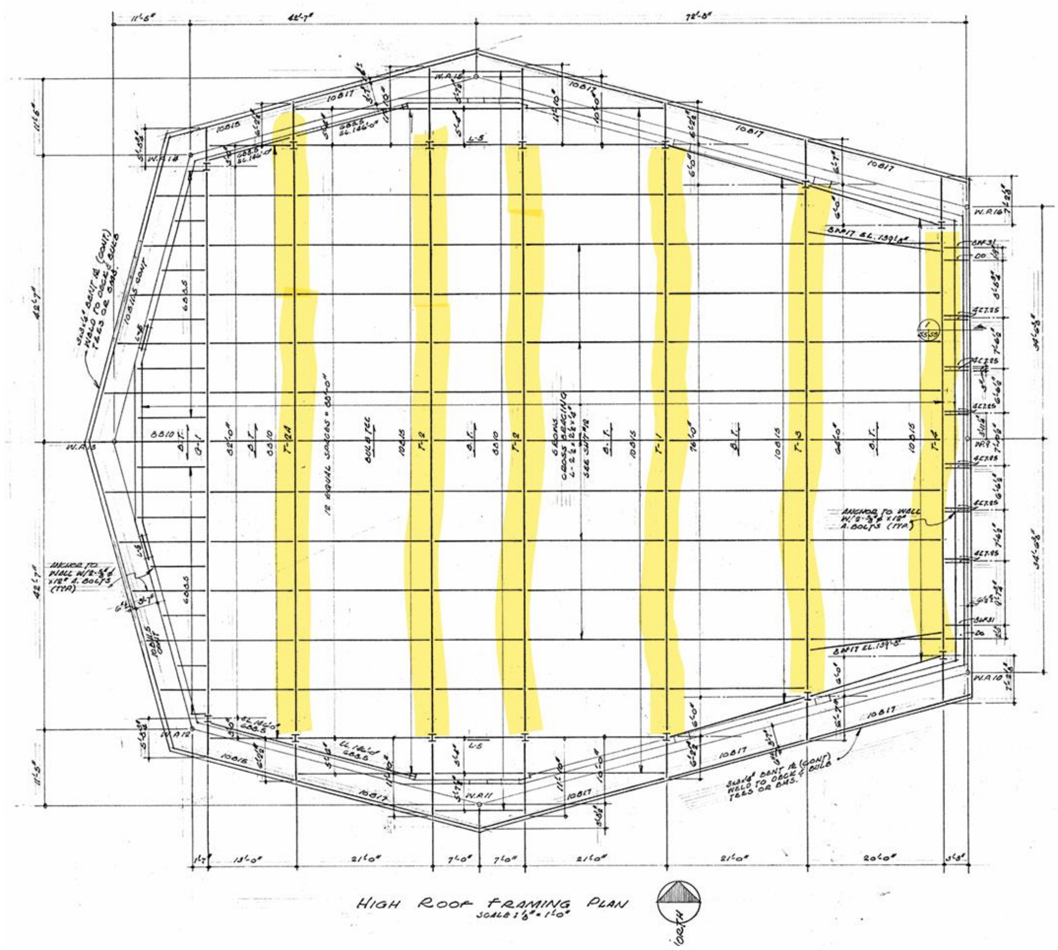


Figure 3. Roof 4 of the CFAC with Trusses Highlighted.

3.5 – CFAC Roof 5

This section of the CFAC was selected for analysis due to its large area and unobstructed south facing roof. The west section of the roof is comprised of a custom truss, Truss C, which spans the width of the roof. The east section of the roof is comprised of a custom framing plan, Frame D. Frame D is made up of a variety of W Beams connected to each other and supported perpendicularly by a larger W Beam with supports on some internal walls. The structural plans and truss details for this section of the roof can be found in Appendix A and in the attached files, and calculations can be found in the attached files.

The structural plans for Truss C show that the truss was only specified to support the weight of the roof and the anticipated snow load, with no extra capacity for solar panels. In order to assess if there is capacity for solar panels in the safety factors, a rigorous and detailed analysis of the truss would have to take place. This analysis would require information beyond what is in the structural plans and is highly time and resource intensive. It would require detailed measurements to be taken of the trusses, a thorough inspection of the welds and connections, and detailed modeling of the truss in STAADPro. Our team determined that given the other alternatives, the need for this rigorous analysis precludes this section of roof from viability for solar panels. Unless rigorous analysis is performed and overseen by licensed Professional Engineer in the state of Michigan, the western part of this roof is **NOT** recommended for solar panels.

The eastern half of the roof, supported by framing plan D, consists of a variety of W beams. It was found that the beams forming the framing plan (running north/south) do have the capacity to hold solar panels. However, these beams distribute their load onto a W18x50 running east/west, which does not have any extra capacity to support solar. Given the constraints with the cross-bracing beam on the eastern half of the roof and the custom truss on the western half, it was determined that this roof section is **NOT** viable for solar panels unless the girder was reinforced.

4 – Prince Conference Center and Hotel

The Prince Conference Center and Hotel were analyzed for their feasibility as a site for rooftop solar panels. The three roofs shown below in Figure 4 were selected for structural analysis. Roof 1 and Roof 2 are connected as part of the western portion of the Prince Conference Center. Roof 3 is the Hotel. These roofs were chosen because they are flat, relatively large, and could be connected to a larger system that includes the Business Building and carpark solar systems in Lots 14, 15, and 16 of Calvin's campus.

Full calculations and load tables used to support those calculations can be found in the zip file that was sent alongside this report.

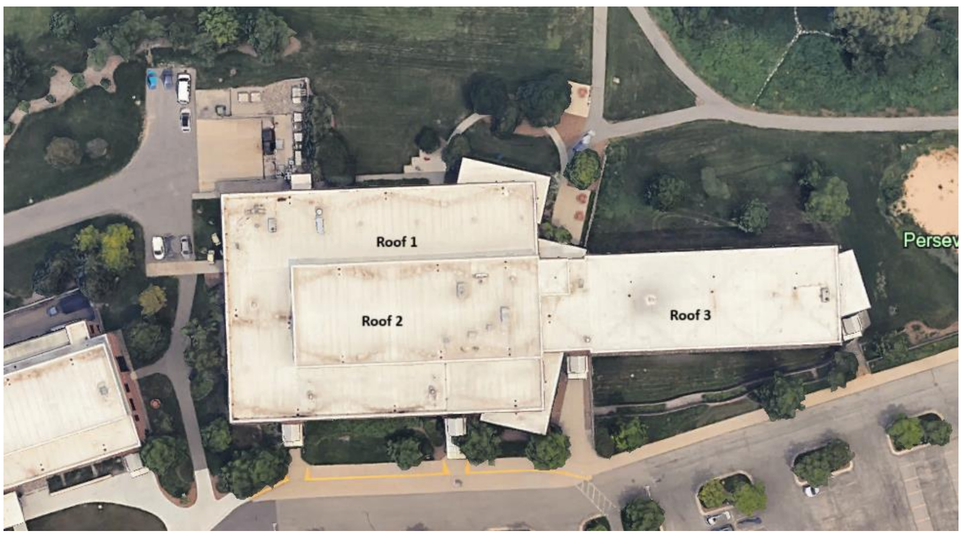


Figure 4. Prince Conference Center and Hotel Roof Labels for Analysis.

4.1 – Prince Roof 1

Roof 1 is above the first floor of the western portion of the Prince Conference Center. The roof is made up of 20-gauge metal decking, two layers of rigid insulation, and a waterproof membrane. The roof consists of twelve types of beams and ten types of joists. The dead load of the roof was determined based on the ASCE 7 code book weights for the materials listed above. The live load on the roof was estimated to be 30 psf based on the snow loads found for other buildings on campus. The design capacities for the beams were found in the AISC Steel Construction Manual tables. The design capacities for the joists were found on various joist manufacturer websites. Full structural plans can be found in Appendix B and the attached files, and calculations can be found in the attached files. Upon analysis of Roof 1 it was determined that the beams and joists have **enough extra capacity to support a rooftop solar system**.

4.2 – Prince Roof 2

Roof 2 is the second story of the western portion of the Prince Conference Center. The roof is made up of 20-gauge metal decking, two layers of rigid insulation, and a waterproof membrane. The roof consists of one truss, eight types of beams, and three types of joists. The total load on the truss was found in the structural drawings. The dead load of the roof was determined based on the ASCE 7 textbook weights for the materials listed above. The live load on the roof was estimated to be 30 psf based on the live loads found in other buildings on campus. The design capacities for the beams were found in the AISC Steel Construction Manual tables. The design capacities for the joists were found on various joist manufacturer websites. Structural plans and calculations can be found in Appendix B and the attached files. Upon analysis of Roof 2 it was determined that the truss, beams, and joists have **enough extra capacity to support a rooftop solar system**.

4.3 – Prince Roof 3

Unfortunately, we were unable to locate any structural drawings of the hotel portion of the Prince Conference Center, in either the files shared to us by Professor De Rooy and facilities or in the physical construction plans located in the Physical Plant on campus. Due to this reality, **we cannot make any determination on the extra capacity of Roof 3 of the Prince Conference Center and therefore its ability to hold solar panels**.

