Farquhar Park Aquatic Center

York, PA

Technical Report #1

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Structural Option

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Executive Summary

The Structural Concepts / Structural Existing Conditions Report describes the structural system of the Farquhar Park Aquatic Center natatorium. This state-of-the-art natatorium complex located in York, PA, consists of a 53-foot high natatorium, a 12-foot deep indoor swimming pool, an outdoor swimming pool, and a 3,600 square foot masonry bath house. Large triangular-shaped trusses made of HSS members span 130'-0" and are supported by triangular, tapered columns. These long spans create a very open area around the indoor pool. A precast concrete grandstand is supported by sloped and bent W-shape beams and HSS columns. A lower roof is supported by smaller trusses that are spaced 15'-0" on-center. Wind columns help transfer lateral loads to the roof diaphragm and to the steel moment-resisting system. Gravity and lateral load calculations were performed on the building. Both wind and seismic analysis for this report were performed using ASCE 7-05 and compared to the results obtained by Nutec Design Associates, Inc. Calculations confirmed that the building was adequately designed to handle the required forces.

Base shear due to wind loads was determined to be 2509.76 kips, which was controlled by the East/West direction. Net wind uplift was calculated to be 10.3 psf, which is very close to Nutec's net wind uplift value of 10 psf. Base shear due to seismic loads was found to be 264.25 kips, which is fairly close to the 300 kip base shear determined by Nutec. Discrepancies may be due to differences in assumptions made and differences in estimations used for calculations.

Gravity checks were performed on the members of a large truss spanning 130'-0" over a large indoor swimming pool area. Another spot check was conducted on a truss above the lobby that supports the lower roof, mechanical support framing, and mechanical units. Snow drift loads were also taken into account. The results determined using ASCE 7-05 were relatively close to those calculated by Nutec, and some internal forces were within a few kips of Nutec's design values. Besides differences in assumptions, variations in results may also be due to the fact that this report only accounts for gravity loads when conducting spot checks, whereas Nutec Design Associates, Inc. would have accounted for lateral forces in addition to gravity loads.

Introduction

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. 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. 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.

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 walls 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.

Nutec Design Associates designed the facility to comply with certain LEED prerequisites and 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 sustainability 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 2 – View of Main Entrance of Natatorium

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. 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.

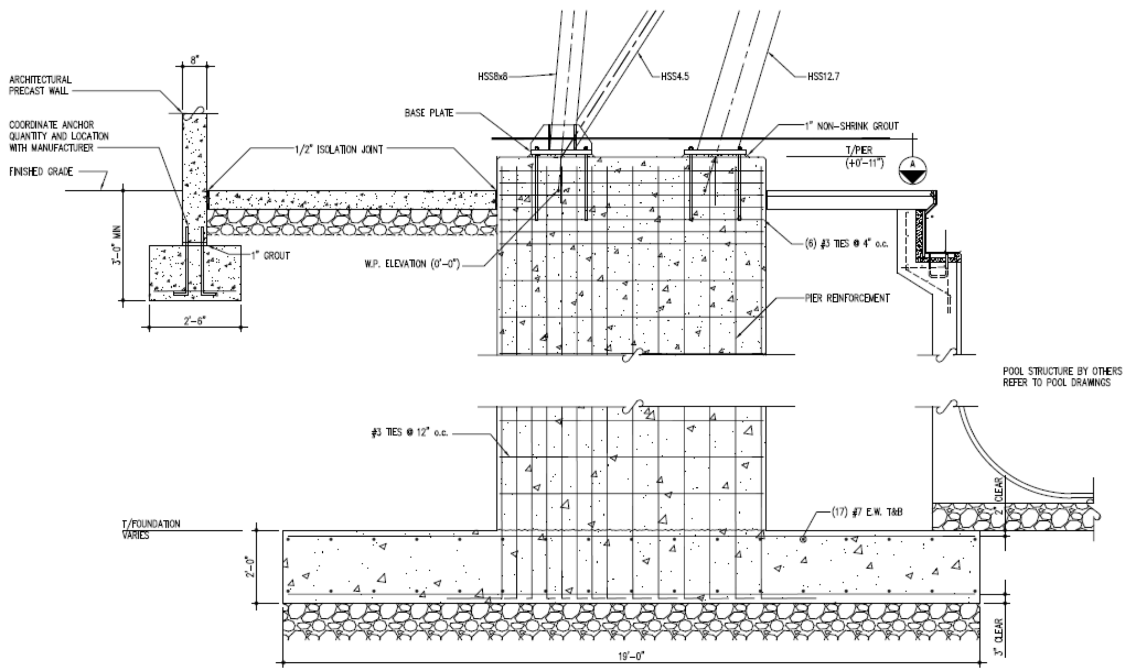


Figure 3 – Detail of Pier Supported Large Tapered Truss Column

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. Strip footing 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.

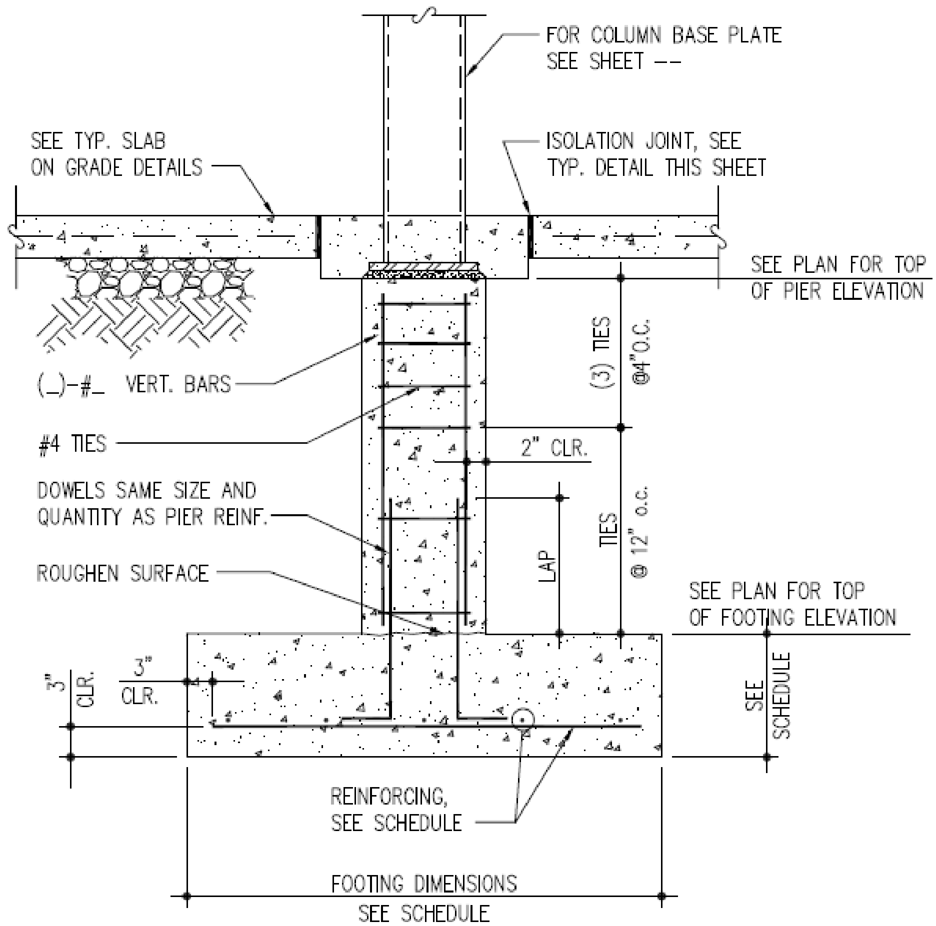


Figure 4 – Typical Pier Detail

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. 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.

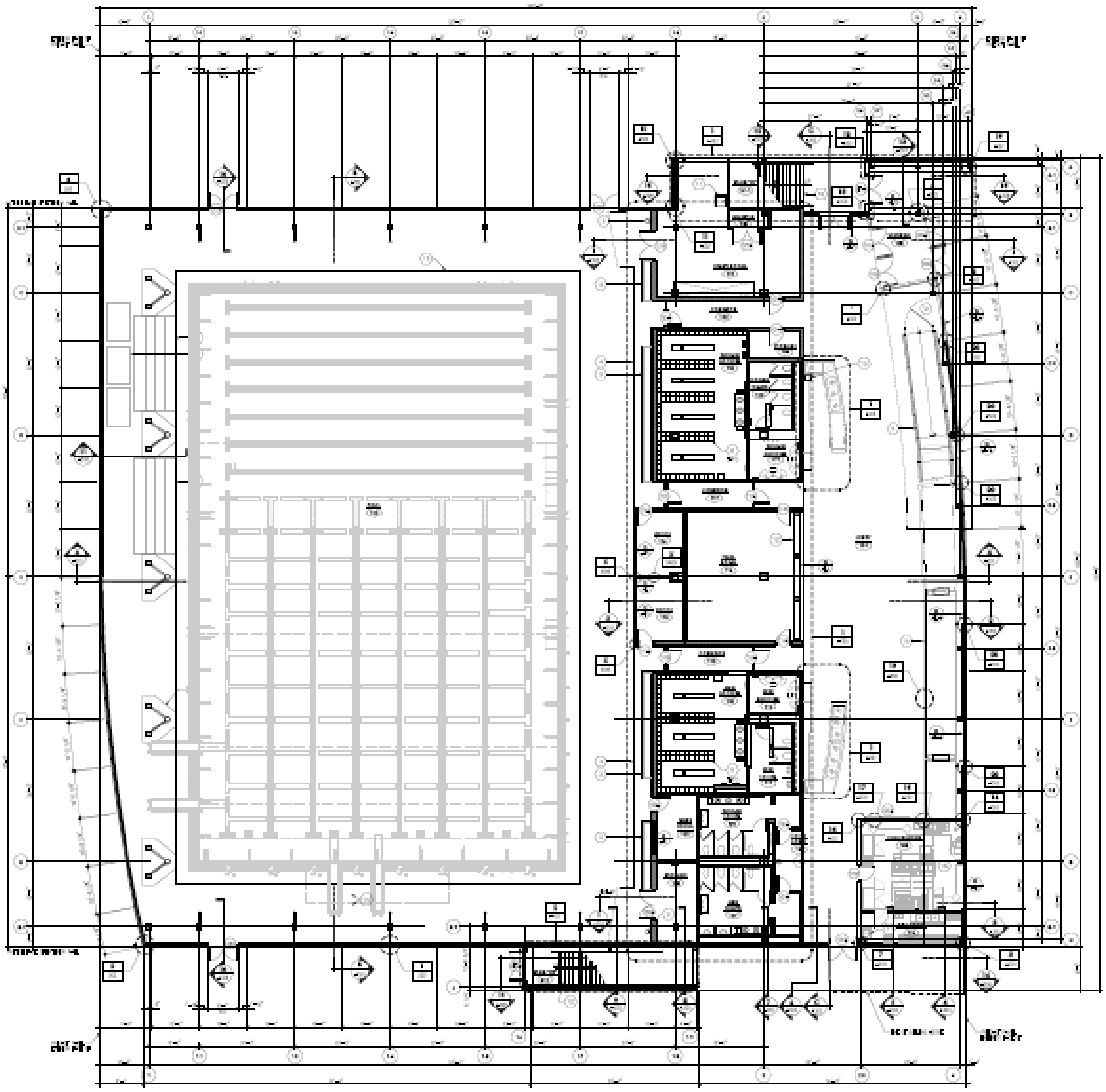


Figure 5 – Entry Level Floor Plan

Triangular HSS trusses spanning 130'-0" support the large curved roof above the indoor swimming pool area. 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 HSS 8x8 members.

