FINAL THESIS REPORT

COMPTON FAMILY ICE ARENA

haley mcclernon|structural option

ADVISOR: KEVIN PARFITT | APRIL 4, 2012

general information

occupant type arena

size 203,000sf

total levelS 3

dates of construction may 2010-october2011

project delivery design-build

project team

owner University of Notre Dame

architect Rossetti

design builder Barton Malow

structural engineer SDI, Inc.

MEP engineer Peter Basso Associates

LEED Heapy Engineering

code FP&C

mechanical

- -Utilizes ammonia chiller for ice generation
- -One large AHU to serve main arena and second to serve community arena with both an enthalpy wheel for heat recovery and a desiccant wheel for ddehu midification.
- -Club level suites served by individual fan coil units
- -General spaces on the event level(locker rooms, class rooms etc.) served by an array of AHU's, some with enthalpy wheels for heat recovery.

structure

- -foundation of the structure comprised of a combination of mat foundations and typical spread footings
- -Steel framing utlized throughout superstructure with 5'' slab on grade and 6'' concrete floor slabs throughout.
- -Precast concrete stadia supported by W21 rakers and long span construction accomplished by barrel vaulted trusses.

architecture

Classic architecture and vision with the functionality of a state of the art facility, the CFIA features an elegant cast stone facade and interior gothic vernacular. The event level contains two full size ice arenas: one standard and one olympic, surrounded by more than 5000 spectator seats throughout the concourse and club levels. In addition, the facility contains impressive locker rooms, classrooms, student athlete food and study areas, offices, confer-ence rooms, and a media center.

h a l e y m c c l e r n o n l s t r u c t u r a l o p t i o n

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EXECUTIVE SUMMARY

The Compton Family Ice Arena is 200,000 square foot complex located on the University of Notre Dame's campus in South Bend, Indiana. It is a three story space comprised of a main event level, a concourse level, and a club level housing two ice rinks, locker rooms, learning and office spaces, and a grand entrance foyer. A steel framing system supports a lightweight composite floor system and lateral loads are resisted by braced frames pinned to mat foundations.

This report opens with an overview of the existing building summarizing the design criteria and confirming structural adequacy. Analysis reveals a logical and appropriate layout characterized by cost effective design thus leading to an investigation into the long span roof design of the arena. Research then led to the consideration of a roof system popularized by structural engineering firm Walter P. Moore for use in many of their arena design projects. This system, deemed the table top truss, replaces traditional long span design with a unique configuration of box trusses carrying load out to four super columns to accomplish roof support at significantly smaller spans.

Guided by existing buildings utilizing the same system, and adapted model is designed to service the Compton Family Ice Arena while remaining as minimally invasive to the current structure as possible. The analysis includes redistribution of loads and the introduction of prominent super columns, and ultimately requires a re-assessment of the lateral system for adjustments in roof height and column layout. Geometry and arrangement of the truss system is manipulated and iterated in order to find the most efficient and effective layout and framing members are optimized for the most economic design.

In addition, the impacts of the proposal stretch beyond just structural verification having influences across all phases of design. Thus, two additional studies are performed to address architectural and construction management impacts as a result of the table top truss.

The architectural investigation looks to accomplish both functional and aesthetically pleasing design of the seating bowl with the incorporation of the new columns at corner sections of the club level. In addition, it addresses changes in floor plans due to column shifts at certain points throughout the building. In all cases of architectural coordination the structure is found to be nominally disruptive to the function of the building. Furthermore, the seating at the upper club level sections is redistributed and the arena gains a bold and exaggerated aesthetic appeal.

The erection procedure for the system is also affected as the new design calls for a specific and marginally unconventional process. The introduction of shoring towers and added effects of additional bolting and welding at truss connections calls for an assessment of construction schedule and cost and is found to add little to no construction time but rather create changes to crane procedure and layout.

The following report addresses and details each of these design phases and provides all necessary supplemental material in the appendices thereafter.

ACKNOWLEDGEMENTS

I would like to extend my gratitude to the following companies and individuals for their support and consultation in the completion of this report:

The University of Notre Dame for granting project and owner permission and offering continued assistance in providing documentation and information. In particular, I would like to thank Craig Tiller for his continued support.

SDI Structures and most notably Andy Greco for his assistance in providing valuable information and insight on the existing structure of the Arena.

Dennis Wittry at Walter P. Moore for his guidance in the proposed design of this thesis.

The AE faculty for their continued support and assistance with this thesis and over the past four years.

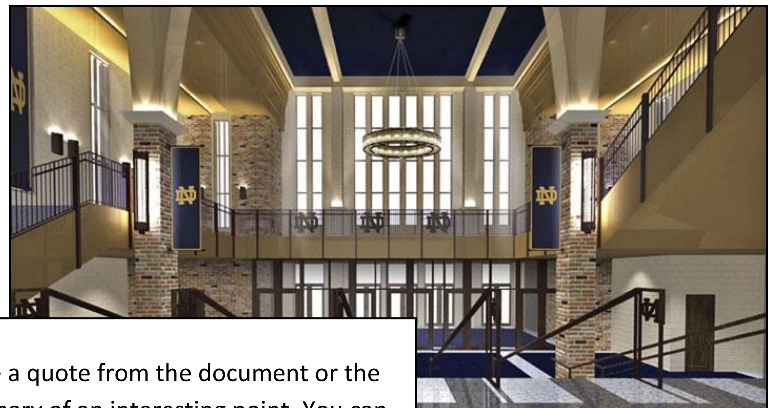
My classmates for their help and support in the completion of this thesis.

BUILDING INTRODUCTION

BUILDING OCCUPANT NAME: *Notre Dame University*
OCCUPANCY OR FUNCTION TYPE: *Arena*
SIZE: *203,000 SF*
TOTAL LEVELS: *3*

OWNER	N O T R E D A M E U N I V E R S I T Y Notre Dame, IN 46556
ARCHITECT	R O S S E T T I Two Towne Square, Suite 200 Southfield, Michigan 48076
DESIGN BUILDER	B A R T O N M A L O W 26500 American Drive Southfield, MI 48034
STRUCTURAL ENGINEER	S D I I N C . 275 East Liberty Ann Arbor, Michigan 48103
MEP ENGINEER	P E T E R B A S S O A S S O C I A T E S 5145 Livernois, Suite 100 Troy, MI 48098
LEED	H E A P Y E N G I N E E R I N G 1400 West Dorothy Lane Dayton OH 45409
CODE	F P & C One Ward Parkway, suite 200 Kansas City, MO 64112
AUDIO/ VIDEO	A C O U S T I C

D I M E N S I O N S
15505 Wright Brothers Drive
Addison, Texas 75001



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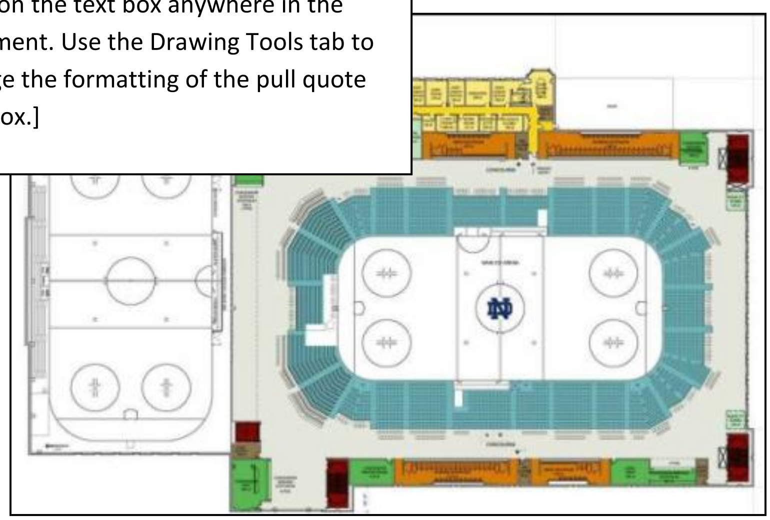


FIGURE 2: CONCOURSE LEVEL FLOOR LAYOUT

Situated on the University of Notre Dame's Campus in South Bend, Indiana, The Compton Family Ice Arena has just opened its doors as the home stadium of the Fighting Irish. Inspired by famous gothic architecture and in staying true to the style of Notre Dame's Campus, the stadium features an elegant cast stone façade and interior gothic vernacular. The sophistication required by the University standard as interpreted by the vision of Rossetti Architects produces a classic building

FIGURE 3: AERIAL RENDINERING FROM EAST SIDE OF ARENA

with all the functionality of a state of the art arena.

Featuring a full size collegiate arena as well as an Olympic sized arena, a full concessions layout, and more than 5000 spectator seats, the new space is not only magnificent in appearance but truly versatile in meeting the needs of the program and community. Adding to amenities will be locker rooms, lounges, classrooms, and study areas, as well as offices, conference areas and media space necessary to facilitate a division one program.

As mandated by the university in conjunction with their sustainability goals across campus, the building was designed with great concern and adherence to LEED requirements. Upon official evaluation it will hopefully acquire LEED silver rating and include a number of state of the art sustainability features.

Cost of Construction for the project is approximately $50 million dollars and at the end of the physical process that began in May of 2010, the University and community will have over 200,000 square feet of recreational and multi-use space.

EXISTING STRUCTURAL OVERVIEW

The Compton Family ice Arena is a three level complex supported by a primarily steel frame structure resting on concrete piers. The long span construction necessary for the open atmosphere required of an arena is accomplished through upwards of 15’ deep barrel trusses across the ice surface and a supporting framework surrounding the central focus. The incredible load induced by the precast risers characteristic of the bowl structure is addressed with sizable rakers fanning the event level as well as precast concrete walls below the stadia. The lateral system as will be discussed is comprised of concentric braced frames throughout the building.

FOUNDATIONS

Earth Exploration Inc. was contracted to investigate subsurface conditions to gain a geotechnical perspective with regard to foundations. What they found was primarily granular type soil extending 75ft with the exception of a particular location of cohesive soil at 22-32 ft. below surface. Despite these weak spots though, the site was deemed adequate to carry the loads of the facility of this nature as long as typical low bearing pressures were maintained. In this case, proposed spread footings were approved without the need of any deep foundation system.

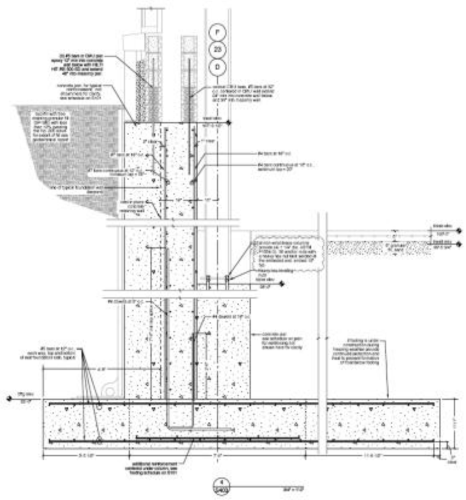


FIGURE 4: TYPICAL DETAIL AT CONCRETE PIER

In following, the ice arena utilizes a combination of mat foundations, grade beams, and typical footings, with step footings typical around the perimeter of the building. The mat foundations, at 18'' and 23'' thicknesses, require #5 bars at 12'' o.c. each way at both the top and bottom as well as additional #5 bars centered under the columns at 23'' mats (See Figure 1). The grade beams primarily span the perimeter of the ice at a minimum thickness of 12'' and require (2) #9 bars through the entire footing. The remaining footings vary in thickness from 18'' to 42'' and require typical bottom reinforcement in both directions. A typical detail at a concrete pier can be seen in Figure 4.

ROOF SYSTEM

The profile of the Compton Family ice Arena is distinguished by 4 different roof levels as can be seen in Figure 5. In blue is the Olympic Ice roof level sharing an elevation with the offices and conference rooms also shown in blue on the north side of the Arena. These two sections extend 32 ft. above grade. The area shown in green has a roof level of 48 ft. and the yellow area represents the curved roof that reaches a maximum height of 63ft. All sections but the yellow are flat roofs utilizing 1-1/2'' 18gage type B wide rib metal roof deck with a vapor retarder, two layers of rigid roof insulation and a single ply roofing membrane. The main arena sloped roof, however, demands 7 ½'' 18gage type N wide rib metal rood deck with the same roof.

FLOOR CONSTRUCTION

At grade, typical floor construction calls for all interior slabs to be 5'' slab on grade with 6x6 W1.4xW1.4 WWF on a 10mm vapor barrier, on compacted granular fill. At both the concourse and club levels the floor support consists of 3" 18 gage composite metal deck with 3 ¼'' light weight concrete above the deck and 6x6 W2.9xW2.9 WWF. Also required are 3/4 '' diameter by 5'' long headed studs on all steel beams supporting concrete slabs.

All metal deck was designed to be continuous over three spans minimum with the requirement of at least 2'' bearing on steel supports. The framing of the arena can be broken in to two sections separated by the building expansion joint between column lines 5 and 6 (refer to plan views in appendices). The first is the main arena on the east wing and the second the Olympic sized arena on the west end.

MAIN ARENA

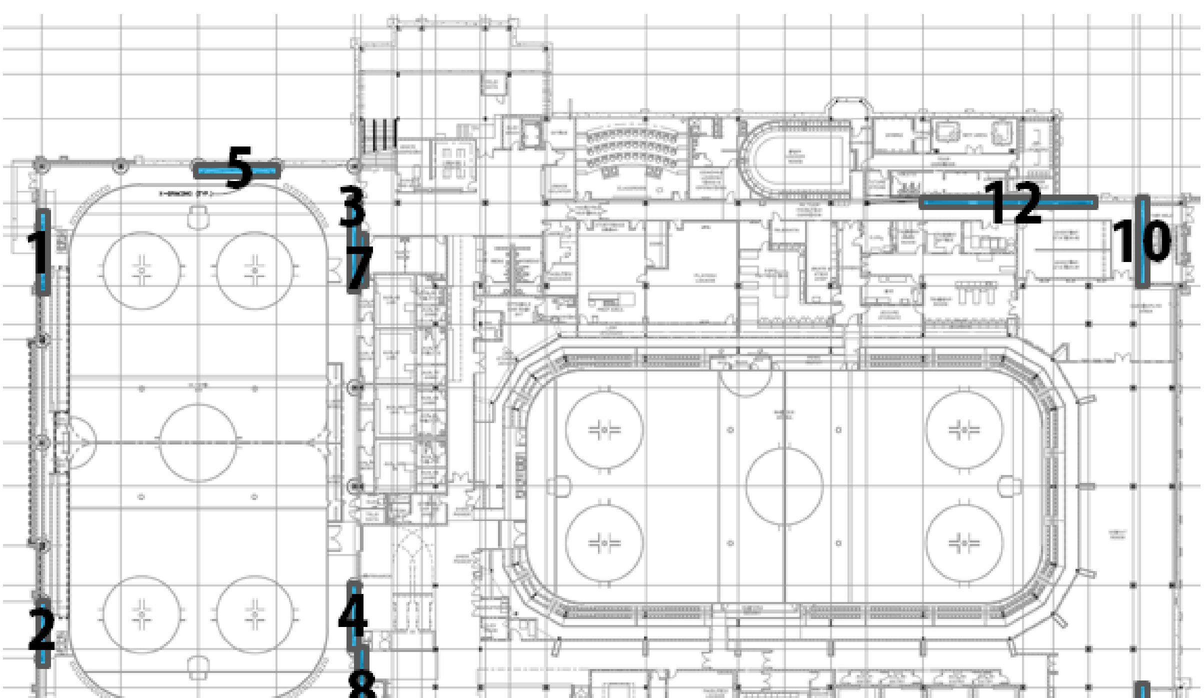
While the central ring on the arena is comprised of rakers based at slab on grade supporting the risers, the surrounding portions of the structure experience fairly regular framing with typical bay sizing. W21s and W18s framing into W24 and W33 girders create roughly 13’x32’ bays on the north, south and west sides. Along the east side you find W12s framing in to W21s to make up roughly 12’x29’ bays.

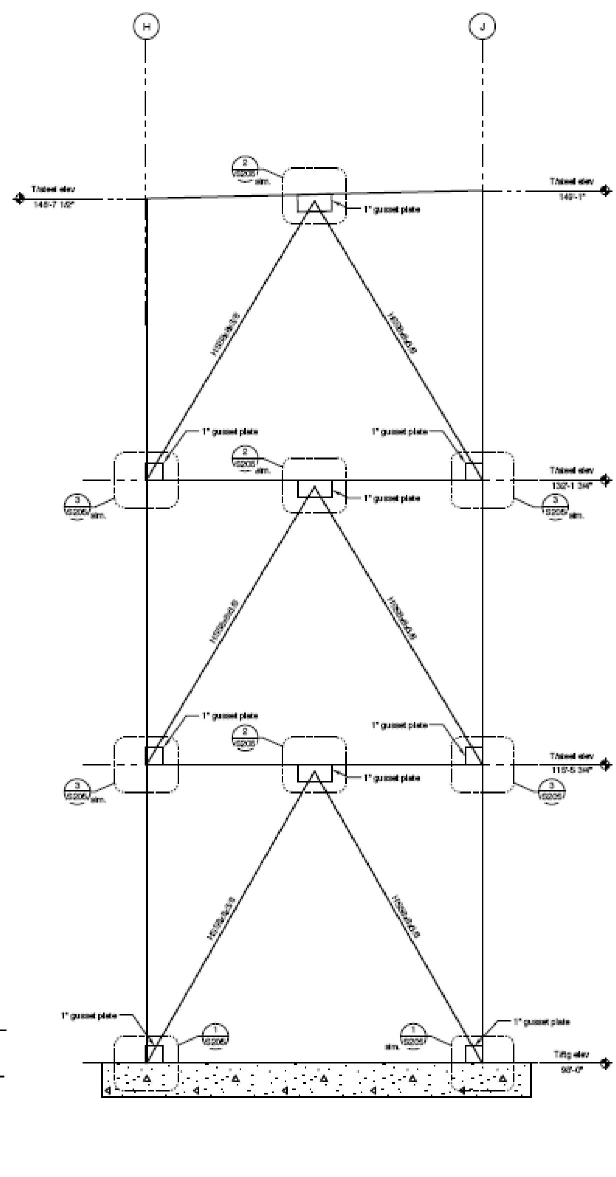
OLYMPIC ARENA

The Olympic rink has no concourse level and extends only to its roof level at the concurrent of the main arena’s club level. This roof structure present at this location will be discussed in following sections.

LATERAL SYSTEM

Steel braced frames are used to resist lateral loads placed on the structure. The west end of the building uses primarily X bracing with C12s. The remaining braced frames are a combination of concentric and eccentric chevron bracing with HSS members at either 8x8x3/8 or 12x8x3/8. An example can be seen in Figure 7 below. The columns connecting these braced franes range in size from W10 to W12 and require additional threaded anchor bolts embedded at least 20’’ into concrete with heavy hex nuts at the embedded end. The locations of each of these frames can be found in the Figure 6.





steel brace frame BF-7

STRUCTURAL MATERIALS

Concrete		
Usage	Weight (PCF)	Strength(PSI)
foundations	150	3500
interior slab	150	3500
interior slab	110	3500
exterior slab	150	3500
columns	150	4000
piers	150	4000
walls	150	4000
Masonry		
Usage	Standard	Strength
CMU	ASTM C90 & C145	f'_m=2000
Mortar Typical	ASTM C270	-
Grout	ASTM C476	f'_c=3000
Steel		
Usage	Standard	Grade
W-Shaped Structural Steel	ASTM A992	-
Channels,Angles, Plates	ASTM A36	-
HSS Round	ASTM A500	-
HSS Rectangular, Square	ASTM A500	-
Structural Steel Pipes	ASTM A53	-
Structural Steel Bolts	ASTM A325	-
Washers	ASTM F436	-
Nuts	ASTM A536	-
Steel Roof Deck/ Comsposite Floor Decl	ASTM A653-94	33, G-60 Galvanized
Anchor Bolts	ASTM F1554	-
Headed Steel Studs	ASTM A108	1010-1020
Soils		
Usage	Strength	
Soils Supporting Foundations	5000psf min allowable bearing capacity	

Figure 8: Building Materials Used

DESIGN CODES

Sheets S001 and LS101 indicate that the Building was designed to comply with the following:

- 2006 International Building Code (IBC) with local amendments
- 2006 international Mechanical Code (IMC) with local amendments
- 2006 International Plumbing Code (IPC) with local amendments
- 2006 International Fire Code (IFC) with local amendments
- 2005 National Electric Code with local amendments
- 2003 ASME A17.1 Elevator Safety Code
- American Disabilities Act Accessibility Guidelines (ADAAG)
- Minimum Design Loads for Buildings and other Structures (ASCE7-10)
- Building Code Requirements for Structural Concrete (ACI 318-08)
- Specifications for Structural Concrete for Buildings (ACI 301-05)
- Masonry Construction for Buildings (ACI 530)
- Technical Notes of Brick Construction (BIA)
- Specification for Structural Steel Buildings (AISC)

**This list also reflects the codes used for analysis in this technical report

GRAVITY LOADS

A major component of this technical report was the calculation of dead, live, and snow loads acting on the building. Below is a summary of the gravity checks performed and load cases found. Supporting Calculations can be found in Appendix A of this report.

DEAD AND LIVE LOADS

An actual summary of dead and live loads was not provided for the analysis done on the building but the following tables provide the values used in this analysis as calculated by code or relevant assumptions. In calculating the overall building weight, a number of elements were taken into consideration including slabs, steel beams and columns, façade, stadia seating, and roofing. What resulted was the following breakdown by floor and an overall building weight of 47880k.

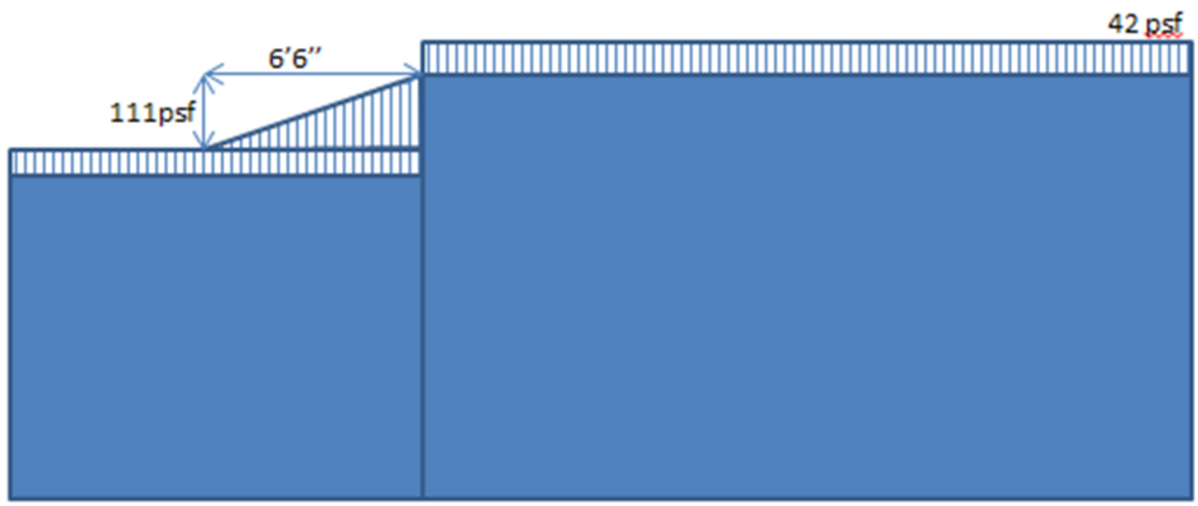
Building Weights by Floor (psf)	
Event Level	261.89
Concourse Level	176.09
Club Level	234.06
Main Roof level	72.70

Live Loads	
Occupancy	Uniform psf
Stadiums/Arenas 1st floor	100
Stadiums/ArenasUpper floor	60
Retail Stores	100
Catwalks for maintenance	40
Roof	20

Dead Loads	
Material	Load (psf)
Slab	46
Façade	60
Superimposed	15

SNOW LOADS

The roof snow loads were calculated in accordance with chapter 7 of ASCE 7-05. The resulting uniform load as found from the maps outlined in chapter 7 was found to be 42 psf. In addition to this load, the snow drift on the lower roof of the Olympic stadium was calculated as shown in the figure 13 below. Full calculations for this analysis can be found in Appendix A.



LATERAL LOADS

WIND LOADS

ASCE 7-10 was used to determine wind pressures on the Arena in both North-South and East-West directions to show forces transferred to the Main Wind-Force Resisting System. Assumptions were made to simplify the shape of the building and the roof was taken to be flat due to the small magnitude of the slope angle. In addition, the roof elevation was set to one value at 48ft. For this analysis, the red box shown in figure 14 represents the shape used to analyze wind loads. As you will see in figure, the wind pressures for windward, leeward, sidewall and internal pressures were all calculated using an excel spreadsheet and then used to find story forces at each level. The results for both north-south and east-west directions can be seen below.

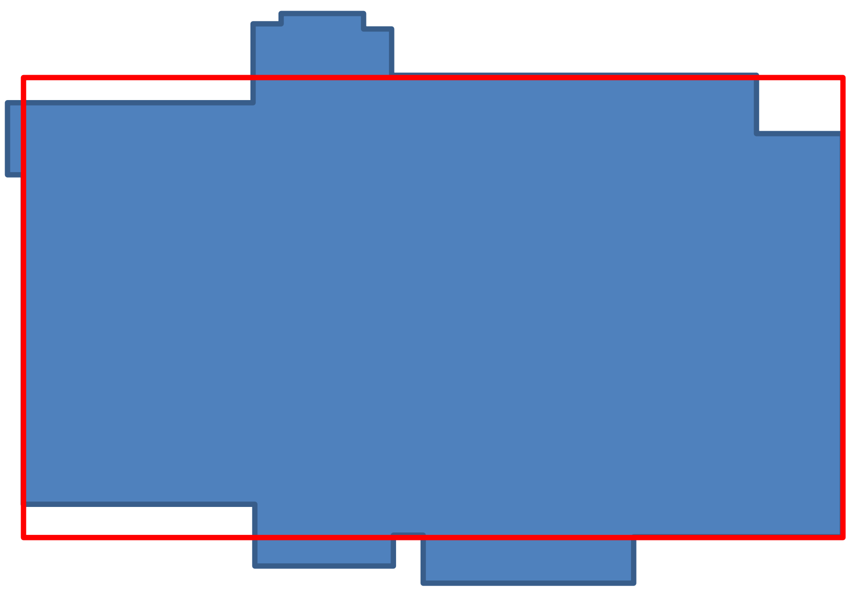
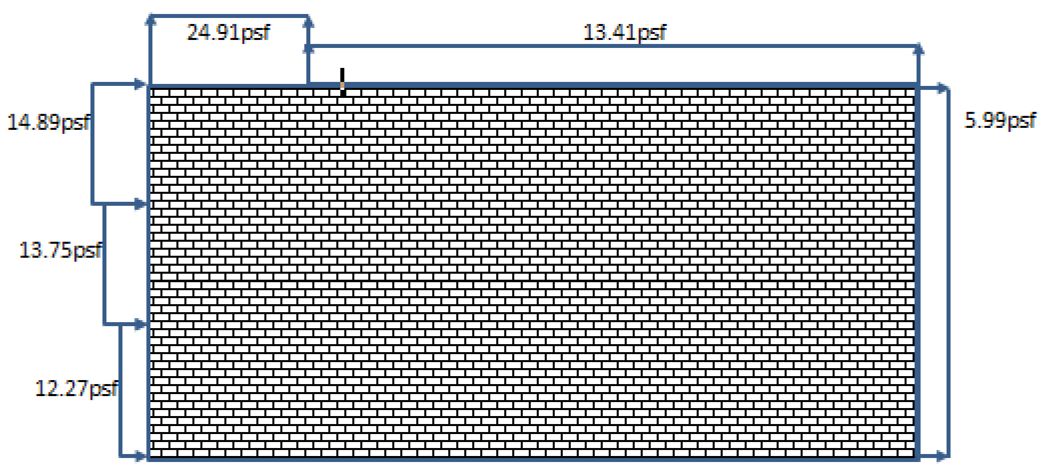
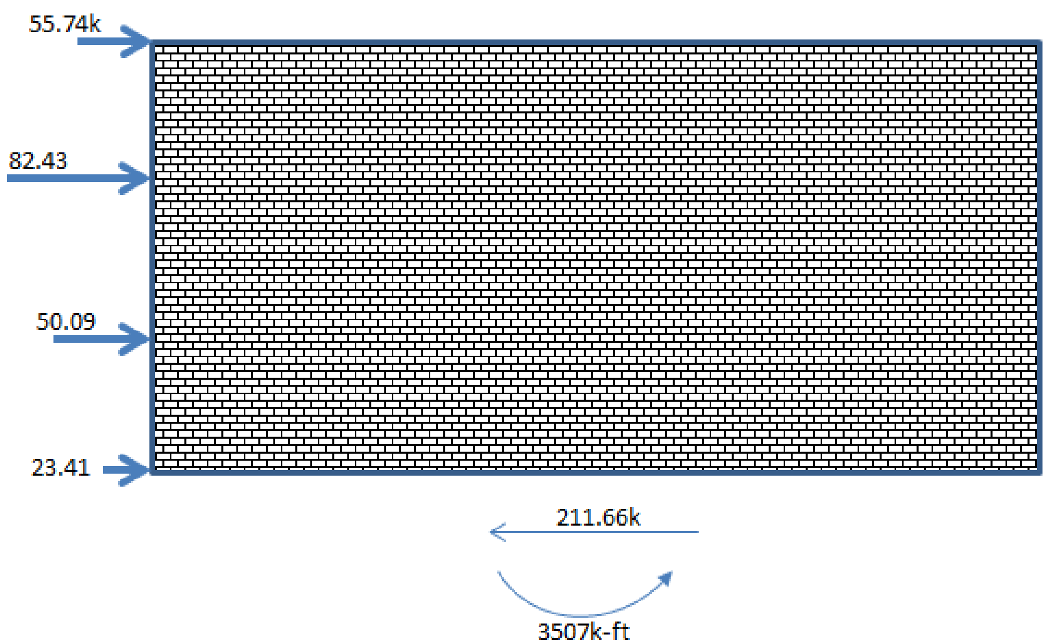