5 – North Hall

When tasked with analyzing the structure of the North Hall, we decided to only focus on the northern circular part, as the other part would receive partial shade, and thus is not optimal for installing solar panels. The North Hall was analyzed in two sections. We analyzed beams and columns to check for strength. We knew that North Hall was built to be as cheap as possible, so it would likely not have significant additional strength. The roof construction includes metal decking, a 5-inch concrete slab, and waterproofing materials that are found in built up membrane roofs. The 5-inch concrete slab is atypical for buildings of this type and added 63 pounds per square foot of load. The cryptic and old-fashioned style of the plans also made analyzing beams precisely a challenge and obstructed the best level of accuracy

5.1 Beams

The roof beams were analyzed by finding the maximum actual moment in the building and comparing that to the maximum moment found in ASD 16th edition. The formula shown below was used to calculate the maximum moment experienced in a beam for most cases.

$$M_{max} = \frac{\omega L^2}{8}$$

The joists have a distributed load from the weight of the roof and thus used this formula to determine the maximum moment using the formula above. Girders and beams were analyzed on a case by case basis where point loads from other beams were added to distributed loads and analyzed in MD Solids. These moments were compared in a table to the maximum loads given for LRFD design in the ASD 16th edition. The roof framing plan can be found below in Figure 5, and full structural plans and calculations can be found in the attached files.

The results showed that for beams only, the first beam would fail once a uniform load greater than 11.5 pounds per square foot is applied. Due to the cryptic nature of the plans, they were not able to be analyzed in great detail, Thus the calculations were calculated in a manner that overestimated the load, so the roof likely has more strength than calculated, but we are **unable to verify any available capacity over 11.5 pounds per square foot**.

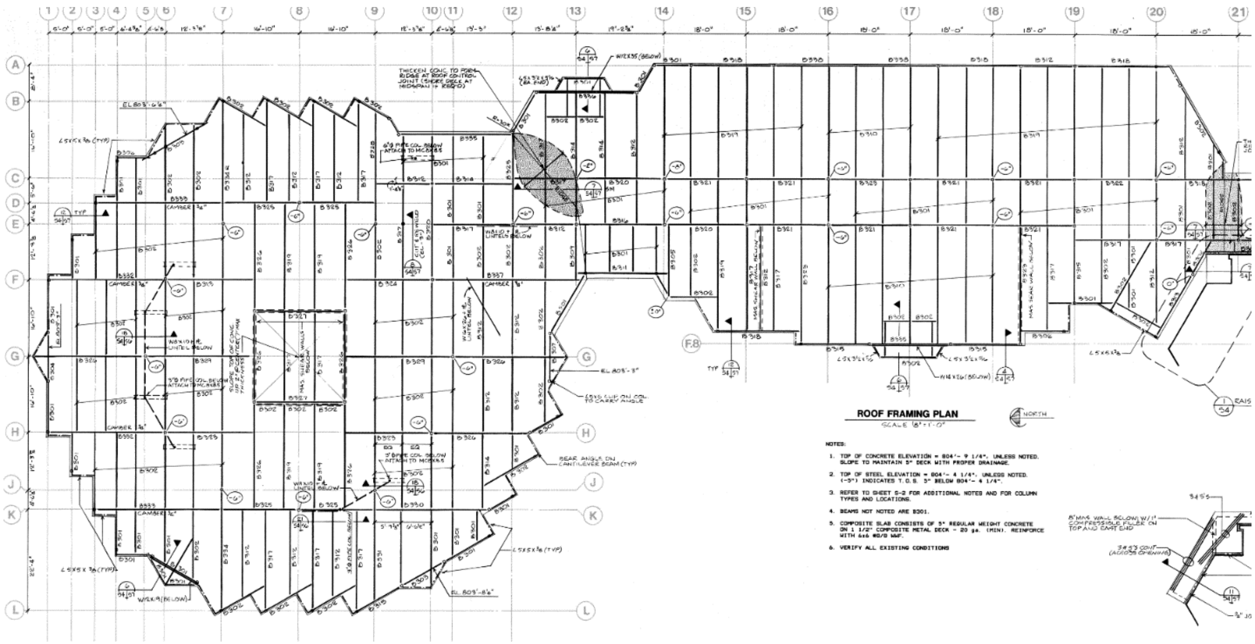


Figure 5. North Hall.

5.2 Columns

The North Hall column structure was spot-checked using 5 columns outlined in blue in Figure 6 below. These columns were chosen for the varying location and loads.

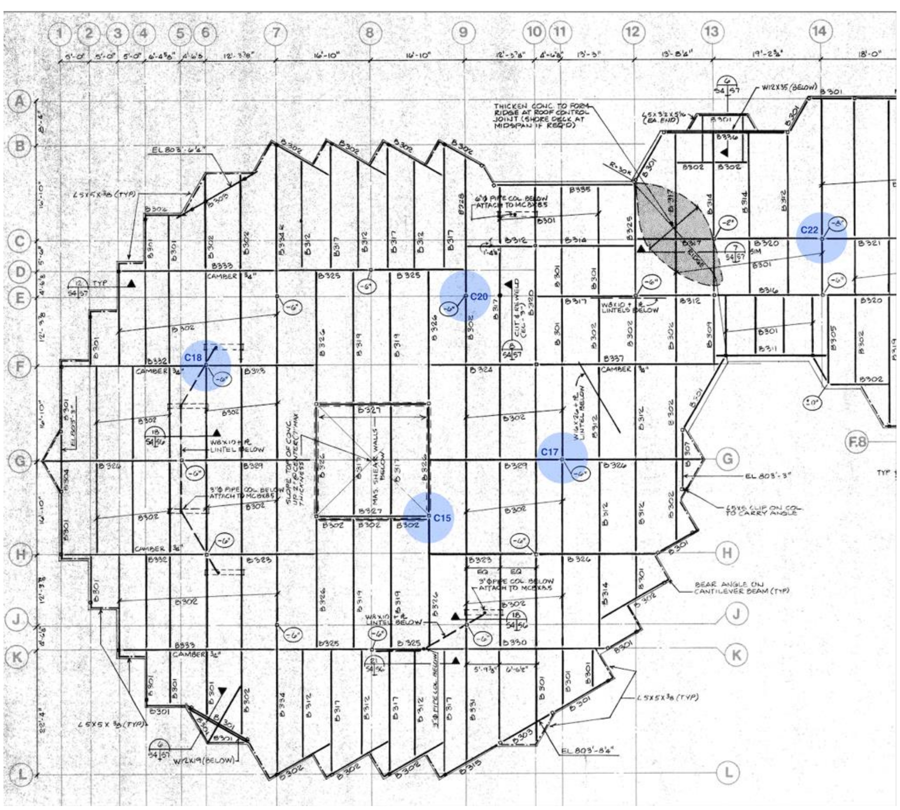


Figure 6. North Hall Spot Column Selection.

Each column has a pinned-pinned connection type, indicating a k value of k = 1. The lengths of the columns analyzed extended only from the 2nd floor to the roof, for a length of 154" according to the North Hall Column Schedule. The values of the radius of gyration (r_y) and area were supplied by the AISC 16th edition according to the column type. At the time that this building was constructed, the value F_y = 43,000 psi was used for the grade of steel in the columns.

This analysis was based on flexural buckling and aimed to find the potential additional load capacity of each column. The critical load (F_{cr}) calculator in Appendix B. The critical capacity (psi) was then multiple by the area of the column to find the critical load value. The LRFD factored reaction forces of the beams were then applied to each column, providing the additional load capacity for each column. At this point in the analysis, it became clear that there is a significant difference in capacity between different areas in the building. The columns in the circular section of North Hall are able to support the additional uniform load of 11.5 psf along any beam. However, the columns in the section of North Hall which is rectangular and connects the circular end section to the Science Building have much lower additional load capacity, which cannot support the additional load of solar panels. Calculations can be found in Appendix C and the attached files.

Therefore, North Hall is likely to support the additional weight of the photovoltaic system in the circular area of the building, but it is recommended that the existing structure and potential implementation of the system is professionally assessed before installation.

6 – DeVos Communication Center

The roof of the Devos Communication Center was also considered (Figure 2). The sections that were analyzed are highlighted in Figure 7 below; for the lower section of the roof, only the part highlighted in yellow was analyzed because this section of the lower roof receives maximum sunlight. Figure 8 below shows a side view of the building roofs.

Figure 5. Roof Sections of Devos Communication Center.

The roof is a built-up membrane roof consisting of a membrane on top of two 2.5" sections of polisocyanurate insulation on top of a 20-gage 1 ½" metal decking. The loading information that was used to perform our calculations for each section is shown in Table 2.

Table 2. Loading information used in Devos Communications Center Calculations.

Original Loading Information		
Live Load	**35**	**psf**
Solar Panel System	5	psf
Snow Load [4]	30	psf
dead load:	**12**	**psf**
Membrane [1]	0.5	psf
0.5" recovery board	0.5	psf
2.5" polisocyanurate insulation [2]	4.5	psf
2.5" polisocyanurate insulation [2]	4.5	psf
metal decking (20-gage) 1 1/2") [3]	2	psf
1. The specific membrane for the roof was also not specified, most membranes I found online weigh 0.5psf.		
2. 1" Polisocyanurate insulation weighs 1.5-2psf. A 2.5" section then weighs about 4.5psf		
3. Taken from 31-S301-DC.		
4. Taken from ASCE 7-10, snow loads.		

Figure 6. Side View of Communications Building Roof.

To find the weight of solar panels that each roof could sustain, the roof was split into sections based on their respective members and were analyzed using ASD (allowable stress design) methodology. Our method of analysis started by finding the ultimate moment for each section under the current loading configuration and comparing that to the allowable moment. The difference between the ultimate moment and the allowable moment can be expressed as a weight per square foot, which is the allowable weight of solar panels that each roof can sustain in its current condition. Our analysis found that the Devos Communications Center would be a **suitable building to fit with solar panels**.

6.1 – Lower Roof

The section of the lower roof that was analyzed was split into four separate sections based on the structural members of the roof. The four sections are shown in Figure 9 below.