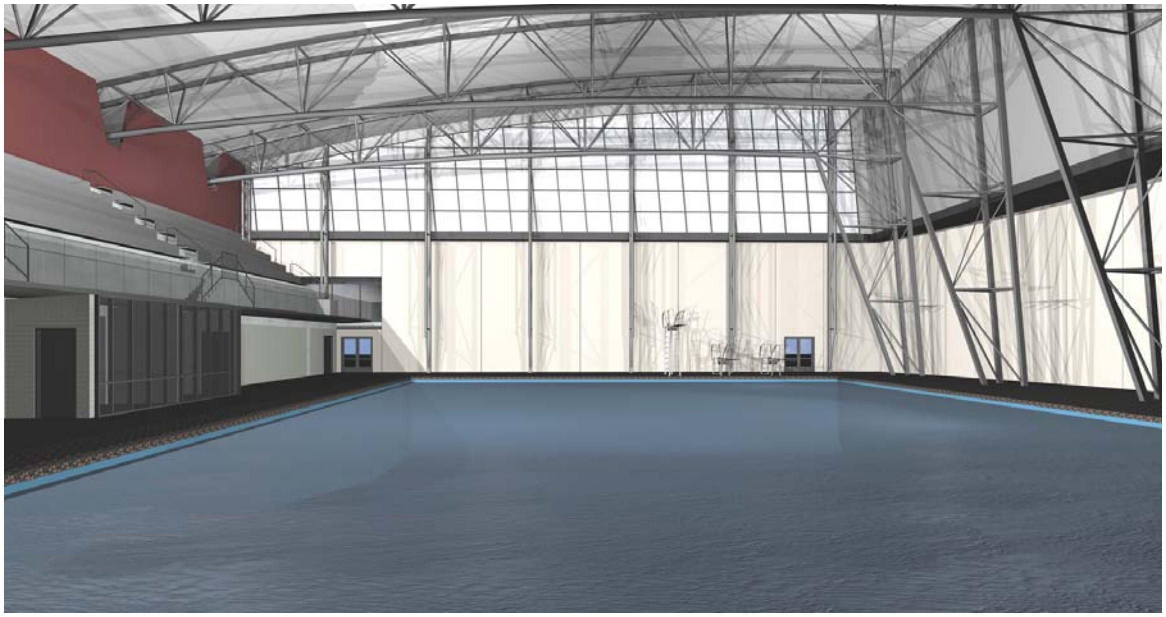


Figure 6 – Rendering of Indoor Pool Area Showing Large Curved Trusses

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 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. 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" 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.

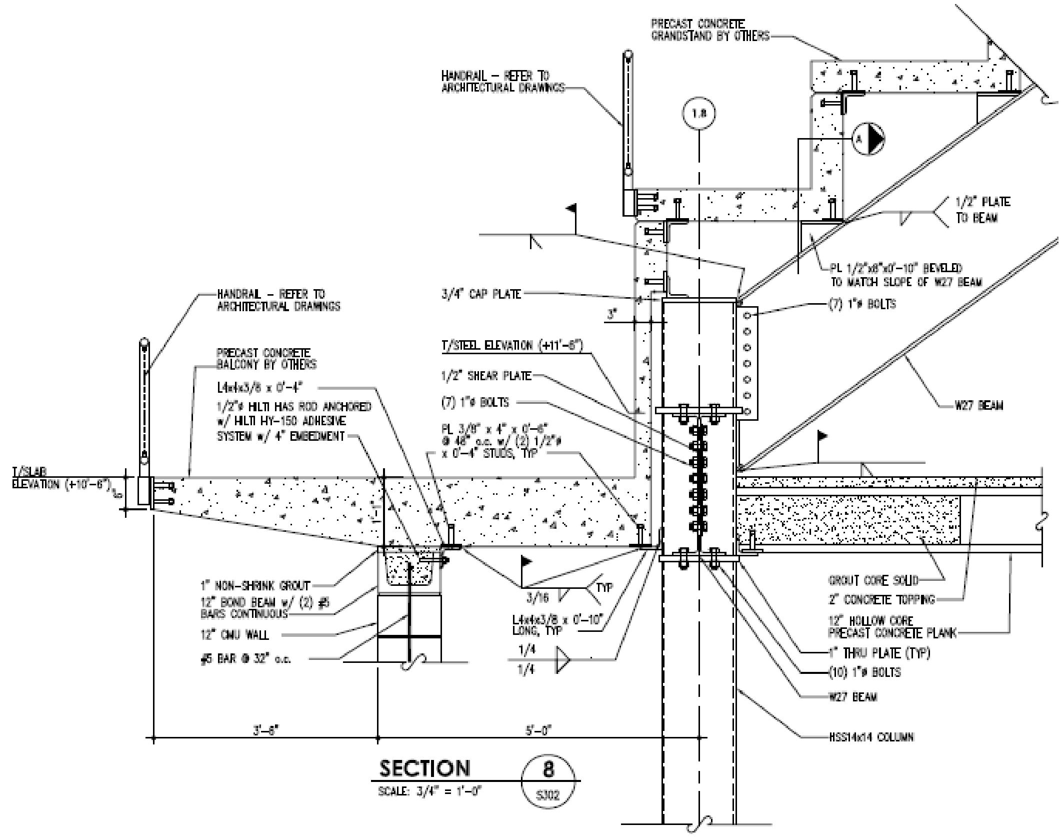


Figure 7 – Section Showing the 12" Hollow Core Precast Concrete Plank, the Precast Concrete Balcony, and the W27x94 Beams Supporting the Concrete Grandstand

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 support 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 one direction, while the truss columns, a frame between the pool and lobby, and frame at the front of the lobby handle the lateral load in the other direction. Some lateral load from the mezzanine goes into the CMU walls, but the steel moment frame provides most of the lateral support. The wind columns are designed to simply take the wind force and transfer it to the roof diaphragm. 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 send part of the load to the five large braced truss columns and the rest of the load to mezzanine moment frame system. The large truss columns are laterally braced by HSS3.500x0.216 X-bracing. The two chord of the truss columns are offset by four feet at the bsae, provided a rather rigid support that can handle high lateral loads.

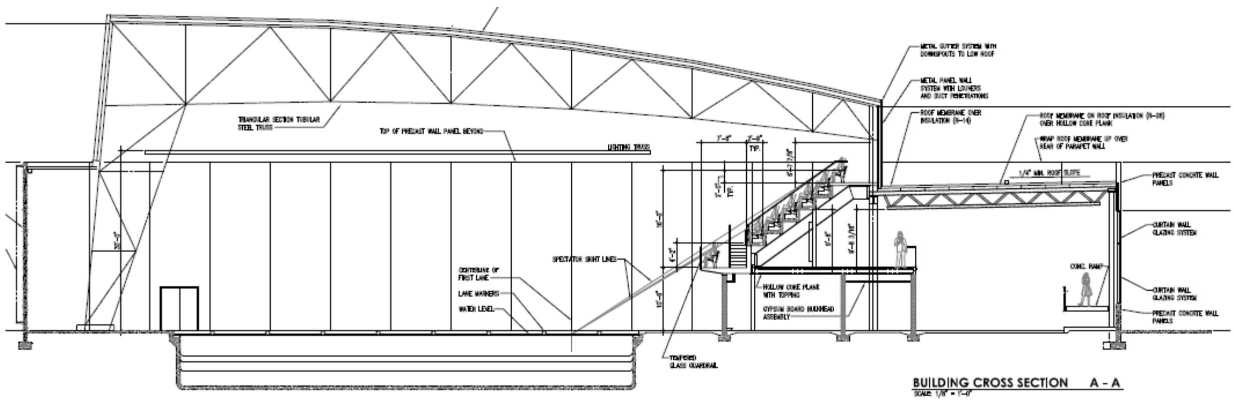


Figure 8 – Building Cross Section

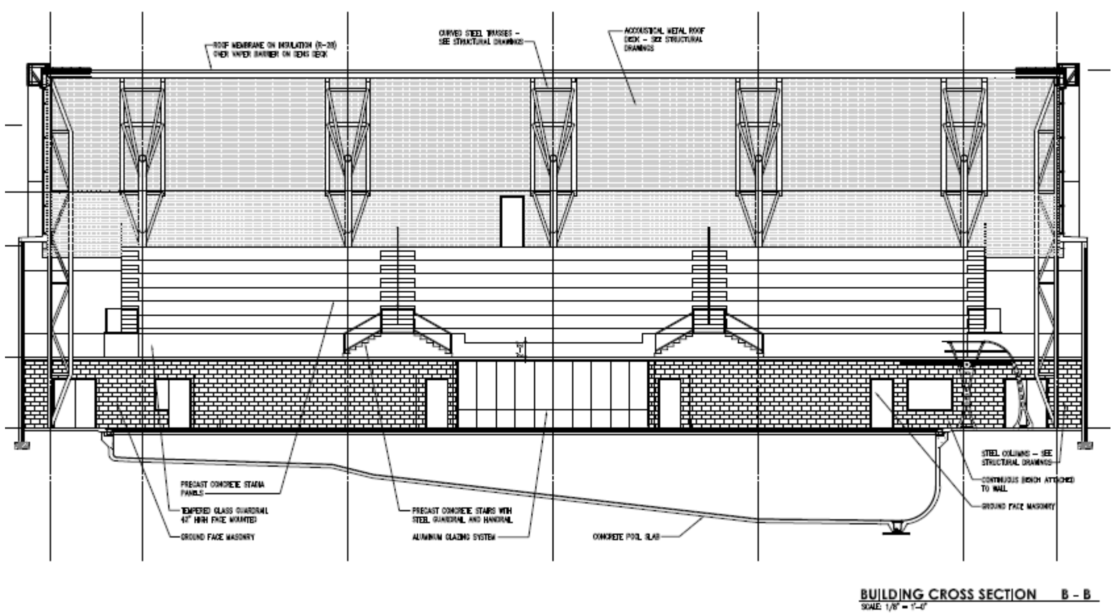


Figure 9 – Building Cross Section Showing the Wind Columns

Codes and Standards

Applied to Original Design:

International Building Code – 2003

"Building Code Requirements for Reinforced Concrete, ACI-318-99", American Concrete Institute

"ACI Manual of Concrete Practice – Parts 1 through 5, 2002", American Concrete Institute

"Manual of Standard Practice", Concrete Reinforcing Steel Institute

"Manual of Steel Construction – Load and Resistance Factor Design", Third Edition, American Institute of Steel Construction (including specification for structural steel buildings, specification for steel hollow structural sections, specification for single-angle members, specification for structural joints using ASTM A325 or A490 bolts, and AISC Code of Standard Practice)

"Hollow Structural Sections Connections Manual", American Institute of Steel Construction

"Detailing for Steel Construction", American Institute of Steel Construction

"Structural Welding Code ANSI/AWS D1.1-98", American Welding Society

"Building Code Requirements for Masonry Structures", (ACI 530-99/ASCE 5-99)

"Specifications for Masonry Structures", (ACI 530.1-99/ASCE 6-99)

Substituted for Thesis Analysis:

International Building Code – 2006

ASCE 7-05

Material Strength Requirement Summary:

Cast-in-Place Concrete

Foundations:	4,000 psi
Slabs on Grade:	4,000 psi
Exposed to Freezing:	4,000 psi
Reinforcing Bars:	60 ksi

Structural Steel

Channels, Angles, and Plates:	36 ksi
Wide Flange Shapes:	50 ksi
Structural Tubing (Rectangular):	46 ksi
Structural Tubing (Round):	42 ksi
Structural Pipe:	35 ksi

Masonry

Compressive Strength:	2,000 psi
Reinforcing Bars:	60 ksi

Building Load Summary

Gravity Loads

Nutec Design Associates, Inc., used the 2003 International Building Code and the American Society of Civil Engineers (ASCE) 7-98 to determine gravity loads, while ASCE 7-05 was used to determine the gravity loads in this report. All reported loads are noted in Table 1.