Figure 1: Assumed Shape for Wind Analysis

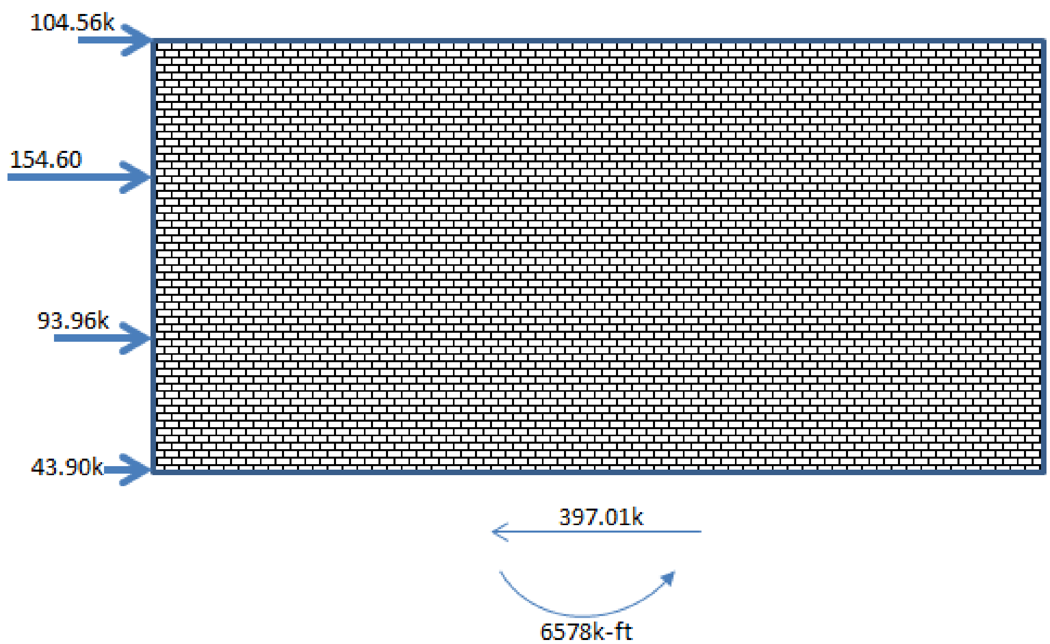
Wind Pressures (East-West Direction)							
				Internal Pressure(psf)		Net Pressure(psf)	
Type	Level	Distances(ft)	Wind Pressure (psf)	(+)(Gcpi)	(-)(Gcpi)	(+)(Gcpi)	(-)(Gcpi)
	Event Level	0	11.72	4.06	-4.06	15.77	7.66
	Concourse Level	16	12.27	4.06	-4.06	16.32	8.21
	Club level	32.3	13.73	4.06	-4.06	17.79	9.67
Windward Walls	Roof Level	48	14.89	4.06	-4.06	18.94	10.83
Leeward Walls	ALL	ALL	-5.99	4.06	-4.06	-1.93	-10.05
Side Walls	ALL	ALL	-13.41	4.06	-4.06	-9.36	-17.47
		0-24	-24.91	4.06	-4.06	-20.85	-28.96
		24-48	-13.41	4.06	-4.06	-9.35	-17.47
		48-96	-13.41	4.06	-4.06	-9.35	-17.47
Roof		>96	-13.41	4.06	-4.06	-9.35	-17.47



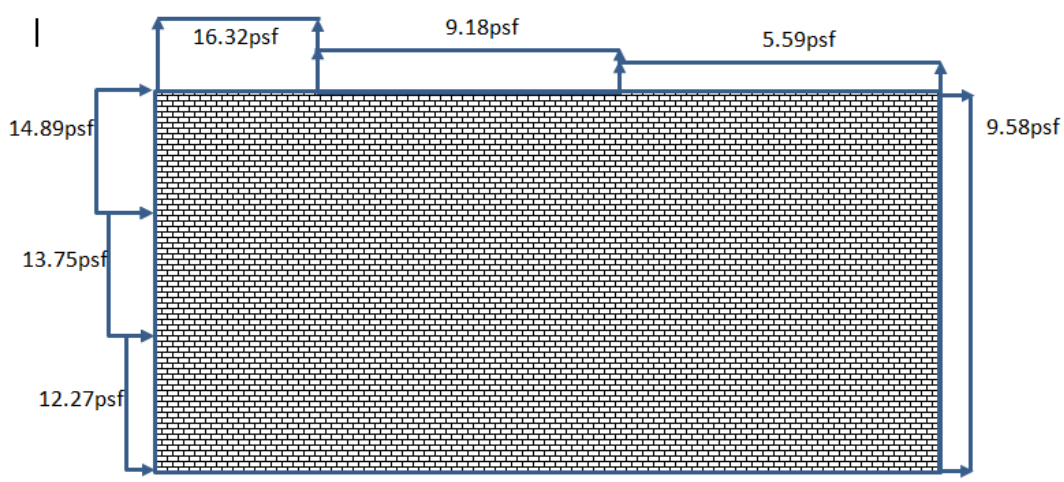
Wind Pressures (East-West Direction)								
		Trib Below		Trib Above				
Floor Level	Elevation	Height(Ft)	Area(ft2)	Height(Ft)	Area(ft2)	Story Force(k)	Story Shear (k)	Overturning moment(k-ft)
Event Level	0.00		0.00	8.00	1908.00	23.41	211.66	0.00
Concourse Level	16.00	8.00	1908.00	8.15	1943.78	50.09	188.26	1506.06
Club level	32.30	8.15	1943.78	15.70	3744.45	82.43	138.17	1126.05
Roof Level	48.00	15.70	3744.45		0.00	55.74	55.74	875.11
							Total Base Shear:	**211.66**
							Total Overturning Moment	**3507.22**



Wind Pressures (North-South Direction)								
Floor Level	Elevation	Trib Below		Trib Above		Story Force(k)	Story Shear (k)	Overturning moment(k-ft)
		Height(Ft)	Area(ft2)	Height(Ft)	Area(ft2)			
Event Level	0.00		0.00	8.00	3578.40	43.90	397.01	0.00
Concourse Level	16.00	8.00	3579.20	8.15	3645.50	93.96	353.12	2824.93
Club level	32.30	8.15	3646.31	15.70	7022.61	154.60	259.16	2112.15
Roof Level	48.00	15.70	7024.18		0.00	104.56	104.56	1641.61
							Total Base Shear:	**397.01**
							Total Overturning Moment	**6578.69**



Wind Pressures (North-South Direction)							
				Internal Pressure(psf)		Net Pressure(psf)	
Type	Level	Distances(ft)	Wind Pressure (psf)	(+)(Gcpi)	(-)(Gcpi)	(+)(Gcpi)	(-)(Gcpi)
	Event Level	0	11.72	4.06	-4.06	15.77	7.66
	Concourse Level	16	12.27	4.06	-4.06	16.32	8.21
	Club level	32.3	13.73	4.06	-4.06	17.79	9.67
Windward Walls	Roof Level	48	14.89	4.06	-4.06	18.94	10.83
Leeward Walls	ALL	ALL	-9.58	4.06	-4.06	-5.52	-13.64
Side Walls	ALL	ALL	-13.41	4.06	-4.06	-9.35	-17.47
		0-24	-16.32	4.06	-4.06	-12.27	-20.38
		24-48	-16.32	4.06	-4.06	-12.27	-20.38
		48-96	-9.18	4.06	-4.06	-5.12	-13.23
Roof		>96	-5.59	4.06	-4.06	-1.54	-9.65

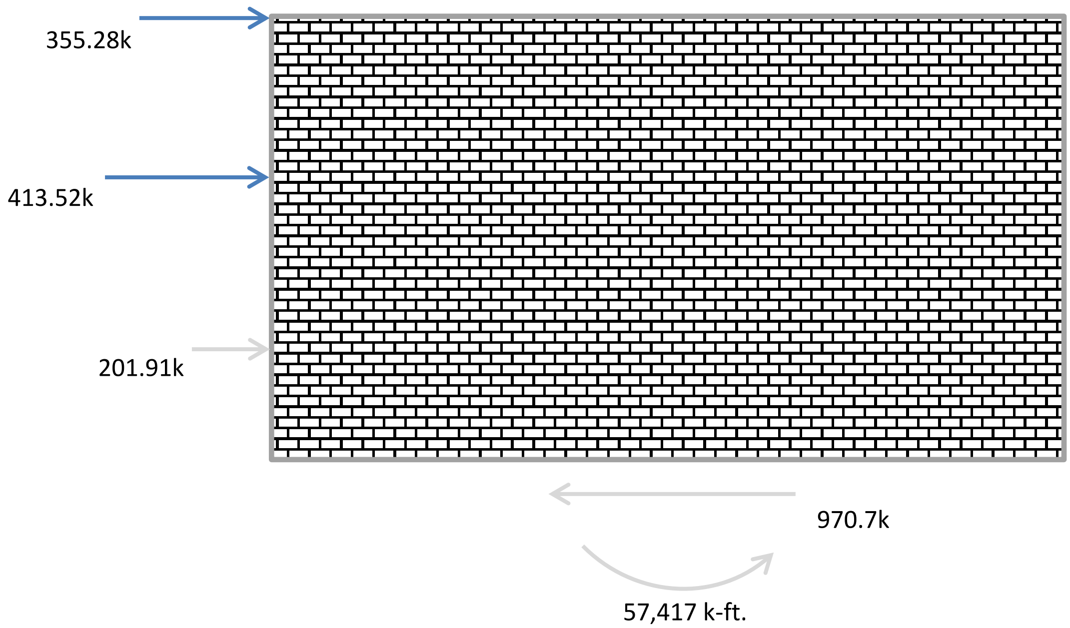


SEISMIC DESIGN

Seismic load calculations were performed in accordance with chapters 11 and 12 of ASCE 7-05 as outlined by the Equivalent Lateral Force Procedure. For use of this method the building footprint was again simplified to a rectangular form similar to that used in the wind load calculation. Because the same lateral bracing system is used in both the north south and east west directions, the resulting forces were equivalent.

It was here that the building floor weights were taken in to account and as can be seen in the charts below, the exceedingly heavy nature of the structure plays a large part in the dominance of the seismic forces. With a soil classification, D, and the combination of heavy building weight and low roof height, it is unsurprising that seismic forces outweigh wind load by as much as 6 times in the east-west direction and 2.5 in the north-south direction.

SeismicForces (East-West and North-South Directions)							
Level	Story Weight, wx(k)	Story Height, hx(ft)	wxhxk	Cvx	Story Force(k) Fx=CvxV	Story Shear (k)	Overturning Moment(k-ft)
Event Level	29000.00	0.00	0.00	0.00	0.00	970.70	0
Concourse Level	11798.00	16.00	188768.00	0.21	201.91	970.70	15531.2
Club level	5790.76	32.30	187041.55	0.43	413.52	768.79	24832.05912
Roof Level	1291.00	48.00	61968.00	0.37	355.28	355.28	17053.2576
						Total Base Shear:	970.70
						Total Overturning Moment	57416.52



ROOF STRUCTURE

The current high roof framing consists of eight barrel trusses spanning across the 156 ft. open bowl at 36 feet on centers. The curved trusses are comprised of W36x210 bottom chords and W14x146 top chords with W8x35 vertical and diagonal elements. Between each truss spans a curved W12x16 shape and east west bracing utilizes W21x44's at approximately every twelve feet. The trusses frame into W14x90 columns at both ends of the span. A typical truss can be seen in image below

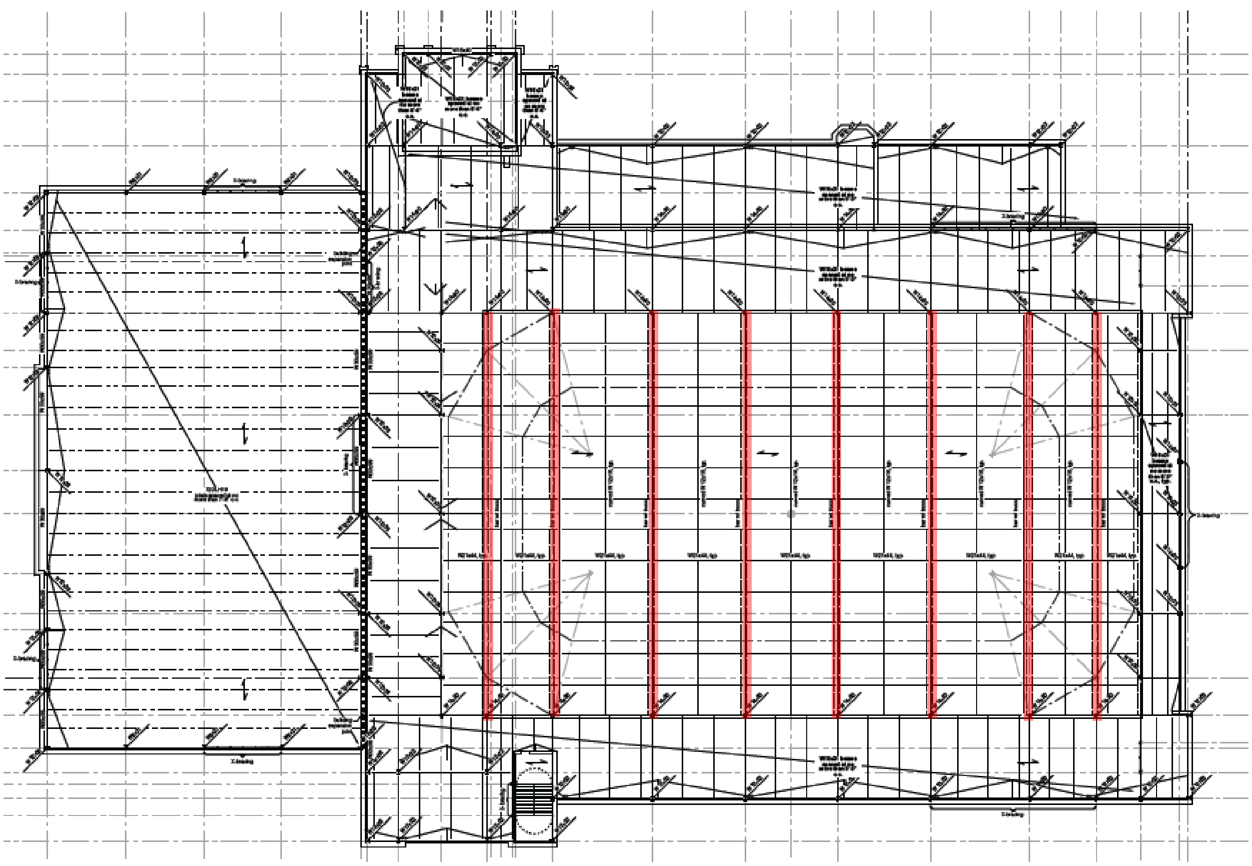


Figure 9: Current High Roof Plan with barrel trusses highlighted

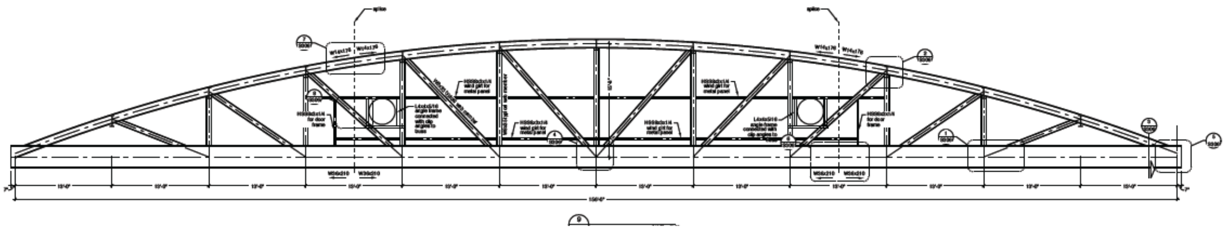


Figure 10: Typical barrel truss detail

A complete framing system take off and cost estimate can be found within the structural depth of this thesis as it will later be compared to alternative systems for economy.

PROBLEM STATEMENT

As confirmed by the preceding analysis, the Compton Family Ice Arena has been built to meet sufficient strength and serviceability requirements as well as impressively match and enhance the landscape of the University of Notre Dame campus while maximizing functionality. It was designed under the University budget with great emphasis on cost control yielding nearly the most economical design possible.

In searching for the most practical and effective redesign, it was natural to rule out concrete design and as proven by the floor re-designs in technical report three, unlikely to find a more efficient floor system. O great interest to the author of this report though and of particular relevance to this building was an investigation of the long span design of the arena.

Of large consideration in an Olympic scale arena project like this is effectively and efficiently addressing the long spans necessary to maintain the open bowl of the arena. The existing structure is defined by barrel trusses spanning 157 ft. in the North-South direction over the 252 ft. length of the main arena. At a maximum, the distance between bottom and top chord is 15 ft. creating a sloped roof over the length of the main arena comprised of W14 and W36 sized chords.

While this design creates an aesthetically adequate structure satisfactory to carry the roof loads and maintain structural stability and serviceability, there are a considerable number of possibilities in designing a roof like this based on further limiting criteria such as cost, constructability, and schedule impact. While these were all considered in the design of the

Compton Family Ice Arena, it appears that the final design was based largely on economy thus ruling out a number of possibilities leading to advantages in aesthetic, constructability, usability and efficiency.

So while there is no true “problem” as far as the structure of the Compton Family Ice Arena is concerned, the basis for this report is an exploration into long span alternatives for the anticipated improvement of the comprehensive design.

PROPOSED SOLUTION

In addressing long span design there were a number of options for ideal configurations. Considered in this case were two of particular interest. The first of which was an alternative to the original design using glue laminated wood framing for hybrid design of the long span steel trusses. The second was an investigation into a trend in Stadia design popularized by lead engineers at Walter P. Moore known as the table top truss system. Championed for its span reduction capabilities, coordination advantages and serviceability, it has been implemented in a number of projects across the country including the Reed Arena at Texas A&M University and the Toyota Center in Houston, Texas.

The design calls for central rectangular shaped box truss supported by four truss legs spanning to the outside of the arena bowl. The entire system then comes down on eight columns which effectively support the entire roof load. A representation of this system can be seen in Figure 11.

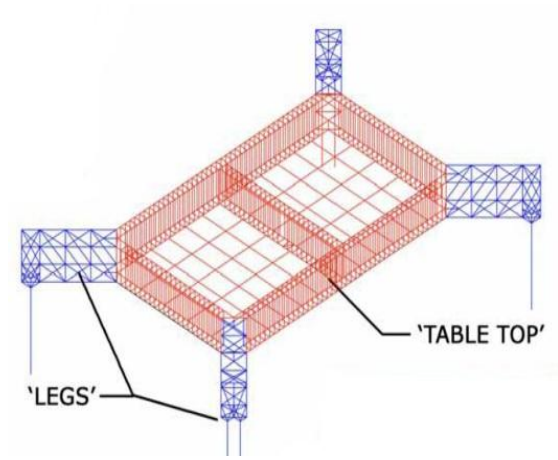


Figure 11: Table Top Truss Formation Loosely adapted from
Reed Arena in Houston, Texas

The Goals of this redesign are to decrease span lengths thus decreasing steel usage while maintaining strength and serviceability requirements and explore the possibility of the table top truss on a slightly smaller scale structure. It will require significant attention to the design and seating requirements of the arena as it will displace existing seating and obstruct sight lines if not properly considered.

BREADTH STUDIES

In maintaining the integrated nature of the Architectural Engineering curriculum, this report has been enhanced by two breadths aimed at supplementing the structural redesign.

With such large scale change comes architectural and aesthetic alterations to both the roof and dome structure. The newly sized dome will be visible from the exterior, as presumed, and thus must not throw off the balance and movement of the overall structure. In addition, the box trusses will bring the columns within the bowl of the arena requiring a re-sculpting of the seating layout with attention to site lines and visibility. It will change the overall look of the interior and reworking of the layout to incorporate an equal number of seats so as not to compromise to economy of the building.

In addition, the unique nature of the table top truss requires a specific erection procedure not often utilized in projects of this scale. The cost and feasibility of utilizing shoring towers for erection is analyzed and the effect on the construction schedule assessed.

STRUCUTRAL DEPTH: TABLE TOP TRUSS DESIGN

PREFACE

In re-designing the roof structure of the Compton Family Ice Arena, significant changes were required throughout the building. For the purposes of the analysis, only the eastern portion containing of the facility containing the main arena was considered. The existing structure contains an expansion joint at this juncture making this a practical approach. The Image below show the building used within the scope of this thesis as separated between column lines six and seven. A larger representation of this and other typical floor plans can also be found in appendix C.

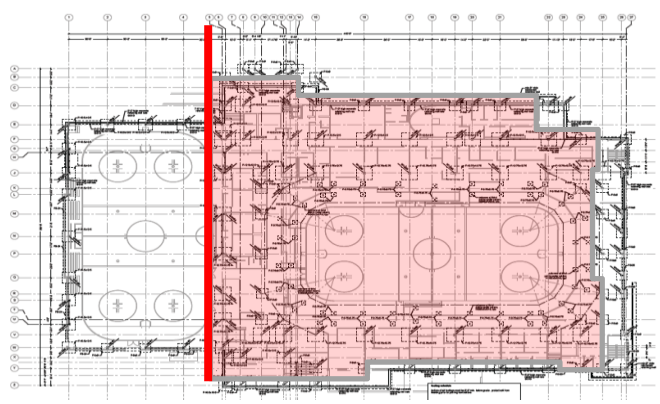


Figure 12: Foundation plan highlighting the portion of the complex focused on in this thesis

BACKGROUND

The table top truss utilizes a rectangular system of box trusses pulled in over the main ice sheet. Central loads are then distributed to super columns at the corners of the bowl by four leg trusses extending radially outward from the corners of the rectangle. The system strives to substantially shorten spans without obstructing the necessary clear span thus reducing necessary steel weight and ultimately achieving cost savings. While the economic advantages are incredibly attractive, this system strives to do more in working to the greater good of the entire building through coordination among all design factions.

When approaching arena design or stadia design of any sort it is import to assess the usage of the building in accordance with the structural and functional design. While the roof system primarily supports the roof live and dead loads it is also responsible for supporting the cat walk necessary for lighting and maintenance as well as loads from rigging beams and substantially sized scoreboards.

Striving to make each of these systems work in harmony with one another, the table top truss aligns the major functions of the space creating a central grid mapped by the rectangle of the table top truss. The box formation of the trusses allows easy access to the catwalks which in turn act as the frame of the central rigging grid necessary for all lighting staging and sound equipment erection for events within the arena. In the case of this re-design, these loads account for almost 100,000 pounds of additional dead load on the trusses plus associated live loads. Aligning the system not only simplified analysis but added long term value.

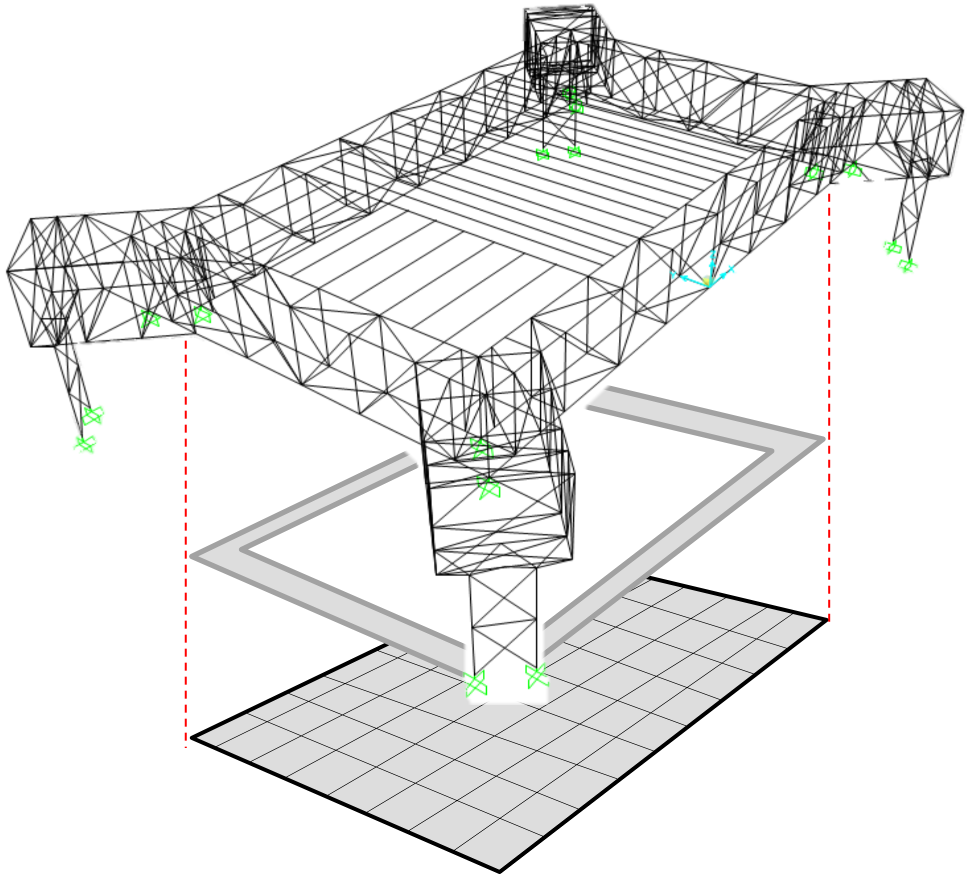
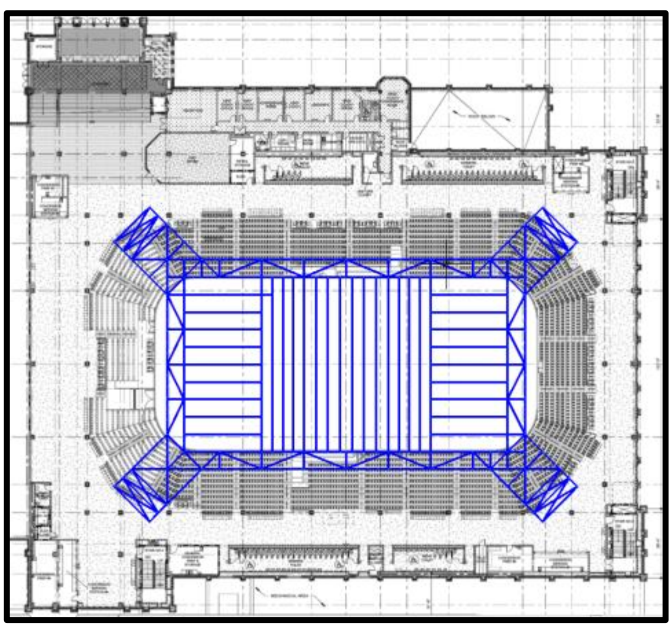
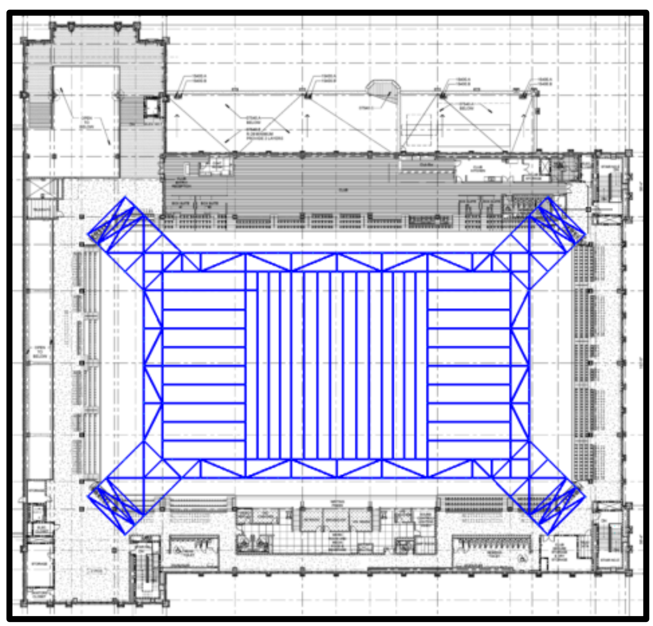


Figure 13: (Above) Schematic showing alignment among table top truss, cat walk and rigging grid.

Figure 14: (Below) Existing architectural plans at concourse (left) and club (right) levels with superimposed truss. Drawn to Scale.

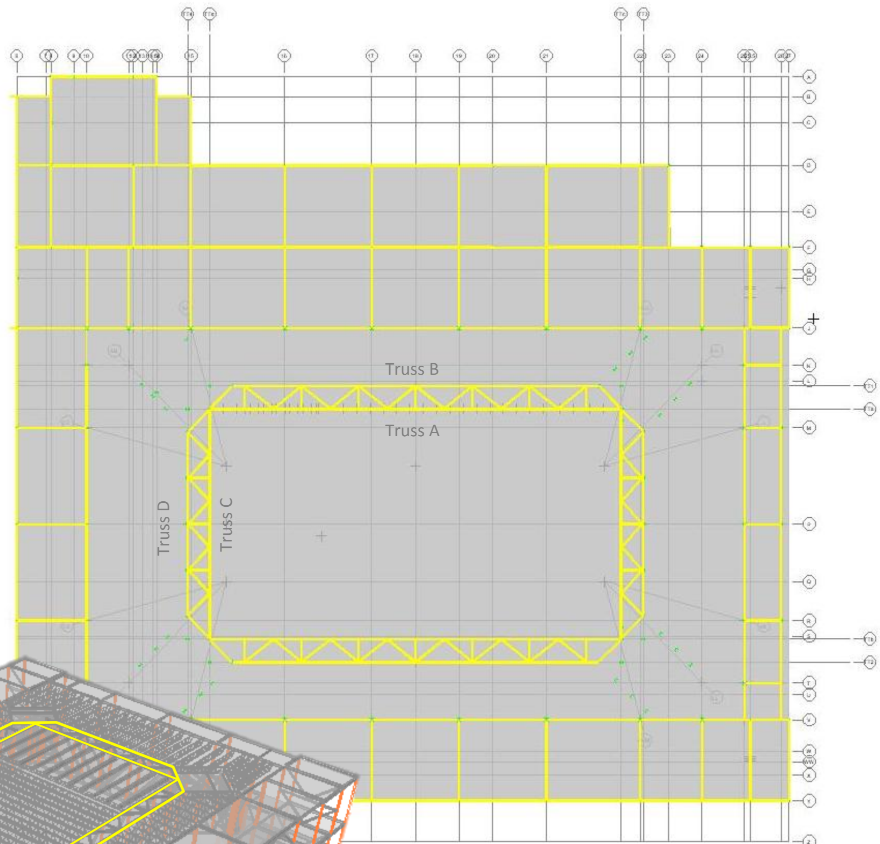




TRUSS LOCATION

The trusses are designed to maintain the fairly simple grid of the existing structure, align with the catwalk and rigging plans, and shorten spans as much as possible. The image in figure 15 shows the box truss in plan view superimposed over the existing low roof plan. As made visible, the trusses fit naturally and efficiently into the space and create an entirely new visual experience at the roof level. With the exception of two column shifts discussed here-after, the remainder of the roof framing is not only unchanged but incorporated into the new load path.

The core of the table top truss is comprised of eight individual trusses framing the inner and outer perimeters of the configuration. Trusses A and C span the inside taking loads from the central space while trusses B and D are removed 9’ -2’’ from A and C respectively and designed to carry the loads from the perimeter.