Figure 7. Sections of lower roof. Section 2 is ignored.

Our calculations determined that sections 1, 3, and 4 can hold solar panels with each section having an allowable weight for the panels shown in Table 1. Section 2 in Figure 9 was ignored because it will not receive as direct of sunlight as sections 1, 3, and 4. Table 3 below summarizes our findings for this section of the roof.

Table 3. Allowable weights for solar panels, Lower Roof.

Section	Additional Weight for Panels (psf)
1	58
2	ignore
3	58
4	78

6.2 – Penthouse of Communications Center

The penthouse was split into two sections for analysis based on the structural members that constitute each section (Figure 10).

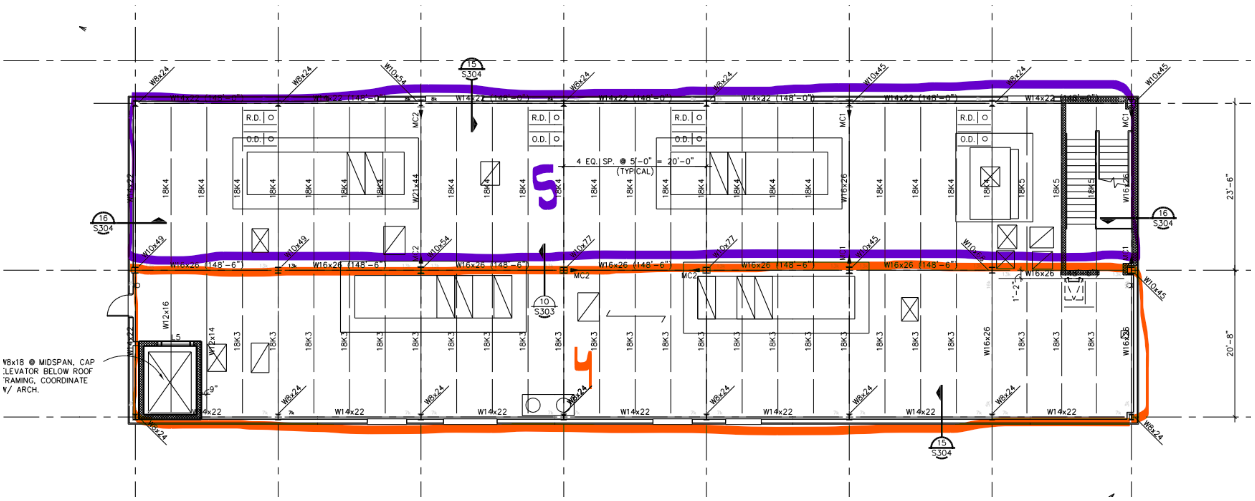


Figure 80. Sections of Penthouse Roof.

Our calculations show that each roof can sustain an allowable weight for solar panels shown in Table 4. The calculations are shown in Appendix D.

Table 4. Allowable Weight for Solar Panels, Penthouse Roof.

Section	Allowable Weight for Panels (psf)
4	43
5	33

7 – Business Building

The Business Building was a recent addition to Calvin University and was therefore built with many considerations for the future. The roof is built with a new, modern membrane with a warm roof construction. It was coated with a lap sealant and flashing to waterproof the edges. The proposed attachment for the solar panels is through a ballast system which would involve penetration of the roof. The roof was designed to support a ballast system with solar panels of 50 pounds per square foot, found in Figure 11. Since the building was designed with intentions to eventually install solar panels, there were no calculations required. The Business Building **CAN** support solar panels.

ROOF DEAD LOAD:

ROOF FRAMING (NO FUTURE FLOOR)	
ROOF DECK	3 psf
ROOFING	2 psf
INSULATION	2 psf
STRUCTURE	4 psf
MEP / FIRE PROTECTION	5 psf
MISC	2 psf
CEILING	2 psf
+ SOLAR PANEL (BALLASTED)	50 psf
	70 psf

Figure 11. Roof Deadload for the Roof Framing Plan.

8 – Venema Aquatic Center

STAAD Pro modelling was used to determine if the roof trusses of the Venema Aquatic Center could safely support the extra load. Safely supporting the extra load required the deflection of the trusses in the y-direction (in inches) to be less than the length of the span (in inches) divided by 240 (L/240) as per specification in the AISC manual. The Venema Aquatic center roofing structure was primarily composed of eleven (11) identical roof trusses. This truss was modelled in STAADPro (see Fig. 12), after hand calculations and consultation of the building structural plans (please see Appendix E for hand calcs and truss details).

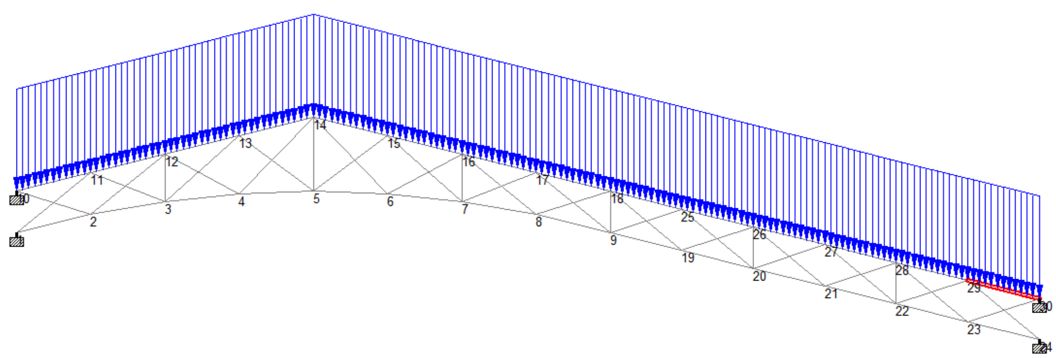


Figure 12. Venema Truss with Loads.

The ends of the truss were assumed to be fixed (as they ran into the wall). Loading was found to be 1.84 kips/ft. This was determined by multiplying the live and dead loads (47 psf combined) by the area of the roof (40,000 sf), after which the solar panel load (63,504 kg or 140,002.35 lbs.) was added. The total load for the whole roof was determined to be 2,020,002.35 pounds. This number was divided by eleven (11), to give the average loading for each individual truss – 252,500.294 pounds or 253 kips. The load was then divided by the length of the truss (127.82 ft) to give the load in kips/ft (1.84). This load was applied to the truss modeled in StaadPro. STAADPro analysis (please see the Venema Reference Items for analysis results) determined that under these loading conditions, the truss deflected only 1.47 inches. The maximum allowable deflection was determined to be 6.2 inches, using the length of the truss span in inches (L = 1,488 in) divided by 240. Given the analysis done in STAADPro, we determined that this **truss can easily support the additional weight of the solar panels.**

9 – Van Noord Arena

The Van Noord Arena was split up into different sections of trusses. The analysis was first started by dividing up the areas that each truss type would experience loading from, using the given number of 21.3 kg per panel. The area of the roof is nearly split in half with a triangular shaped truss and more bridge looking truss, named Arena Truss C and Arena Truss B, respectively (See these in Appendix F). There is a similar truss to Truss B that is called Arena Truss D, which has one extra member in it. After finding the load each section would experience, we started modeling Truss C. After modeling, a report is generated by the program and can be seen in the provided project folder, labeled as 'Truss C Report'. We found that the deflection shown in the report is significantly below the calculated allowable deflection. This was found using the equation L/240, which ends up giving an allowable deflection of 3.2 inches. Because of this knowledge, we can go on to assume that Truss C is **able to hold the load of the new solar panels.**

At this point, the analysis got a little murky. Truss B and D are very similar and ended up with similar results post-modeling. It can be found in the provided project folder labeled as 'Truss B Report' that it is assumed that Truss B is to deflect about 7 inches, while the allowable was only 5.5 inches. This does not make much sense. Our team deduced that Truss C is experiencing a higher load than Truss B, due to the area distribution. For this reason, we believe that this building can support solar panels. However, given that the Mechanical class requested a structural response weeks earlier than initially planned, we didn't have enough time to obtain a more professional assessment of potential inaccuracies in the model. Once again, Truss C contains more solar panels than Truss B and Truss C was successful. Therefore, we would cautiously like to assume that the model has a discrepancy and that the Van Noord Arena is capable of withstanding the addition of solar panels.

10 – Hekman Library

Hekman Library was initially only a four-story building, and in 1994 a fifth story was completed. This roof is a simply supported steel beam section, that has the necessary capacity to add the required number of panels (see attached spreadsheet in the zip folder). The capacity was found using LRFD method. After finding the additional moment capacity, the additional allowable psf was found, and Hekman library **will be able to hold the extra capacity**. These numbers range from about 17.5 psf to 25 psf.

11 – Hiemenga Hall

The first section of Hiemenga Hall was built in 1961, and an additional section was built in the late eighties. The first building was built using a simply supported steel beam section, that has plenty of additional capacity. Drawings are a bit unclear in quite a few spots, especially weak on the dimensioning side, which led our group to assume worst case scenario, and even in worst case scenario, this **building has the necessary capacity**. In the attached excel (found in the zip folder), equations are given. The addition in Hiemenga, which completed the block and gives that area its courtyard, was built using concrete slaps. It was again found there was plenty of additional capacity for solar panels. An exact analysis of available loading in pounds per square foot is available on the excel in the zip file, but the figures are smallest at about 30 psf, with a rather large uncertainty due to unclear sheet plans and poor dimensioning.