Gravity Loads			
Description	*Nutec*	*ASCE 7-05*	*Design Value*
Dead (DL)			
Concrete	145 pcf	150 pcf	150 pcf
Live (LL)			
Roofs	30 psf + Drifted Snow	20 psf	20 psf + Drifted Snow
Grandstands	100 psf	100 psf	100 psf
Ramps, Corridor	100 psf	100 psf	100 psf
Mechanical Rooms	100 psf	?	100 psf
Snow (S)			
Snow	21 psf	23.1 psf	23.1 psf

Table 1 – Building Gravity Loads

Building Loads	
Large Trusses and Supporting Columns	146.78 kips
Concrete Grandstand	331.52 kips
Concrete Balcony	129.89 kips
Concrete Ramp	107.04 kips
Hollow Core Concrete Planks	315.71 kips
(2) Stairs at Grandstand	28.48 kips
Concrete Stairs by Lobby	41.97 kips
Roofing	242.02 kips
Wind Column Trusses	30.25 kips
Trusses Above Lobby	22.23 kips
Gallery Level Framing (above lobby)	51.75 kips
Mechanical Unit Support Framing	18.92 kips
Mechanical Units	54.50 kips
Interior Walls (Ground Level)	271.77 kips
Interior Walls (Concourse Level)	179.81 kips
Precast Concrete Panels	1577.84 kips
Roofing above Lobby	304.20 kips
Precast Sill by Wind Trusses	66.89 kips
Roofing along Large Trusses	44.02 kips
Roofing along West Edge	59.21 kips
Columns in Lobby	37.22 kips
Sloped Beams Supporting Concrete Seating Area	9.09 kips
TOTAL	4071.12 kips

Table 2 – Building Loads

Wind Loads

Method 2 – Analytical Procedure of ASCE 7-05 Section 6.5 was used to determine wind loads. The wind analysis from this report shows similar results to those obtained from Nutec's design. Net wind uplift pressure is the only main aspect of wind design that I could really compare to at this time. My value is within 1 psf of the value determined by Nutec. I am currently waiting for more design values, like base shear, to compare to. Variables used in the wind calculation are located in Table 2 and wind loads are noted in Tables 3 and 4.

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	22.337	Eq. 6-15
Equivalent Height of Structure	z	31.8	Table 6-2
Intensity of Turbulence	I_z	0.201	Eq. 6-5
Integral Length Scale of Turbulence	L_z	496.31'	Eq. 6-7
Background Response Factor (North/South)	Q	0.8468	Eq. 6-6
Background Response Factor (East/West)	Q	0.8558	Eq. 6-6
Gust Effect Factor (North/South)	G_f	0.956	Eq. 6-4
Gust Effect Factor (East/West)	G_f	0.966	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 3 – Wind Variables

The maximum uplift wind pressure on the roof that I calculated was -23.45 psf (for the East/West Direction). The dead weight of the roof that I calculated came out to be 13.15 psf. Hence, the net uplift wind pressure when I subtract the dead weight from the maximum uplift wind pressure is 10.3 psf.

23.45 psf – 13.15 psf = 10.3 psf

Nutec Design Associates, Inc. calculated a net uplift wind pressure of 10 psf. Therefore, my maximum net uplift pressure almost exactly matches that calculated by Nutec Design Associates, Inc. This can explain that there are relatively minor differences between ASCE 7-05 and IBC – 2003 in the category of wind load design.

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							
4	53.0	25.0	1.102	22.34	20.42	-14.27	-18.97	-23.24	34.69	161.37	274.14	161.37	274.14	8552.50	14529.35
3	28.0	14.0	0.964	19.54	18.37	-14.27	-18.97	-23.24	32.63	145.14	257.91	306.51	532.05	8582.21	14897.37
2	14.0	14.0	0.85	17.23	16.67	-14.27	-18.97	-23.24	30.94	131.73	244.50	438.24	776.55	6135.37	10871.74
1	0.0	0.0	0.00	0.00	0.00	0.00	-18.97	-23.24	0.00	0.00	0.00	438.24	776.55	0.00	0.00
sum(Story Shear (Windward))=1344.36 k							sum (Story Shear (Total))=2359.29 k								
sum(Moment (Windward))=23270.08 ft-k							sum (Moment (Total))=40298.46 ft-k								

Table 4 – Wind Loads (North/South Direction)

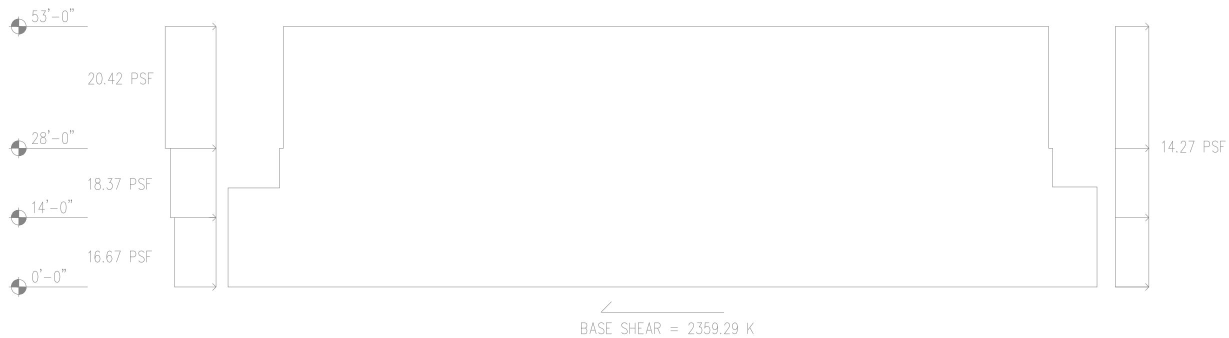


Figure 10 – North/South Wind Pressures

Wind Loads (East/West 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							
4	53.0	25.0	1.102	22.34	20.50	-13.61	-19.13	-23.45	34.11	175.46	291.92	175.46	291.92	9299.32	15471.67
3	28.0	14.0	0.964	19.54	18.44	-13.61	-19.13	-23.45	32.05	157.80	274.25	333.25	566.17	9331.11	15852.84
2	14.0	14.0	0.85	17.23	16.74	-13.61	-19.13	-23.45	30.35	143.20	259.66	476.46	825.84	6670.40	11561.70
1	0.0	0.0	0.00	0.00	0.00	0.00	-19.13	-23.45	0.00	0.00	0.00	476.46	825.84	0.00	0.00
sum(Story Shear (Windward))=1461.63 k							sum (Story Shear (Total))=2509.76 k								
sum(Moment (Windward))=25300.84 ft-k							sum (Moment (Total))=42886.21 ft-k								

Table 5 – Wind Loads (East/West Direction)

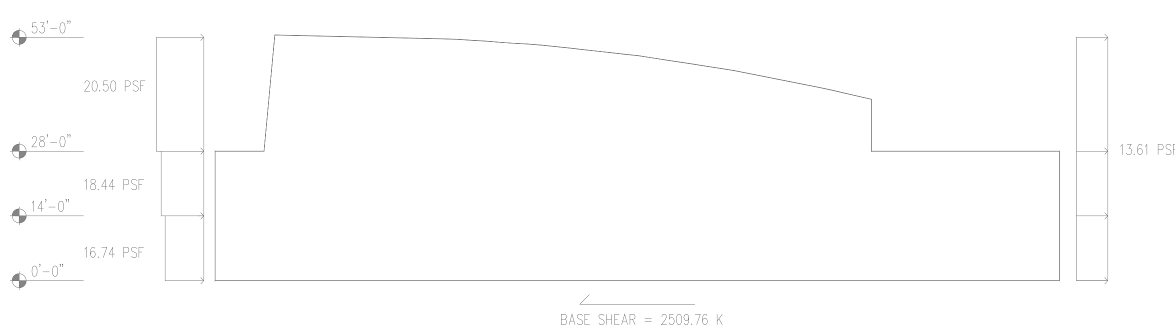


Figure 11 – East/West Wind Pressures

When performing initial wind load calculations for the Farquhar Park Aquatic Center natatorium, the building was considered to be rigid. The building has a steel moment resisting frame system, however some walls take lateral load as well. Therefore, Equations C6-17 and C6-18 from ASCE 7-05 Commentary for Wind Loads was used to determine the value of n_1 (n_1 = H/1000 = average value and n_1 = H/75 = lower bound value). These equations are applicable to all building in steel and concrete, and hence the equations were used to find that the building is rigid. However, after later hearing from Nutec, I found out the building is flexible and that steel moment frames provide most of the lateral support. Therefore, Equation C6-14 was used to find n_1 and it was found to be less than 1 Hz, hence meaning that the building was flexible. Then values for G_f were found, and calculations continued, eventually ending up with a maximum base shear of 2509.76 kips.

Seismic Loads

Chapters 11 and 12 from ASCE 7-05 were used to calculate the seismic loads on the Farquhar Park Aquatic Center natatorium. The equivalent lateral force method was used for the analysis, and the seismic design variables used in the calculations are located in Table 6. The base shear that was calculated (264.25 kips) is fairly close to the base shear calculated by Nutec Design Associates, Inc. (300 kips). Variations in base shear could be due to differences in the ways we calculated the weight of the building, for it is somewhat difficult to account for the weight of every single part. Estimates are often required for this analysis, which could easily result in deviations between final results calculated by two different people.