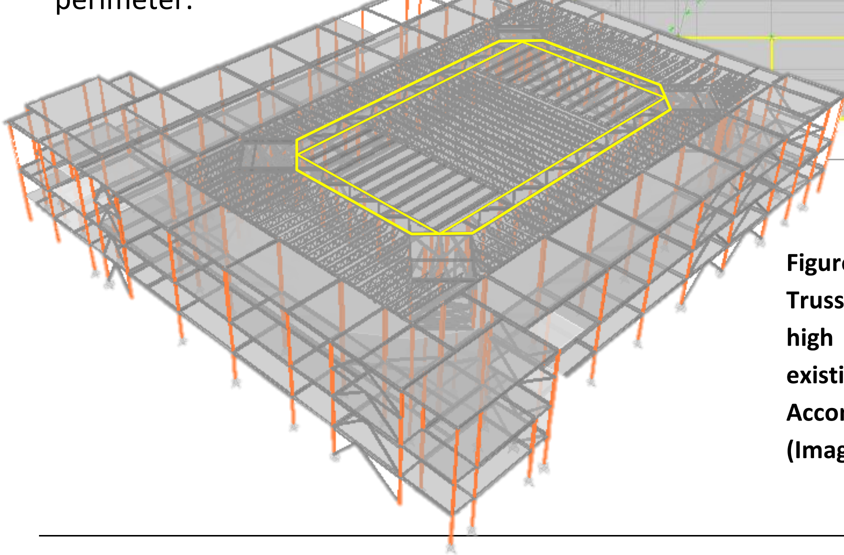


Figure 15: The eight trusses of the “Table Top Truss” overlaying the existing framing grid at the high roof level. (3D model at left) as well as existing low roof (Plan View at Right) Accompanying three dimensional representation. (Images extracted from ETABS)

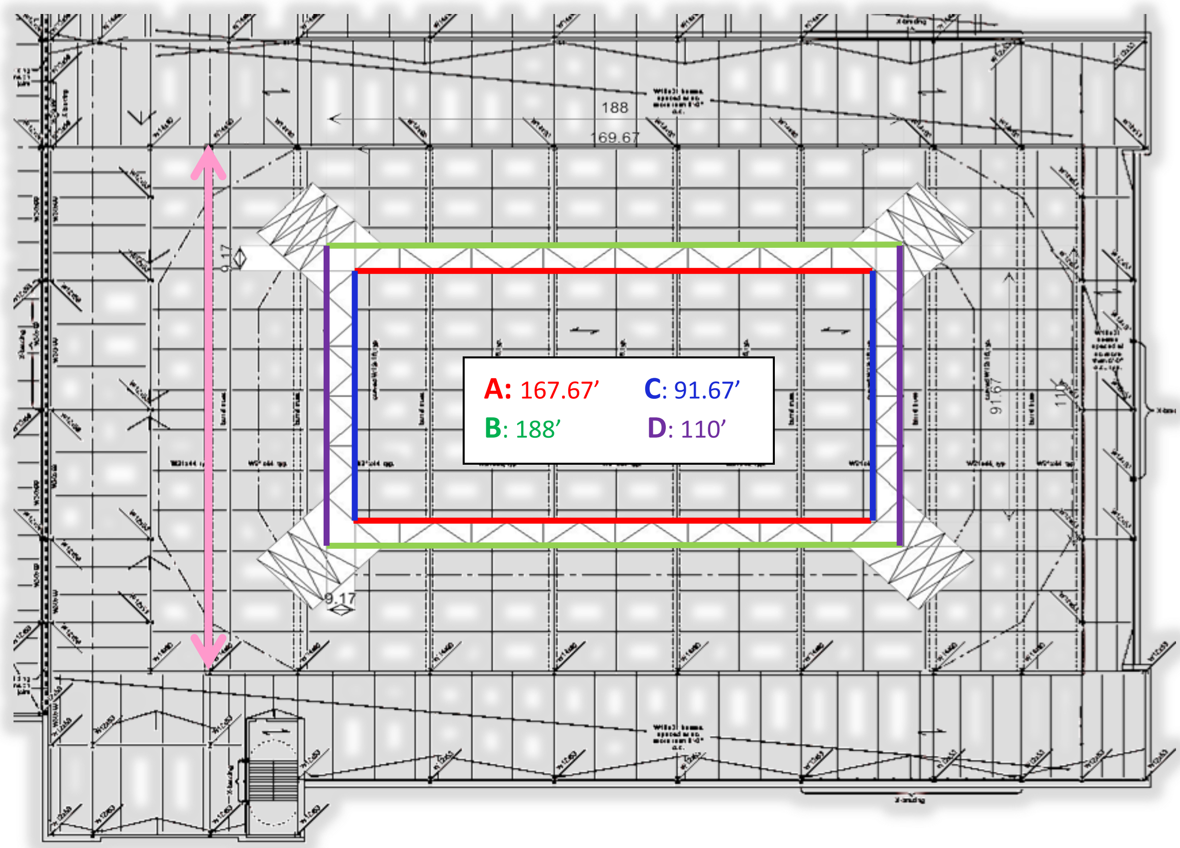


Figure 16: Comparison of span lengths between Proposed and existing systems. Truss spans color coded and called out in pan view.

This configuration allowed the designer to shorten the existing long span by almost 65’-0’’ decreasing the distance from 156 ft. to 91’-8’’ as can be seen in Figure 16. While the longest truss in the table top, Truss B, extends 188’-0’’, this design calls for only two trusses as this length as opposed to the existing eight trusses currently in place.

The most important factor effecting location though was the ultimate column placement at the four corners of the structure. These columns interrupt the bowl seating at the club level requiring seat relocation and sight line analysis (further information provided in the Architectural Breadth of this paper) as well as structural coordination. The eight trusses distribute their load to the four legs of table top to then be carried to the outside of the bowl where the loads are taken by eight super columns. Figure 17 shows the location of the entire roof structure superimposed over the initial structural plan at each level. It highlights the interaction between the existing structure and new roof design so as to highlight the logical integration of the two. The super columns have been highlighted at each level showing their unobstructed path to the ground level. This structure was designed particularly to clear the concourse level so as to avoid losing excessive seating. Additionally, this allowed for the bowl design to remain undisturbed in the design process. Provided in Figure 18 are typical corner sections at the concourse and club levels showing the exact location of the newly introduced super-columns as they line up with the existing structure.

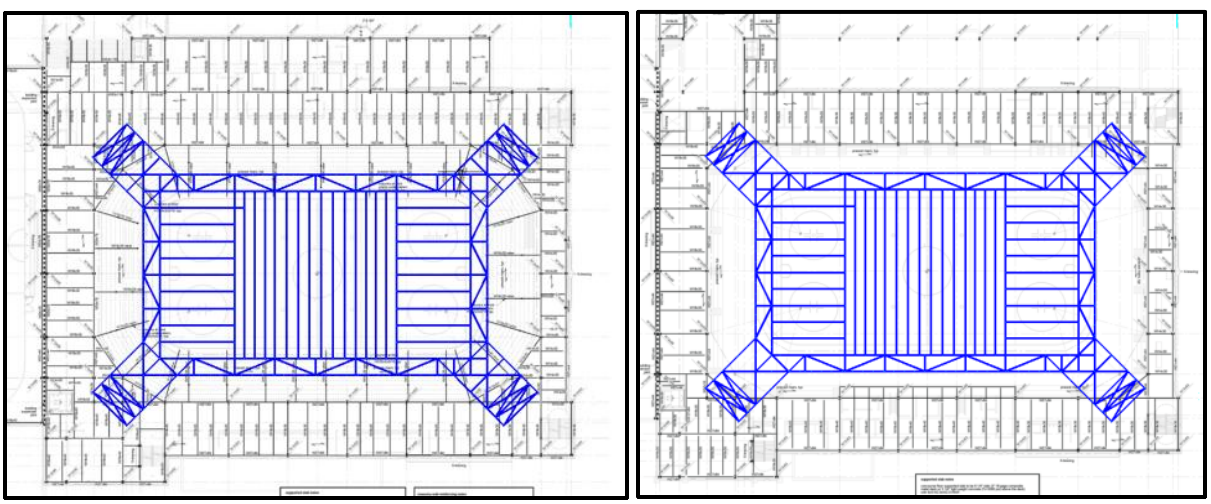
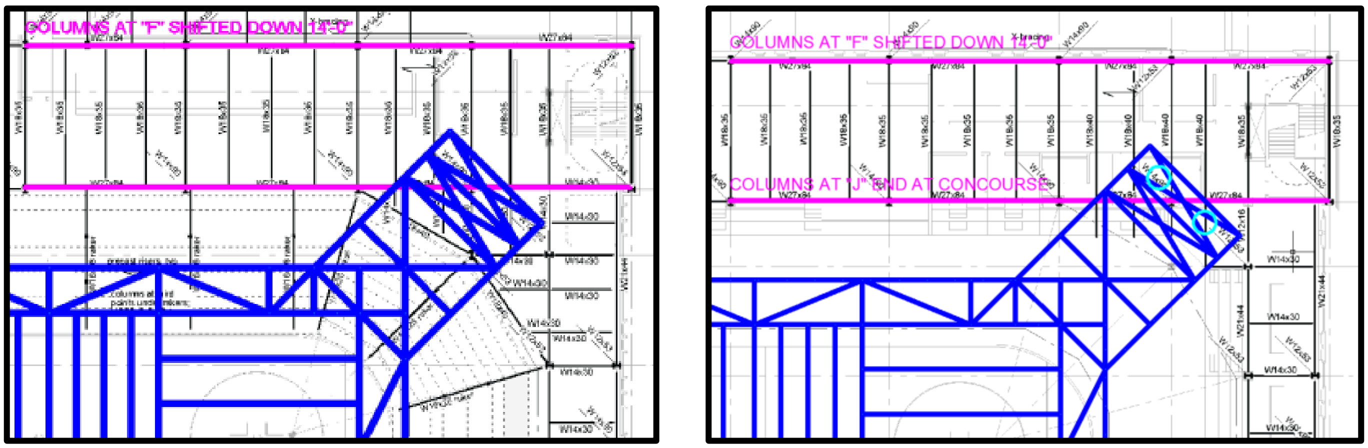


Figure 17: Table Top Truss Superimposed over structural plans at concourse (LEFT) and club (RIGHT) levels.

While much of the surrounding structure was left untouched, there were adjustments necessary to accommodate the changes. In order to maintain the symmetry of the structure necessary for stability, the columns at grid lines F and Y were pulled in by fourteen feet as seen in Figure 19. This change moved the columns from one side of a hallway to the other thus requiring very minimal architectural adaptation. In addition, the columns at the exterior of the building were found to adequately carry the increased loads.



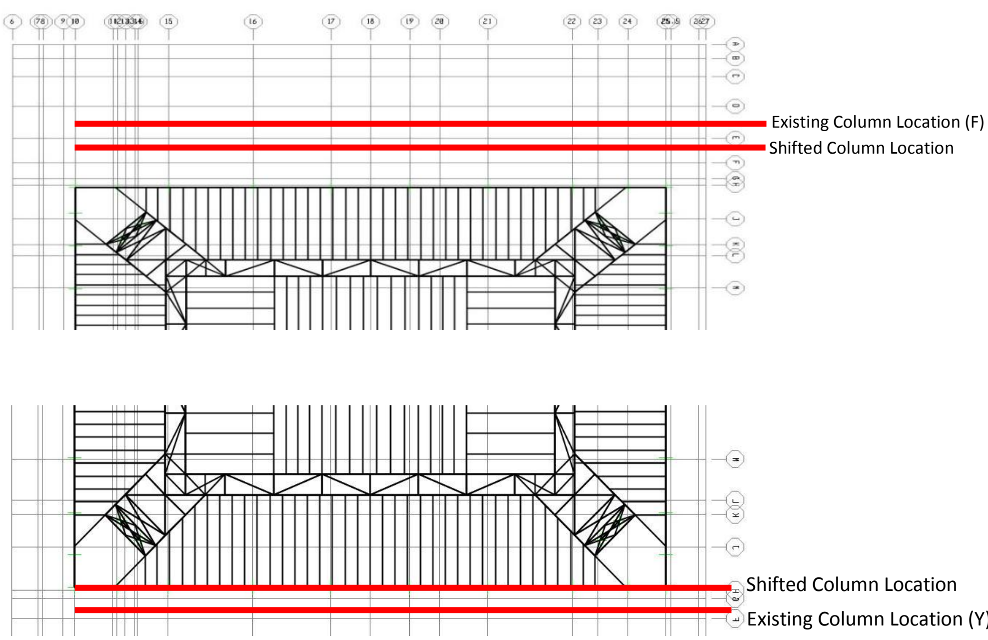


Figure 18: (Above) Typical corner column impact on surrounding structure at Club (Top) and Concourse (Bottom) levels.

Figure 19: (Left) Depiction of column shifts necessary to accommodate roof structure.

The East and West ends of the roof structure were designed to line up exactly with the columns at grid lines 10 and 25.5 and thus seamlessly fit into the existing structural layout. These column lines were restricted by surrounding code required egress thus posing a greater challenge if altered.

TRUSS ANALYSIS

LOADS

Loads for the table top truss structure were calculated in accordance with the initial design loads. Each truss was analyzed under a number of combinations at which point it was determined that 1.2D + 1.6L would be the controlling load case for the design. The results of each analysis can be found in appendix A justifying this conclusion. The member forces were calculated by hand using the joint method for truss analysis and altered per load case through utilization of an excel table. Each truss was then further checked for force and deflection using a RISA2D model. A summary of the loads can be found in Figure 20.

Live Loads	
Occupancy Type	**Load (PSF) (Per IBC2009 & ASCE7-10)**
Roof	20PSF
Dead Loads	
Material	**Load (PSF)**
Roof Deck	4.46
Single Ply membrane	2
Rigid Roof Insulation(2 layers)	4.5
Vapor Retardent	0.7
Exterior Roof Sheething	2
Catwalk/Rigging Loads	7

Figure 20: Summary of Loads used for Gravity Analysis

DESIGN

Once loads were calculated the members were analyzed and designed to comply. All trusses are Pratt trusses as is typical in this type of structure. The design process was multi step beginning with a hand selection of appropriate section choices based on force and deflection calculations acquired in analysis. Appendix B contains all calculations used to define sections chosen to fulfill minimum sizing requirements based on the AISC Steel Construction Manual. The trusses were then re-modeled in SAP with the assigned sections and run to utilize the programs optimization capabilities. In the final steps of the process, all checks were taken into account and typical member sizes were chosen for each truss so as to develop efficient and economic trusses. All trusses were designed with W14 chords, and 2L shapes for diagonals and posts.

A break-down of this procedure is shown below for each truss to represent the complete process by which design sets were chosen. Information includes final truss dimensions, chord and bracing choices, and deflection and optimization models.

TRUSS A

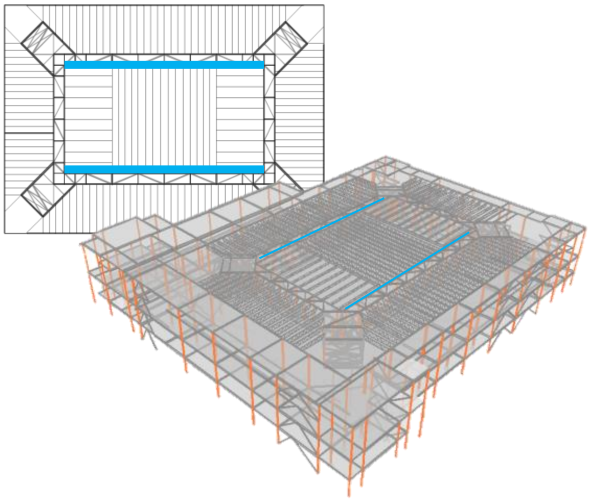
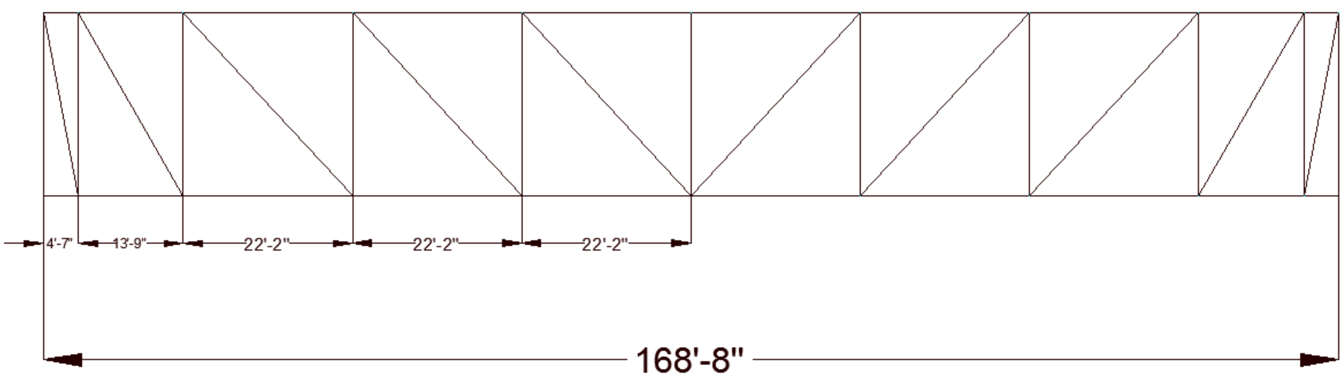
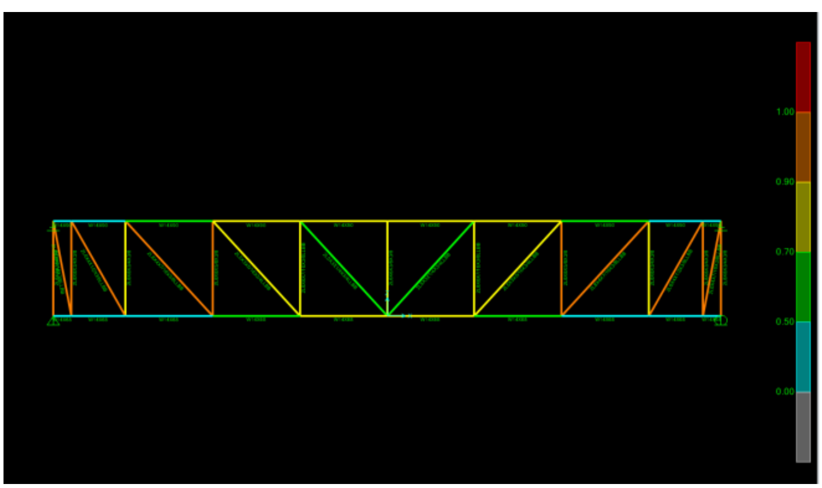
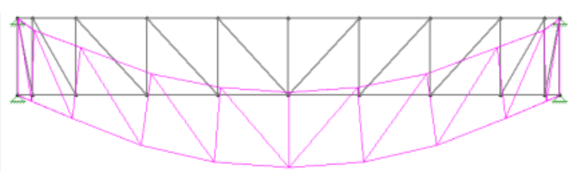


Table 15: Material List By Section Property				
Section	Quantity	Total Length (in)	Total Length (ft)	Total Weight
2L5X3X1/4X3/4LLBB	2	784.092	65.341	0.863
2L5X3X5/16X3/4LLBB	2	784.092	65.341	1.069
2L5X5X7/16X3/8	2	586.409	48.867	1.403
2L6X4X1/2X3/8LLBB	1	331.917	27.660	0.894
2L6X6X3/8X3/8	1	288	24.000	0.715
2L8X4X7/16X3/8LLBB	2	784.092	65.341	2.268
2L8X6X7/16X3/8LLBB	2	576	48.000	1.96
2L8X8X3/4X3/8	5	1440	120.000	9.392
2L8X8X5/8X3/8	2	576	48.000	3.169
W14X68	10	2036	169.667	11.547
W14X90	12	2655.917	221.326	19.958

Final Design Take-Off's for Truss A







Top: Exaggerated deflected shape modeled in RISA2-D *Left*: SAP2000 optimization output showing capacity of each designed truss member. Red indicates an overstressed member vs. grey representing a member carrying little to none of its load capacity.

TRUSS B

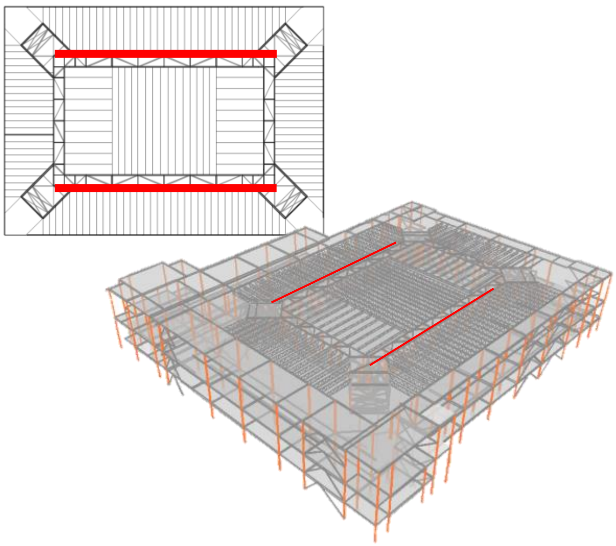
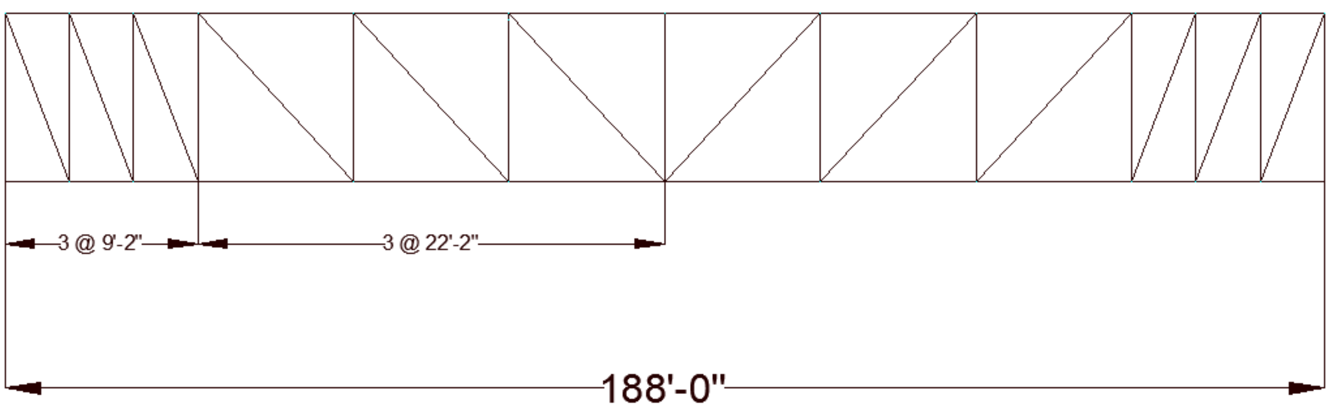
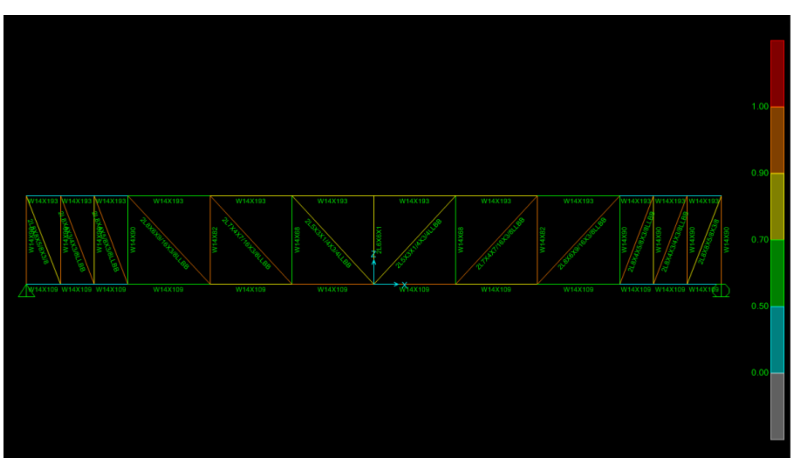
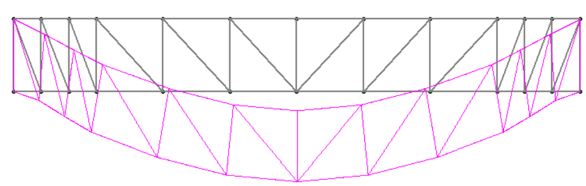


Table 23: Material List By Section Property				
Section	Quantity	Total Length(in)	Total Length (ft)	TotalWeight
W14X68	2	576	48	3.267
W14X82	2	576	48	3.92
W14X90	8	2304	192	17.313
W14X109	12	2256	188	20.471
W14X193	12	2256	188	36.336
2L5X3X1/4X3/4LLBB	2	784.092	65.341	0.863
2L6X6X1	1	288	24	1.797
2L7X4X7/16X3/8LLBB	2	784.092	65.341	2.061
2L8X4X3/4X3/8LLBB	2	616.584	51.382	2.972
2L8X4X5/8X3/8LLBB	2	616.584	51.382	2.5
2L8X6X9/16X3/8LLBB	2	784.092	65.341	3.38
2L8X8X5/8X3/8	2	616.584	51.382	3.392

Final Design Take-Offs for Truss B







Top: Exaggerated deflected shape modeled in RISA2-D *Left*: SAP2000 optimization output showing capacity of each designed truss member. Red indicates an overstressed member vs. grey representing a member carrying little to none of its load capacity.

TRUSS C

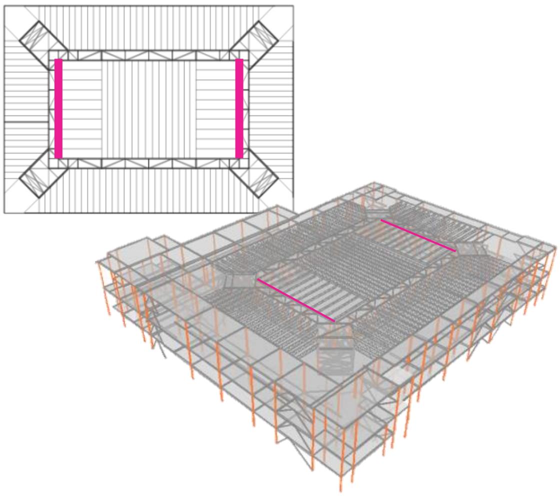
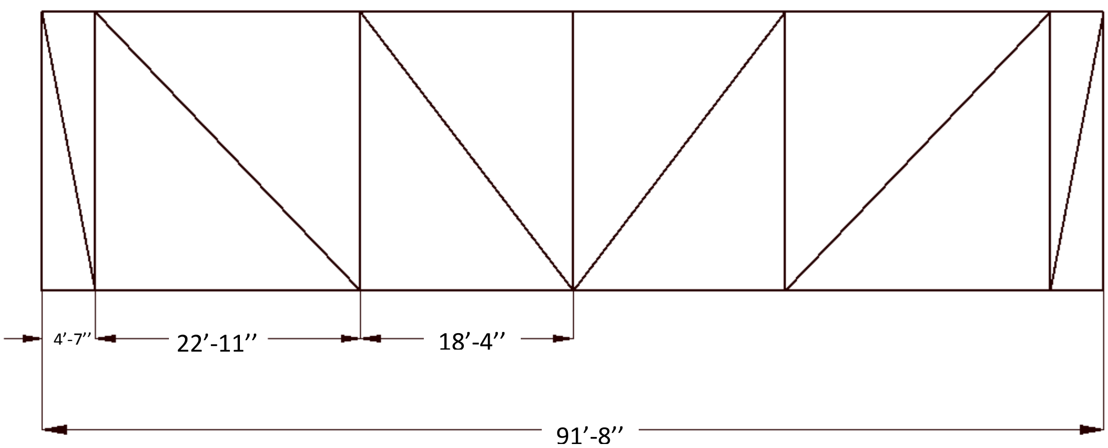
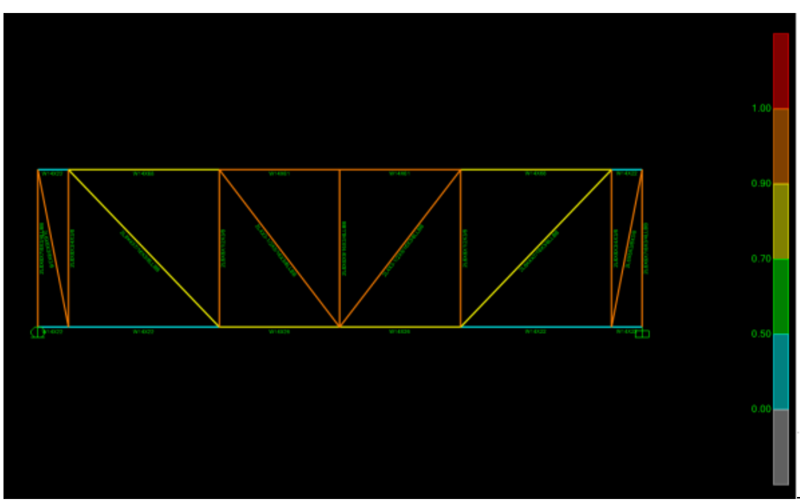
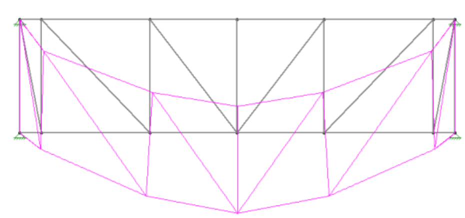


Table 18: Material List 2 - By Section Property				
Section	Quantity	Total Length (in)	Total Length (ft)	Total Weight (K)
W14X22	8	1210	100.833	2.227
W14X48	1	288	24.000	1.152
W14X61	6	1566	130.500	7.949
W14X68	2	576	48.000	3.267
W14X74	1	288	24.000	1.78
W14X82	1	288	24.000	1.96
2L4X3X1/4X3/8LLBB	2	724.828	60.402	0.695
2L5X3X1/2X3/8LLBB	2	586.409	48.867	1.249
2L7X4X3/8X3/8LLBB	2	796.414	66.368	1.807

Final Design Take-Offs for Truss C







Top: Exaggerated deflected shape modeled in RISA2-D *Left*: SAP2000 optimization output showing capacity of each designed truss member. Red indicates an overstressed member vs. grey representing a member carrying little to none of its load capacity.