Appendix A: CFAC

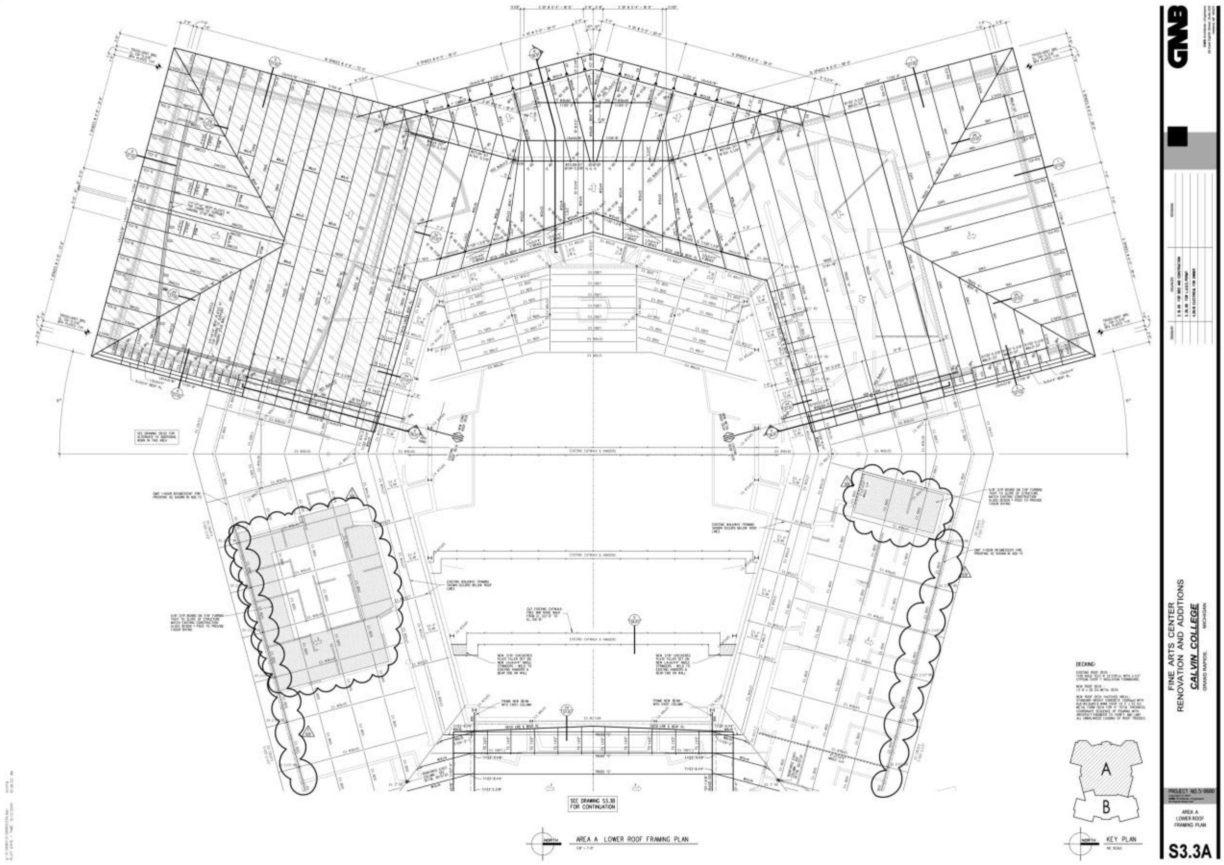


Figure 93. CFAC Structural Plans for Roofs 1,2,3.

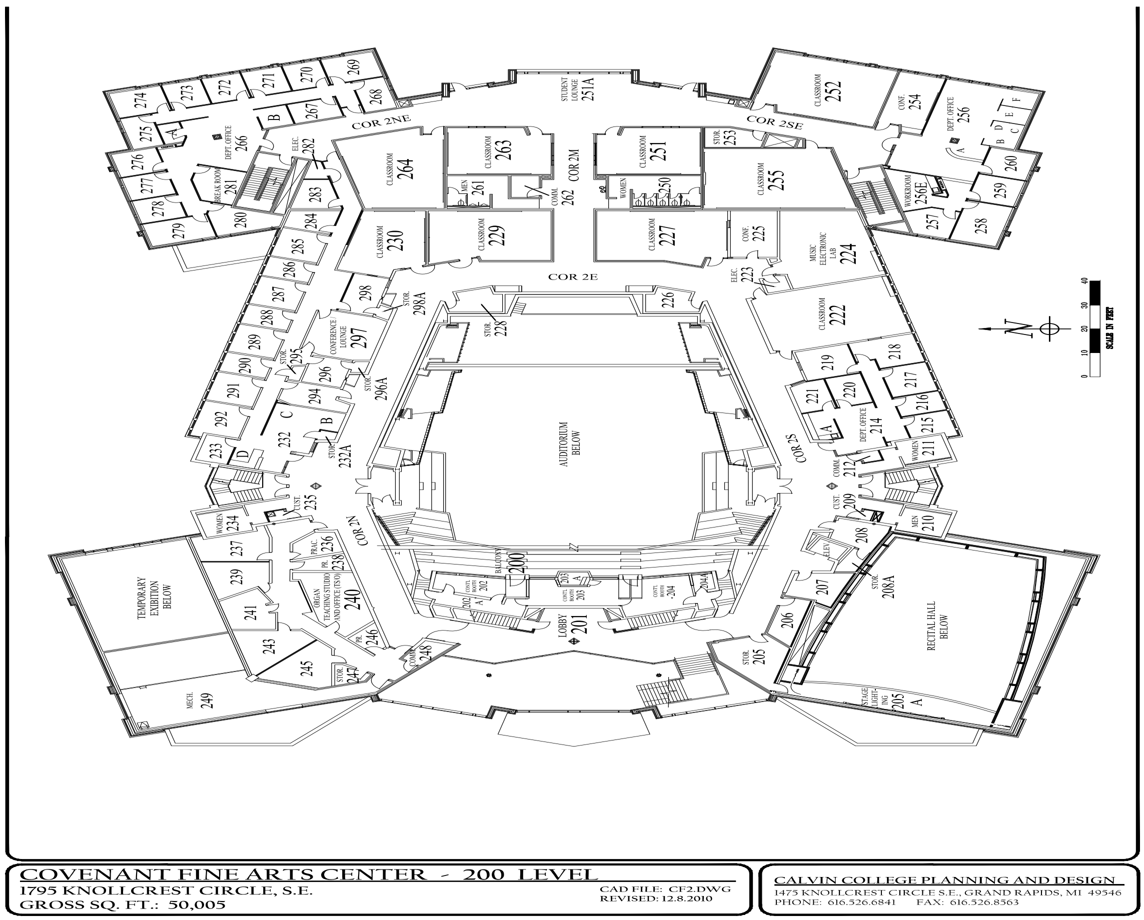


Figure 104. CFAC Upper-Level Floor Plan Roofs 1, 2,3.

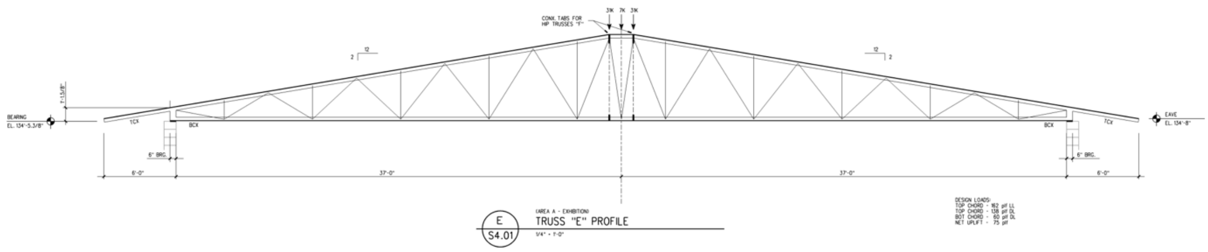


Figure 115. Truss E Profile from CFAC Roof 1.

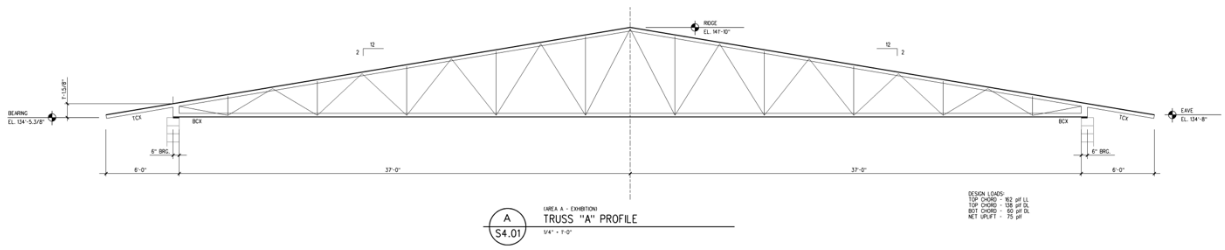


Figure 126. Truss A Profile from CFAC Roof 1.