Seismic Design Variables			**ASCE Reference**
Site Classification		B	
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	0.028	Table 12.8-2
Building Height (above grade)	h_n	53 ft	
Approximate Period Parameter	x	0.8	Table 12.8-2
Approximate Fundamental Period	T_a	0.671 sec	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	1.140 sec	
Seismic Response Coefficient	C_s	0.038	Eq. 12.8-2
Structure Period Exponent	k	1.0	

Table 6 – Seismic Design Variables

Seismic Loads								
Level	**Story Weight w_x (kips)**	**Height h_x (ft)**	**h_x^k**	**$w_x h_x^k$**	**C_{vx}**	**Lateral Force F_x (kips)**	**Story Shear V_x (kips)**	**Moments M_x (ft-k)**
4	439.95	53.00	53.00	23317.49	0.336	88.82	0.00	4707.37
3	1094.54	24.00	24.00	26268.84	0.379	100.06	88.82	2401.45
2	1884.53	10.50	10.50	19787.59	0.285	75.37	188.88	791.41
1	649.10	0.00	0.00	0.00	0.000	0.00	264.25	0.00
sum($w_i h_i^k$)=	**69373.92**	**sum(F_x)=V=**	**264.25 kips**				**sum(M_x)=**	7900.23
Total Building Weight (Above Grade) = 4071.12 kips								

Table 7 – Seismic Loads

$$V = CsW = (0.06491)(4071.12 \text{ kips}) = 264.25 \text{ kips}$$

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

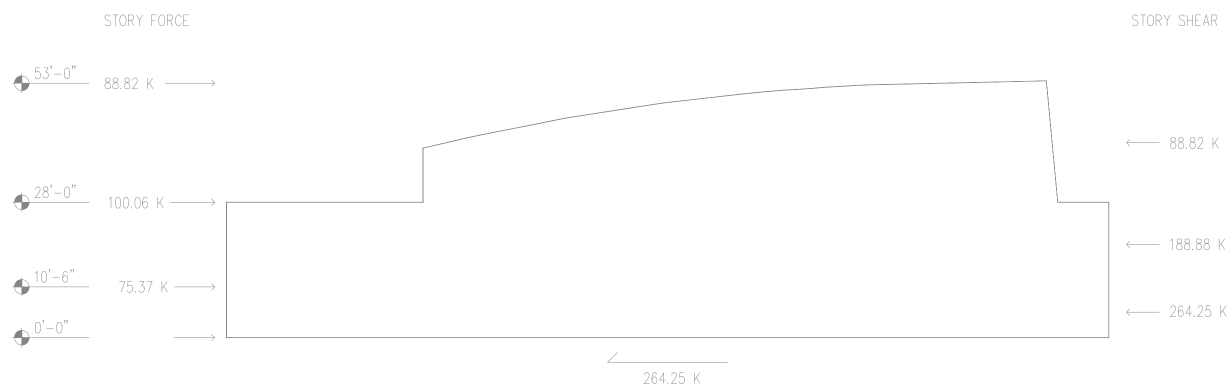


Figure 12 – Seismic Loading on Natatorium

Spot Checks

Spot checks of the large trusses spanning over the indoor swimming pool and of the smaller trusses spanning over the lobby and supporting the lower roof were performed to check the adequacy of their designs. Loads were calculated based on ASCE 5-07 and appropriately applied at the joints. STAAD was used to determine the internal axial forces of each member. These axial forces were then compared to those calculated by Nutec Design Associates, Inc. Overall the results were fairly close to those determined by Nutec. Differences may be due to the fact that Nutec used a roof live load of 30 psf, whereas ASCE 7-05 suggested a roof live load of 20 psf. Plus, this analysis only accounted for gravity loads, whereas Nutec accounted for lateral loads in addition to gravity loads.

Large Truss

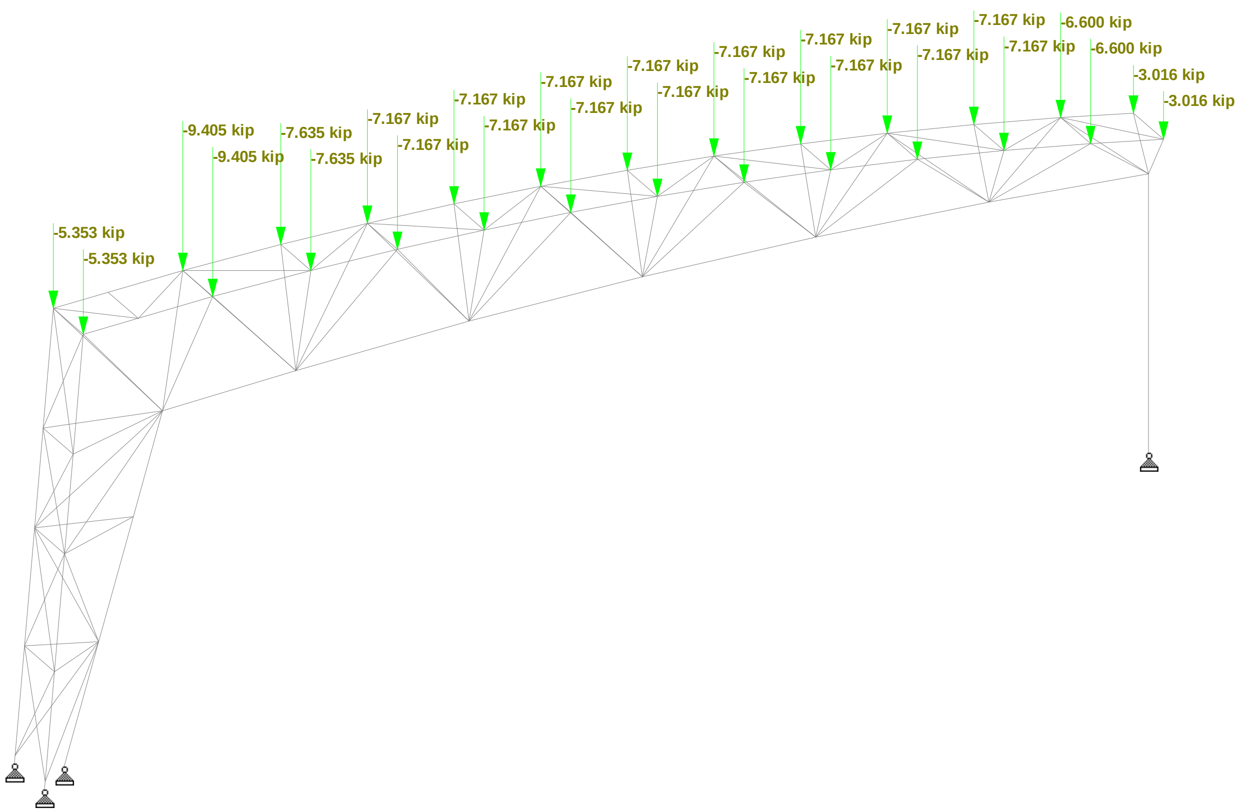


Figure 13 – Large Truss Modeled in STADD with Loads Applied

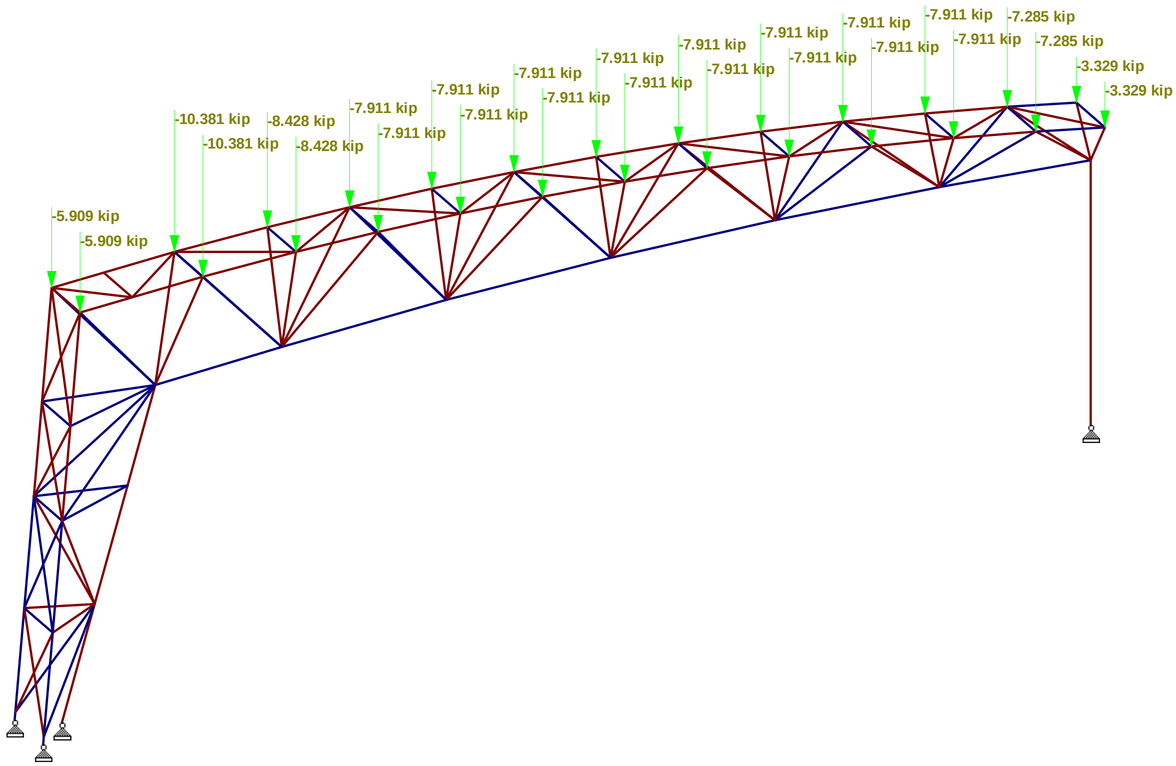


Figure 14 – Large Truss Modeled in STAAD and Showing Compressive vs Tensile Axial Forces

Red = Compression

Blue = Tension

The results of the STAAD analysis were fairly close to the axial forces determined by Nutec Design Associates, Inc. Differences between results, again, are probably due to the use of different loads for the building. This report did not account for lateral loads when performing spot checks, whereas Nutec's design would have accounted for lateral forces as well as gravity forces.

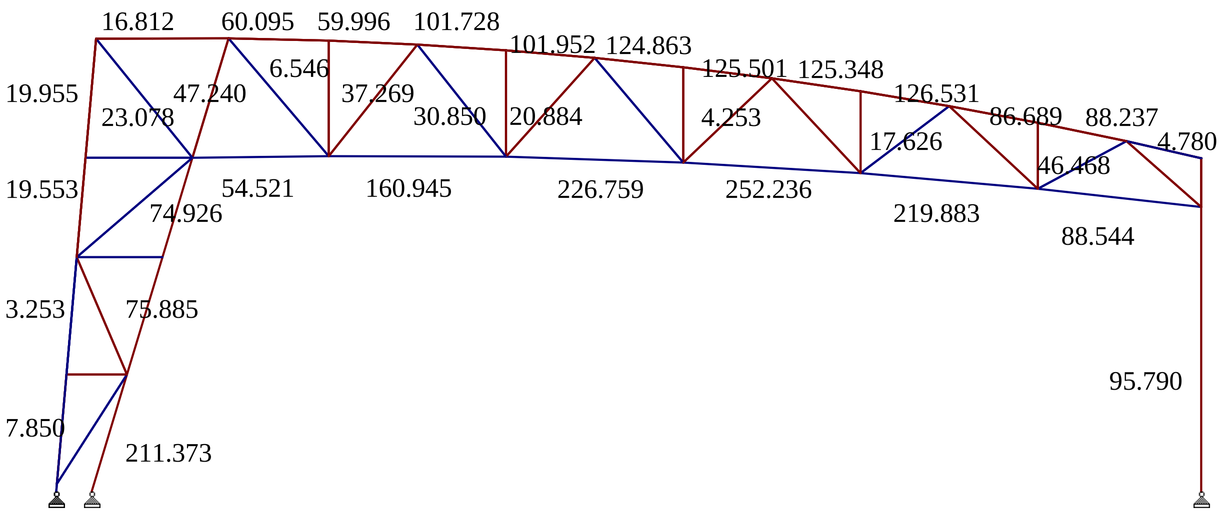


Figure 15 – Axial Forces from STADD (in kips)