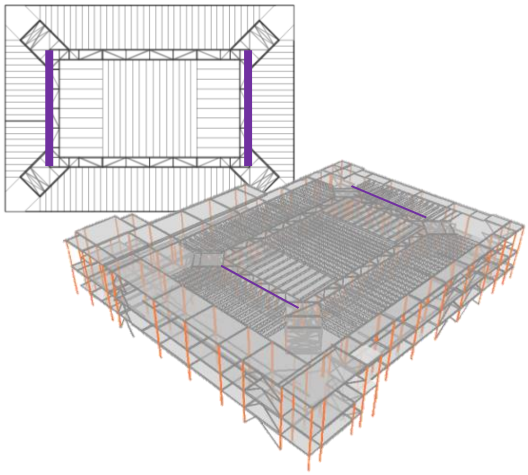
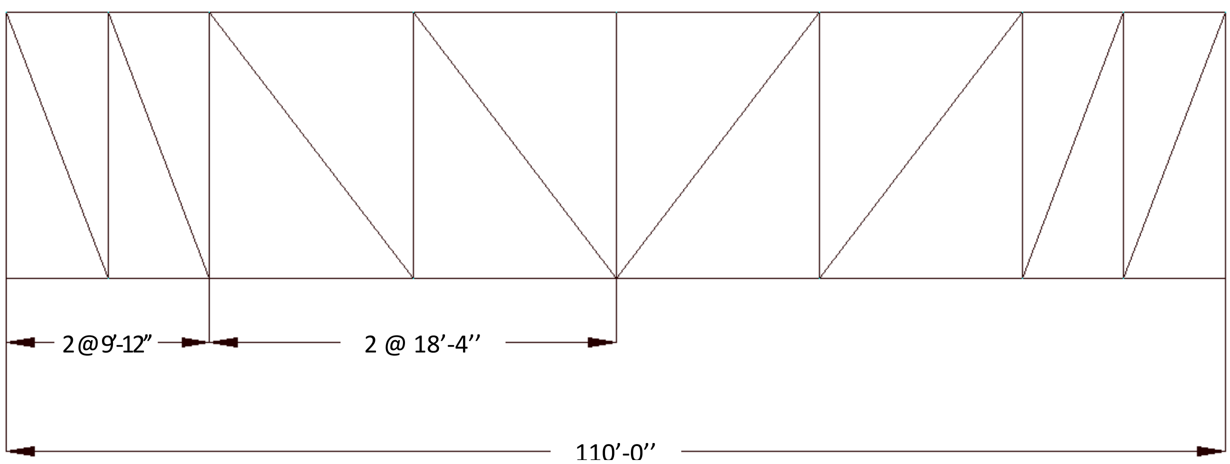
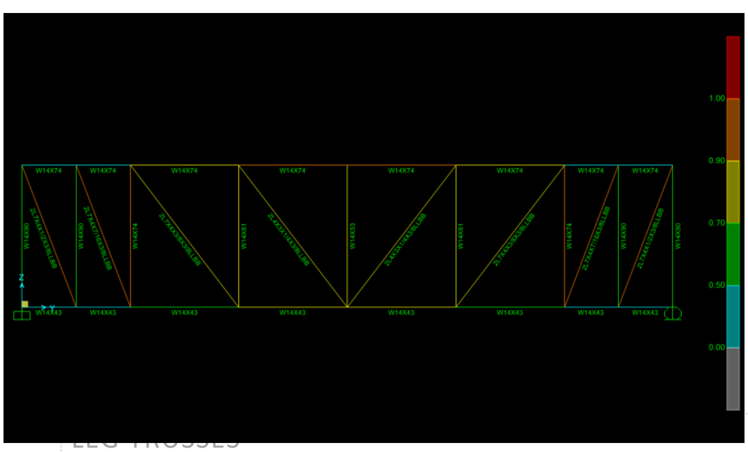
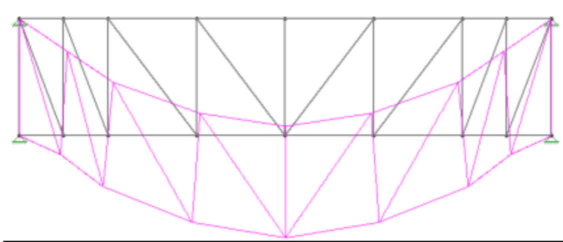


Table 23: Material List By Section Property				
Section	Quantity	Total Length (in)	Total Length (ft)	Total Weight (K)
W14X43	8	1320	110.000	4.716
W14X53	1	288	24.000	1.274
W14X61	2	576	48.000	2.924
W14X74	10	1896	158.000	11.721
W14X90	4	1152	96.000	8.657
2L4X3X1/4X3/8LLBB	2	724.828	60.402	0.695
2L7X4X1/2X3/8LLBB	2	616.584	51.382	1.836
2L7X4X3/8X3/8LLBB	2	724.828	60.402	1.644
2L7X4X7/16X3/8LLBB	2	616.584	51.382	1.621

Final Design Take-Offs for Truss D







Top: Exaggerated deflected shape modeled in RISA2-D *Left*: SAP2000 optimization output showing capacity of each designed truss member. Red indicates an overstressed member vs. grey representing a member carrying little to none of its load capacity.

The leg trusses are responsible for carrying the load to super columns at the exterior of the frame. Connecting directly to the trusses through a combination of bolting and welding, a rigid connection is created between the framing elements. While the design of this connection was beyond the scope of this thesis, engineers at Walter P. Moore lended their expertise in suggesting a suitable connection.

To achieve necessary strength, full penetration welds must be created at the flanges of the connection between Trusses A and C as well as Trusses B and D. The leg trusses frame in at this connection as well requiring both full penetration welds and bolting at every location. This includes a connection to Truss B, Truss D and the intersection of Trusses A and B. Additional

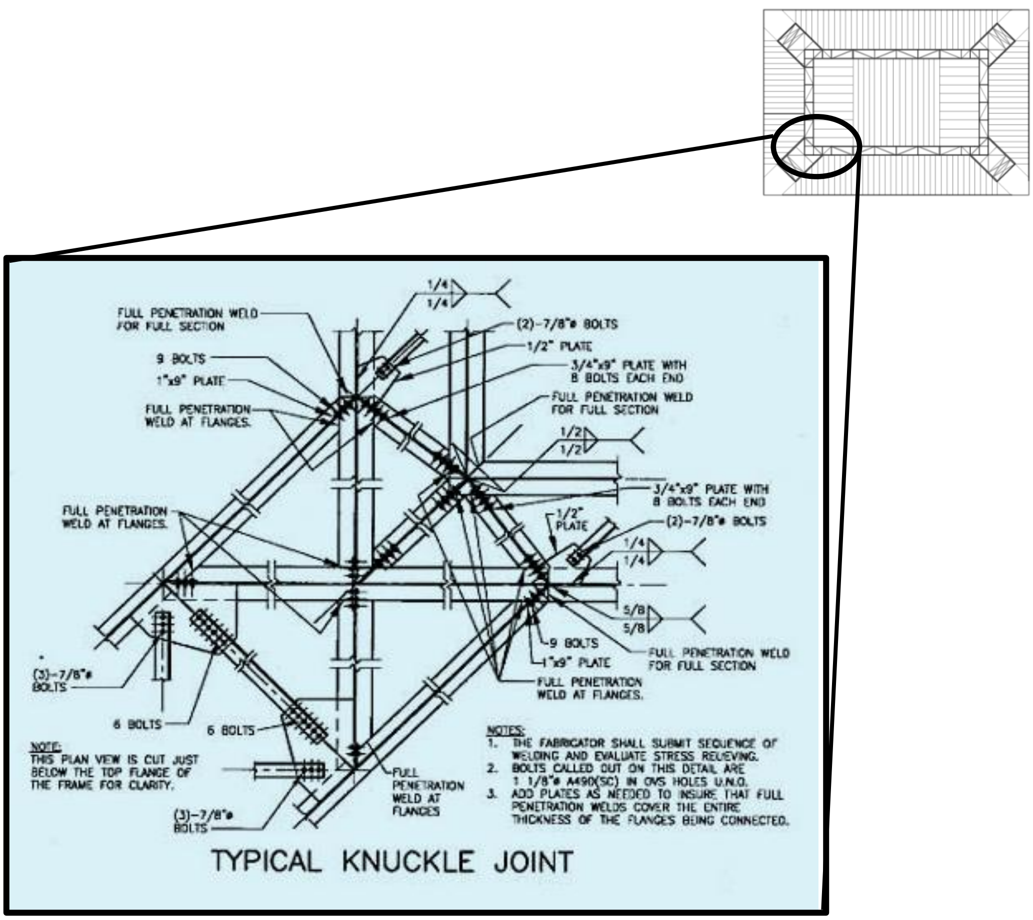


Figure 21: Typical Connection for Knuckle Joint as Provided by Walter P. Moore Engineers

plates ensure that full penetration welds cover the entire thickness of all flanges being connected. A typical detail as provided by Walter P. Moore can be seen in figure 21.

The sizing of the leg trusses was conducted in the same manner as the eight primary trusses. To carry the combined loading of the emerging trusses, the chords of the leg trusses required W27's at the top chord and typical 2L8x8x1 diagonals. A complete material list can be found in the material take-off conducted for cost analysis later in this section.

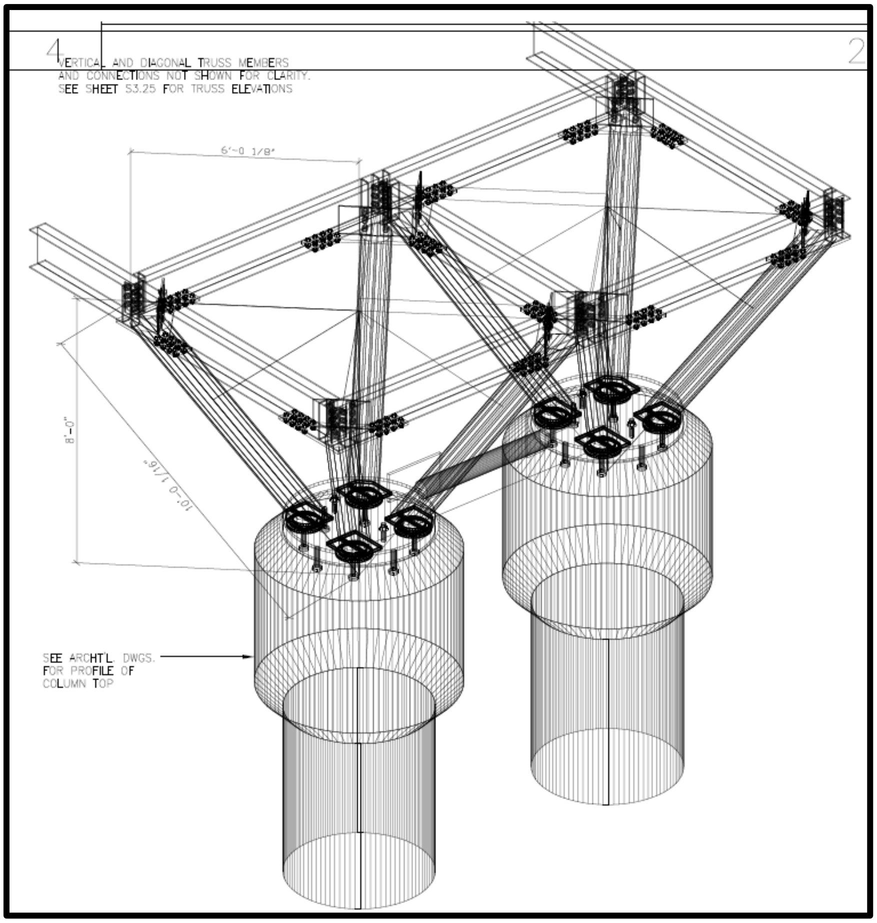
COLUMN CONNECTIONS

The connections between the leg trusses and the columns are critical to the system and achieved through the use of pot bearings. These connections allow for the movement of the trusses without compromising strength of connection. These bearings are capable of transmitting forces while absorbing deformations and rotations. A larger depiction of a pot bearing can be found in Appendix E.

This type of bearing can take exceedingly large vertical loads ranging from 5,000-30,000 kN making it a desirable approach to this system.

As can be seen in Figure, The legs of the table top come down on 10'' pipes to the pot bearing where the load is then transferred to the columns. Three dimensional representations of this schematic can be seen on the following page. In addition, the image below shows the basic geometry of the members.

Figure 22: Extruded SAP model highlighting structural geometry



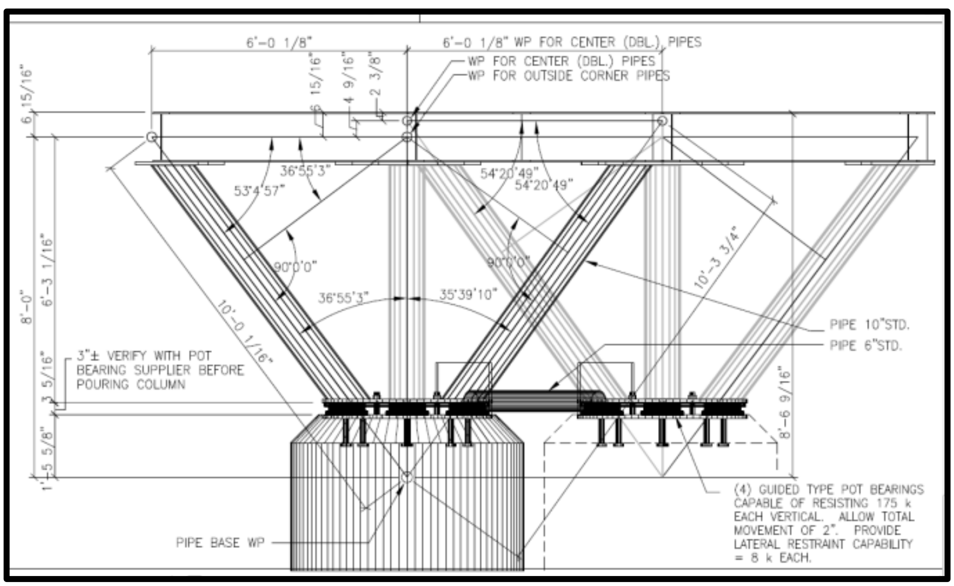


Figure 23: Typical connection at corner column locations.

SUPPLEMENTAL FRAMING MEMBERS

The infill steel spanning both the inside and outside of the table top truss is comprised of conventional deep long span joists. Joists were found to be more efficient than beams in this case due to the long spans and lack of restrictions or depth requirements. Joist loads were calculated using the same loads specified above in Figure 20 and sized for necessary spans using the Vulcraft Steel Joist and Joist Girder manual. All hand calculations can be found in appendix C.

The interior joists were designed so as to distribute load evenly to each side of the rectangular box truss. For this reason, center joists span in North-South direction framing into Truss A, while the outer joists are turned 90° to frame into truss C. A number of layouts were investigated to determine the most efficient spacing before the arrangement in figure 23 was decided most efficient. 60DLH16 joists span in the long direction across the eighty eight foot center section at 5'-6'' O.C. . 26LH13 joists, also at 5'-6'' O.C, frame into truss C across the two forty foot outside sections. To carry the interior point loads from the 26LH13 joists as well as the standard roof loads, joists girders were designed to span the 91'-8'' on either side of the 60DLH16 grouping. While it is typical practice to design and special order a joist girder of this size and specificity, a 100G10N19F joist girder was chosen from the Vulcraft Steel Joist and Joist Girder manual in order to better estimate final costs.

The joists selected to span from outside trusses to the surrounding columns were 44LH17 joists. These were sized in the same manner as the previous joists and calculations can be found in appendix C.

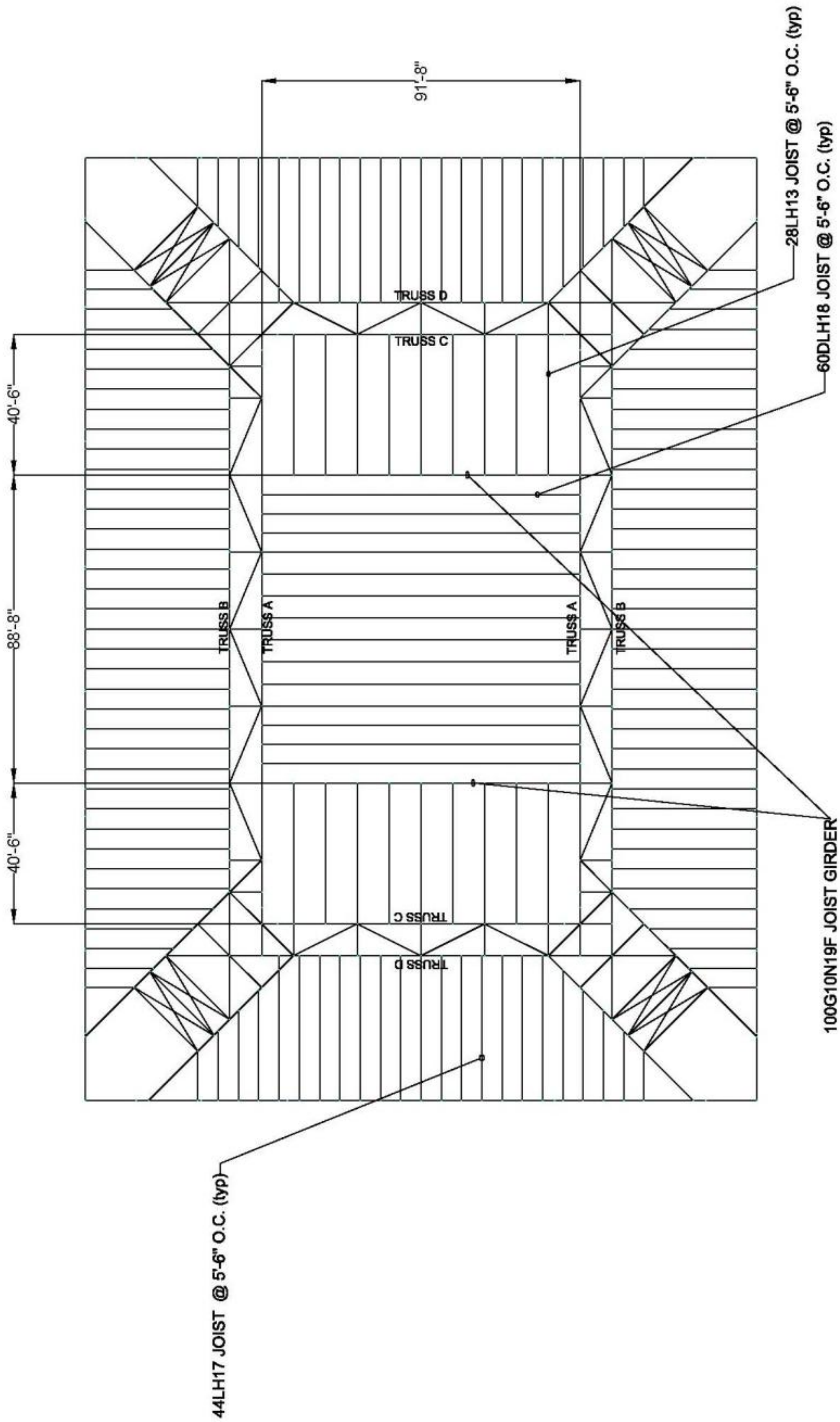


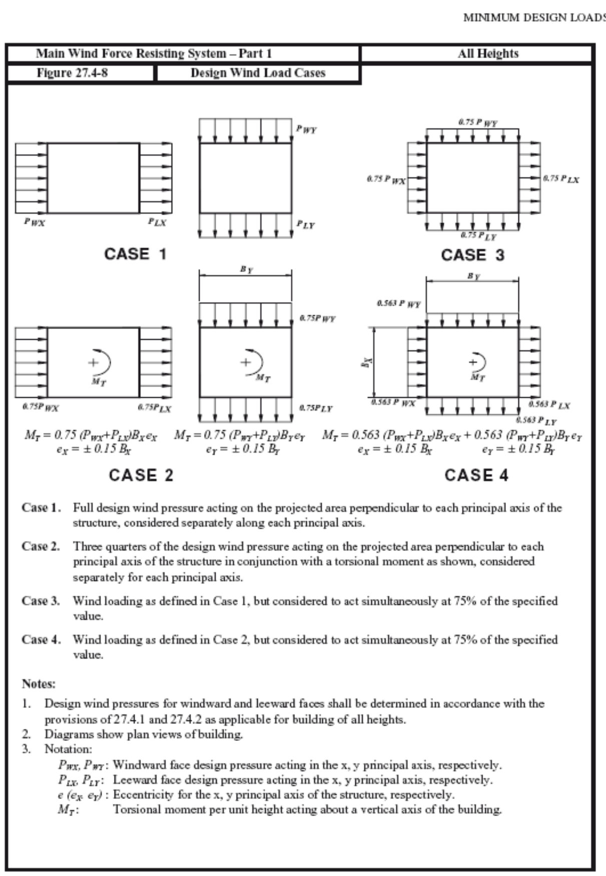
Figure 24: Joist layout and spacing

LATERAL DESIGN

The lateral force resisting system had to be adjusted to account for the new roof height as well as the column shifts made to accommodate the roof system. Column lines were pulled inward at grid lines F and Y thus creating a new shape for the roof structure and inherently adjusting the roof diaphragm created by the decking spanning this section. So as to adequately absorb the lateral loads incurred, the braced frames at these gird lines were also shifted inward.

In order to assess to capacity of the system with the given adjustments, Wind pressures were re-calculated (see Appendix A) and applied to an adjusted model created in ETABS. The new model accounted for frame shifts, diaphragm changes and height adjustments but left original framing in place.

Load cases were applied in compliance with ASCE7-05 pictures below to find the controlling load case.


MINIMUM DESIGN LOADS

Main Wind Force Resisting System – Part 1 | All Heights

Figure 27.4-8 | Design Wind Load Cases

Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

Case 2. Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3. Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notes:

1. Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of 27.4.1 and 27.4.2 as applicable for building of all heights.
2. Diagrams show plan views of building.
3. Notation:
   - P_{WX}, P_{WY} : Windward face design pressure acting in the x, y principal axis, respectively.
   - P_{LX}, P_{LY} : Leeward face design pressure acting in the x, y principal axis, respectively.
   - e (e_X, e_Y) : Eccentricity for the x, y principal axis of the structure, respectively.
   - M_T : Torsional moment per unit height acting about a vertical axis of the building.

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Figure 25: Load Cases defined by ASCE7-05

	Total Force X-Direction	Total Force Y Direction	Total Force
	Load Case 1		Load Case 3
ROOF	73.16	111.85	137.25
CLUB	110.52	171.75	210.75
CONCOURSE	72.19	115	140.25
	Load Case 2		Load Case 4
ROOF	54	83.25	103.1
CLUB	82.5	128.25	158.2
CONCOURSE	54	86.25	105.7

Figure 26: Maximum force calculations for each of the four defined load cases

The controlling load case was then decided upon as load case three because it produced the greatest forces of the four cases. Once this was decided, columns beams and braces were checked for combined capacity. Axial force, bending in the X direction and Bending in the Y direction were taken from the Etabs Analysis and checked against member capacity. A sample of this calculation can be seen in Figure 26 below and full calculations can be found in Appendix A. Any interaction capacity resulting in a value greater than one required redesign to carry the new loads.

Lateral Column Checks									
Section	Level	KL	P	bx	by	P	Mx	My	Capacity Check
w10x45	1(J)	16	3.27	5.19	11.70	57.15	2.82	74.31	0.94
w10x45	2	16	3.27	5.19	11.70	20.76	0.54	11.04	0.16
w10x45	3	29	10.70	7.82	11.70	0.14	0.12	17.40	0.22
w10x45	1(H)	16	3.27	5.19	11.70	57.23	3.89	82.73	1.05
w10x45	2	16	3.27	5.19	11.70	20.83	0.62	12.36	0.17
w10x45	3	29	10.70	7.82	11.70	0.15	0.19	19.11	0.24

Figure 27: Sample calculation used in determining column capacity at braced frames.

It was found that the columns failed at the base of many of the braced frames. These columns were upsized to account for this new load and reanalyzed for sufficiency. Columns at braced frames 7, 8, 9, 10, and 11 failed as shown in Figure 28. A typical change is shown at braced frame seven. The member in red indicates the over stressed original designation and the member label in black shows the adjusted size.

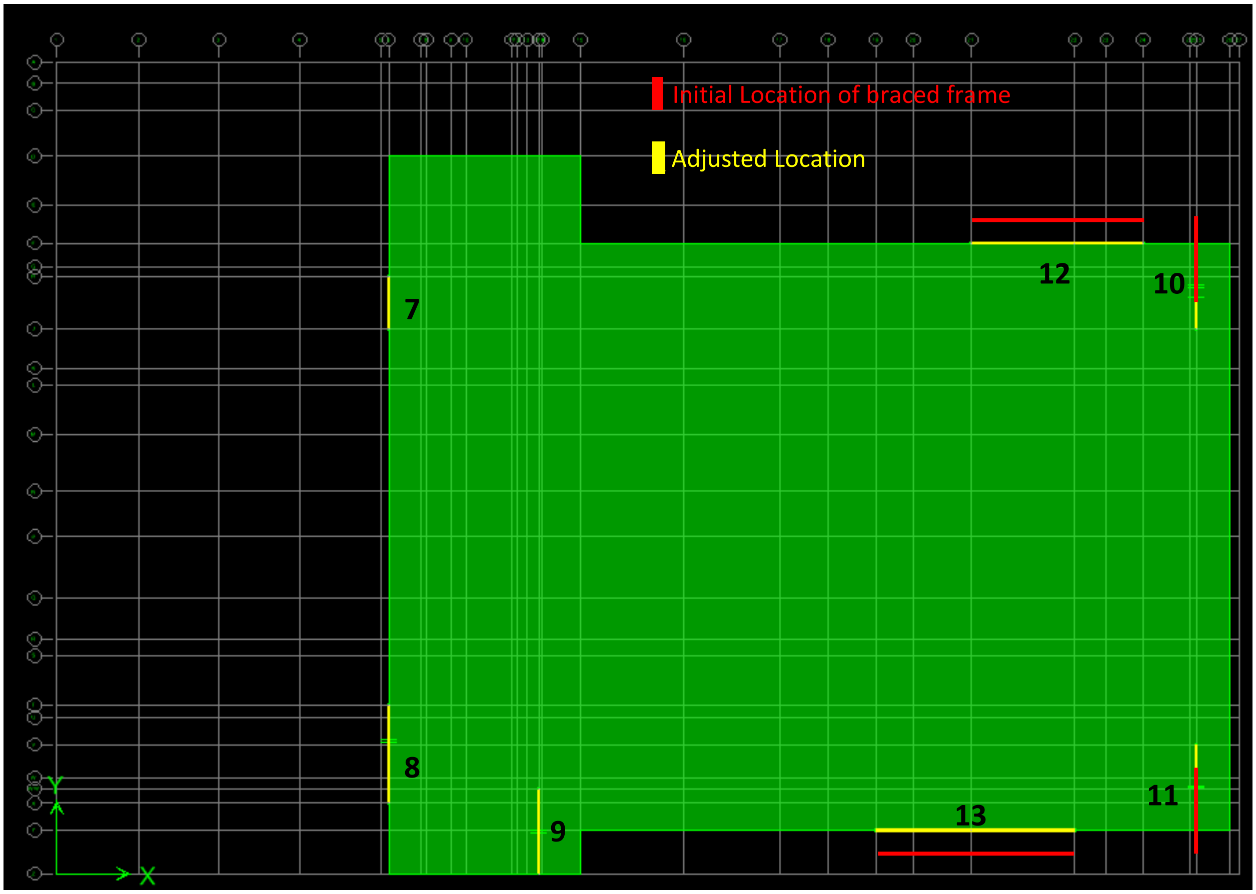
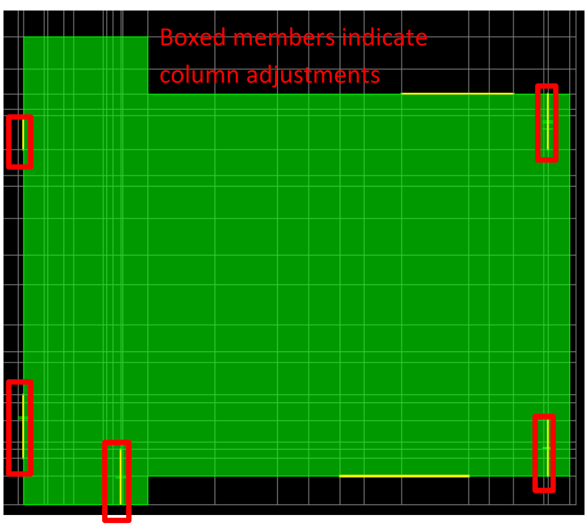
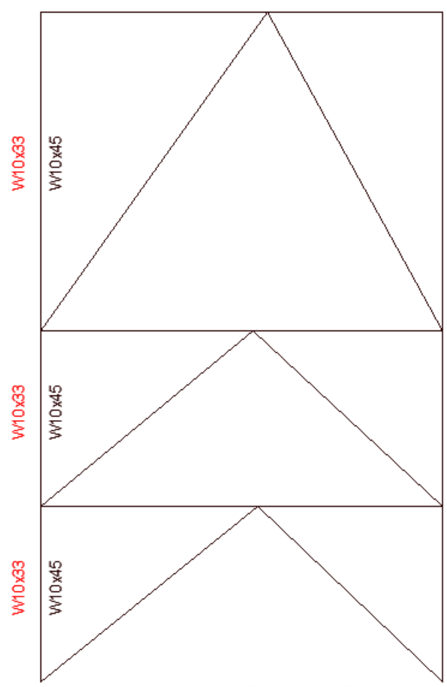


Figure 28: (Above) Shifted Locations of braced frames. Figure 29: (Below) Plan view of lateral system with inadequate frames shown in red. Figure 30: (Right) Typical braced frame with upsized columns to account for additional lateral load.





A complete summary of column changes can be seen in Figure 30. In addition, extended calculations can be found in Appendix A.