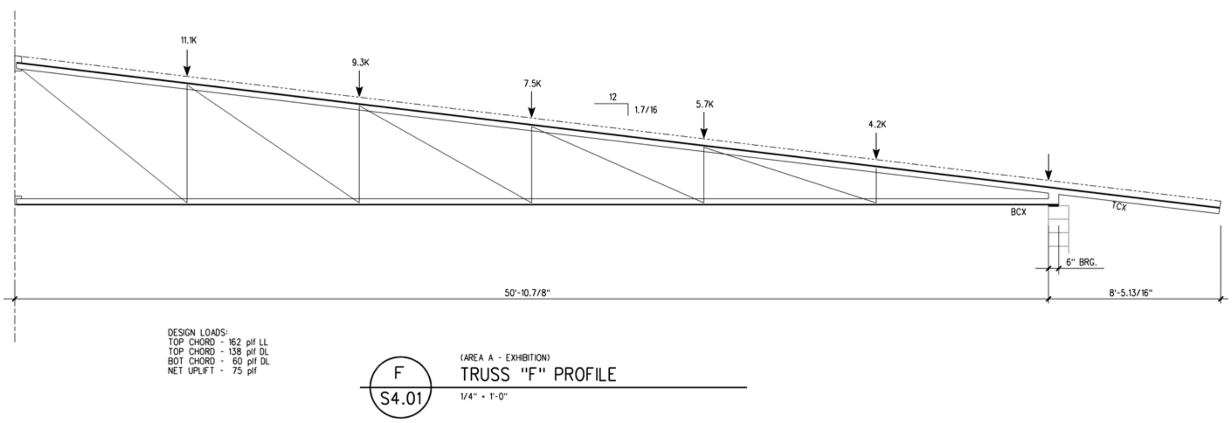


Figure 137. Truss F Profile from CFAC Roof 1.

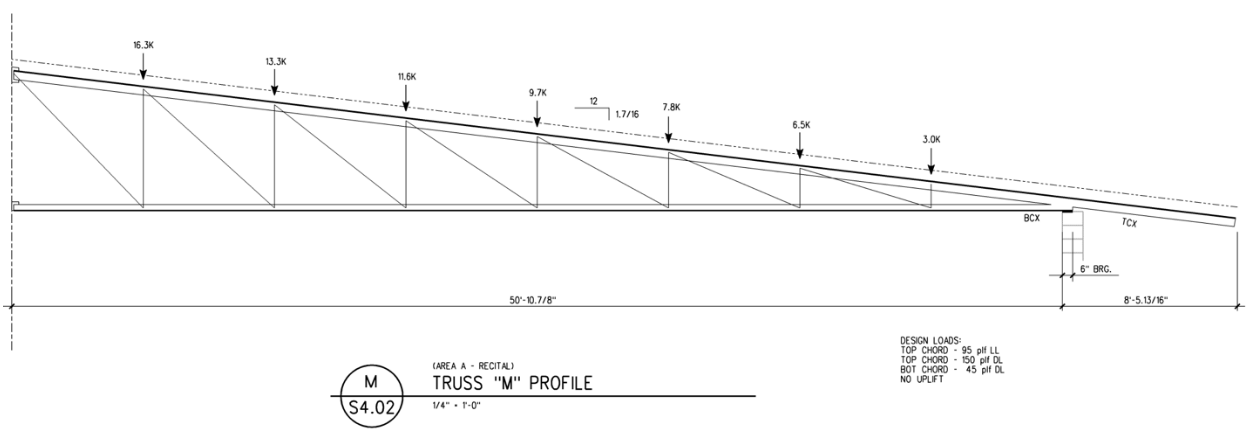


Figure 148. Truss M Profile from CFAC Roofs 2 and 3.

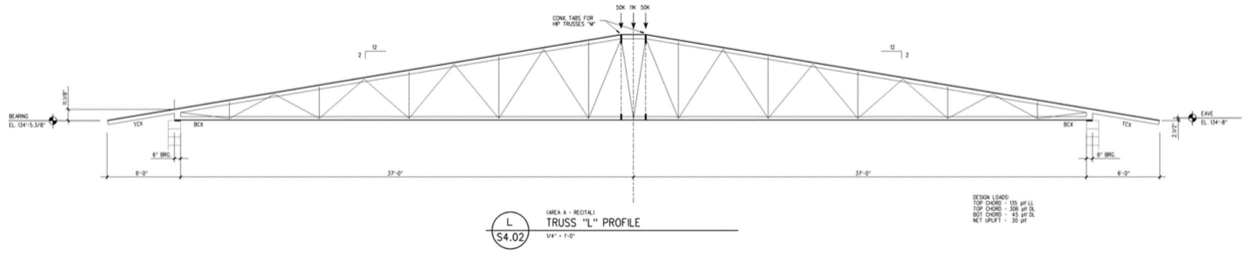


Figure 159. Truss L Profile from CFAC Roof 3.

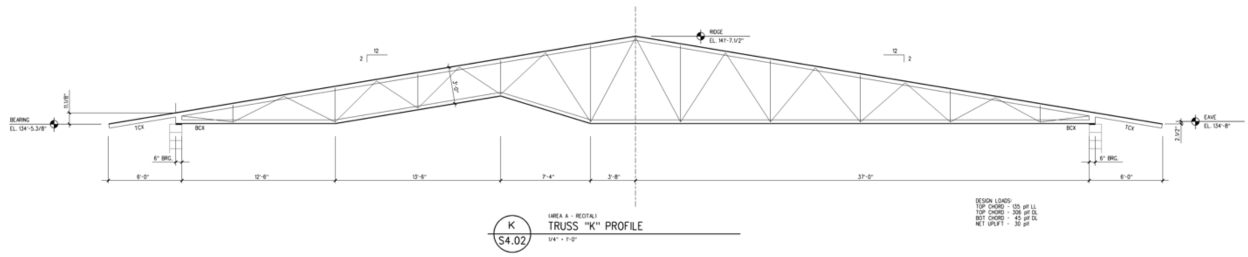


Figure 160. Truss K Profile from CFAC Roof 3.

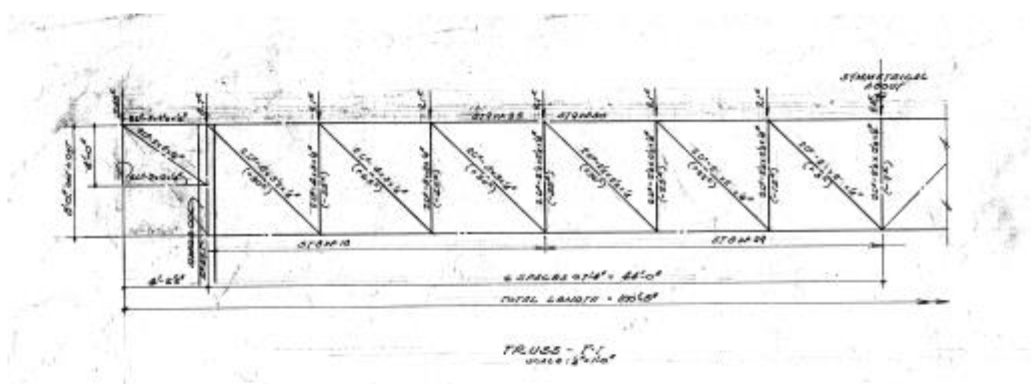


Figure 171 Truss T-1 Profile from CFAC Roof 4.

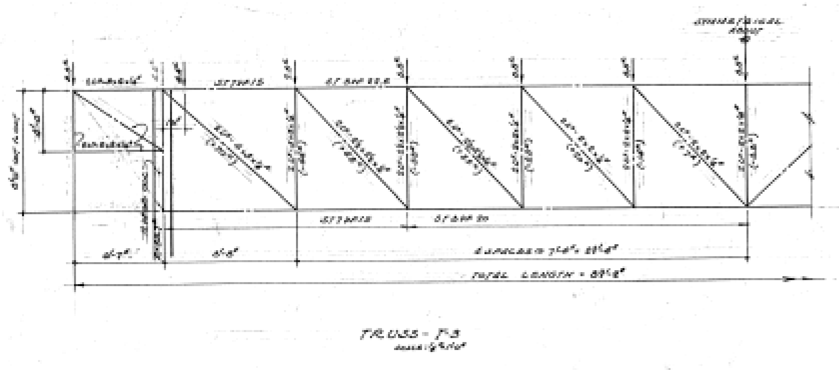


Figure 182. Truss T-3 Profile from CFAC Roof 4.

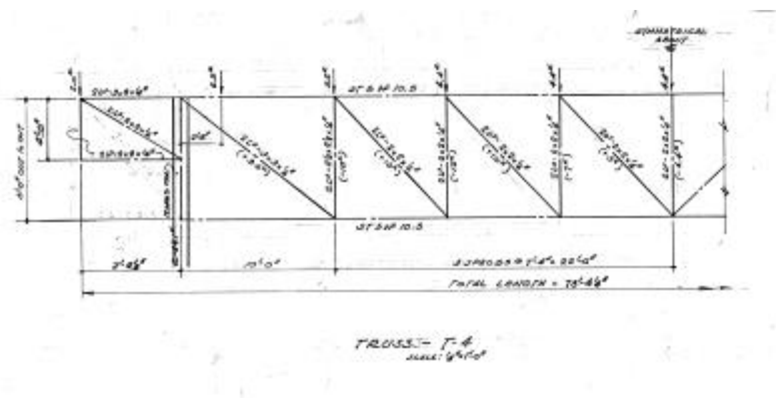


Figure 193. Truss T-4 Profile from CFAC Roof 4.