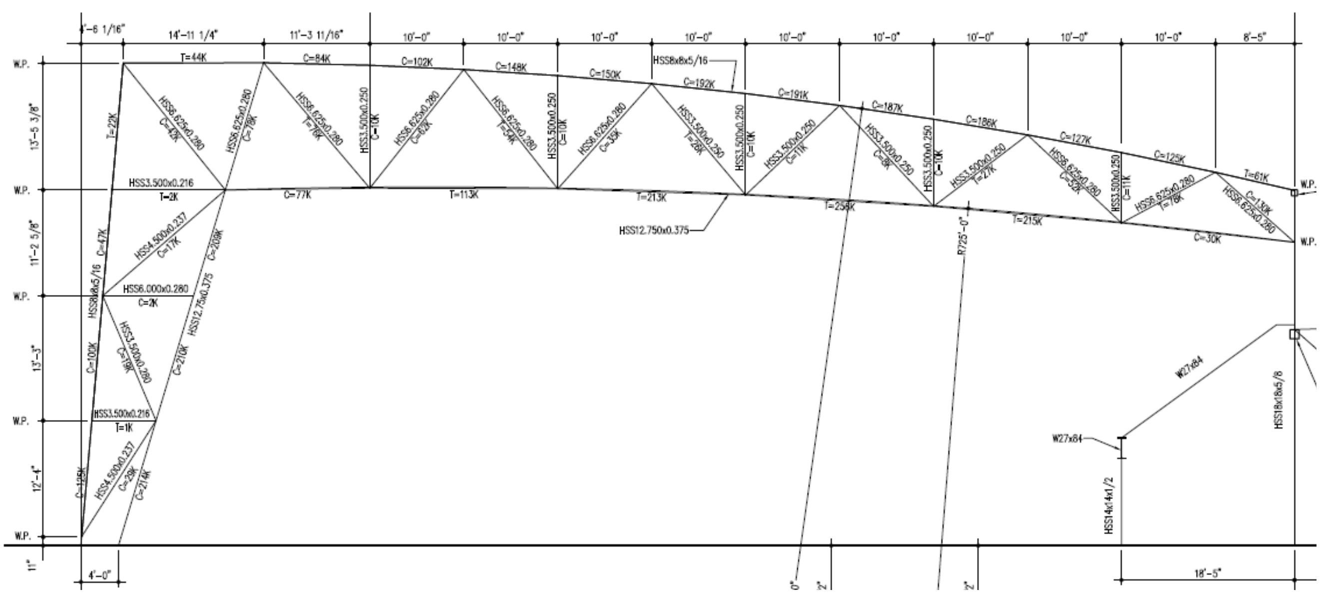


Figure 16 – Axial Forces Determined by Nutec for Large Truss

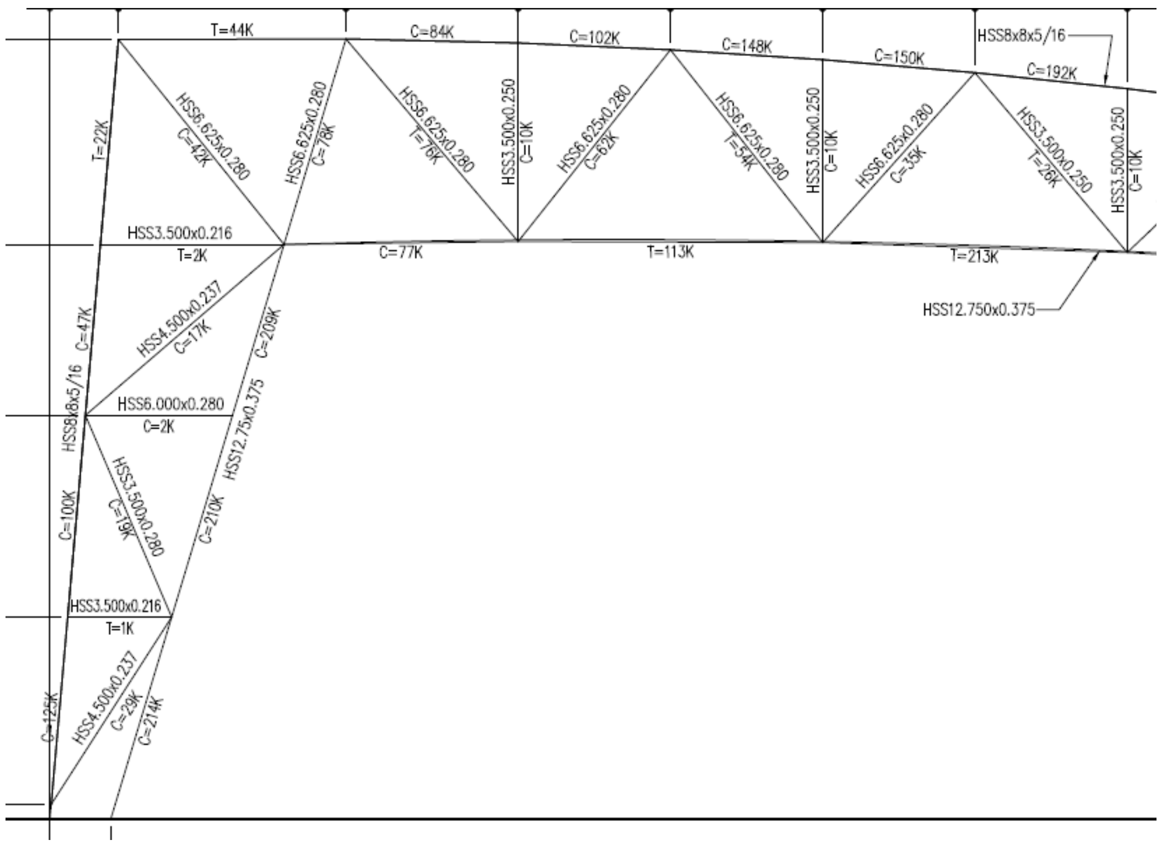


Figure 17 – Close-up View of Axial Forces in Large Truss Determined by Nutec

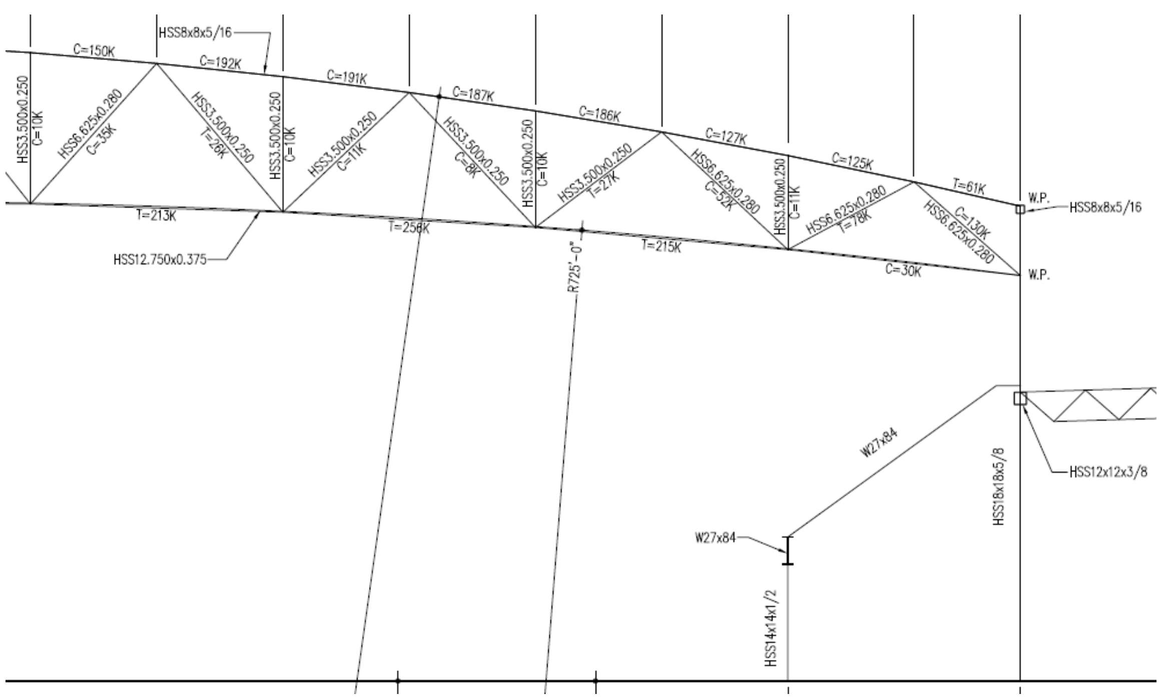


Figure 18 – Close-up view of Axial Forces in Large Truss Determined by Nutec

Truss Above Lobby (Smaller Truss)

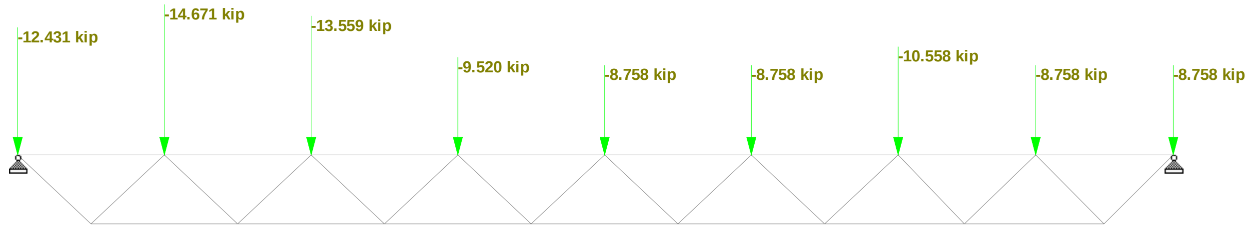


Figure 19 – STAAD Model of Smaller Truss Showing Loading Diagram

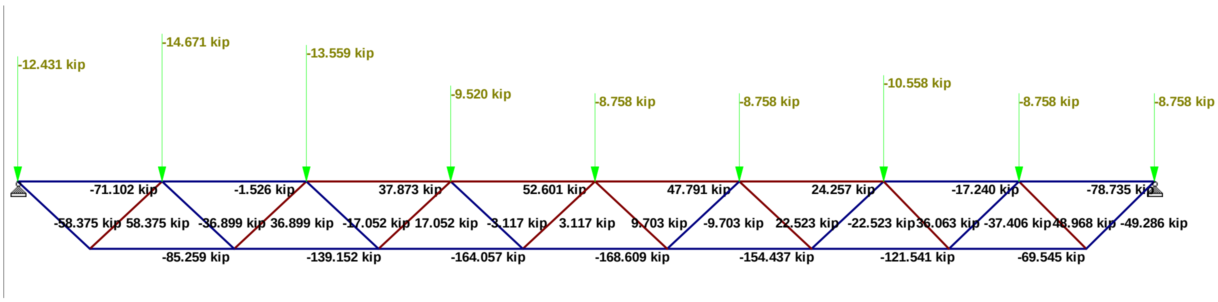


Figure 20 – STAAD Model Showing Compressive vs Tensile Axial Forces (Axial Forces Indicated in kips)