Member	Level	KL	p	bx	by	P	Mx	My	Initial Capacity		Member	p	bx	by	P	Mx	My	Adjusted Capacity
w10x33	1(V)	16	4.68	7.89	16.90	89.42	0.94	75.89	1.38	REDESIGHED TO:	w10x45	3.27	5.19	11.70	89.42	0.94	75.89	0.99
w10x33	1(Y)	16	4.68	7.89	16.90	66.50	1.08	56.44	1.03		w10x45	3.27	5.19	11.70	66.50	1.08	56.44	0.74
w10x33	1(F)	16	4.68	7.89	16.90	89.77	1.56	73.47	1.35		w10x45	3.27	5.19	11.70	89.77	1.56	73.47	0.96
w10x33	1(J)	16	4.68	7.89	16.90	84.03	1.03	74.03	1.35		w10x45	3.27	5.19	11.70	89.77	1.56	73.47	0.96
w10x33	1	16	4.68	7.89	16.90	67.56	1.17	54.68	1.01		w10x45	3.27	5.19	11.70	67.56	1.17	54.68	0.72
w10x33	1	16	4.68	7.89	16.90	67.56	1.63	56.09	1.03		w10x45	3.27	5.19	11.70	67.56	1.63	56.09	0.74
w10x45	1(X)	16	3.27	5.19	11.70	71.25	1.91	88.49	1.12		w10x49	2.43	4.43	8.38	71.25	1.91	88.49	0.82
w10x45	1(H)	16	3.27	5.19	11.70	57.23	3.89	82.73	1.05		w10x49	2.43	4.43	8.38	57.23	3.89	82.73	0.77

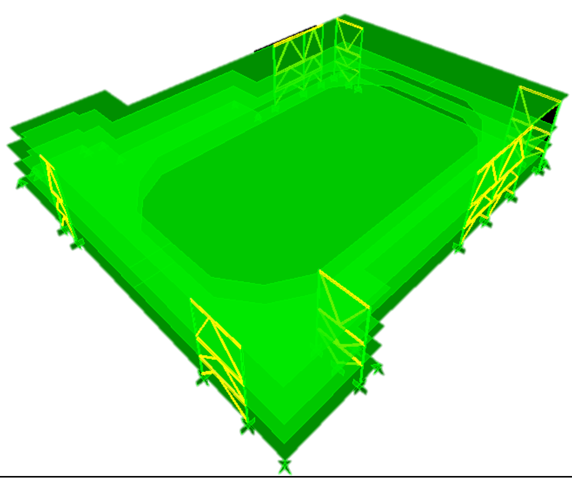
Figure 31: Summary of columns found inadequate to carry additional loads and corresponding adjusted sections

A similar analysis was performed as a check for bracing members to confirm axial capacity. All members were HSS8x8x3/16 except for braced frames 12 and 13 which utilized HSS12x8x3/16 shapes. A comparison showed that all tension members adequately carried their adjusted loads but the compression members fell just short at multiple locations at the concourse and club levels. Rather than resize each member individually, it was decided to upsize HSS8x8x3/16 braces to HSS8x8x1/4 members at the club and concourse level for braced frames 7, 8, 9, 10 & 11. A sample calculation can be seen in Figure 31 as well as complete analysis in Appendix A.

Bracing Member Checks						
Member	Section	Axial Load	KL(in)	KL(ft)	1.6P	ϕPn
D32	HSS8x8x.375	-44.12	240	20	-70.592	431
D33	HSS8x8x.376	66.04	308.914	25.74283	105.664	151
D34	HSS8x8x.377	-40.67	273.805	22.81708	-65.072	431
D35	HSS8x8x.378	41.94	282.136	23.51133	67.104	134
D36	HSS8x8x.379	-21.21	399.891	33.32425	-33.936	431
D37	HSS8x8x.380	20.98	396.011	33.00092	33.568	73.8
D14	HSS8x8x.375	-56.64	247.386	20.6155	-90.624	431
D15	HSS8x8x.376	76.67	343.641	28.63675	122.672	105
D16	HSS8x8x.377	-36.61	390.427	32.53558	-58.576	431
D17	HSS8x8x.378	41.08	436.807	36.40058	65.728	65.9
D18	HSS8x8x.379	43.49	297.692	24.80767	69.584	128
D19	HSS8x8x.380	-43.57	297.692	24.80767	-69.712	431

Figure 32: Sample calculations used to determine adequacy of existing braced members.

Once all members were redesigned to carry the lateral loads, the model was updated and run once again to verify period and drift requirements. The new system was found to be adequate for supporting lateral loads. The results can be seen below in figure 32.



Wind Story Displacement E-W Direction				
Floor	Displacement (in)			Allowable Displacement (in)
	Story Height (in)	Story Drift Ratio	Story Drift	
Concourse	192	0.000274	0.052608	0.48
Club	392	0.000256	0.100352	0.98
Roof	740	0.000196	0.14504	1.85

Wind Story DisplacementN-S Direction				
Floor	Displacement (in)			Allowable Displacement (in)
	Story Height(in)	Story Drift Ratio	Story Drift	
Concourse	192	0.000712	0.136704	0.48
Club	392	0.000769	0.301448	0.98
Roof	740	0.000779	0.57646	1.85

Figure 33: Wind Story Displacements taken from ETABS analysis.

**Wind was found to be controlling in technical report three and the same held true for the new building design. An adjusted building weight was calculated though to account for the new design and incorporated into the seismic analysis. These calculations can be found in Appendix A.

COST COMPARISON

As an economy comparison between the existing and re-designed roof structure, steel take-offs were taken from both high roof framing plans and priced based on the RS Means Cost Works.

EXISTING STRUCTURE

High Roof Framing System Take-Off & Cost Estimate

	Structural Member	Quantity	Weight/ft. (PLF)	Length (ft)	Total Weight (lb)	Total Weight (tons)	Cost/ft.	Cost
S151	W12x14	30	14	13	5460	2.73	$25.21	$9,833.07
S151	W12x26	12	26	13	4056	2.028	$41.06	$6,405.36
S151	W12x26	6	26	17.417	2717.052	1.358526	$41.06	$4,290.85
S151	W21x44	4	44	25.583	4502.608	2.251304	$65.86	$6,739.59
S151	W12x26	4	26	25.583	2660.632	1.330316	$41.06	$4,201.75
S151	W18x40	3	40	38.667	4640.04	2.32002	$61.15	$7,093.46
S151	W24x55	3	55	38.667	6380.055	3.1900275	$80.03	$9,283.56
S151	W18x35	3	35	36	3780	1.89	$54.65	$5,902.20
S151	W24x55	3	55	36	5940	2.97	$80.03	$8,643.24
S151	W12x14	1	14	14.75	206.5	0.10325	$25.21	$371.85
S151	W16x26	1	26	24.833	645.658	0.322829	$40.27	$1,000.02
S151	L4x4x3/8	12		15.647			$40.80	$7,660.77
S151	W24x55	1	55	38.417	2112.935	1.0564675	$80.03	$3,074.51
S152	W24x55	1	55	38.417	2112.935	1.0564675	$80.03	$3,074.51
S152	W16x26	1	26	24.833	645.658	0.322829	$40.27	$1,000.02
S152	W12x14	1	14	14.75	206.5	0.10325	$25.21	$371.85
S152	W12x14	30	14	13	5460	2.73	$25.21	$9,831.90
S152	W12x26	12	26	13	4056	2.028	$41.06	$6,405.36
S152	W12x26	5	26	17.417	2264.21	1.132105	$41.06	$3,575.71
S152	W21x44	3	44	25.583	3376.956	1.688478	$65.86	$5,054.69
S152	W12x26	2	26	25.583	1330.316	0.665158	$41.06	$2,100.88
S152	W18x40	2	40	38.667	3093.36	1.54668	$61.15	$4,728.97
S152	W24x55	3	55	38.667	6380.055	3.1900275	$80.03	$9,283.56
S152	W18x35	2	35	36	2520	1.26	$54.65	$3,934.80
S152	L4x4x3/8	12		15.647			$40.80	$7,660.77
S152	W24x55	3	55	36	5940	2.97	$80.03	$8,643.24
S153	W12x14	23	14	13	4186	2.093	$25.21	$7,537.79
S153	W12x26	13	26	13	4394	2.197	$41.06	$6,939.14
S153	W21x44	3	44	25.583	3376.956	1.688478	$65.86	$5,054.69
S153	W12x26	2	26	25.583	1330.316	0.665158	$41.06	$2,100.88
S153	W18x40	2	40	38.667	3093.36	1.54668	$61.15	$4,728.97
S153	W24x55	3	55	38.667	6380.055	3.1900275	$80.03	$9,283.56
S153	W18x35	4	35	36	5040	2.52	$54.65	$7,869.60
S153	L4x4x3/8	12		18.238			$40.80	$8,929.32
S153	W24x55	6	55	36	11880	5.94	$80.03	$17,286.48
S154	W12x14	23	14	13	4186	2.093	$25.21	$7,537.79
S154	W12x26	13	26	13	4394	2.197	$41.06	$6,939.14
S154	W12x26	2	26	25.583	1330.316	0.665158	$41.06	$2,100.88
S154	W21x44	4	44	25.583	4502.608	2.251304	$65.86	$6,739.59
S154	W18x40	3	40	38.667	4640.04	2.32002	$61.15	$7,093.46
S154	W24x55	3	55	38.667	6380.055	3.1900275	$80.03	$9,283.56
S154	W18x35	6	35	36	7560	3.78	$54.65	$11,804.40
S154	W24x55	6	55	36	11880	5.94	$80.03	$17,286.48
S154	L4x4x3/8	12		18.238			$40.80	$8,929.32

SUPPLEMENTARY FRAMING MEMBERS

TOTAL **$287,611.55**

S307 - V	W8x35	2	35	8.333	583.31	0.291655	$56.98	$949.63		BARRELL TRUSS (TYPICAL)
S307 - V	W8x35	2	35	11.177	782.39	0.391195	$56.98	$1,273.73		
S307 - V	W8x35	2	35	13.167	921.69	0.460845	$56.98	$1,500.51	VERTICALS	
S307 - V	W8x35	2	35	14.375	1006.25	0.503125	$56.98	$1,638.18		
S307 - V	W8x35	1	35	14.771	516.985	0.2584925	$56.98	$841.65		
S307 - D	W8x35	2	35	13.792	965.44	0.48272	$56.98	$1,571.74		
S307 - D	W8x35	2	35	15.417	1079.19	0.539595	$56.98	$1,756.92		
S307 - D	W8x35	2	35	17.146	1200.22	0.60011	$56.98	$1,953.96	DIAGONALS	
S307 - D	W8x35	2	35	18.51	1295.7	0.64785	$56.98	$2,109.40		
S307 - D	W8x35	2	35	19.375	1356.25	0.678125	$56.98	$2,207.98		
S307 - T	W14x145	1	145	159.708	23157.66	11.57883	$201.50	$32,181.16	TOP & BOTTOM CHORD	
S307 - B	W14x120	2	120	45.5	10920	5.46	$167.99	$15,287.09		
S307 - B	W14x120	1	120	65	7800	3.9	$167.99	$10,919.35	*PER TRUSS*	$74,191.29
									QUANTITY	7
									TOTAL	**$519,339.03**
S306 - V	W8x35	2	35	8.333	583.31	0.291655	$56.98	$949.63		BARRELL TRUSS (END)
S306 - V	W8x35	2	35	11.177	782.39	0.391195	$56.98	$1,273.73		
S306 - V	W8x35	2	35	13.167	921.69	0.460845	$56.98	$1,500.51	VERTICALS	
S306 - V	W8x35	2	35	14.375	1006.25	0.503125	$56.98	$1,638.18		
S306 - V	W8x35	1	35	14.771	516.985	0.2584925	$56.98	$841.65		
S306 - D	W8x35	2	35	13.792	965.44	0.48272	$56.98	$1,571.74		
S306 - D	W8x35	2	35	15.417	1079.19	0.539595	$56.98	$1,756.92		
S306 - D	W8x35	2	35	17.146	1200.22	0.60011	$56.98	$1,953.96	DIAGONALS	
S306 - D	W8x35	2	35	18.51	1295.7	0.64785	$56.98	$2,109.40		
S306 - D	W8x35	2	35	19.375	1356.25	0.678125	$56.98	$2,207.98		
S306 - T	W14x176	1	176	159.708	28108.608	14.054304	$243.05	$38,817.03	TOP & BOTTOM CHORD	
S306 - B	W36x210	2	210	45.5	19110	9.555	$285.85	$26,012.35		
S306 - B	W36x210	1	210	65	13650	6.825	$285.85	$18,580.25	*PER TRUSS*	$99,213.32
									QUANTITY	1
									TOTAL	**$99,213.32**
*INCLUDES OVERHEAD & PROFIT IN ESTIMATE					TOTAL WEIGHT (TONS)	298.356402		**TONS**	**GRAND TOTAL**	**$906,163.90**

Figure 34: High roof framing system take-off and analysis

PROPOSED STRUCTURE

Proposed High Roof Framing Take-Off and Cost Estimate

Zone	Structural Member	Quantity	Weight/ft(PLF)	Length (ft)	Total Weight (lb)	Total Weight (tons)	Cost/ft.	Cost
S151	60DLH18	7	59	45.83	18929.17	9.46	$49.12	$15,759.33
S151	28LH13	4	30	40.50	4860	2.43	$23.90	$3,871.80
S151	44LH17	24	47	41.67	47000	23.5	$37.37	$37,370.00
S151	W24x84	6	84	38.67	19488	9.744	$119.23	$27,661.36
S151	W24x84	1	84	25.58	2149	1.0745	$119.23	$3,050.30
S151	W24x84	1	84	24.83	2086	1.043	$119.23	$2,960.88
S151	W24x84	1	84	38.42	3227	1.6135	$119.23	$4,580.42
S151	w18x35	3	35	16.67	1750	0.875	$54.65	$2,732.50
S152	60DLH18	7	59	45.83	18929.17	9.46	$49.12	$15,759.33
S152	28LH13	4	30	40.50	4860	2.43	$23.90	$3,871.80
S152	44LH17	24	47	41.67	47000	23.5	$37.37	$37,370.00
S152	W24x84	6	84	38.67	19488	9.744	$119.23	$27,661.36
S152	W24x84	1	84	25.58	2149	1.0745	$119.23	$3,050.30
S152	W24x84	1	84	24.83	2086	1.043	$119.23	$2,960.88
S152	W24x84	1	84	38.42	3227	1.6135	$119.23	$4,580.42
S152	W18x35	3	35	16.67	1750	0.875	$54.65	$2,732.50
S153	60DLH18	7	59	45.83	18929.17	9.46	$49.12	$15,759.33
S153	28LH13	4	30	40.50	4860	2.43	$23.90	$3,871.80
S153	44LH17	24	47	41.67	47000	23.5	$37.37	$37,370.00
S153	W24x84	6	84	38.67	19488	9.744	$119.23	$27,661.36
S153	W24x84	1	84	25.58	2149	1.0745	$119.23	$3,050.30
S153	W24x84	1	84	24.83	2086	1.043	$119.23	$2,960.88
S153	W24x84	1	84	38.42	3227	1.6135	$119.23	$4,580.42
S153	w18x35	3	35	16.67	1750	0.875	$54.65	$2,732.50
S154	60DLH18	7	59	45.83	18929.17	9.46	$49.12	$15,759.33
S154	28LH13	4	30	40.50	4860	2.43	$23.90	$3,871.80
S154	44LH17	24	47	41.67	47000	23.5	$37.37	$37,370.00
S154	W24x84	6	84	38.67	19488	9.744	$119.23	$27,661.36
S154	W24x84	1	84	25.58	2149	1.0745	$119.23	$3,050.30
S154	W24x84	1	84	24.83	2086	1.043	$119.23	$2,960.88
S154	W24x84	1	84	38.42	3227	1.6135	$119.23	$4,580.42
S154	W18x35	3	35	16.67	1750	0.875	$54.65	$2,732.50
D	2L2-1/2X2-1/2X1/2	4	7.7	25.7	791.56	0.39578	$ 10.66	$1,095.85
D	2L2X2X3/8	4	4.7	30.2	567.76	0.28388	$ 10.66	$1,287.73
D	2L6X6X5/8	9	24.2	24	5227.2	2.6136	$ 16.86	$3,641.76
D	w14x109	2	90	110	19800	9.9	$127.78	$28,111.60
C	2L2-1/2X2-1/2X1/2	2	3.9	24.43	190.554	0.095277	$ 10.66	$520.85
C	2L2-1/2X2-1/2X1/2	2	3.9	33.18	258.804	0.129402	$ 10.66	$707.40
C	2L2X2X3/8	2	3.9	30.2	235.56	0.11778	$ 10.66	$643.86
C	2L6x6x1/2	4	19.6	24	1881.6	0.9408	$ 16.86	$1,618.56
C	2L5x5x5/8	3	20	24	1440	0.72	$ 16.86	$1,213.92
C	W14x109	2	84	91.67	15400.56	7.70028	$127.78	$23,427.19

SUPPLEMENTAL FRAMING MATERIALS

Total	$391,946.37

Truss D

PER TRUSS	$34,136.94
QUANTITY	2
TOTAL	$68,273.87

Truss C

PER TRUSS	$28,131.77
QUANTITY	2
TOTAL	$56,263.55

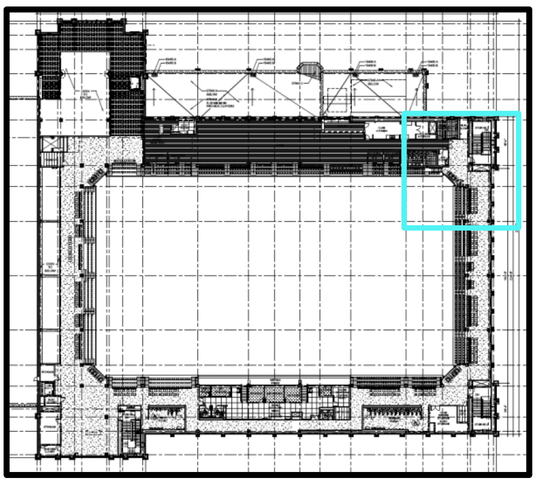
B	2L3-1/2X2-1/2-1/2	2	9.8	25.7	503.72	0.25186	$ 10.66	$547.92	Truss B	
B	2L3-1/2X2-1/2-1/2	2	9.8	32.7	640.92	0.32046	$ 10.66	$697.16		
B	2L2-1/2X2X3/8	2	9.8	32.7	640.92	0.32046	$ 10.66	$697.16		
B	2L2X2X3/8	2	2.4	32.7	156.96	0.07848	$ 10.66	$697.16		
B	2L4X3X5/8	4	11.1	25.7	1141.08	0.57054	$ 10.66	$1,095.85		
B	2L6X6X1	6	37.4	24	5385.6	2.6928	$ 16.86	$2,427.84		
B	2L6X6X1/2	3	19.6	24	1411.2	0.7056	$ 16.86	$1,213.92		
B	2L8X8X1/2	4	26.4	24	2534.4	1.2672	$ 16.86	$1,618.56		
B	W14x211	2	146	188	54896	27.448	$195.78	$73,613.28	PER TRUSS	$82,608.86
									QUANTITY	2
									TOTAL	**$165,217.73**
A	2L4x4x3/4	2	18.5	24.43	903.91	0.451955	$ 10.66	$520.85	Truss A	
A	2L4x4x3/4	2	18.5	27.66	1023.42	0.51171	$ 10.66	$589.71		
A	2L4x4x3/4	2	18.5	32.67	1208.79	0.604395	$ 10.66	$696.52		
A	2L2-1/2X2-1/2X1/2	2	9.8	32.67	640.332	0.320166	$ 10.66	$696.52		
A	2L2-1/2X1-1/2X1/4	2	9.8	32.67	640.332	0.320166	$ 10.66	$696.52		
A	2L6X6X1/2	3	19.6	24	1411.2	0.7056	$ 16.86	$1,213.92		
A	2L8X8X1/2	8	26.4	24	5068.8	2.5344	$ 16.86	$3,237.12		
A	W14x211	2	217	169.67	73636.78	36.81839	$195.78	$66,435.99	PER TRUSS	$74,087.16
									QUANTITY	2
									TOTAL	**$74,087.16**
	W14X145	4	145	9.167	5316.86	2.65843	$188.55	$6,913.75	HORIZONTALS	TOP CHORD BRACING
	W14X22	16	22	9.167	3226.784	1.613392	$81.59	$11,966.97		
	W14X26	4	26	9.167	953.368	0.476684	$81.59	$2,991.74		
	W14X68	4	68	9.167	2493.424	1.246712	$111.65	$4,093.98		
	W14X61	6	61	9.167	3355.122	1.677561	$98.30	$5,406.70		
	2L8X8X1X3/8	12	32.7	23.9	9378.36	4.68918	$16.86	$4,835.45	DIAGONALS	
	2L8X8X1X3/8	8	32.7	12.9	3374.64	1.68732	$16.86	$1,739.95		
	2L8X4X1/2X3/8	8	28.4	20.4	4634.88	2.31744	$16.86	$2,751.55		**$40,700.09**
	2L5X5X5/16X3/4	24	16.2	9.167	3564.1296	1.7820648	$16.86	$3,709.33	HORIZONTALS	BOTTOM CHORD BRACING
	2L5X5X5/16X3/4	16	16.2	23.9	6194.88	3.09744	$16.86	$6,447.26	DIAGONALS	
	2L5X5X5/16X3/4	4	16.2	28.9	1872.72	0.93636	$16.86	$1,949.02		
	2L5X5X5/16X3/4	4	16.2	20.42	1323.216	0.661608	$16.86	$1,377.12		
	2L5X5X5/16X3/4	4	16.2	12.9	835.92	0.41796	$16.86	$869.98	TOTAL	$55,052.81
	2L4x4x3/4	2	18.5	24.43	903.91	0.451955	$ 10.66	$520.85	Leg Truss	
	2L4x4x3/4	2	18.5	27.66	1023.42	0.51171	$ 10.66	$589.71		
	2L4x4x3/4	2	18.5	32.67	1208.79	0.604395	$ 10.66	$696.52		
	2L2-1/2X2-1/2X1/2	2	9.8	32.67	640.332	0.320166	$ 10.66	$696.52		
	2L2-1/2X1-1/2X1/4	2	9.8	32.67	640.332	0.320166	$ 10.66	$696.52		
	2L6X6X1/2	3	19.6	24	1411.2	0.7056	$ 16.86	$1,213.92		
	2L8X8X1/2	8	26.4	24	5068.8	2.5344	$ 16.86	$3,237.12		
	W27x217	2	217	149.42	64848.28	32.42414	$127.78	$38,185.78	PER TRUSS	$45,836.95
									QUANTITY	4
									TOTAL	**$183,347.79**
*INCLUDES OVERHEAD & PROFIT IN ESTIMATE					Total Weight (lb)	358.93 Tons			**GRAND TOTAL**	**$979,836.55**

Figure 35: Proposed high roof framing system take-off and analysis

ARCHITECTURAL BREADTH

In the redesign of the structural roof concept, a number of changes took place that require architectural iterations in order to maintain the functions of the building. Of primary concern is the new presence of the super columns at the corners of the seating bowl. As designed, the columns have minimal interference with the structure of the building but do in fact alter important aesthetics. In the case of the bowl seating, seats had to be removed at the corner section of the club level seating to make room for the columns thus impacting the calculated capacity of the arena.

While cost considerations for game day revenue would be an issue if seats were not relocated, the primary issue is the type of seating in obstructed locations. The corner spaces of the club level housed a majority of the handicap seating in the arena, a category regulated by code not revenue.



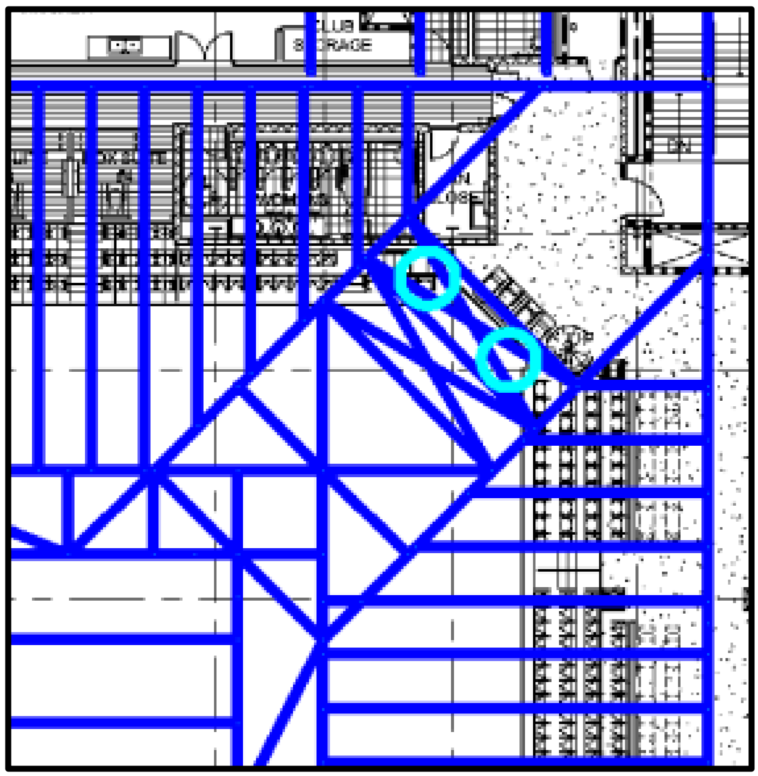
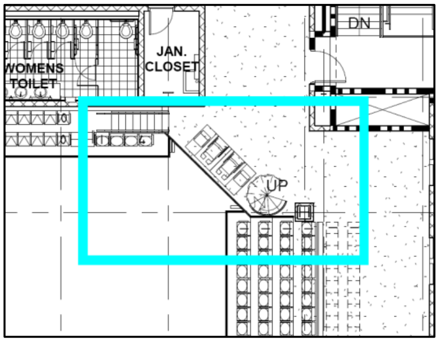


Figure 36: (Right) Table top roof structure superimposed over existing architectural drawing at club level. Images scaled and aligned for exact location.

A worst case scenario has been assumed in seating loss and cumulative seat losses can be seen in figure 36 below. A total of seventy five seats in addition to eleven handicap seats were



Locoation	Typical Seat	Handicap Seat
Northeast Corner	16	2
South East Corner	17	3
South West Corner	21	3
North West Corner	21	3
TOTAL	75	11

Figure 37: Largest possible area effected by super columns. Table 36: Seating loss totals broken down by section.

displaced by the super columns. According to the 2006 International Building Code, the total required number of handicap seats is “6 plus one for each 150 between 500-5000.” This cumulates to a code requirement of 30 handicap seats throughout the arena. With 45 handicap seats on the concourse level and an additional six remaining on the club level, the arena still complies with the code even in the absence of the 11 displaced seats. A visual comparison of the existing vs. proposed arena can be found on the next page in figures 37 & 38.

TABLE 1108.2.2.1
ACCESSIBLE WHEELCHAIR SPACES

CAPACITY OF SEATING IN ASSEMBLY AREAS	MINIMUM REQUIRED NUMBER OF WHEELCHAIR SPACES
4 to 25	1
26 to 50	2
51 to 100	4
101 to 300	5
301 to 500	6
501 to 5,000	6, plus 1 for each 150, or fraction thereof, between 501 through 5,000
5,001 and over	36 plus 1 for each 200, or fraction thereof, over 5,000

Figure 38: IBC Regulation for handicap seating. IBC Table 1108.2.2.1

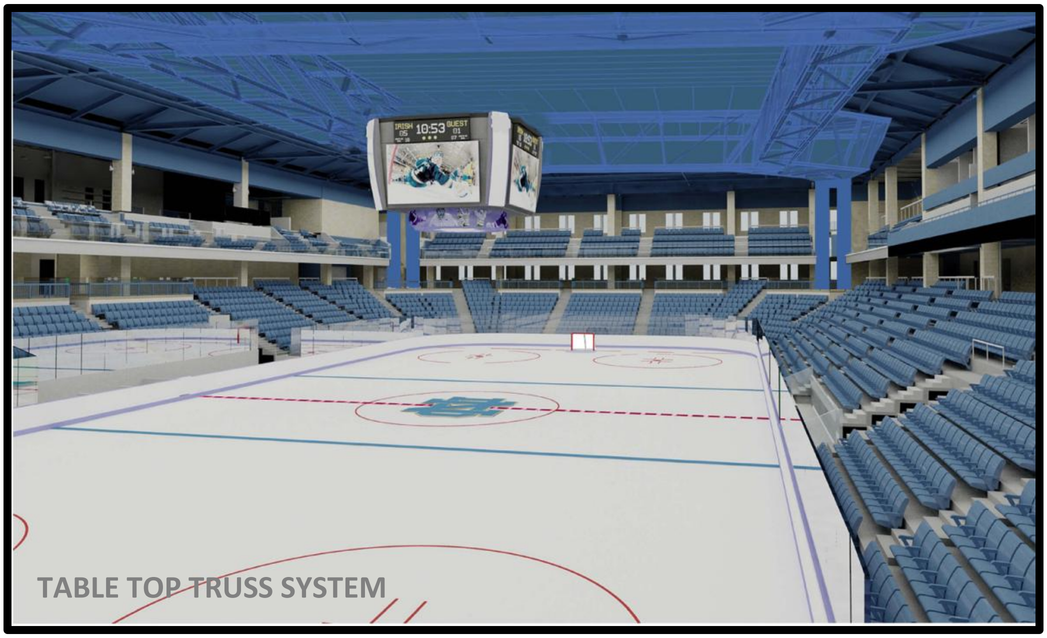


Figure 39: (ABOVE) Interior rendering with adjusted table top truss in place.
Figure 40: (BELOW) Interior rendering showing existing roof structure

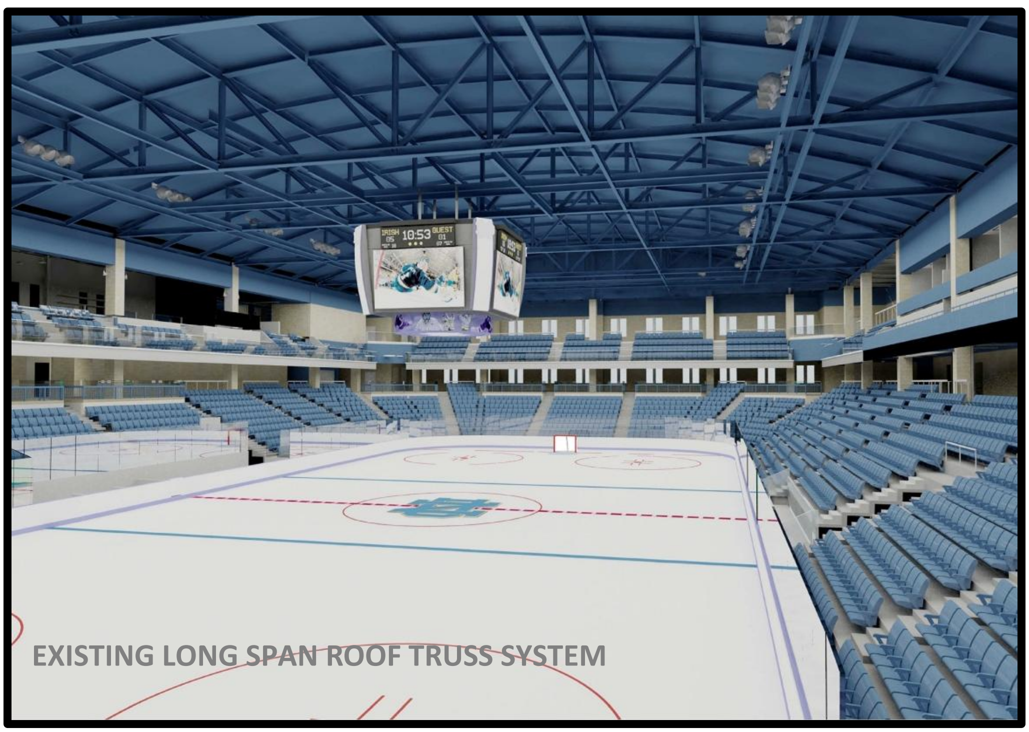


Figure 41: Aerial Rending displaying adjusted roof shape Figure

Figure 42: Aerial view of existing roof structure

CONSTRUCTION MANAGEMENT BREADTH

The construction of the table top truss system differs from that of the conventional long span truss and requires a specialized erection procedure. The suggested method is to first erect the legs of the table top truss. Each leg would rest on its respective corner column and a shoring tower at the location of the connection. Once in place, the sides of the rectangular interior could be placed one by one. The introduction of the shoring towers would obviously add cost to the procedure but in looking into the existing construction sequence it seems that the trade is nominal.