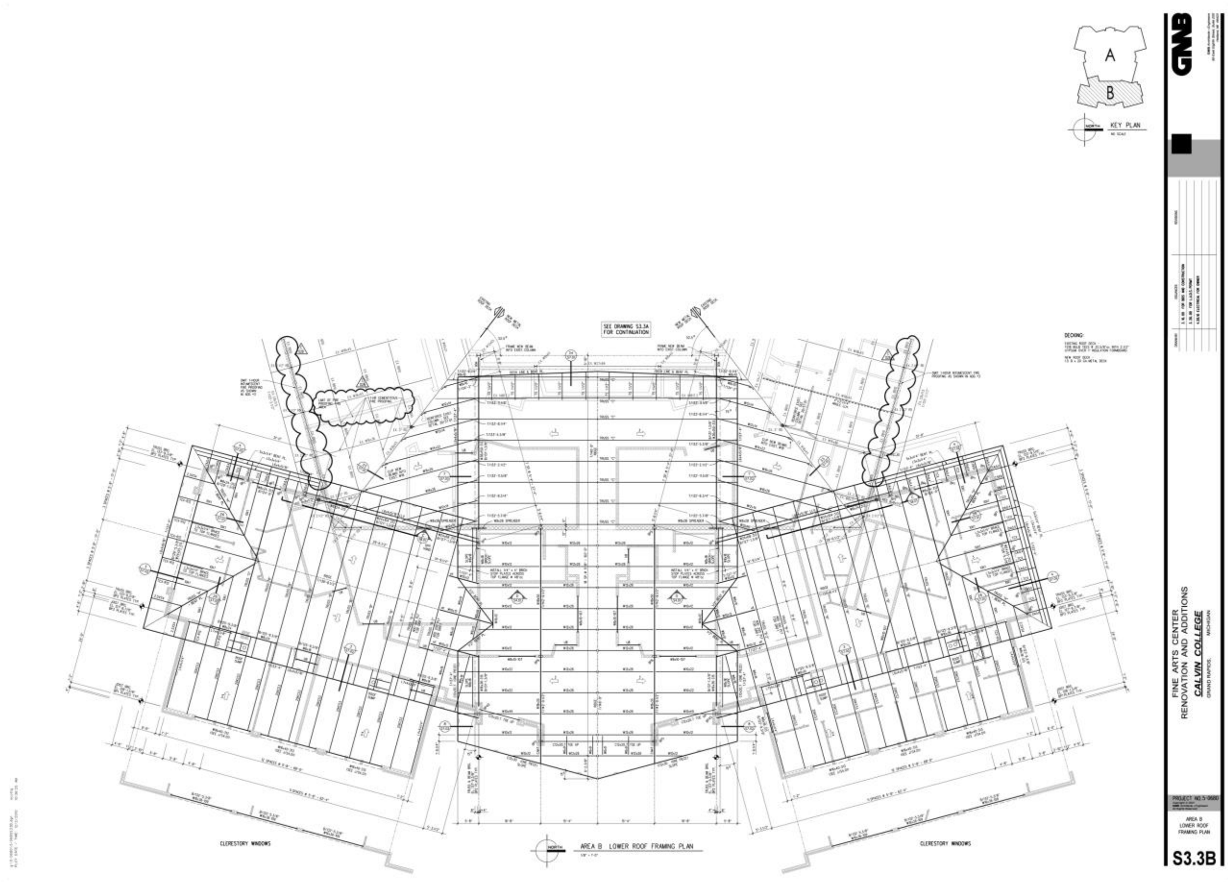


Figure 204. CFAC Structural Plans for Roof 5.

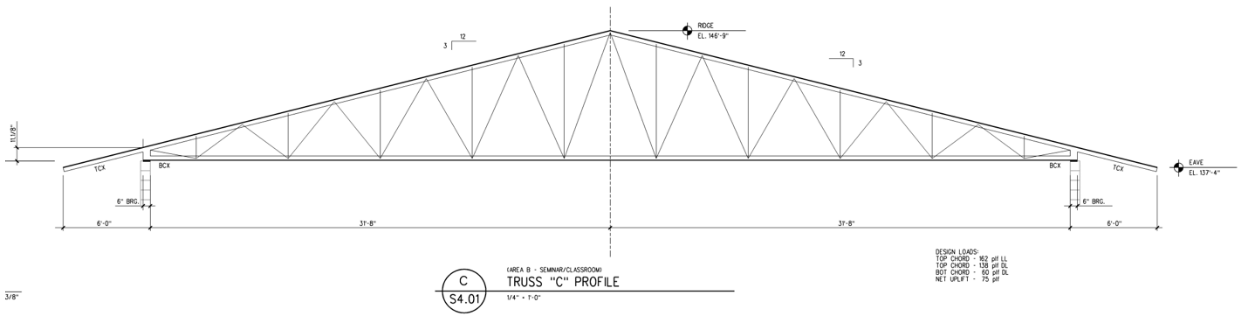


Figure 215. Truss C Profile for CFAC Roof 5.

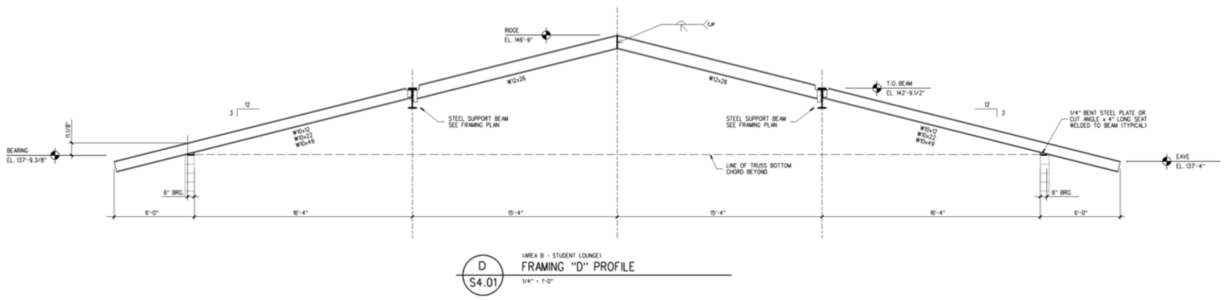


Figure 226. Framing Plan D Profile for CFAC Roof 5.

Appendix B: Prince Conference Center and Hotel

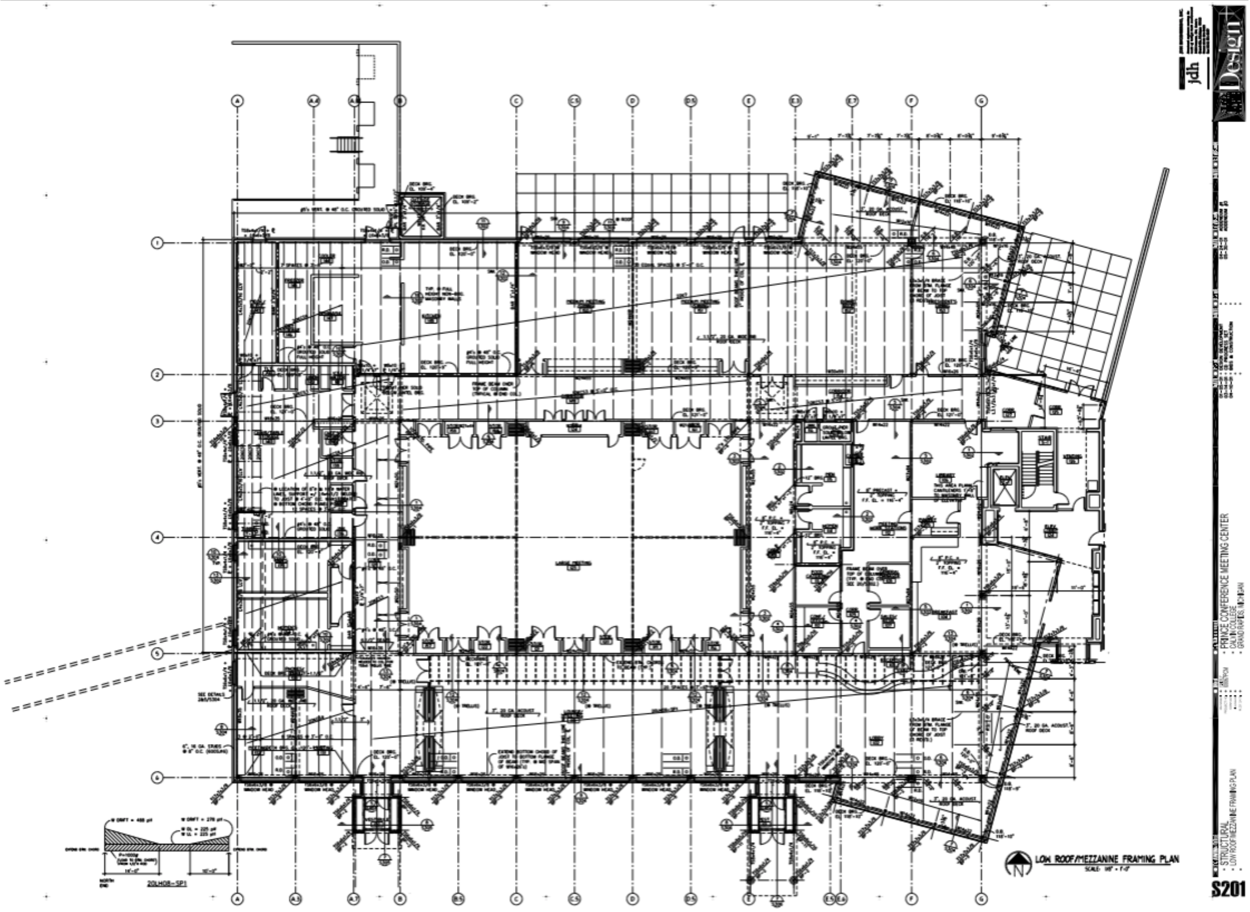


Figure 27. Prince Conference Center Roof 1 Framing Plan.