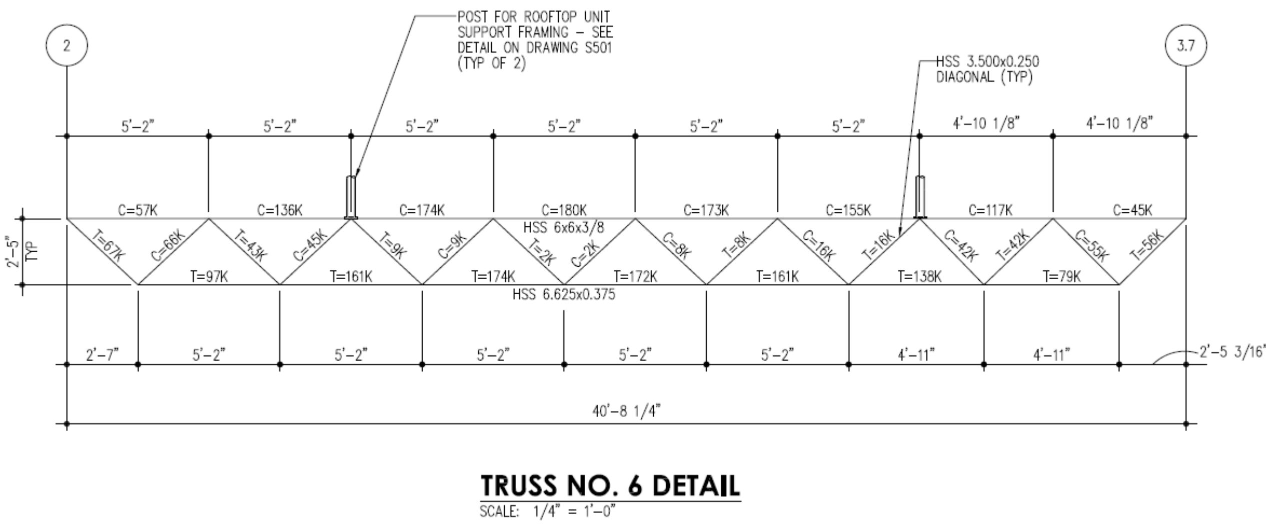


Figure 21 – Axial Forces Determined By Nutec for Truss No. 6

The axial forces in the bottom chord and web members from STAAD seem fairly similar to the bottom chord axial forces and web member axial forces calculated by Nutec. However, there seems to be large variations in the results for the top chord forces from those determined by Nutec. Perhaps this is due to differences in the way the truss was modeled. Also, this report does not account for lateral loads when performing spot checks, whereas Nutec would have accounted for lateral loads and gravity loads in their design.

Conclusion

After analyzing the Farquhar Park Aquatic Center natatorium and performing gravity and lateral load calculations, it was determined that the building was adequately designed to carry the required loads. Following the procedures described in ASCE 7-5, wind loads were calculated using Method 2 and the resulting net wind uplift on the roof was found to be 10.3 kips, which almost exactly matched Nutec's design value of 10 psf (the difference between the two was only 0.3 psf). The base shear due to wind following ASCE 7-5 procedures was found to be 2509.76 kips, which was controlled by the East/West wind direction. Nutec's base shear results are not yet readily available to compare this value to. The seismic load according to ASCE 7-05 was found to cause a base shear of 264.25 kips, which is somewhat comparable to Nutec's design value of 300 kips. Differences are most likely due to variations in building weight calculations. The 300 kips determined by Nutec may also be a rounded value, or perhaps a lower base shear was calculated but was just bumped up to 300 kips to be conservative. Overall, the calculated design values according to ASCE 7-05 were very close to those determined by Nutec.

Spot checks on a large roof truss and a lower roof truss also showed that the building was adequately designed. Results between the axial forces determined from the spot checks and Nutec's axial forces were fairly similar. Some forces were within a few kips of each other while others some were more than 40 kips apart. This may be due to the fact that the analysis for this report did not account for lateral loads when performing the spot checks. There may have also been variations in ways to account for the loading on the lower roof trusses due to the mechanical unit support framing and mechanical units on top of the roofing material itself. Several estimates and assumptions were required for the calculations in this report. However, the results determined using ASCE 7-05 were generally very close to those calculated by Nutec.

Appendix A – Gravity Loads

Snow Load

Ground Snow Load $\rightarrow$ $p_g = 30$ psf (Figure 7-1)

Flat Roof Snow Load $\rightarrow$ $p_f = 0.7\, C_e\, C_t\, I\, p_g > 20\,(I)$

C_e = Exposure Factor = 1.0 (Table 7-2)

Terrain Category C, Exposure of Roof Partially Exposed

C_t = Thermal Factor = 1.0 (Table 7-3)

I = Importance Factor = 1.1 (Table 7-4)

Occupancy Category III

$p_f = (0.7)(1.0)(1.0)(1.1)(30 \text{ psf}) = 23.1 \text{ psf} > 20(1.1) = 22 \text{ psf}$

$\therefore\ p_f = 23.1 \text{ psf}$

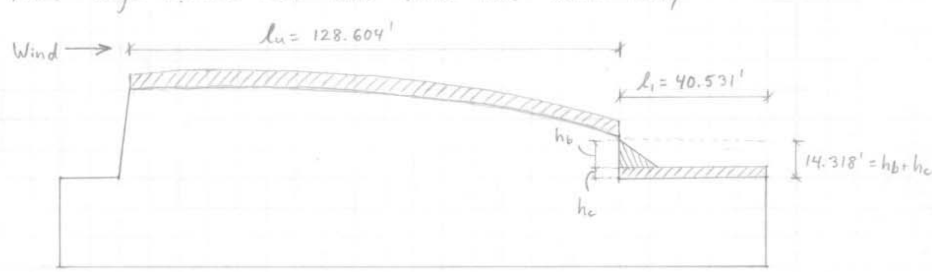
Sloped Roof Snow Load $\rightarrow$ $p_s = C_s\, p_f$

Use $C_s = 1.0$ since slope of large roof is nearly flat toward highest point (only 1-2° slope) $\rightarrow$ worst case scenario

$p_s = (1.0)(23.1 \text{ psf}) = 23.1 \text{ psf}$

Snow Drift

From large curved roof onto lower roof above lobby



7.7.1 (ASCE 7-05)

γ = Snow Density = $0.13\, p_g + 14 = (0.13)(30 \text{ psf}) + 14 = 17.9 \text{ pcf} < 30 \text{ psf}$

$\therefore\ \gamma = 17.9 \text{ pcf}$

$h_b = \frac{p_s}{\gamma} = \frac{23.1 \text{ psf}}{17.9 \text{ pcf}} = 1.291'$ $\leftarrow$ height of balanced snow load

$h_c = 14.318' - 1.291' = 13.027'$

$\frac{h_c}{h_b} = \frac{13.027'}{1.291'} = 10.095 > 0.2$ ∴ Drift calculation is required

$l_u = 128.604'$ ← length of the roof upwind from the drift

$l_l = 40.531'$ ← length of lower roof

$h_d = [0.43\sqrt[3]{l_u}\sqrt[4]{p_g+10} - 1.5$ ← height of snow drift

Leeward Side Drift Height → $h_d = [0.43\sqrt[3]{128.604'}\sqrt[4]{30+10}] - 1.5 = 3.958'$

Windward Side Drift Height → $h_d = [0.43\sqrt[3]{40.531}\sqrt[4]{30+10} - 1.5] = 2.215'$

$h_d = (0.75)(2.215) = 1.661'$

Width of Snow Drift (using highest value of h_d)

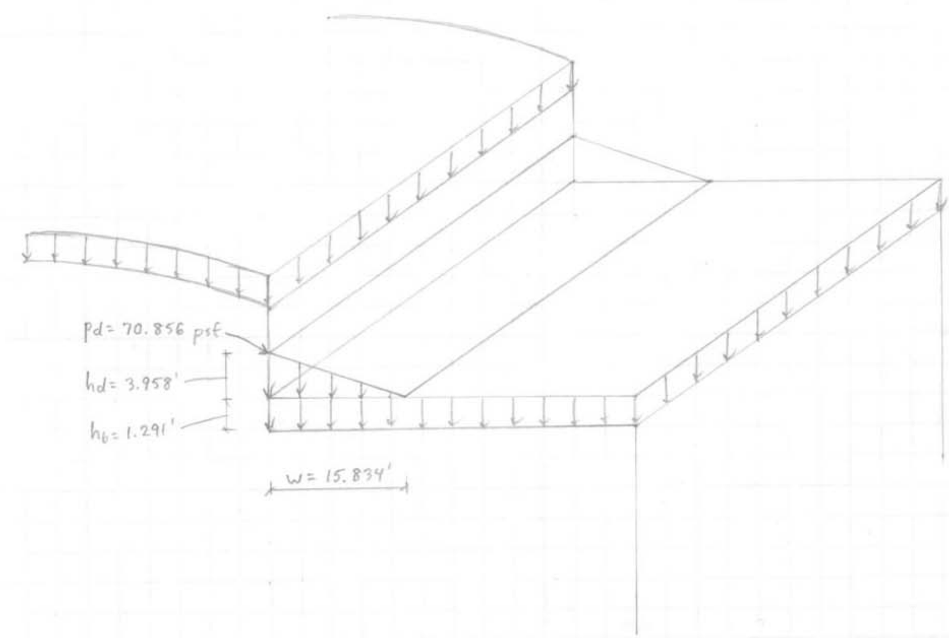
If $h_d \leq h_c$, then $w = 4h_d$ and drift height $= h_d$

$h_d = 3.958' < 13.027' = h_c$ ∴ $w = 4h_d = (4)(3.958') = 15.834'$

$w = 15.834' < 8h_c = (8)(13.027') = 104.218'$

Drift Load → p_d = maximum intensity of drift surcharge load

$p_d = h_d\gamma = (3.958')(17.9\ pcf) = 70.856\ psf$



Appendix B – Wind Calculations

Wind Calculations

Method 2 – Analytical Procedure

Building Natural Frequency → n_1

For steel Moment-Resisting-Frames → $n_1 = \frac{22.2}{H^{0.8}}$

H = Building Height = 53'

$n_1 = \frac{22.2}{(53)^{0.8}} = \boxed{0.927 \text{ Hz}} < 1 \text{ Hz}$ ∴ Structure is flexible

$\boxed{g_Q = g_v = 3.4}$

$g_R = \sqrt{2\ln(3{,}600\,n_1)} + \frac{0.577}{\sqrt{2\ln(3600\,n_1)}} = \sqrt{2\ln[3600(0.927)]} + \frac{0.577}{\sqrt{2\ln[3600(0.927)]}} = \boxed{4.171}$

$\bar{z} = 0.6h = (0.6)(53') = \boxed{31.8'} > z_{min} = 15'$ (Table 6-2) (Exposure C)

Use maximum roof height (most conservative) instead of trying to estimate mean roof height of curved roof

$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = (0.20)\left(\frac{33}{31.8}\right)^{1/6} = \boxed{0.201}$

from Table 6-2, Exposure C

$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = (500')\left(\frac{31.8'}{33}\right)^{1/5.0} = \boxed{496.309}$

from Table 6-2, Exposure C

$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$

North / South:

B = 183'

L = 156'

$Q_{N/S} = \sqrt{\frac{1}{1 + 0.63\left(\frac{183' + 53'}{496.309'}\right)^{0.63}}} = \boxed{0.8468}$

East / West:

B = 156'

L = 183'

$Q_{E/W} = \sqrt{\frac{1}{1 + 0.63\left(\frac{156' + 53'}{496.309'}\right)^{0.63}}} = \boxed{0.8558}$

$\boxed{V = 90 \text{ m.p.h.}}$ (Figure 6-1)

$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) = (0.65)\left(\frac{31.8'}{33}\right)^{1/6.5}(90)\left(\frac{88}{60}\right) = \boxed{85.312 \text{ mph}}$

from Table 6-2, Exposure C

$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{(0.927)(496.309)}{85.312} = \boxed{5.391}$

$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}} = \frac{(7.47)(5.391)}{[1 + (10.3)(5.391)]^{5/3}} = \boxed{0.0484}$

$R_h = \frac{1}{n} - \frac{1}{2n^2}\left(1 - e^{-2n}\right)$ for $n > 0$

$n = \frac{4.6 n_1 h}{\bar{V}_{\bar{z}}} = \frac{(4.6)(0.927)(53)}{85.312} = 2.648 > 0$

$R_h = \frac{1}{2.648} - \frac{1}{(2)(2.648)^2}\left(1 - e^{-2(2.648)}\right) = \boxed{0.307}$

$R_B = \frac{1}{n} - \frac{1}{2n^2}\left(1 - e^{-2n}\right)$ for $n > 0$

North / South:

$n = \frac{4.6 n_1 B}{\bar{V}_{\bar{z}}} = \frac{(4.6)(0.927)(183)}{85.312} = 9.144 > 0$

$R_{B(N/S)} = \frac{1}{9.144} - \frac{1}{2(9.144)^2}\left(1 - e^{-2(9.144)}\right) = \boxed{0.103}$

East / West:

$n = \frac{4.6 n_1 B}{\bar{V}_{\bar{z}}} = \frac{(4.6)(0.927)(156)}{85.312} = 7.795 > 0$

$R_{B(E/W)} = \frac{1}{7.795} - \frac{1}{2(7.795)^2}\left(1 - e^{-2(7.795)}\right) = \boxed{0.120}$

$R_L = \frac{1}{n} - \frac{1}{2n^2}\left(1 - e^{-2n}\right)$ for $n > 0$

North / South:

$n = \frac{15.4 n_1 L}{\bar{V}_{\bar{z}}} = \frac{(15.4)(0.927)(156)}{85.312} = 26.095 > 0$

$R_{L(N/S)} = \frac{1}{26.095} - \frac{1}{2(26.095)^2}\left(1 - e^{-2(26.095)}\right) = \boxed{0.0376}$

East / West:

$n = \frac{15.4 n_1 L}{\bar{V}_{\bar{z}}} = \frac{(15.4)(0.927)(183)}{85.312} = 30.612 > 0$

$R_{L(E/W)} = \frac{1}{30.612} - \frac{1}{(2)(30.612)^2}\left(1 - e^{-2(30.612)}\right) = \boxed{0.0321}$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)}$$ ← $\beta = 0.01$ for steel buildings

North / South:

$$R_{(N/S)} = \sqrt{\left(\frac{1}{0.01}\right)(0.0484)(0.307)(0.103)[0.53 + 0.47(0.0376)]} = \boxed{0.290}$$

East / West:

$$R_{(E/W)} = \sqrt{\left(\frac{1}{0.01}\right)(0.0484)(0.307)(0.120)[0.53 + 0.47(0.0321)]} = \boxed{0.312}$$

$$G_f = 0.925\left(\frac{1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right)$$

North / South:

$$G_{f(N/S)} = 0.925\left(\frac{1 + 1.7(0.201)\sqrt{(3.4)^2(0.8468)^2 + (4.171)^2(0.290)^2}}{1 + 1.7(3.4)(0.201)}\right) = \boxed{0.956}$$

East / West:

$$G_{f(E/W)} = 0.925\left(\frac{1 + 1.7(0.201)\sqrt{(3.4)^2(0.8558)^2 + (4.171)^2(0.312)^2}}{1 + 1.7(3.4)(0.201)}\right) = \boxed{0.966}$$

Velocity Pressure

$V = 90$ mph

$K_d = 0.85$ (Table 6-4)

$I = 1.15$ (Table 6-1) (Occupancy Category III)

Exposure Category → C

$K_{zt} = 1.0$

Level	Height	K_z ← (Table 6-3) (Exposure C)
1	0'	0
2	14'	0.85
3	28'	0.964 ← from interpolation
4	53'	1.102

$K_h = 1.102$

$q_z = 0.00256 K_z K_{zt} K_d V^2 I$

Level 1 → $q_z = 0.00256(0)(1.0)(0.85)(90)^2(1.15) = 0$

Level 2 → $q_z = 0.00256(0.85)(1.0)(0.85)(90)^2(1.15) = 17.229$

Level 3 → $q_z = 0.00256(0.964)(1.0)(0.85)(90)^2(1.15) = 19.540$

Level 4 → $q_z = 0.00256(1.102)(1.0)(0.85)(90)^2(1.15) = 22.337 = q_h$

$q_h = 0.00256\, K_h K_{zt} K_d V^2 I = 0.00256\,(1.102)(1.0)(0.85)(90)^2(1.15) = 22.337$

Level 4

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 → $\frac{L}{B} = \frac{156'}{183'} = 0.852$ → $C_p = -0.5$

Side Wall → $C_p = -0.7$

East / West:

Windward Wall → $C_p = 0.8$

Leeward Wall → $\frac{L}{B} = \frac{183'}{156'} = 1.173$ → $C_p = -0.4654$ (interpolation)

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 θ<10 and Parallel to ridge for all θ"

North / South:

$\frac{h}{L} = \frac{53'}{156'} = 0.3397 < 0.5$

Horizontal Distance from Windward Edge	C_p
0 to h/2	−0.9, −0.18
h/2 to h	−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:

$\frac{h}{L} = \frac{53'}{183'} = 0.2896 < 0.5$

Same chart (above, for North / South) applies

Use worst case scenario → $C_p = -0.9$ for entire roof

Internal Pressure Coefficients (GC_{pi}) (Figure 6-5)

Enclosed Buildings → $GC_{pi} = +0.18$
-0.18

Design Wind Pressures

Windward Walls → $p_z = q_z G_f C_p - q_h (GC_{pi})$

Leeward Walls, Side Walls, and Roofs → $p_h = q_h G_f C_p - q_h (GC_{pi})$

North / South:

Windward Walls → $p_z = (q_z)(0.956)(0.8) - (22.337)(-0.18)$

$= \boxed{0.765\,(q_z) + 4.021 \text{ psf}}$

(Varies by level → See Table)

Leeward Walls → $p_h = (22.337)(0.956)(-0.5) - (22.337)(0.18) =$

$= \boxed{-14.699 \text{ psf}}$

Side Walls → $p_h = (22.337)(0.956)(-0.7) - (22.337)(0.18) =$

$= \boxed{-18.970 \text{ psf}}$

Roof → $p_h = (22.337)(0.956)(-0.9) - (22.337)(0.18) = \boxed{-23.242 \text{ psf}}$

East / West

Windward Walls → $p_z = (q_z)(0.966)(0.8) - (22.337)(-0.18)$

$= \boxed{0.773\,(q_z) + 4.021 \text{ psf}}$

(Varies by level → See Table)

Leeward Walls → $p_h = (22.337)(0.966)(-0.4654) - (22.337)(0.18) =$

$= \boxed{-14.065 \text{ psf}}$

Side Walls → $p_h = (22.337)(0.966)(-0.7) - (22.337)(0.18) =$

$= \boxed{-19.129 \text{ psf}}$

Roof → $p_h = (22.337)(0.966)(-0.9) - (22.337)(-0.18) = \boxed{23.446 \text{ psf}}$

* Forces, Base Shears, and Moments are shown in spreadsheet

Appendix C – Seismic Calculations

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)

$S_{MS} = F_a S_S = (1.2)(0.20) = 0.24$ (Eq. 11.4-1)

$S_{M1} = F_v S_1 = (1.7)(0.054) = 0.0918$ (Eq. 11.4-2)

$S_{DS} = \frac{2}{3} S_{MS} = (\frac{2}{3})(0.24) = 0.16$ (Eq. 11.4-3)

$S_{D1} = \frac{2}{3} S_{M1} = (\frac{2}{3})(0.0918) = 0.0612$ (Eq. 11.4-4)

Seismic Design Category based on S_{DS} (Table 11.6-1)

$S_{DS} < 0.167$, Occupancy Category III $\rightarrow$ SDC A

Seismic Design Category based on S_{D1}

$S_{D1} < 0.067$, Occupancy Category III $\rightarrow$ SDC A

Use most severe of the two Seismic Design Categories (same in this case)

Seismic Design Category $\rightarrow \boxed{A}$

Could use methods of 11.7 $\rightarrow$ Design Requirements for Seismic Design Category A (Lateral Forces $\rightarrow F_x = 0.01 w_x$) but continue to solve for C_s instead

$R = 3$ (Table 12.2-1) $\rightarrow$ Steel Systems Not Specifically Detailed for Seismic Resistance, Excluding Cantilever Column Systems

$I = 1.25$ (Table 11.5-1) $\rightarrow$ Occupancy Category III

$T_a = C_t h_n^{x}$

$C_t = 0.02$ (Table 12.8-2)

$h_n = 53'$

$x = 0.75$ (Table 12.8-2)

$T_a = (0.02)(53')^{0.75} = 0.3929$

$T_L = 6$ seconds (Figure 22-15)

$T = T_a = 0.3929$ (this is allowed per Section 12.8.2 ASCE 7-05)

$< C_u T_a = (1.7)(0.3929) = 0.6679$

$$C_s = \min \begin{bmatrix} \dfrac{S_{DS}}{\left(\frac{R}{I}\right)} = \dfrac{0.16}{(3/1.25)} = 0.06667 \\ \dfrac{S_{D1}}{T\left(\frac{R}{I}\right)} = \dfrac{0.0612}{(0.3929)(3/1.25)} = 0.06491 \end{bmatrix}$$

$W =$

$V = C_s W$

Appendix D – Spot Checks

Spot Check

Large Truss Supporting Curved Roof

· Loads:

Dead → Roofing → Zinc Standing Seam Roof Panels ≈ 1.5 psf
1/2" Moisture Resistant Gysum Wall Board ≈ 2.5 psf
4 1/2" Rigid Insulation ≈ (1.5 psf/in.)(4 1/2") ≈ 6.75 psf
7 1/2" Metal Acoustical Roof Deck ≈ 2.4 psf
D = 13.15 psf

Roof Live Load = L_r = 20 psf

Snow Load = S = 23.1 psf

· Load Combinations (LRFD)

$1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R) = (1.2)(13.15\text{ psf}) + (0.5)(23.1\text{ psf}) = 27.33\text{ psf}$

$1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (0.5L \text{ or } 0.8W) = (1.2)(13.15\text{ psf}) + (1.6)(23.1\text{ psf}) =$

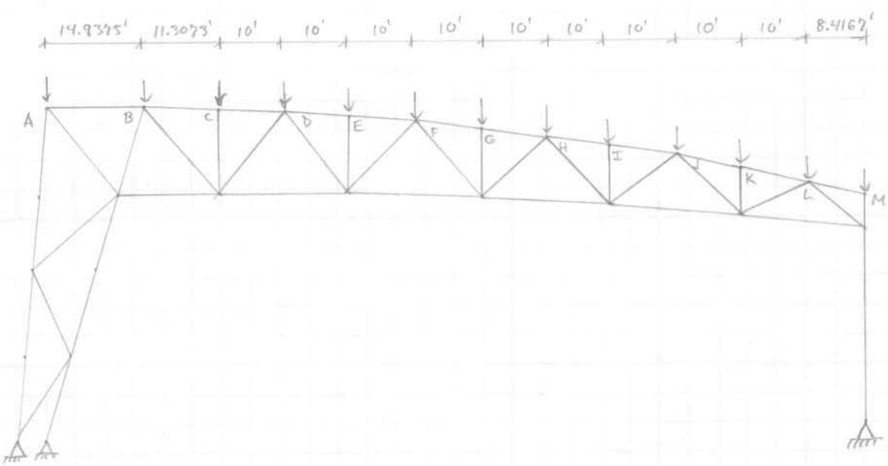
$= 52.74\text{ psf}$

· Large Trusses Spaced @ 30'-0" o.c.