The current erection called for two cranes on site during the erection of the trusses. With the shoring towers in place, one crane would be capable of making each pick with a guide line. Given the existing schedule with a truss erection period of 68 days (See schedule below), the comparative cost breakdown shows savings in the alternate approach.

PHR0T1-8P4D1	Build-up Roof Trusses - Arena High Roof Truss Seq #14	2	11-Sep-12	12-Sep-12
PHR0T2-8P4D1	Build-up Roof Trusses - Arena High Roof Truss Seq #17	2	13-Sep-12	14-Sep-12
PHR0T8-8P4D1	Build-up Roof Trusses - Arena High Roof Truss Seq #18	2	17-Sep-12	18-Sep-12
PHR0T7-8P4D1	Build-up Roof Trusses - Arena High Roof Truss Seq #21	2	19-Sep-12	20-Sep-12
PHR0T6-8P4D1	Build-up Roof Trusses - Arena High Roof Truss Seq #22	2	21-Sep-12	24-Sep-12
PHR0T3-8P4D1	Build-up Roof Trusses - Arena High Roof Truss Seq #23	2	25-Sep-12	26-Sep-12
PHR0T4-8P4D1	Build-up Roof Trusses - Arena High Roof Truss Seq #24	2	27-Sep-12	28-Sep-12
PHR0T5-8P4D1	Build-up Roof Trusses - Arena High Roof Truss Seq #25	2	01-Oct-12	02-Oct-12
PHR0T1-8P4D2	Erect Roof Trusses - Arena High Roof Truss Seq #14	1	13-Sep-12	13-Sep-12
PHR0T2-8P4D2	Erect Roof Trusses - Arena High Roof Truss Seq #17	1	21-Sep-12	21-Sep-12
PHR0T8-8P4D2	Erect Roof Trusses - Arena High Roof Truss Seq #18	1	09-Oct-12	09-Oct-12
PHR0T7-8P4D2	Erect Roof Trusses - Arena High Roof Truss Seq #21	1	17-Oct-12	17-Oct-12
PHR0T6-8P4D2	Erect Roof Trusses - Arena High Roof Truss Seq #22	1	24-Oct-12	24-Oct-12
PHR0T3-8P4D2	Erect Roof Trusses - Arena High Roof Truss Seq #23	1	01-Nov-12	01-Nov-12
PHR0T4-8P4D2	Erect Roof Trusses - Arena High Roof Truss Seq #24	1	09-Nov-12	09-Nov-12
PHR0T5-8P4D2	Erect Roof Trusses - Arena High Roof Truss Seq #25	1	16-Nov-12	16-Nov-12
PHR0T1-8P4D4	Truss Infill & Plumb/Bolt/Weld - Arena High Roof Truss Seq #17	5	24-Sep-12	28-Sep-12
PHR0T2-8P4D4	Truss Infill & Plumb/Bolt/Weld - Arena High Roof Truss Seq #19 & #20	5	10-Oct-12	16-Oct-12
PHR0T7-8P4D4	Truss Infill & Plumb/Bolt/Weld - Arena High Roof Truss Seq #21	4	18-Oct-12	23-Oct-12
PHR0T6-8P4D4	Truss Infill & Plumb/Bolt/Weld - Arena High Roof Truss Seq #22	5	25-Oct-12	31-Oct-12
PHR0T3-8P4D4	Truss Infill & Plumb/Bolt/Weld - Arena High Roof Truss Seq #23	5	02-Nov-12	08-Nov-12
PHR0T4-8P4D4	Truss Infill & Plumb/Bolt/Weld - Arena High Roof Truss Seq #24	4	12-Nov-12	15-Nov-12
PHR0T5-8P4D4	Truss Infill & Plumb/Bolt/Weld - Arena High Roof Truss Seq #25	5	19-Nov-12	27-Nov-12

Figure 43: Typical Truss Erection Schedule

Existing				
Truss Weight: 25.79 Tons				
Crane	Quantity	Cost per Day	# Days	Total Cost
25 Ton	2	$1,650.00	68	$224,400.00

Table Top Truss				
Truss Weights: 33.66Tons				
Crane	Quantity	Cost per Day	# Days	Total Cost
40 Ton	1	$1,900.00	68	$129,200.00
Shoring				
Truss Height:37'				
Equipment	Quantity/Tower	# of Towers	Cost	
Tower 5'x7'x10'	1	4	$1,500.00	$6,000.00
5' added sections	6	4	$228.00	$5,472.00
			TOTAL:	$140,672.00

Figure 44: Cost Breakdown taken From RS Means Building Construction Cost Data

**See Appendix F for CostWorks References

CONCLUSION

Analysis and design led to the effective implementation of the table top truss system in the Compton Family Ice Arena. Working in conjunction with the existing structure, the new design was able to fit nearly seamlessly into the layout of the building without disrupting critical structural elements. It was able to adopt the existing lateral load resisting system comprised of braced frames with only minimal changes and in the end lead to a comprehensive and effective design.

In addition, the system coordination and comprehensive design established by the table top truss allowed for increased functionality in the building and ease of management. The architectural impacts brought a positive and dramatic presence to the interior space while remaining minimally invasive to the seating capacity requirements of the space.

As prefaced in the introduction though, the primary focus of the system was to effectively shorten spans as to recover economic value on the system. While the spans were in fact shortened by nearly 60', the cost analysis shows that the table top truss system is still a more expensive alternative to the traditional long span trusses in the existing structure. It is believed by the author of this report that further iteration could be made in reducing costs, primarily by adjusting the depth and dimensions of the box trussing, but for all intents and purposes it has been concluded that the Compton Family Ice Arena is comparatively too small to benefit from a system like this. The capacity of the system in comparison to the load demand suggests that the strength of the table top truss exceeds the needs of this space thus adding cost to an underutilized system.

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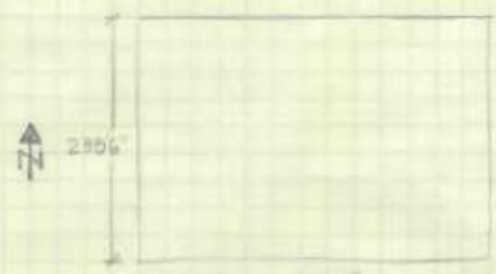
APPENDIX A

WIND LOAD CALCULATIONS

WINDLOAD

main wind force resisting system (MWFRS)

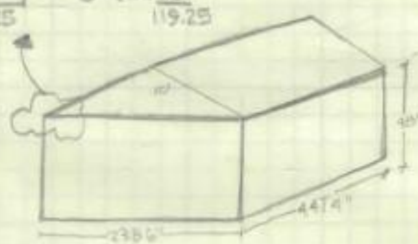
simplifying assumptions



- assumed shape (already aligned with north)
- assumed gabled
- assumed all roof heights @ 148'

→ NORTH SOUTH WINDS
L= 238.5ft B= 447.3ft
→ EAST-WEST WINDS
L= 447.3ft B= 238.5ft

$\theta = \tan \frac{15}{119.25}$



low rise → < 60ft

$P = q\,GC_p - q_i\,(GC_{pi})$ (psf)/(N/m²)

q_z= windward walls

q_h= leeward walls, sidewall roof

location: south bend, indiana
topography: homogeneous
terrain: flat, open

→ BASIC WIND SPEED: V=90MPH (FIGURE 6-1C)
→ EXPOSURE: C (SECTION 6.5.6.3)
→ BUILDING CATEGORY: III (TABLE 1-1)
→ IMPORTANCE FACTOR: I=1.15 (TABLE 6-1)

→ VELOCITY PRESSURE: $q_z = .00256 K_z K_{zt} K_d V^2 I$

K_z → interpolate table 6-3
40ft 1.04
48ft —
50ft 1.09

$y_2 = \frac{(x_2-x_1)(y_3-y_1)}{(x_3-x_1)} + y_1 \rightarrow \underline{\underline{1.08}}$

same for all levels
→ REFER TO EXCEL SHEET

WIND LOAD

GUST EFFECT FACTOR

→ G for a rigid structure

$$G = .925\left(\frac{(1+1.7 g_Q I_z Q)}{1+1.7 g_v I_z}\right)$$

$z = .6h = .6(48') = 28.8$

$C = .20$, $z_{min} = 15$ } TBL 6.2

$g_Q = 3.4$, $g_v = 3.4$ } 6.5.8.1

$$I_z = c\left(\frac{33}{z}\right)^{1/6} = .20\left(\frac{33}{28.8}\right)^{1/6} = .205$$

$$Q(N\text{-}S) = \sqrt{\frac{1}{1+.63\left(\frac{447.3+55.5}{486.6}\right)^{.63}}}$$

$B = 447.3$ (N-S), 288.5 (E-W)

$\ell = 500$

$$L_z = \ell\left(\frac{z}{33}\right)^{\epsilon} = 500\left(\frac{28.8}{33}\right)^{.20} = 486.6$$

$= .78$

$$Q(E\text{-}W) = \sqrt{\frac{1}{\left(1+.63\left(\frac{288.5+55.5}{486.6}\right)^{.63}\right)}} = .828$$

$$G(N\text{-}S) = .925\left(\frac{(1+1.7(3.4)(.205)(.78))}{1+1.7(3.4)(.205)}\right) = .925\left(\frac{1.924}{2.1849}\right)$$

$= .8145$

$$G(E\text{-}W) = .925\left(\frac{1+1.7(3.4)(.205)(.828)}{1+1.7(3.4)(.205)}\right) = .925\left(\frac{1.981}{2.1849}\right)$$

$= .8387$

WIND LOAD

*fully enclosed building
* parapets disregarded [$p_r = q_p(GC_p - GC_{pi})$]

EXTERNAL PRESSURE COEFICIENTS [FIGURE 6-6]

$\theta = \tan^{-1} 15/119.25 \Rightarrow \theta = 7.2°$

WALLS: windward: $C_p = .8$
Leeward: $C_p \Rightarrow$ interpolate → see excel
Side: $C_p = -.7$

ROOF: $\theta = 7.2°$ ROOF AREA = 238.5(447.3) = 106681
$h/2 = 24$ ≥ 1000
$h = 48$ RED FACTOR = .8
$2h = 96$

internal pressure coeficients [FIG. 6-5]
enclosed: $GC_{pi} = \pm .18$

DESIGN WIND PRESSURES

① WINDWARD: $P_z = q_z G_f C_p - q_h GC_{pi}$

② LEEWARD/SIDE/ROOF: $P_h = q_h(G_f C_p - GC_{pi})$

$q_h \rightarrow K_z = 1.112$

$= 22.5396$

General Wind Load Design Criteria		
Design Wind Speed	90 mph	ASCE7-05, Fig6-1C
Directionality Factor	0.85	ASCE7-05, Fig6-4
Importance Factor	1.15	ASCE7-05, Tbl.6-1
Exposure Category	C	ASCE7-05,Sec. 6.5.6.3
Topographic Factor	1	ASCE7-05, Sec. 6.5.7.1
Internal Pressure Coefficient	0.18	ASCE7-05, Fig6-5

Velocity Pressure Coeficients(Kz) and Velocity Pressure(qz)			
Level	**Elevation**	**Kz**	**qz**
Event Level	0	0.85	17.23
Concourse Level	16	0.89	18.04
Club Level	32.67	0.996	20.19
Roof Level	61.67	1.08	21.89

External pressure Coefficient(Cp)		
Description	**North-South Wind**	**East-West Wind**
L/B	0.533	1.875
Windward Walls	0.8	0.8
Leeward Walls	-0.5	-0.3125
Side Walls	-0.7	-0.7
h/L	0.2327	0.124
Roof: 0-h/2	-0.852	-1.3
Roof: h/2-h	-0.852	-0.7
Roof: h-2h	-0.479	-0.7
Roof: >2h	-0.292	-0.7

Wind Pressures (North-South Direction)							
				Internal Pressure(psf)		Net Pressure(psf)	
Type	Level	Distances(ft)	Wind Pressure (psf)	(+)(Gcpi)	(-)(Gcpi)	(+)(Gcpi)	(-)(Gcpi)
	Event Level	0	11.72	4.06	-4.06	15.77	7.66
	Concourse Level	16	12.27	4.06	-4.06	16.32	8.21
	Club level	32.3	13.73	4.06	-4.06	17.79	9.67
Windward Walls	Roof Level	61.3	14.89	4.06	-4.06	18.94	10.83
Leeward Walls	ALL	ALL	9.58	4.06	-4.06	13.64	5.52
Side Walls	ALL	ALL	-13.41	4.06	-4.06	-9.35	-17.47
		0-24	-16.32	4.06	-4.06	-12.27	-20.38
		24-48	-16.32	4.06	-4.06	-12.27	-20.38
		48-96	-9.18	4.06	-4.06	-5.12	-13.23
Roof		>96	-5.59	4.06	-4.06	-1.54	-9.65

Story Forces (North-South Direction)								
		Trib Below		Trib Above				
Floor Level	Elevation	Height(Ft)	Area(ft2)	Height(Ft)	Area(ft2)	Story Force(k)	Story Shear (k)	Overturning moment(k-ft)
Event Level	0.00		0.00	8.00	2522.40	55.11	453.72	0.00
Concourse Leve	16.00	8.00	2522.40	8.15	2569.70	115.00	398.61	3188.89
Club level	32.30	8.15	2569.70	14.50	4571.85	171.75	283.61	2311.39
Roof Level	48.00	14.50	4571.85		0.00	111.85	111.85	1621.89
							Total Base Shear:	453.72
							Total Overturning Moment	7122.16

Wind Pressures (East-West Direction)							
				Internal Pressure(psf)		Net Pressure(psf)	
Type	Level	Distances(ft)	ind Pressure (p	(+)(Gcpi)	(-)(Gcpi)	(+)(Gcpi)	(-)(Gcpi)
	Event Level	0	11.72	4.06	-4.06	15.77	7.66
	Concourse Level	16	12.27	4.06	-4.06	16.32	8.21
	Club level	32.3	13.73	4.06	-4.06	17.79	9.67
Windward Walls	Roof Level	48	14.89	4.06	-4.06	18.94	10.83
Leeward Walls	ALL	ALL	5.99	4.06	-4.06	10.05	1.93
Side Walls	ALL	ALL	-13.41	4.06	-4.06	-9.36	-17.47
		0-24	-24.91	4.06	-4.06	-20.85	-28.96
		24-48	-13.41	4.06	-4.06	-9.35	-17.47
		48-96	-13.41	4.06	-4.06	-9.35	-17.47
Roof		>96	-13.41	4.06	-4.06	-9.35	-17.47

Story Forces (East-West Direction)								
		Trib Below		Trib Above				
Floor Level	Elevation	Height(Ft)	Area(ft2)	Height(Ft)	Area(ft2)	Story Force(k)	Story Shear (k)	Overturning moment(k-ft)
Event Level	0.00		0.00	8.00	1908.00	33.78	289.66	0.00
Concourse Level	16.00	8.00	1908.00	8.15	1943.78	73.16	255.88	2047.05
Club level	32.30	8.15	1943.78	14.50	3458.25	110.52	182.72	1489.15
Roof Level	48.00	14.50	3458.25		0.00	72.19	72.19	1046.81
							Total Base Shear:	289.66
							Total Overturning Moment	4583.02

SEISMIC CALCULATIONS

seismic design

thermal plunge S309

- location south bend, indiana (46556)
- S_{DS} = .127
- S_{D1} = .088
- occupancy category = III (table 1-1)
- per table 11.6-1 → S_{DS} < .167 → seismic design categ. A
- per table 11.6-2 → .067 ≤ S_{D1} < .133 → seismic design categ B

design category B controls

- per geotechnical report → soil site class D
- per structural dwgs (ref. S311) EOR design does not indicate any special considerations or influence zones ∴ per ASCE7-05 table 12.2-1, building frame assumed

"ordinary steel concentrically braced frames w/ response mod. factor 3.25"

→ no special/seismic considerations required for braced fram conx.

seismic use group	II
importance factor	I_e = 1.25
soil classification	D
1 sec. spectral resp.	S_1 = .055 F_v = 2.4 S_{D1} = .088
seismic design categ.	B
response mod. factor	R = 3.25
long period transition	T_L = 12
fundament period	.1N = .3

S_s = 10% g
F_a = 1.0
S_1 = 6% g
$S_{MS} = F_a S_s = 1.0(.1) = .1$
$S_{M1} = 2.4(.06) = .144$

SEISMIC DESIGN

SEISMIC BASE SHEAR: $V = C_s W$

$T < T_L \rightarrow C_s = \frac{S_{DS}}{R/I} \leq S_{D1} / (R/I)T$

$= \frac{.127}{\left(\frac{3.25}{1.25}\right)} \leq \frac{.088}{\left(\frac{3.25}{1.25}\right)(.3)}$

$= .0488 \leq .1128$ ✓

*TABLE 12.8-2 → $C_t = .03$

$x = .75$

$h_n = 48'$

$T = .03(48)^{.75} = .547$

ROOF DEAD LOAD = 1291^K
CLUB LEVEL DEAD LOAD: 5791^K
CONCOURSE LEVEL DL: 11798^K
EVENT LEVEL DL: $29,000^K$
(GROUND)

SEE SPREADSHEETS

SNOW

$\rightarrow W_{roof} = 1291^K + .6(42PSF)(40192sf) = 2303^K$

$W_{TOTAL} = 19,892$

$V = C_s W = .0488(19,892) = 970.7^K$

SEISMIC DESIGN

VERTICLE DIST. OF SEISMIC FORCES: $F_x = C_{vx} V$

$$C_{vx} = \frac{W_x h_x^k}{\Sigma w_i h_i^2} \qquad K=2 \ (\text{FOR } T=.547)$$

$$C_{ROOF}: \frac{2303(48)^2}{(11798)(16)^2 + (5791)(32.67)^2 + (2303)(48)^2}$$

$$= \frac{5306112}{14507302} = .366$$

$$C_{CLUB}: \frac{5791(32.67)^2}{14507302} = .426$$

$$C_{CONC}: \frac{11798(16)^2}{14507302} = .208$$

$F_{ROOF} = .366(970.7^K) = 355.3^K$

$F_{CLUB} = .426(970.7^K) = 413.5^K$

$F_{CONC} = .208(970.7^K) = 201.9^K$

**Adjusted building weight for new roof weight

SeismicForces (East-West and North-South Directions)							
Level	Story Weight, wx(k)	Story Height, hx(ft)	wxhxk	Cvx	Story Force(k) Fx=CvxV	Story Shear (k)	Overturning Moment(k-ft)
Event Level	29000.00	0.00	0.00	0.00	0.00	970.70	0
Concourse Level	11798.00	16.00	188768.00	0.21	201.91	970.70	15531.2
Club level	5790.76	32.30	187041.55	0.43	413.52	768.79	24832.05912
Roof Level	1291.00	61.77	79741.20	0.37	355.28	355.28	21944.34505
						Total Base Shear:	970.70
						Total Overturning Moment	62307.60

New Roof Loads				
Quantity	Weight/ft(PLF)	Length (ft)	Total Weight (lb)	Total Weight (tons)
7	59	45.83	18929.17	9.46
4	30	40.50	4860	2.43
24	47	41.67	47000	23.5
6	84	38.67	19488	9.744
1	84	25.58	2149	1.0745
1	84	24.83	2086	1.043
1	84	38.42	3227	1.6135
3	35	16.67	1750	0.875
7	59	45.83	18929.17	9.46
4	30	40.50	4860	2.43
24	47	41.67	47000	23.5
6	84	38.67	19488	9.744
1	84	25.58	2149	1.0745
1	84	24.83	2086	1.043
1	84	38.42	3227	1.6135
3	35	16.67	1750	0.875
7	59	45.83	18929.17	9.46
4	30	40.50	4860	2.43
24	47	41.67	47000	23.5
6	84	38.67	19488	9.744
1	84	25.58	2149	1.0745
1	84	24.83	2086	1.043
1	84	38.42	3227	1.6135
3	35	16.67	1750	0.875
7	59	45.83	18929.17	9.46
4	30	40.50	4860	2.43
24	47	41.67	47000	23.5
6	84	38.67	19488	9.744
1	84	25.58	2149	1.0745
1	84	24.83	2086	1.043
1	84	38.42	3227	1.6135
3	35	16.67	1750	0.875
4	7.7	25.7	791.56	0.39578
4	4.7	30.2	567.76	0.28388
9	24.2	24	5227.2	2.6136
2	90	110	19800	9.9
			TOTAL	212.17

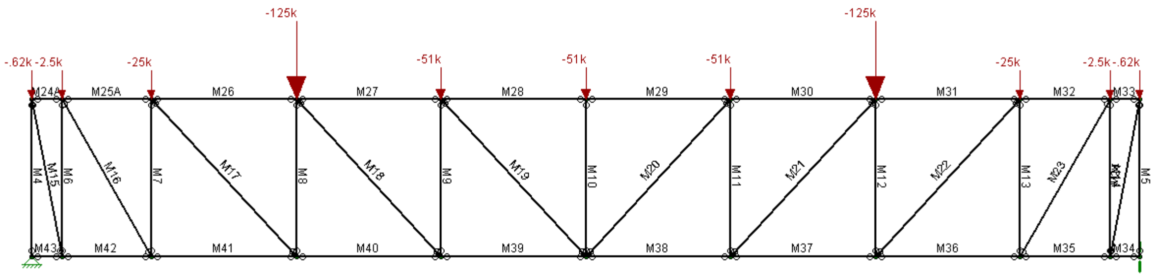
BRACED FRAME CHECKS

Lateral Column Checks										
Section	Level	KL	P	bx	by	P	Mx	My	Capacity Check	
w10x45	1(J)	16	3.27	5.19	11.70	57.15	2.82	74.31	0.94	BF7
w10x45	2	16	3.27	5.19	11.70	20.76	0.54	11.04	0.16	
w10x45	3	29	10.70	7.82	11.70	0.14	0.12	17.40	0.22	
w10x45	1(H)	16	3.27	5.19	11.70	57.23	3.89	82.73	1.05	
w10x45	2	16	3.27	5.19	11.70	20.83	0.62	12.36	0.17	
w10x45	3	29	10.70	7.82	11.70	0.15	0.19	19.11	0.24	
w10x45	1(X)	16	3.27	5.19	11.70	71.25	1.91	88.49	1.12	BF8
w10x45	2	16	3.27	5.19	11.70	33.14	0.26	54.69	0.68	
w10x45	3	29	10.70	7.82	11.70	0.09	0.07	24.37	0.30	
w10x45	1(T)	16	3.27	5.19	11.70	70.13	1.14	72.40	0.93	
w10x45	2	16	3.27	5.19	11.70	33.07	0.27	23.19	0.31	
w10x45	3	29	10.70	7.82	11.70	0.18	0.00	26.89	0.33	
w10x33	1	16	4.68	7.89	16.90	67.56	1.63	56.09	1.03	BF9
w10x33	2	16	4.68	7.89	16.90	36.44	0.31	16.01	0.31	
w10x33	3	29	15.70	14.50	16.90	13.86	0.09	9.22	0.19	
w10x33	1	16	4.68	7.89	16.90	67.56	1.17	54.68	1.01	
w10x33	2	16	4.68	7.89	16.90	36.49	0.41	15.47	0.31	
w10x33	3	29	15.70	14.50	16.90	13.85	0.01	10.32	0.20	
w10x33	1(J)	16	4.68	7.89	16.90	84.03	1.03	74.03	1.35	BF10
w10x33	2	16	4.68	7.89	16.90	48.39	-0.34	21.37	0.41	
w10x33	3	29	15.70	14.50	16.90	18.51	0.05	12.76	0.25	
w10x33	1(F)	16	4.68	7.89	16.90	89.77	1.56	73.47	1.35	
w10x33	2	16	4.68	7.89	16.90	48.42	-0.21	20.73	0.40	
w10x33	3	29	15.70	14.50	16.90	18.49	0.13	13.28	0.26	
w10x33	1(Y)	16	4.68	7.89	16.90	66.50	1.08	56.44	1.03	BF11
w10x33	2	16	4.68	7.89	16.90	35.59	0.21	17.93	0.34	
w10x33	3	29	15.70	14.50	16.90	13.57	0.06	9.55	0.19	
w10x33	1(V)	16	4.68	7.89	16.90	89.42	0.94	75.89	1.38	
w10x33	2	16	4.68	7.89	16.90	47.56	0.31	24.84	0.47	
w10x33	3	29	15.70	14.50	16.90	18.20	0.01	12.40	0.24	
w12x58	1	16	2.00	3.13	7.29	43.97	1.43	86.05	0.68	BF12
w12x58	2	16	2.00	3.13	7.29	19.04	0.28	40.72	0.32	
w12x58	3	29	5.34	4.36	7.29	0.08	0.01	13.52	0.10	
W10x49	1	16	2.43	4.43	8.38	2.90	1.78	59.29	0.51	
W10x49	2	16	2.43	4.43	8.38	5.26	0.21	45.95	0.39	
W10x49	3	29	6.16	5.96	8.38	0.07	0.02	12.83	0.11	
w10x33	1	16	4.68	7.89	16.90	37.80	1.02	36.83	0.67	
w10x33	2	16	4.68	7.89	16.90	13.57	0.09	12.23	0.23	
w10x33	3	29	15.70	14.50	16.90	0.04	0.01	2.88	0.06	
W10x45	1(19)	16	3.27	5.19	11.70	36.16	12.77	34.13	0.51	BF13
W10x45	2	16	3.27	5.19	11.70	15.85	1.92	12.41	0.17	
W10x45	3	29	10.70	7.82	11.70	0.05	0.20	7.18	0.10	
W10x45	1(21)	16	3.27	5.19	11.70	0.09	13.40	42.04	0.56	
W10x45	2	16	3.27	5.19	11.70	0.53	1.53	32.17	0.39	
W10x45	3	29	10.70	7.82	11.70	0.02	0.14	12.36	0.16	
W10x45	1(22)	16	3.27	5.19	11.70	36.15	13.50	34.30	0.51	
W10x45	2	16	3.27	5.19	11.70	16.39	2.00	13.12	0.18	
W10x45	3	29	10.70	7.82	11.70	0.04	0.28	8.05	0.11	

Bracing Member Checks						
Member	Section	Axial Load	KL(in)	KL(ft)	1.6P	ϕPn
D20R	HSS8x8x.375	21.45	367.462	30.62183	34.32	153
D20CL	HSS8x8x.376	41.86	232.215	19.35125	66.976	298
D20CO	HSS8x8x.377	66.01	225.362	18.78017	105.616	310
D21R	HSS8x8x.378	-21.43	367.462	30.62183	-34.288	431
D21CL	HSS8x8x.379	-41.77	232.215	19.35125	-66.832	431
D21CO	HSS8x8x.380	-65.91	225.362	18.78017	-105.456	431
D22R	HSS8x8x.375	21.45	367.462	30.62183	34.32	153
D22CL	HSS8x8x.376	41.86	232.215	19.35125	66.976	298
D22CO	HSS8x8x.377	66.01	225.362	18.78017	105.616	310
D23R	HSS8x8x.378	-21.43	367.462	30.62183	-34.288	431
D23CL	HSS8x8x.379	-41.77	232.215	19.35125	-66.832	431
D23CO	HSS8x8x.380	-65.91	225.362	18.78017	-105.456	431
D32	HSS8x8x.375	-44.12	240	20	-70.592	431
D33	HSS8x8x.376	66.04	308.914	25.74283	105.664	151
D34	HSS8x8x.377	-40.67	273.805	22.81708	-65.072	431
D35	HSS8x8x.378	41.94	282.136	23.51133	67.104	134
D36	HSS8x8x.379	-21.21	399.891	33.32425	-33.936	431
D37	HSS8x8x.380	20.98	396.011	33.00092	33.568	73.8
D14	HSS8x8x.375	-56.64	247.386	20.6155	-90.624	431
D15	HSS8x8x.376	76.67	343.641	28.63675	122.672	105
D16	HSS8x8x.377	-36.61	390.427	32.53558	-58.576	431
D17	HSS8x8x.378	41.08	436.807	36.40058	65.728	65.9
D18	HSS8x8x.379	43.49	297.692	24.80767	69.584	128
D19	HSS8x8x.380	-43.57	297.692	24.80767	-69.712	431

APPENDIX B

TRUSS A CALCULATIONS

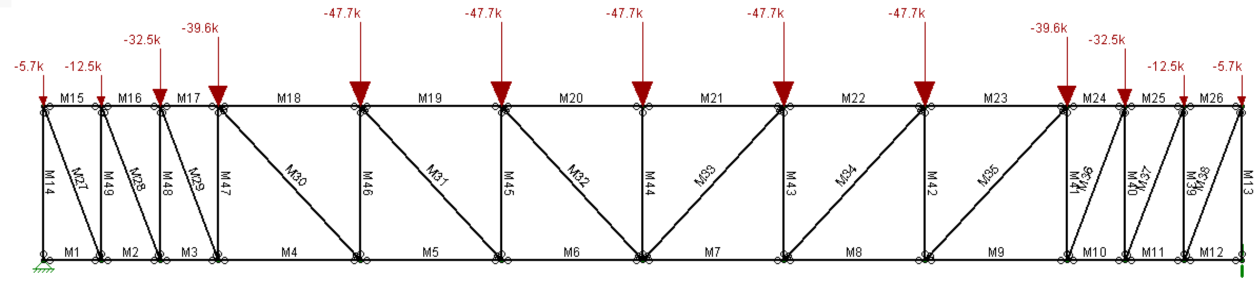


Member Forces (Tensile)					
Member	Length(in)	Length(ft)	Axial(K)	Minimum 2L Shape	Axial Capacity
15	293.21	24.43	-233.14	2l4x4x3/4	353.00
16	331.92	27.66	-261.04	2l4x4x3/4	353.00
17	392.05	32.67	-274.30	2l4x4x3/4	353.00
18	392.05	32.67	-104.14	2L2-1/2X2-1/2X1/2	146.00
19	392.05	32.67	-34.71	2L2-1/2X1-1/2X1/4	60.00
20	392.05	32.67	-34.71	2L2-1/2X1-1/2X1/4	60.00
21	392.05	32.67	-104.14	2L2-1/2X2-1/2X1/2	146.00
22	392.05	32.67	-274.30	2l4x4x3/4	353.00
23	331.92	27.66	-261.04	2l4x4x3/4	353.00
24	293.21	24.43	-233.14	2l4x4x3/4	353.00
34	55	4.58	0.00	Chord Members	
35	165	13.75	-43.73		
36	266	22.17	-173.50		
37	266	22.17	-359.61		
38	266	22.17	-430.26		
39	266	22.17	-430.26		
40	266	22.17	-359.61		
41	266	22.17	-173.50		
42	165	13.75	-43.73		
43	55	4.58	0.00		