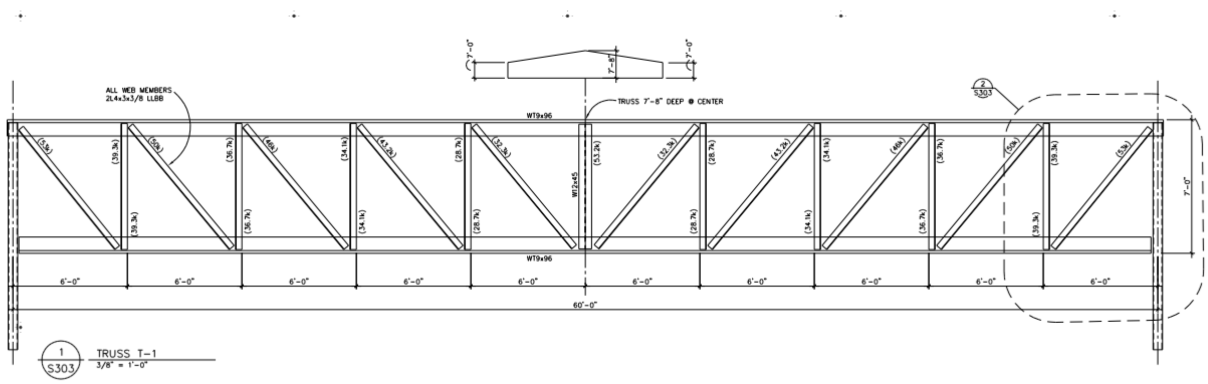


Figure 28. Truss T-1 Profile for Prince Conference Center Roof 1.

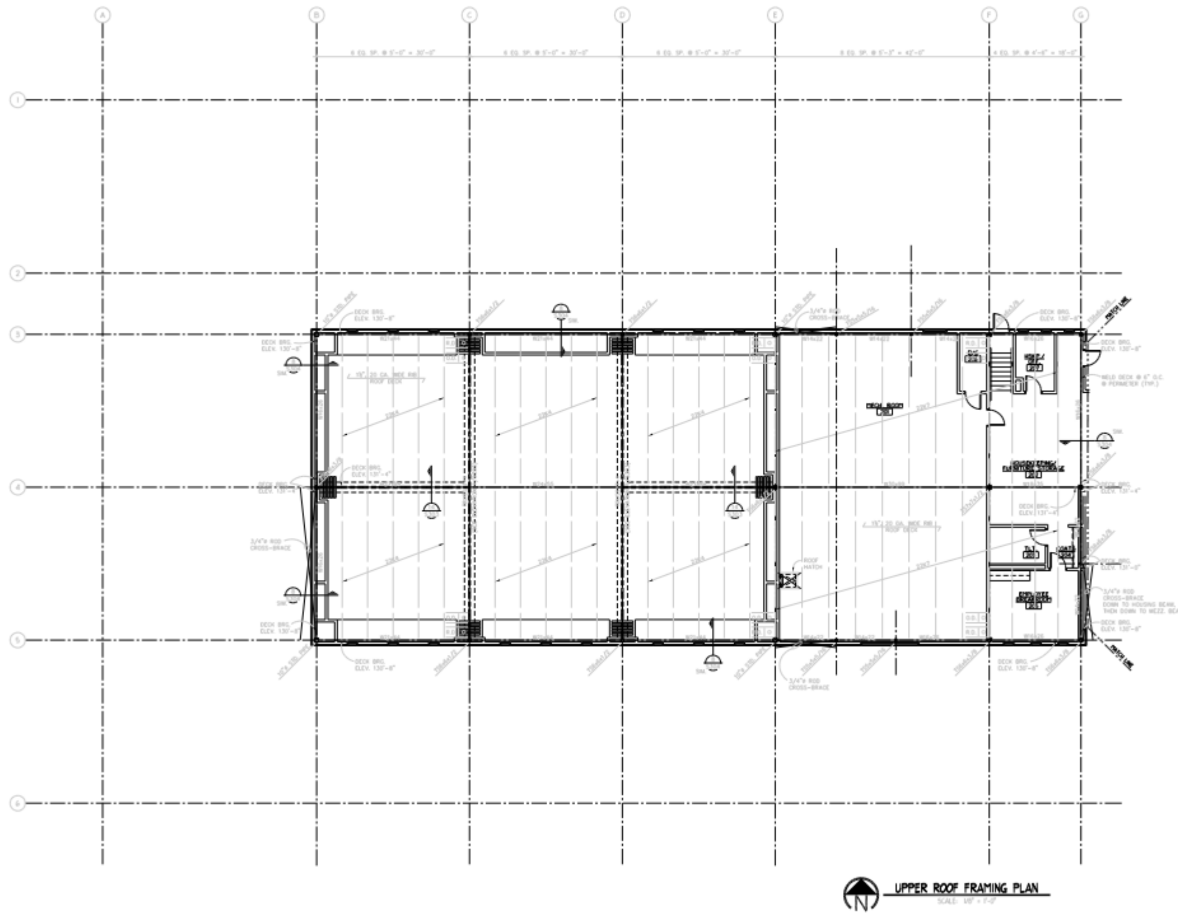


Figure 29. Prince Conference Center Roof 2 Framing Plan.

Appendix C: Column Calculator for North Hall

Table 4 – Column Limit State of Flexural Buckling (F_{cr}) Calculator

Inputs		
K	effective length factor	1
L	lateral unbraced length of a member (in)	154
r	radius of gyration (in)	1.55
Fy	specifiied minimum yield stress (ksi)	50
E	modulus of elasticity (ksi)	29000
Fcr	24.29	

Calculated Values		
	Formula	**Value**
λc	$\sqrt{Fy/Fe}$	1.31
Fe	$\frac{\pi^2 E}{(\frac{KL}{r})^2}$	28.99
Intermediate Fcr ($\lambda c > 1.5$)	$\left(\frac{0.877}{\lambda^2}\right) Fy$	25.43
Long Fcr ($\lambda c \leq 1.5$)	$(0.658^{\lambda c^2})$Fy	24.29

Table 5 – Column Additional Capacity Calculations

Column Calculations								
Column No.	**Size**	**Area (in^2)**	**Ry (in)**	**Slenderness Ratio L_c/r_y**	**Calculated F_{cr} (ksi)**	**Load Capacity (kips)**	**Load from Reactions at Top of Column (kips)**	**Additional Capacity (kips)**
C20	HSS 4x4x3/16	2.58	1.55	8.3	24.29	62.6682	29.96	32.71
C18	HSS 5x5x3/16	3.28	1.96	6.5	31.84	104.435 2	84.13	20.30
C17	HSS 5x5x3/16	3.28	1.96	6.5	31.84	104.435 2	57.46	46.98
C15	HSS 6x6x3/16	3.98	2.37	5.4	36.72	146.145 6	39.39	106.76

Appendix D: Calculations for Devos Communications Center

For this level, we are only focusing on the portion of the roof that is to the west of the penthouse. The parts of the roof that are north and south of the penthouse will not receive as good of sunlight as to the west.

Section	Membe	Length (ft	Tributary Width (ft)	ω_U (kip/ft)	M_U (kip*ft)	Yield Strength (kip/ft)	ΦM_N (kip*ft)	Φ*Capacity (psf)	allowable weight (psf
1	22K4	23.5	5	0.352	24.299	0.712	44.235225	128.16	57.76
3	22K5	23.5	5	0.352	24.299	0.712	44.235225	128.16	57.76
4	10K1	10	5	0.352	4.4	0.825	9.28125	148.5	78.1

1. https://vulcraft.com/catalogs/JoistGirder/Vulcraft-Steel-Joist-Joist-Girder-Systems-Manual-V2020.1J.pdf

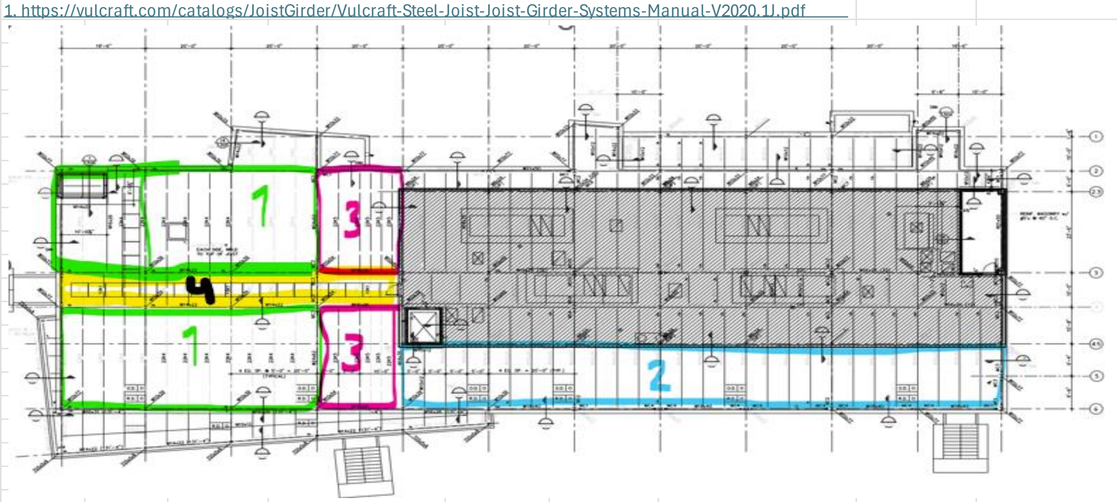
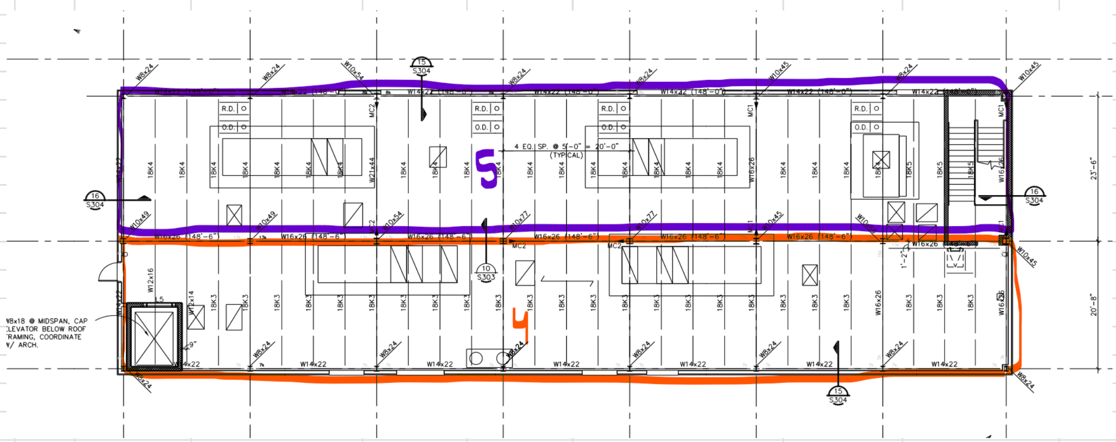