$(52.74\text{ psf})(30') = 1582.2\text{ lb/ft}$

Two top chords (6'-0" apart) → each top chord takes $\frac{1582.2\text{ lb/ft}}{2} = 791.1\text{ lb/ft}$

· Turn Distributed Load into Point Loads Applied at Joints



A) $(791.1 \ ^{lb}/_{ft})(\frac{14.9375'}{2}) = 5908.53 \text{ lb} = 5.909^k$

B) $(791.1 \ ^{lb}/_{ft})(\frac{14.9375' + 11.3073'}{2}) = 10381.13 \text{ lb} = 10.381^k$

C) $(791.1 \ ^{lb}/_{ft})(\frac{11.3073' + 10'}{2}) = 8428.10 \text{ lb} = 8.428^k$

D) $(791.1 \ ^{lb}/_{ft})(\frac{10' + 10'}{2}) = 7911 \text{ lb} = 7.911^k$ (same for E to K)

L) $(791.1 \ ^{lb}/_{ft})(\frac{10' + 8.4167'}{2}) = 7284.71 \text{ lb} = 7.285^k$

M) $(791.1 \ ^{lb}/_{ft})(\frac{8.4167'}{2}) = 3329.21 \text{ lb} = 3.329^k$

* See STAAD Results

Spot Check

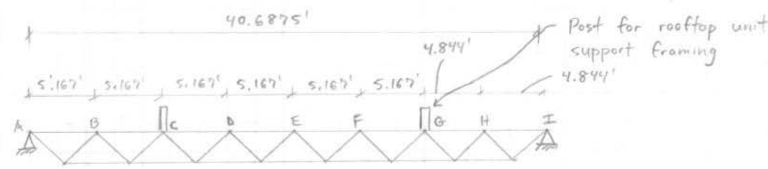
Truss Above Lobby (supporting lower roof and mechanical units)

· Loads:

Dead → 6" Hollow-Core Precast Concrete Planks ≈ 48.75 psf
4½" Rigid Insulation = 6.75 psf
Mechanical Unit Framing ≈ 10 psf
D = 70.5 psf

Roof Live Load → Assume L = 100 psf to account for mechanical equipment, + Drift Load from snow load

Snow Load → S = 23.1 psf



Truss 6

Trusses spaced 15' o.c.

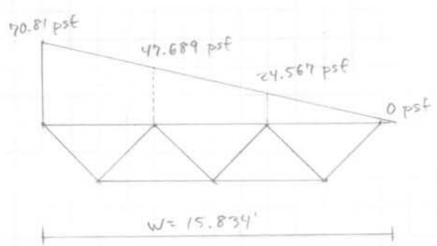
Dead Load per Post → (10 psf)(15') = 150 lb/ft

$\frac{(150 \text{ lb/ft})(20')}{2}$ = 1500 lb = 1.5^k — average length of mech. framing area

Roof Live Load → (100 psf)(15') = 1500 lb/ft

$\frac{(15.1667 \text{ lb/ft})(5.1667')}{2} = 6.2^k$

Drift Load



$\left(\frac{15.834' - 5.1667'}{15.834'}\right)(70.81 \text{ psf}) = 47.689 \text{ psf}$

$\left(\frac{15.834' - (2)(5.1667')}{15.834'}\right)(70.81 \text{ psf}) = 24.567 \text{ psf}$

A,B) $\left(\frac{70.81 \text{ psf} + 47.689 \text{ psf}}{2}\right)(15') = 888.74 \text{ lb/ft}$

$\frac{(888.74 \text{ lb/ft})(5.1667')}{2} = 2295.908 \text{ lb} = 2.296^k$

B, C) $\left(\frac{47.689 \text{ psf} + 24.567 \text{ psf}}{2}\right)(15') = 541.9161 \text{ lb/ft}$

$\frac{(541.9161 \text{ lb/ft})(5.1667')}{2} = 1399.950 \text{ lb} = 1.40^k$

C,D) $\frac{(24.567 \text{ psf})}{2}(15') = 184.252 \text{ lb/ft}$

$\frac{(184.252 \text{ lb/ft})(5.1667')}{2} = 475.985 \text{ lb} = 0.476^k$

· Load Combination

$1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (0.5L \text{ or } 0.8W) \rightarrow 1.2D + 1.6L_r$

All Joints $\rightarrow 1.2D = 1.2(48.75 \text{ psf} + 6.75 \text{ psf}) = 66.6 \text{ psf}$

C, G $\rightarrow 1.2D = 1.2(1.5^k) = 1.8^k$

All Joints $\rightarrow 1.6L = (1.6)(100 \text{ psf}) = 160 \text{ psf}$

A $\rightarrow 1.6L = 1.6(2.296^k) = 3.673^k$

B $\rightarrow 1.6L = 1.6(2.296^k + 1.40^k) = 5.913^k$

C $\rightarrow 1.6L = 1.6(1.40^k + 0.476^k) = 3.001^k$

D $\rightarrow 1.6L = 1.6(0.476^k) = 0.762^k$

$(66 \text{ psf})(15') = 1089 \text{ lb/ft} \rightarrow \frac{(1089 \text{ lb/ft})(5.1667')}{2\,(1000 \text{ lb/kip})} = 2.5575^k \leftarrow$ Each joint

$(160 \text{ psf})(15') = 2400 \text{ lb/ft} \rightarrow \frac{(2400 \text{ lb/ft})(5.1667')}{2\,(1000 \text{ lb/kip})} = 6.2^k$

· Total Joint Loads

A) $2.558^k + 6.2^k + 3.673^k = 12.431^k$

B) $2.558^k + 6.2^k + 5.913^k = 14.671^k$

C) $2.558^k + 6.2^k + 3.001^k + 1.8^k = 13.559^k$

D) $2.558^k + 6.2^k + 0.762^k = 9.520^k$

E) $2.558^k + 6.2^k = 8.758^k$ (same for Joints F, H, and I)

G) $2.558^k + 6.2^k + 1.8^k = 10.558^k$

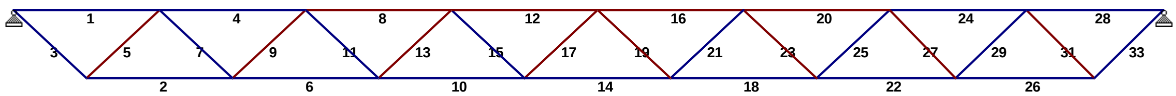


Figure 22 – Beam Labels for Members of Truss Above Lobby

Beam Maximum Axial Forces

Distances to maxima are given from beam end A.

Beam	Node A	Length (ft)	L/C		d (ft)	Max Fx (kip)
1	1	5.167	1:LOAD CASE	Max -ve		
				Max +ve	0.000	-71.102
2	3	5.167	1:LOAD CASE	Max -ve		
				Max +ve	0.000	-85.259
3	1	3.537	1:LOAD CASE	Max -ve		
				Max +ve	0.000	-58.375
4	5	5.167	1:LOAD CASE	Max -ve		
				Max +ve	0.000	-1.526
5	3	3.537	1:LOAD CASE	Max -ve	0.000	58.375
				Max +ve		
6	6	5.167	1:LOAD CASE	Max -ve		
				Max +ve	0.000	-139.152
7	5	3.537	1:LOAD CASE	Max -ve		
				Max +ve	0.000	-36.899
8	7	5.167	1:LOAD CASE	Max -ve	0.000	37.873
				Max +ve		
9	6	3.537	1:LOAD CASE	Max -ve	0.000	36.899
				Max +ve		
10	8	5.167	1:LOAD CASE	Max -ve		
				Max +ve	0.000	-164.057
11	7	3.538	1:LOAD CASE	Max -ve		

				Max +ve	0.000	-17.052
12	9	5.167	1:LOAD CASE	Max -ve	0.000	52.601
				Max +ve		
13	8	3.537	1:LOAD CASE	Max -ve	0.000	17.052
				Max +ve		
14	10	5.167	1:LOAD CASE	Max -ve		
				Max +ve	0.000	-168.609
15	9	3.537	1:LOAD CASE	Max -ve		
				Max +ve	0.000	-3.117
16	11	5.167	1:LOAD CASE	Max -ve	0.000	47.791
				Max +ve		
17	10	3.537	1:LOAD CASE	Max -ve	0.000	3.117
				Max +ve		
18	12	5.167	1:LOAD CASE	Max -ve		
				Max +ve	0.000	-154.437
19	11	3.538	1:LOAD CASE	Max -ve	0.000	9.703
				Max +ve		
20	13	5.167	1:LOAD CASE	Max -ve	0.000	24.257
				Max +ve		
21	12	3.537	1:LOAD CASE	Max -ve		
				Max +ve	0.000	-9.703
22	14	4.917	1:LOAD CASE	Max -ve		
				Max +ve	0.000	-121.541

Beam Maximum Axial Forces Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max Fx (kip)
23	13	3.537	1:LOAD CASE	Max -ve	0.000	22.523
				Max +ve		
24	15	4.844	1:LOAD CASE	Max -ve		
				Max +ve	0.000	-17.240
25	14	3.538	1:LOAD CASE	Max -ve		
				Max +ve	0.000	-22.523
26	16	4.917	1:LOAD CASE	Max -ve		
				Max +ve	0.000	-69.545
27	15	3.359	1:LOAD CASE	Max -ve	0.000	36.063
				Max +ve		
28	17	4.844	1:LOAD CASE	Max -ve		
				Max +ve	0.000	-78.735
29	16	3.485	1:LOAD CASE	Max -ve		
				Max +ve	0.000	-37.406
31	17	3.411	1:LOAD CASE	Max -ve	0.000	48.968
				Max +ve		
33	18	3.433	1:LOAD CASE	Max -ve		
				Max +ve	0.000	-49.286

Table 8 – STAAD Results for Member Axial Forces of Truss Above Lobby