Member Forces (Compressive)					
Member	Length	Length (ft)	Axial(K)	Minimum 2L Shape	Axial Capaci
4	288	24.00	229.62	2L8X8X1/2	241
5	288	24.00	229.62	2L8X8X1/2	241
6	288	24.00	229.00	2L8X8X1/2	241
7	288	24.00	226.50	2L8X8X1/2	241
8	288	24.00	201.50	2L8X8X1/2	241
9	288	24.00	76.50	2L6X6X1/2	108
10	288	24.00	51.00	2L6X6X1/2	108
11	288	24.00	76.50	2L6X6X1/2	108
12	288	24.00	201.50	2L8X8X1/2	241
13	288	24.00	226.50	2L8X8X1/2	241
14	288	24.00	229.00	2L8X8X1/2	241
26	266	22.17	359.61	Chord Members	
27	266	22.17	430.26		
28	266	22.17	453.81		
29	266	22.17	453.81		
30	266	22.17	430.26		
31	266	22.17	359.61		
32	165	13.75	173.50		
33	55	4.58	43.73		
24A	55	4.58	43.73		
25A	165	13.75	173.50		

TABLE: Joint Displacements			
Joint	U1 (in)	U2(in)	U3(in)
1	0.00	0.00	0.00
2	0.44	0.00	-0.10
11	0.19	0.00	-3.62
12	0.19	0.00	-3.64
21	0.37	0.00	0.00
22	-0.07	0.00	-0.10
23	0.44	0.00	-0.60
24	0.00	0.00	-0.50
25	0.42	0.00	-1.31
26	0.01	0.00	-1.21
27	0.35	0.00	-2.26
28	0.04	0.00	-2.17
29	0.27	0.00	-3.05
30	0.11	0.00	-3.00
31	0.10	0.00	-3.05
32	0.27	0.00	-3.02
33	0.02	0.00	-2.27
34	0.34	0.00	-2.18
35	-0.05	0.00	-1.32
36	0.37	0.00	-1.21
37	-0.07	0.00	-0.60
38	0.37	0.00	-0.50

TRUSS B CALCULATIONS

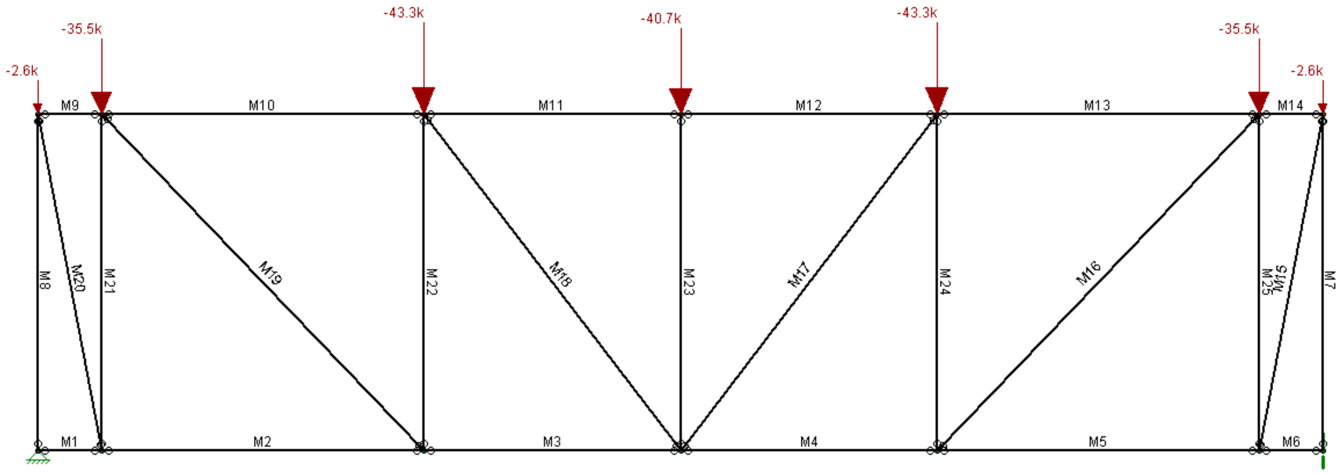


Member Forces (Tensile)					
Memb	Length(i	Length(ft)	Axial(	Minimum 2L Shap	Axial Capacity
38	308.29	25.69	-218.21	2L4X3X5/8	252.00
37	308.29	25.69	-204.83	2L4X3X5/8	252.00
36	308.29	25.69	-170.04	2L3-1/2X2-1/2-1/2	178
35	392.05	32.67	-162.33	2L3-1/2X2-1/2-1/2	178
34	392.05	32.67	-97.40	2L2-1/2X2X3/8	100.00
33	392.05	32.67	-32.47	2L2X2X3/8	88.1
32	392.05	32.67	-32.47	2L2X2X3/8	88.1
31	392.05	32.67	-97.40	2L2-1/2X2X3/8	100.00
30	392.05	32.67	-162.33	2L3-1/2X2-1/2-1/2	178
29	308.29	25.69	-170.04	2L3-1/2X2-1/2-1/2	178
28	308.29	25.69	-204.83	2L4X3X5/8	252.00
27	308.29	25.69	-218.21	2L4X3X5/8	252.00
12	110.00	9.17	0.00	Chord Members	
11	110.00	9.17	-77.86		
10	110.00	9.17	-150.94		
9	266.00	22.17	-211.62		
8	266.00	22.17	-321.76		
7	266.00	22.17	-387.84		
6	266.00	22.17	-387.84		
5	266.00	22.17	-321.76		
4	266.00	22.17	-211.62		
3	110.00	9.17	-150.94		
2	110.00	9.17	-77.86		
1	110.00	9.17	0.00		

TABLE: Joint Displacements			
Joint	X	Y	Z
1	0	0	0
13	0.44	0.00	-3.75
14	0.44	0.00	-3.78
25	0.88	0.00	0.00
28	0.27	0.00	-3.38
31	0.13	0.00	-2.69
34	0.04	0.00	-1.78
37	0.01	0.00	-1.20
40	0.00	0.00	-0.60
43	0.61	0.00	-3.38
46	0.74	0.00	-2.69
49	0.84	0.00	-1.78
52	0.86	0.00	-1.20
55	0.88	0.00	-0.60
344	0.53	0.00	-3.43
387	0.75	0.00	-0.12
390	0.12	0.00	-0.12
391	0.34	0.00	-3.43
392	0.25	0.00	-2.76
393	0.17	0.00	-1.86
394	0.15	0.00	-1.31
395	0.13	0.00	-0.72
396	0.75	0.00	-0.72
397	0.73	0.00	-1.31
398	0.71	0.00	-1.86
399	0.63	0.00	-2.76
400	0.54	0.00	-3.43

Member Forces (Compressive)					
Membe	Length(in)	Length(ft)	Axial(K)	Minimum 2L Shap	Axial Capacity
13	288	24.00	209.55	2L8X8X1/2	241
14	288	24.00	209.55	2L8X8X1/2	241
15	110	9.17	77.86	Chord Members	
16	110	9.17	150.94		
17	110	9.17	211.62		
18	266	22.17	321.76		
19	266	22.17	387.84		
20	266	22.17	409.87		
21	266	22.17	409.87		
22	266	22.17	387.84		
23	266	22.17	321.76		
24	110	9.17	211.62		
25	110	9.17	150.94		
26	110	9.17	77.86		
39	288	24.00	203.85	2L8X8X1/2	241
40	288	24.00	191.35	2L6X6X1	193
41	288	24.00	158.85	2L6X6X1	193
42	288	24.00	119.25	2L6X6X1	193
43	288	24.00	71.55	2L6X6X1/2	108
44	288	24.00	47.70	2L6X6X1/2	108
45	288	24.00	71.55	2L6X6X1/2	108
46	288	24.00	119.25	2L6X6X1	193
47	288	24.00	158.85	2L6X6X1	193
48	288	24.00	191.35	2L6X6X1	193
49	288	24.00	203.85	2L8X8X1/2	241

TRUSS C CALCULATIONS

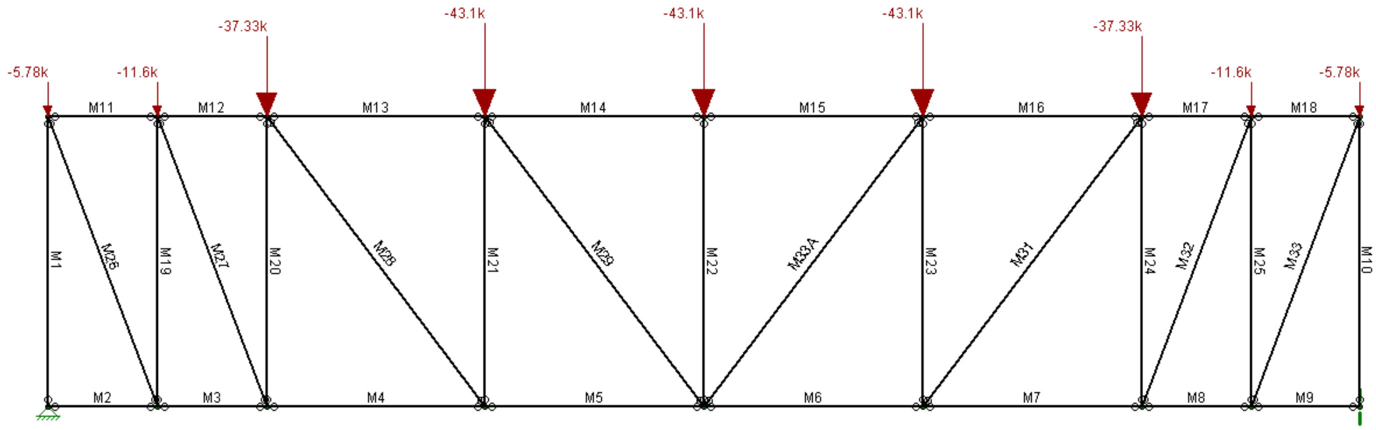


Member Forces(Tensile)					
Member	Length(in)	Length(ft)	Axial(K)	Minimum 2L Shape	Axial Capacity
1	55.00	4.58	0.00	Chord Members	
2	275.00	22.92	-18.94		
3	220.00	18.33	-79.71		
4	220.00	18.33	-79.71		
5	275.00	22.92	-18.94		
6	55.00	4.58	0.00		
15	293.21	24.43	-100.94	2L2-1/2X2-1/2X1/2	146
16	398.21	33.18	-88.01	2L2-1/2X2X3/8	100.00
17	362.41	30.20	-25.61	2L2X2X3/8	88.1
18	362.41	30.20	-25.61	2L2X2X3/8	88.1
19	398.21	33.18	-88.01	2L2-1/2X2X3/8	100.00
20	293.21	24.43	-100.94	2L2-1/2X2-1/2X1/2	146

TABLE: Joint Displacements			
Joint	X	Y	Z
1	0	0	0
2	-1.17E-14	0	-0.075895
3	2.332E-14	0	0
4	1.167E-14	0	-0.075895
5	0.369836	0	-1.120431
6	0.414979	0	-0.371635
7	-1.054E-14	0	-0.45108
8	1.167E-15	0	-0.371635
9	-4.694E-15	0	-1.182041
10	7.002E-15	0	-1.120431
11	4.655E-15	0	-1.182041
12	1.05E-14	0	-0.45108
186	-0.20749	0	-1.557879
196	-0.20749	0	-1.577476

Member Forces (Compressive)					
Member	Length(in)	Length(ft)	Axial(K)	Minimum 2L Shape	Axial Capacit
7	288	24.00	101.75	2L6X6X1/2	108
9	288	24.00	18.94	Chord Members	
10	55	4.58	79.71		
11	275	22.92	95.26		
12	220	18.33	95.26		
13	220	18.33	79.71		
14	275	22.92	18.94		
21	55	4.58	99.15	2L2-1/2X2-1/2X1/2	116
22	288	24.00	63.65	2L5X5X5/8	73.5
23	288	24.00	40.70	2L5X5X5/8	73.5
24	288	24.00	63.65	2L5X5X5/8	73.5
25	288	24.00	99.15	2L6X6X1/2	108

TRUSS D CALCULATIONS



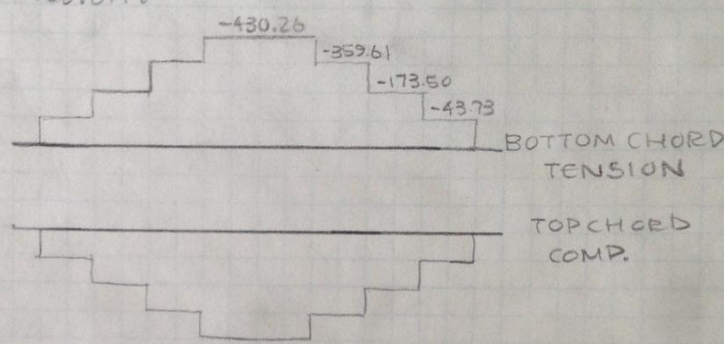
Member Forces(Compressive)					
Membe	Length(in	Length(ft	Axial(K	Minimum 2L Shape	Axial Capacity
1	288	24.00	119.36	2L6X6X5/8	131
10	288	24.00	119.36	2L6X6X5/8	131
11	110	9.17	43.38	Chord Members	
12	110	9.17	82.33		
13	220	18.33	131.72		
14	220	18.33	148.18		
15	220	18.33	148.18		
16	220	18.33	131.72		
17	110	9.17	82.33		
18	110	9.17	43.38		
19	288	24.00	113.58	2L6X6X5/8	131
20	288	24.00	101.98	2L6X6X5/8	131
21	288	24.00	64.65	2L5X5X5/8	73.5
22	288	24.00	43.10	2L5X5X5/8	73.5
23	288	24.00	64.65	2L5X5X5/8	73.5
24	288	24.00	101.98	2L6X6X5/8	131
25	288	24.00	113.58	2L6X6X5/8	131

TABLE: Joint Displacements			
Joint	X	Y	Z
1	0	0	0
57	0.47	0.00	0.00
370	0.24	0.00	-1.80
371	0.24	0.00	-1.84
372	0.10	0.00	-1.47
376	0.00	0.00	-0.46
378	0.37	0.00	-1.47
382	0.47	0.00	-0.46
387	0.43	0.00	-0.08
413	0.04	0.00	-0.08
465	0.02	0.00	-0.91
466	0.45	0.00	-0.91
467	0.42	0.00	-0.54
468	0.40	0.00	-0.98
469	0.32	0.00	-1.53
470	0.15	0.00	-1.53
471	0.07	0.00	-0.99
472	0.05	0.00	-0.54

Member Forces(Tensile)					
Member	Length(in)	Length(ft)	Axial(K)	Minimum 2L Shape	Axial Capacity
2	110.00	9.17	0.00	Chord Members	
3	110.00	9.17	-43.38		
4	220.00	18.33	-82.33		
5	220.00	18.33	-131.72		
6	220.00	18.33	-131.72		
7	220.00	18.33	-82.33		
8	110.00	9.17	-43.38		
9	110.00	9.17	0.00		
26	308.29	25.69	-121.58	2L2-1/2X2-1/2X1/2	146
27	308.29	25.69	-109.17	2L2-1/2X2-1/2X1/2	146
28	362.41	30.20	-81.35	2L2X2X3/8	88.1
29	362.41	30.20	-27.12	2L2X2X3/8	88.1
31	362.41	30.20	-81.35	2L2X2X3/8	88.1
32	308.29	25.69	-109.17	2L2-1/2X2-1/2X1/2	146
33	308.29	25.69	-121.58	2L2-1/2X2-1/2X1/2	146
33A	362.41	30.20	-27.12	2L2X2X3/8	88.1

TRUSS CHORD DESIGN SAMPLE

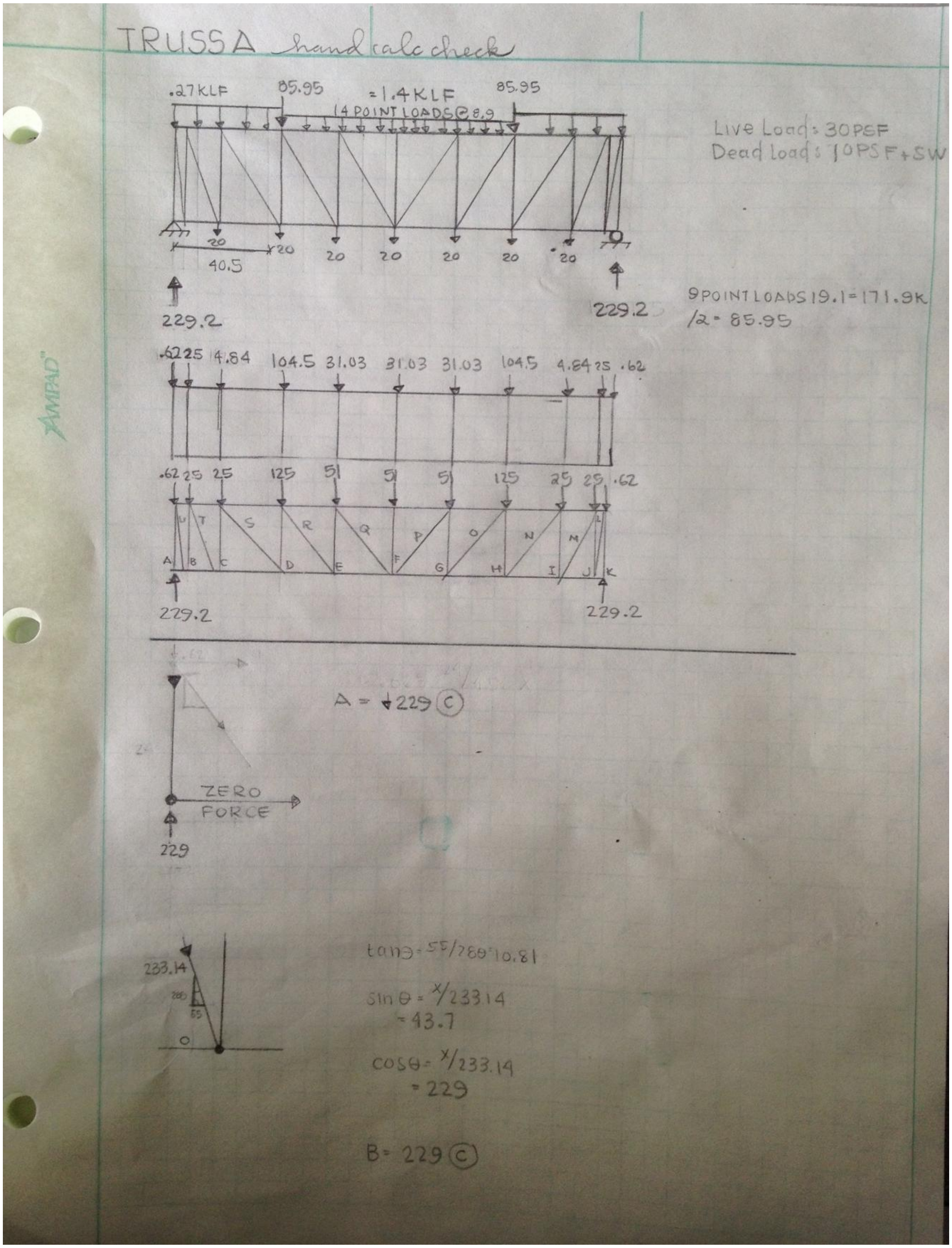
depth: 24ft
tribwidth: 78ft
length: 169.67ft



TOP CHORD → W14X90 $\phi P_{nc} = 821^{k}$

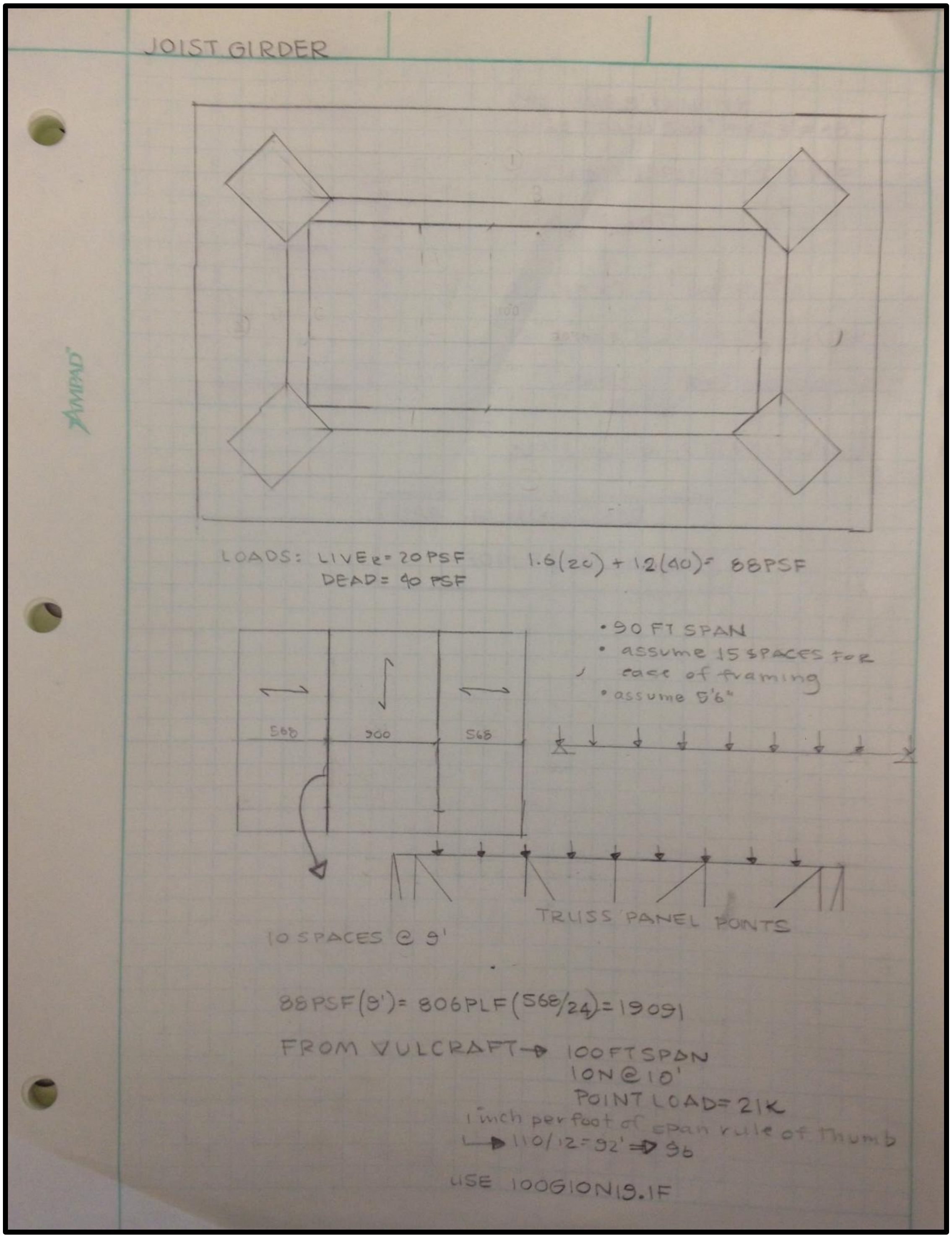
$A = 62\ in^2$
$I = 2660$

BOTTOM CHORD → W14X90 → $\phi P_{nc} = 970^{k}$

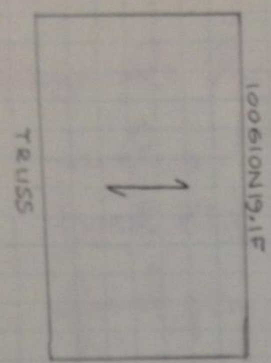

TRUSS A hand calc check
.27KLF
85.95
=1.4KLF
85.95
14 POINT LOADS @ 8.9
Live Load = 30PSF
Dead load = 70PSF + SW
40.5
229.2
229.2
9 POINT LOADS 19.1 = 171.9K
/2 = 85.95
ZERO FORCE
229
A = 229 C
tanθ = 55/280 = 10.81
sin θ = x/233.14
= 43.7
cosθ = x/233.14
= 229
B = 229 C

APPENDIX C

JOIST SELECTION



E-W INT. JOISTS



• ASSUME 9' SPACING

CLEAR SPAN: 568" = 47.3' ≈ 48'

LIVE LOAD = 30PSF(9') = 270 PLF

USE 28LH13

$\ell/180) = 47(12)/180 = 3.13$ in

$\Delta L = 1.15(27/12)(48(12))^4 / 384(29000)$

$I = 26.767(266)(48 \quad 4.67)^3(10^{-6})$

$= 820.8$

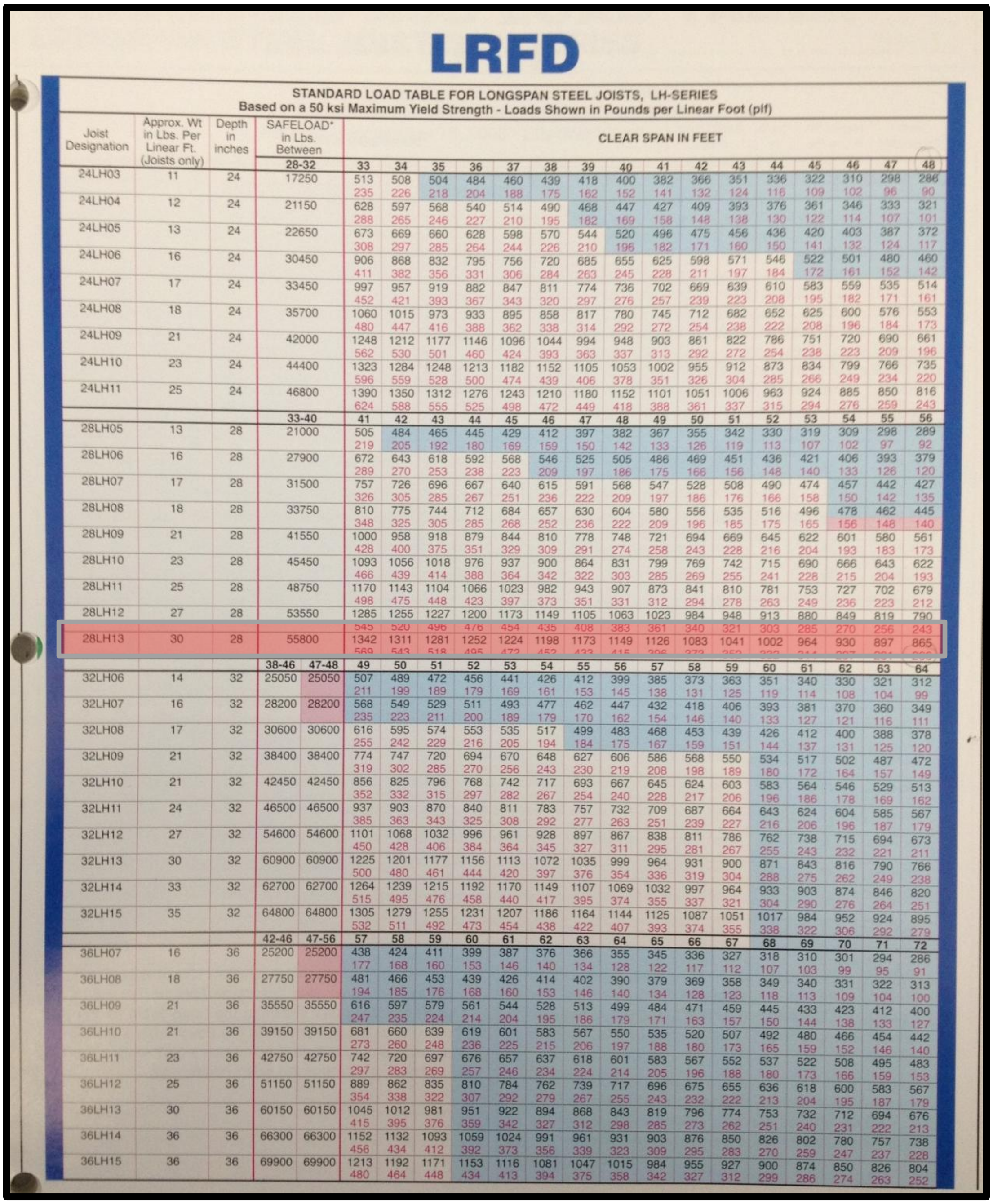
$\Delta L = 3.116 \rightarrow 3.13 > 3.16$ ✓ OKAY

$\boxed{\text{USE 28LH13 JOISTS} = 30\text{PLF (28 in deep)}}$

LRFD

STANDARD LOAD TABLE LONGSPAN STEEL JOISTS, LRFD DLH-SERIES
Based on a 50 ksi Maximum Yield Strength - Loads Shown in Pounds per Linear Foot (plf)