Figure 30 Calculations for Devos Communication Center

Secti	Me	Length (ft	Tributary Width	ω_U (kip/	M_U (kip*	Yield Strength (kip/	ΦM_N (kip	Φ*Capacity (p	allowable weight (p
4	18K3	21	5	0.352	19.404	0.63	31.255875	113.4	43
5	18K4	24	5	0.352	25.344	0.577	37.3896	103.86	33.46



Figures 31. Calculations for Devos Communications Center

Appendix E: Venema Aquatic Center

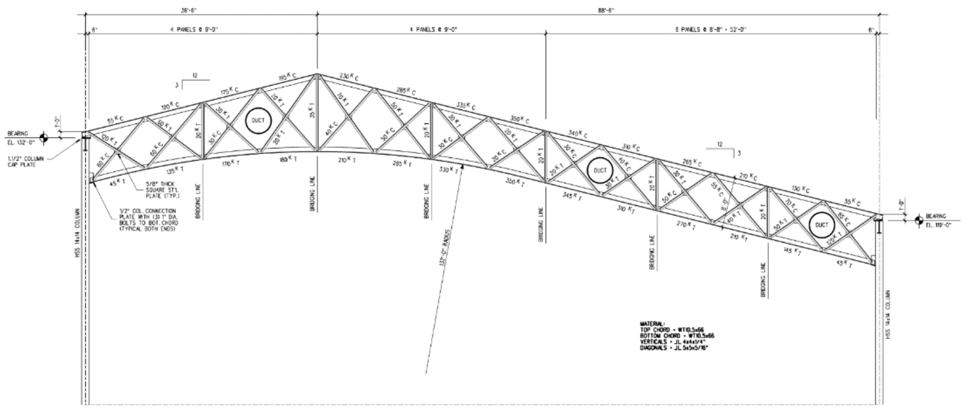


Figure 32. Venema Aquatic Center Truss

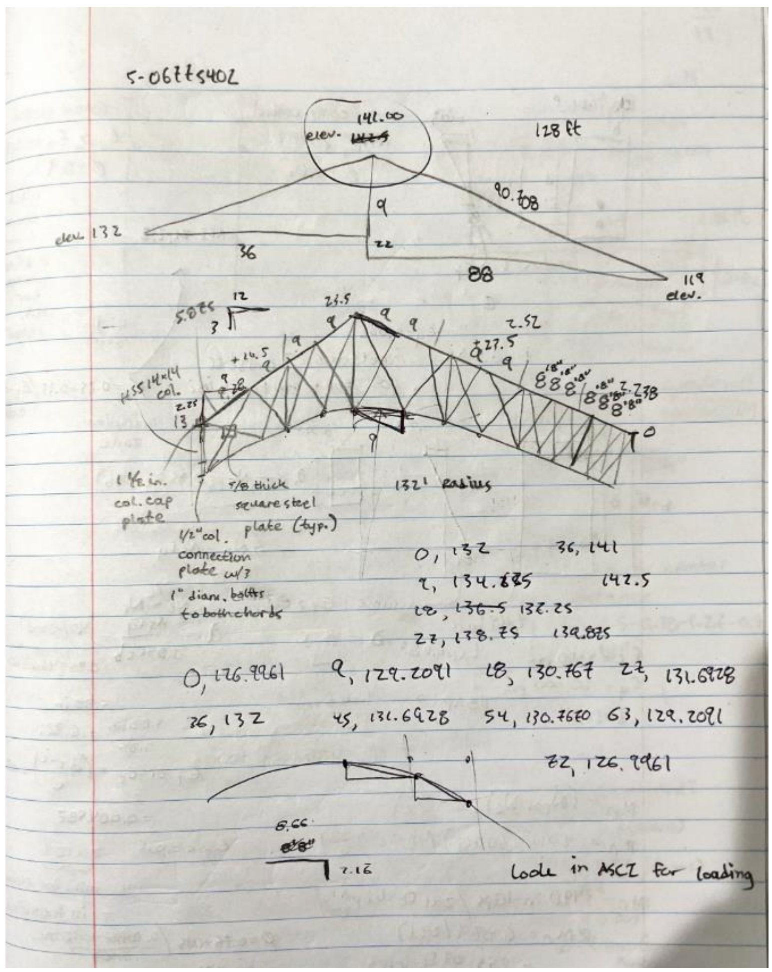


Figure 33. Venema Aquatic Center Hand Calculations

Appendix F: Van Noord Arena

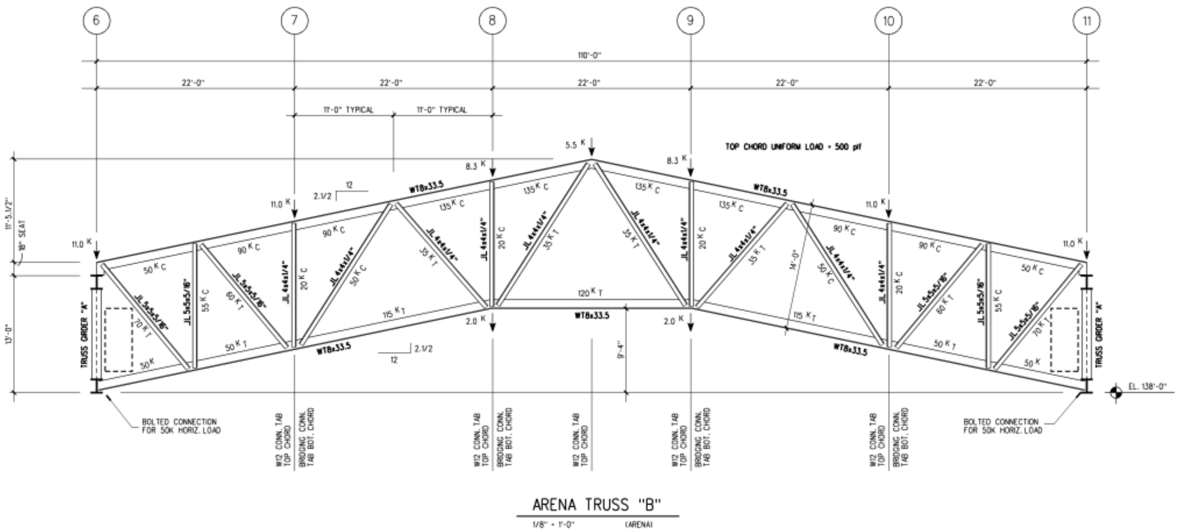


Figure 34. Van Noord Arena Roof Truss B

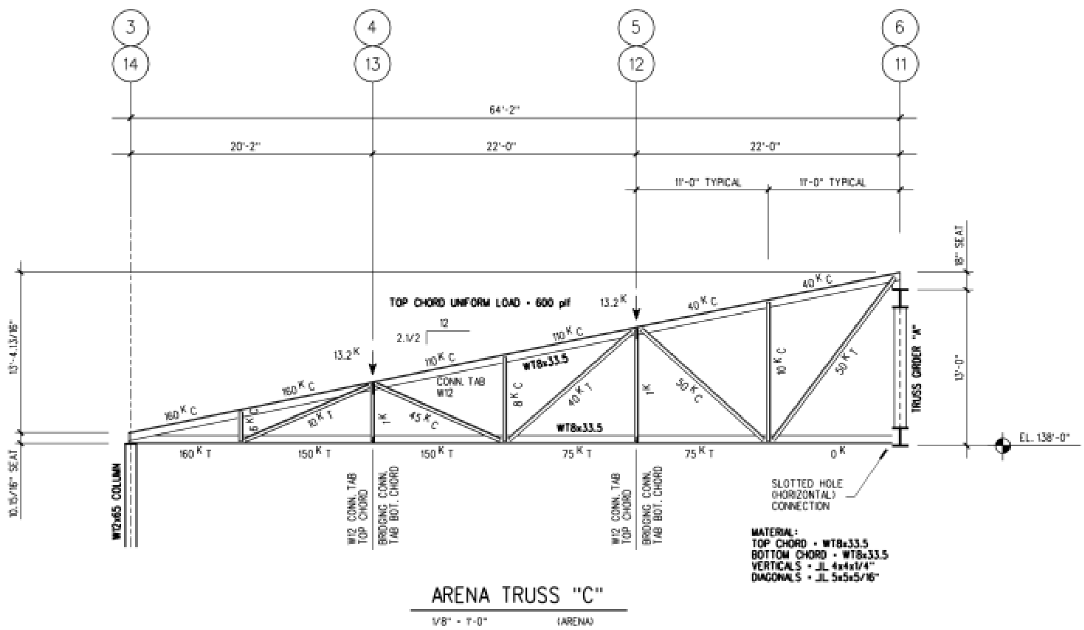


Figure 35. Van Noord Arena Truss C