Joist Designation	Approx. Wt in Lbs. Per Linear Ft (Joists only)	Depth in inches	SAFE LOAD* in Lbs. Between		CLEAR SPAN IN LINEAR FEET															
			70-99	100-104	105	106	107	108	109	110	111	112	113	114	115	116	117	118	119	120
60DLH12	29	60	46650	46650	442	433	426	418	411	405	397	391	384	378	372	366	360	354	348	342
					168	163	158	154	150	146	142	138	134	131	128	124	121	118	115	113
60DLH13	35	60	56700	56700	537	526	517	508	499	490	483	474	466	459	451	444	436	429	423	415
					203	197	191	187	181	176	171	167	163	158	154	151	147	143	139	135
60DLH14	40	60	63000	63000	597	586	574	564	555	544	534	525	516	507	498	490	481	474	465	457
					216	210	205	199	193	189	183	178	173	170	165	161	156	152	149	145
60DLH15	43	60	73950	73950	700	687	675	663	651	640	628	618	607	597	588	577	568	559	550	541
					255	248	242	235	228	223	216	210	205	200	194	190	185	180	175	171
60DLH16	46	60	81300	81300	769	756	741	727	714	702	690	676	666	654	642	631	621	610	600	589
					285	277	269	262	255	247	241	235	228	223	217	211	206	201	196	190
60DLH17	52	60	93450	93450	885	868	853	837	822	807	793	778	765	751	739	726	714	702	690	679
					324	315	306	298	290	283	275	267	261	254	247	241	[illegible]	[illegible]	[illegible]	[illegible]
60DLH18	59	60	107850	107850	1021	1002	984	966	948	931	915	898	883	867	852	838	823	810	796	783
					366	357	348	337	327	319	310	303	294	286	279	272	266	259	252	246
			75-99	100-112	113	114	115	116	117	118	119	120	121	122	123	124	125	126	127	128
64DLH12	31	64	45000	45000	396	388	382	376	370	364	358	352	346	342	336	331	327	321	316	312
					153	150	146	142	138	135	132	129	125	122	119	116	114	111	109	106
64DLH13	34	64	54600	54600	481	472	465	457	450	442	436	429	421	415	409	403	396	390	385	379
					186	181	176	171	168	163	159	155	152	148	144	141	137	134	131	128
64DLH14	40	64	62550	62550	550	540	531	523	514	505	498	489	481	474	466	459	451	444	438	430
					199	193	189	184	179	174	171	166	162	158	154	151	147	143	140	136
64DLH15	43	64	71700	71700	631	621	610	600	591	580	571	562	553	544	537	528	520	511	504	496
					234	228	223	217	211	206	201	196	191	187	182	177	173	170	165	161
64DLH16	46	64	80700	80700	711	699	687	675	664	652	642	631	621	610	601	591	582	573	564	555
					262	254	248	242	235	229	224	218	213	208	203	198	193	189	184	180
64DLH17	52	64	93000	93000	819	804	790	777	763	751	738	726	714	702	691	681	669	658	648	639
					298	290	283	275	268	262	255	248	243	237	231	226	220	215	210	205
64DLH18	59	64	107400	107400	945	928	912	897	880	867	852	838	823	810	798	784	772	760	748	736
					337	328	320	311	304	296	288	282	274	267	261	255	249	243	237	232
			80-99	100-120	121	122	123	124	125	126	127	128	129	130	131	132	133	134	135	136
68DLH13	37	68	52500	52500	432	426	418	412	406	400	394	388	382	378	372	366	361	355	351	346
					171	168	164	159	155	152	149	145	142	138	135	133	130	127	124	121
68DLH14	40	68	60450	60450	498	490	483	475	468	462	454	448	441	435	429	421	415	409	403	399
					184	179	175	171	167	163	159	155	152	148	145	141	138	135	133	130
68DLH15	44	68	67800	67800	558	547	540	531	522	514	505	498	490	483	475	468	462	454	448	441
					206	201	196	191	187	182	178	174	170	166	162	158	155	152	148	145
68DLH16	49	68	80400	80400	661	649	640	630	619	610	600	591	582	573	564	556	547	540	531	523
					242	236	230	225	219	214	209	204	199	195	190	186	182	178	174	171
68DLH17	55	68	90600	90600	745	733	721	711	700	690	679	669	658	649	640	630	621	612	604	595
					275	268	262	256	249	244	238	232	228	222	217	212	208	203	198	194
68DLH18	61	68	104850	104850	862	849	835	823	810	798	786	774	762	751	739	729	718	708	697	688
					311	304	297	289	283	276	269	263	257	251	246	240	234	230	225	219
68DLH19	67	68	120750	120750	993	976	961	946	931	916	901	888	874	861	847	835	822	810	798	787
					353	344	336	328	320	313	305	298	291	285	278	272	266	260	254	248
			84-99	100-128	129	130	131	132	133	134	135	136	137	138	139	140	141	142	143	144
72DLH14	41	72	58800	58800	454	447	441	435	427	421	415	411	405	399	393	388	382	378	372	367
					171	167	163	159	155	152	149	146	143	139	136	133	131	128	125	123
72DLH15	44	72	67350	67350	520	513	504	496	489	483	475	468	462	454	448	442	436	429	423	418
					191	187	183	178	174	171	167	163	160	156	152	150	147	143	140	137
72DLH16	50	72	77850	77850	601	592	585	576	567	559	552	544	537	529	522	514	507	501	493	487
					225	219	214	209	205	200	196	191	188	183	179	175	171	169	165	161
72DLH17	56	72	87600	87600	676	667	657	648	639	630	621	612	603	595	586	579	571	564	556	549
					256	250	245	239	233	228	224	218	213	209	205	200	196	191	188	184
72DLH18	59	72	102600	102600	792	780	768	757	745	735	724	718	705	694	685	675	666	657	648	639
					289	283	276	270	265	258	252	247	242	236	231	227	222	217	212	209
72DLH19	70	72	120300	120300	928	913	900	886	873	859	847	835	823	811	799	789	777	766	756	745
					328	321	313	306	300	293	286	280	274	268	263	257	251	247	241	236



LRFD

STANDARD LOAD TABLE FOR LONGSPAN STEEL JOISTS, LH-SERIES
Based on a 50 ksi Maximum Yield Strength - Loads Shown in Pounds per Linear Foot (plf)

Joist Designation	Approx. Wt in Lbs. Per Linear Ft. (Joists only)	Depth in inches	SAFELOAD* in Lbs. Between 28-32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48
24LH03	11	24	17250	513 / 235	508 / 226	504 / 218	484 / 204	460 / 188	439 / 175	418 / 162	400 / 152	382 / 141	366 / 132	351 / 124	336 / 116	322 / 109	310 / 102	298 / 96	286 / 90
24LH04	12	24	21150	628 / 288	597 / 265	568 / 246	540 / 227	514 / 210	490 / 195	468 / 182	447 / 169	427 / 158	409 / 148	393 / 138	376 / 130	361 / 122	346 / 114	333 / 107	321 / 101
24LH05	13	24	22650	673 / 308	669 / 297	660 / 285	628 / 264	598 / 244	570 / 226	544 / 210	520 / 196	496 / 182	475 / 171	456 / 160	436 / 150	420 / 141	403 / 132	387 / 124	372 / 117
24LH06	16	24	30450	906 / 411	868 / 382	832 / 356	795 / 331	756 / 306	720 / 284	685 / 263	655 / 245	625 / 228	598 / 211	571 / 197	546 / 184	522 / 172	501 / 161	480 / 152	460 / 142
24LH07	17	24	33450	997 / 452	957 / 421	919 / 393	882 / 367	847 / 343	811 / 320	774 / 297	736 / 276	702 / 257	669 / 239	639 / 223	610 / 208	583 / 195	559 / 182	535 / 171	514 / 161
24LH08	18	24	35700	1060 / 480	1015 / 447	973 / 416	933 / 388	895 / 362	858 / 338	817 / 314	780 / 292	745 / 272	712 / 254	682 / 238	652 / 222	625 / 208	600 / 196	576 / 184	553 / 173
24LH09	21	24	42000	1248 / 562	1212 / 530	1177 / 501	1146 / 460	1096 / 424	1044 / 393	994 / 363	948 / 337	903 / 313	861 / 292	822 / 272	786 / 254	751 / 238	720 / 223	690 / 209	661 / 196
24LH10	23	24	44400	1323 / 596	1284 / 559	1248 / 528	1213 / 500	1182 / 474	1152 / 439	1105 / 406	1053 / 378	1002 / 351	955 / 326	912 / 304	873 / 285	834 / 266	799 / 249	766 / 234	735 / 220
24LH11	25	24	46800	1390 / 624	1350 / 588	1312 / 555	1276 / 525	1243 / 498	1210 / 472	1180 / 449	1152 / 418	1101 / 388	1051 / 361	1006 / 337	963 / 315	924 / 294	885 / 276	850 / 259	816 / 243

Joist Designation	Approx. Wt	Depth	33-40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56
28LH05	13	28	21000	505 / 219	484 / 205	465 / 192	445 / 180	429 / 169	412 / 159	397 / 150	382 / 142	367 / 133	355 / 126	342 / 119	330 / 113	319 / 107	309 / 102	298 / 97	289 / 92
28LH06	16	28	27900	672 / 289	643 / 270	618 / 253	592 / 238	568 / 223	546 / 209	525 / 197	505 / 186	486 / 175	469 / 166	451 / 156	436 / 148	421 / 140	406 / 133	393 / 126	379 / 120
28LH07	17	28	31500	757 / 326	726 / 305	696 / 285	667 / 267	640 / 251	615 / 236	591 / 222	568 / 209	547 / 197	528 / 186	508 / 176	490 / 166	474 / 158	457 / 150	442 / 142	427 / 135
28LH08	18	28	33750	810 / 348	775 / 325	744 / 305	712 / 285	684 / 268	657 / 252	630 / 236	604 / 222	580 / 209	556 / 196	535 / 185	516 / 175	496 / 165	478 / 156	462 / 148	445 / 140
28LH09	21	28	41550	1000 / 428	958 / 400	918 / 375	879 / 351	844 / 329	810 / 309	778 / 291	748 / 274	721 / 258	694 / 243	669 / 228	645 / 216	622 / 204	601 / 193	580 / 183	561 / 173
28LH10	23	28	45450	1093 / 466	1056 / 439	1018 / 414	976 / 388	937 / 364	900 / 342	864 / 322	831 / 303	799 / 285	769 / 269	742 / 255	715 / 241	690 / 228	666 / 215	643 / 204	622 / 193
28LH11	25	28	48750	1170 / 498	1143 / 475	1104 / 448	1066 / 423	1023 / 397	982 / 373	943 / 351	907 / 331	873 / 312	841 / 294	810 / 278	781 / 263	753 / 249	727 / 236	702 / 223	679 / 212
28LH12	27	28	53550	1285 / 545	1255 / 520	1227 / 496	1200 / 476	1173 / 454	1149 / 435	1105 / 408	1063 / 383	1023 / 361	984 / 340	948 / 321	913 / 303	880 / 285	849 / 270	819 / 256	790 / 243
28LH13	30	28	55800	1342 / 569	1311 / 543	1281 / 518	1252 / 495	1224 / 472	1198 / 452	1173 / 433	1149 / 415	1126 / [illegible]	1083 / [illegible]	1041 / [illegible]	1002 / [illegible]	964 / [illegible]	930 / [illegible]	897 / [illegible]	865 / [illegible]

Joist Designation	Approx. Wt	Depth	38-46	47-48	49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64
32LH06	14	32	25050	25050	507 / 211	489 / 199	472 / 189	456 / 179	441 / 169	426 / 161	412 / 153	399 / 145	385 / 138	373 / 131	363 / 125	351 / 119	340 / 114	330 / 108	321 / 104	312 / 99
32LH07	16	32	28200	28200	568 / 235	549 / 223	529 / 211	511 / 200	493 / 189	477 / 179	462 / 170	447 / 162	432 / 154	418 / 146	406 / 140	393 / 133	381 / 127	370 / 121	360 / 116	349 / 111
32LH08	17	32	30600	30600	616 / 255	595 / 242	574 / 229	553 / 216	535 / 205	517 / 194	499 / 184	483 / 175	468 / 167	453 / 159	439 / 151	426 / 144	412 / 137	400 / 131	388 / 125	378 / 120
32LH09	21	32	38400	38400	774 / 319	747 / 302	720 / 285	694 / 270	670 / 256	648 / 243	627 / 230	606 / 219	586 / 208	568 / 198	550 / 189	534 / 180	517 / 172	502 / 164	487 / 157	472 / 149
32LH10	21	32	42450	42450	856 / 352	825 / 332	796 / 315	768 / 297	742 / 282	717 / 267	693 / 254	667 / 240	645 / 228	624 / 217	603 / 206	583 / 196	564 / 186	546 / 178	529 / 169	513 / 162
32LH11	24	32	46500	46500	937 / 385	903 / 363	870 / 343	840 / 325	811 / 308	783 / 292	757 / 277	732 / 263	709 / 251	687 / 239	664 / 227	643 / 216	624 / 206	604 / 196	585 / 187	567 / 179
32LH12	27	32	54600	54600	1101 / 450	1068 / 428	1032 / 406	996 / 384	961 / 364	928 / 345	897 / 327	867 / 311	838 / 295	811 / 281	786 / 267	762 / 255	738 / 243	715 / 232	694 / 221	673 / 211
32LH13	30	32	60900	60900	1225 / 500	1201 / 480	1177 / 461	1156 / 444	1113 / 420	1072 / 397	1035 / 376	999 / 354	964 / 336	931 / 319	900 / 304	871 / 288	843 / 275	816 / 262	790 / 249	766 / 238
32LH14	33	32	62700	62700	1264 / 515	1239 / 495	1215 / 476	1192 / 458	1170 / 440	1149 / 417	1107 / 395	1069 / 374	1032 / 355	997 / 337	964 / 321	933 / 304	903 / 290	874 / 276	846 / 264	820 / 251
32LH15	35	32	64800	64800	1305 / 532	1279 / 511	1255 / 492	1231 / 473	1207 / 454	1186 / 438	1164 / 422	1144 / 407	1125 / 393	1087 / 374	1051 / 355	1017 / 338	984 / 322	952 / 306	924 / 292	895 / 279

Joist Designation	Approx. Wt	Depth	42-46	47-56	57	58	59	60	61	62	63	64	65	66	67	68	69	70	71	72
36LH07	16	36	25200	25200	438 / 177	424 / 168	411 / 160	399 / 153	387 / 146	376 / 140	366 / 134	355 / 128	345 / 122	336 / 117	327 / 112	318 / 107	310 / 103	301 / 99	294 / 95	286 / 91
36LH08	18	36	27750	27750	481 / 194	466 / 185	453 / 176	439 / 168	426 / 160	414 / 153	402 / 146	390 / 140	379 / 134	369 / 128	358 / 123	349 / 118	340 / 113	331 / 109	322 / 104	313 / 100
36LH09	21	36	35550	35550	616 / 247	597 / 235	579 / 224	561 / 214	544 / 204	528 / 195	513 / 186	499 / 179	484 / 171	471 / 163	459 / 157	445 / 150	433 / 144	423 / 138	412 / 133	400 / 127
36LH10	21	36	39150	39150	681 / 273	660 / 260	639 / 248	619 / 236	601 / 225	583 / 215	567 / 206	550 / 197	535 / 188	520 / 180	507 / 173	492 / 165	480 / 159	466 / 152	454 / 146	442 / 140
36LH11	23	36	42750	42750	742 / 297	720 / 283	697 / 269	676 / 257	657 / 246	637 / 234	618 / 224	601 / 214	583 / 205	567 / 196	552 / 188	537 / 180	522 / 173	508 / 166	495 / 159	483 / 153
36LH12	25	36	51150	51150	889 / 354	862 / 338	835 / 322	810 / 307	784 / 292	762 / 279	739 / 267	717 / 255	696 / 243	675 / 232	655 / 222	636 / 213	618 / 204	600 / 195	583 / 187	567 / 179
36LH13	30	36	60150	60150	1045 / 415	1012 / 395	981 / 376	951 / 359	922 / 342	894 / 327	868 / 312	843 / 298	819 / 285	796 / 273	774 / 262	753 / 251	732 / 240	712 / 231	694 / 222	676 / 213
36LH14	36	36	66300	66300	1152 / 456	1132 / 434	1093 / 412	1059 / 392	1024 / 373	991 / 356	961 / 339	931 / 323	903 / 309	876 / 295	850 / 283	826 / 270	802 / 259	780 / 247	757 / 237	738 / 228
36LH15	36	36	69900	69900	1213 / 480	1192 / 464	1171 / 448	1153 / 434	1116 / 413	1081 / 394	1047 / 375	1015 / 358	984 / 342	955 / 327	927 / 312	900 / 299	874 / 286	850 / 274	826 / 263	804 / 252

LRFD

STANDARD LOAD TABLE FOR LONGSPAN STEEL JOISTS, LH-SERIES
Based on a 50 ksi Maximum Yield Strength - Loads Shown in Pounds per Linear Foot (plf)

Joist Designation	Approx. Wt in Lbs. Per Linear Ft. (Joists Only)	Depth in inches	SAFELOAD* in Lbs. Between		CLEAR SPAN IN FEET															
			47-59	60-64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80
40LH08	16	40	24900	24900	381	370	361	351	342	333	325	316	309	301	294	288	280	274	267	261
					150	144	138	132	127	122	117	112	108	104	100	97	93	90	86	83
40LH09	21	40	32700	32700	498	484	472	459	447	436	424	414	403	394	384	375	366	358	349	342
					196	188	180	173	166	160	153	147	141	136	131	126	122	118	113	109
40LH10	21	40	36000	36000	550	535	520	507	493	481	469	457	445	435	424	414	403	393	382	373
					216	207	198	190	183	176	169	162	156	150	144	139	134	129	124	119
40LH11	22	40	39300	39300	598	582	567	552	537	523	510	498	484	472	462	450	439	429	418	409
					234	224	215	207	198	190	183	176	169	163	157	151	145	140	135	130
40LH12	25	40	47850	47850	729	708	688	670	652	636	619	603	588	573	559	546	532	519	507	495
					285	273	261	251	241	231	222	213	205	197	189	182	176	169	163	157
40LH13	30	40	56400	56400	859	835	813	792	771	750	730	712	694	676	660	643	628	613	598	585
					334	320	307	295	283	271	260	250	241	231	223	214	207	199	192	185
40LH14	35	40	64500	64500	984	957	930	904	880	856	834	813	792	772	753	735	717	699	682	666
					383	367	351	336	323	309	297	285	273	263	252	243	233	225	216	209
40LH15	36	40	72150	72150	1101	1068	1036	1006	978	949	924	898	874	850	828	807	786	766	747	729
					427	408	390	373	357	342	328	315	302	290	279	268	258	248	239	230
40LH16	42	40	79500	79500	1212	1194	1176	1158	1141	1126	1095	1065	1036	1009	982	957	933	909	886	864
					469	455	441	428	416	404	387	371	356	342	329	316	304	292	282	271
			52-59	60-72	73	74	75	76	77	78	79	80	81	82	83	84	85	86	87	88
44LH09	19	44	30000	30000	408	397	388	379	370	363	354	346	339	331	324	316	310	303	297	291
					158	152	146	141	136	131	127	122	118	114	110	106	103	99	96	93
44LH10	21	44	33150	33150	450	439	429	418	408	399	390	381	373	364	357	349	342	334	327	321
					174	168	162	155	150	144	139	134	130	125	121	117	113	110	106	103
44LH11	22	44	35850	35850	487	475	465	453	442	433	423	414	403	396	387	378	370	363	354	348
					188	181	175	168	162	157	151	146	140	136	131	127	123	119	115	111
44LH12	25	44	44400	44400	603	589	574	561	547	534	520	508	496	484	472	462	450	439	430	420
					232	224	215	207	200	192	185	179	172	166	160	155	149	144	139	134
44LH13	30	44	52650	52650	715	699	681	666	649	634	619	606	592	579	565	553	541	529	519	507
					275	265	254	246	236	228	220	212	205	198	191	185	179	173	167	161
44LH14	31	44	60600	60600	823	801	780	759	739	721	703	685	669	654	637	622	609	594	580	568
					315	302	291	279	268	259	249	240	231	223	215	207	200	193	187	181
44LH15	36	44	70500	70500	958	934	912	889	868	847	826	805	786	768	750	732	714	699	682	667
					366	352	339	326	314	303	292	281	271	261	252	243	234	227	219	211
44LH16	42	44	81300	81300	1105	1078	1051	1026	1002	978	955	933	912	891	870	852	832	814	796	780
					[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	291	282	272	263	255	246
44LH17	47	44	87300	87300	1185	1170	1153	1138	1125	1098	1072	1048	1024	1000	978	957	936	915	895	876
					450	438	426	415	405	390	376	363	351	338	327	316	305	295	285	276
			56-59	60-80	81	82	83	84	85	86	87	88	89	90	91	92	93	94	95	96
48LH10	21	48	30000	30000	369	361	354	346	339	331	325	318	312	306	300	294	288	282	277	271
					141	136	132	127	123	119	116	112	108	105	102	99	96	93	90	87
48LH11	22	48	32550	32550	399	390	382	373	366	358	351	343	337	330	324	318	312	306	300	294
					152	147	142	137	133	129	125	120	117	113	110	106	103	100	97	94
48LH12	25	48	41100	41100	504	493	483	472	462	451	442	433	424	415	408	399	391	384	376	369
					191	185	179	173	167	161	156	151	147	142	138	133	129	126	122	118
48LH13	29	48	49200	49200	603	589	576	564	552	540	529	517	507	498	487	477	468	459	450	441
					228	221	213	206	199	193	187	180	175	170	164	159	154	150	145	141
48LH14	32	48	58050	58050	712	696	681	666	651	637	624	610	598	585	574	562	550	540	529	519
					269	260	251	243	234	227	220	212	206	199	193	187	181	176	171	165
48LH15	36	48	66750	66750	817	799	781	765	748	732	717	702	687	672	658	645	633	619	607	595
					308	298	287	278	269	260	252	244	236	228	221	214	208	201	195	189
48LH16	42	48	76950	76950	943	922	901	882	864	844	826	810	792	777	760	745	730	715	702	688
					355	343	331	320	310	299	289	280	271	263	255	247	239	232	225	218
48LH17	47	48	86400	86400	1059	1035	1012	990	969	948	928	909	889	871	853	837	820	804	787	772
					397	383	371	358	346	335	324	314	304	294	285	276	268	260	252	245

DESIGN GUIDE WEIGHT TABLE FOR JOIST GIRDERS
U. S. CUSTOMARY

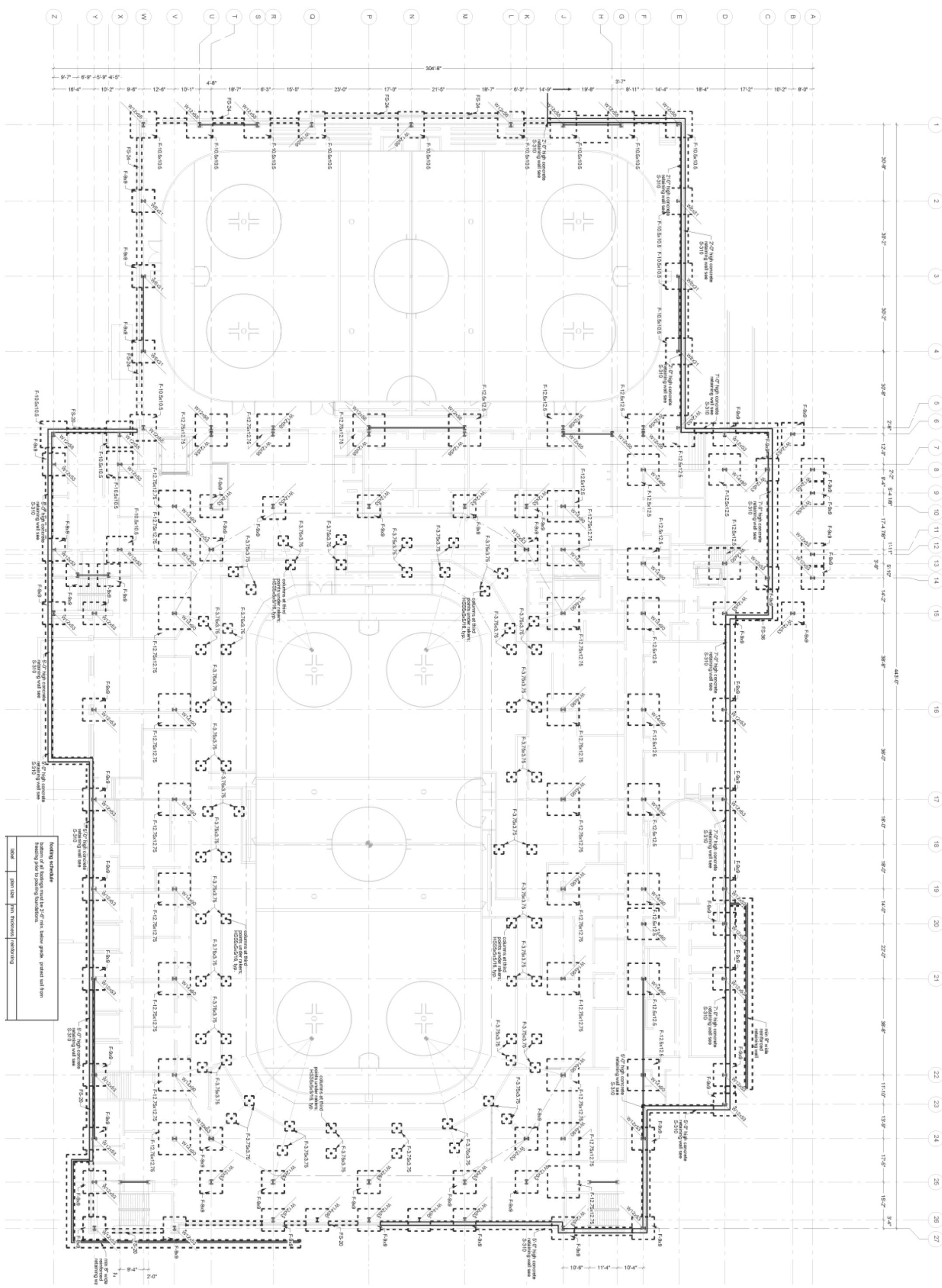
Based on a 50ksi maximum yield strength

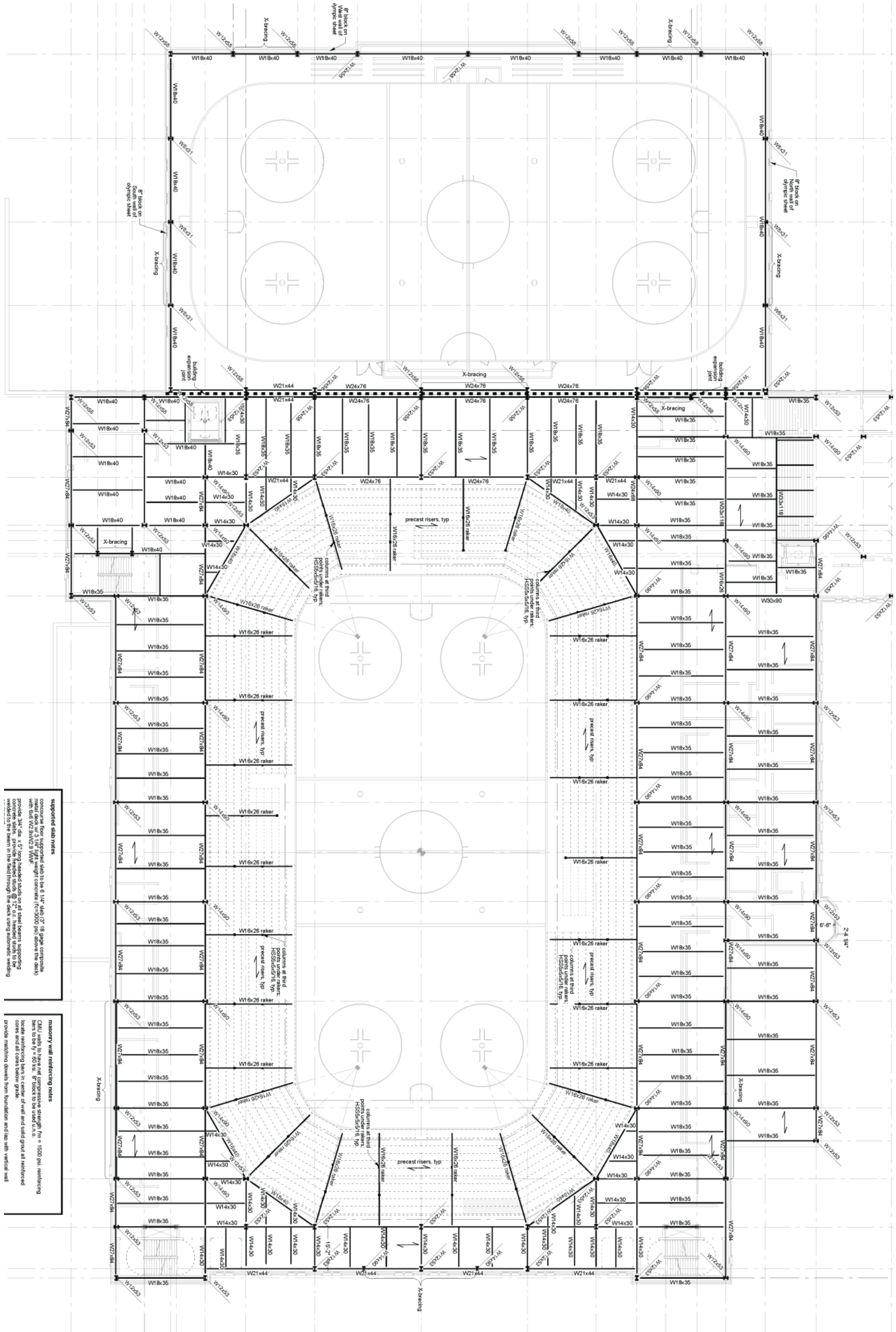
Girder Span (ft)	Joist Spaces (ft)	Girder Depth (in)	Joist Girder Weight – Pounds Per Linear Foot / Load on Each Panel Point																					
		LRFD	6K	7.5K	9K	10.5K	12K	13.5K	15K	16.5K	18K	21K	24K	27K	30K	37.5K	45K	52.5K	60K	75K	90K	105K	120K	150K
		ASD	4K	5K	6K	7K	8K	9K	10K	11K	12K	14K	16K	18K	20K	25K	30K	35K	40K	50K	60K	70K	80K	100K
		84	56	57	58	62	64	72	76	88	90	103	118	129	142	172	200	225	257					
	10N@	96	58	58	59	61	64	67	70	78	88	94	106	120	131	152	180	204	228					
	10.00	108	58	60	60	61	63	68	70	73	77	93	96	111	111	139	170	188	209	258				
		120	60	60	62	64	66	67	68	71	74	85	99	108	113	139	157	[illegible]	[illegible]	[illegible]	[illegible]			
		84	50	54	58	66	70	75	89	92	101	112	129	138	159	187	221	257						
	12N@	96	50	54	57	61	68	70	80	84	96	106	116	123	137	179	205	228	271					
	8.33	108	52	54	58	62	65	72	74	79	89	101	110	121	128	164	193	221	246	299				
		120	54	57	60	62	66	69	77	79	86	92	107	117	126	151	178	206	239	283				
		84	55	60	71	76	83	96	110	112	119	139	161	184	199	235	288							
	16N@	96	56	60	67	75	79	88	102	105	119	128	145	168	191	218	265	301						
100	6.25	108	58	63	67	72	81	87	93	106	111	125	136	157	180	204	251	292						
		120	60	65	68	74	79	90	93	98	110	117	134	147	166	208	248	275	304					
		84	57	65	73	82	92	98	112	114	123	151	164	187	203	250								
	17N@	96	60	65	72	81	89	103	110	123	123	145	177	179	198	256	285							
	5.88	108	64	67	72	76	86	96	108	113	123	135	158	172	182	231	264	308						
		120	67	68	73	80	85	90	99	112	119	133	143	167	178	214	250	281	330					
		84	67	77	87	105	115	122	132	148	159	193	208	226	246									
	20N@	96	67	73	82	95	111	120	126	135	152	177	199	211	227	279								
	5.00	108	66	72	79	91	101	116	125	130	131	162	184	197	207	267	316							
		120	71	75	82	88	96	106	120	123	136	149	170	193	205	246	289	332						
Bearing Depth			7 1/2 in.										10 in.											

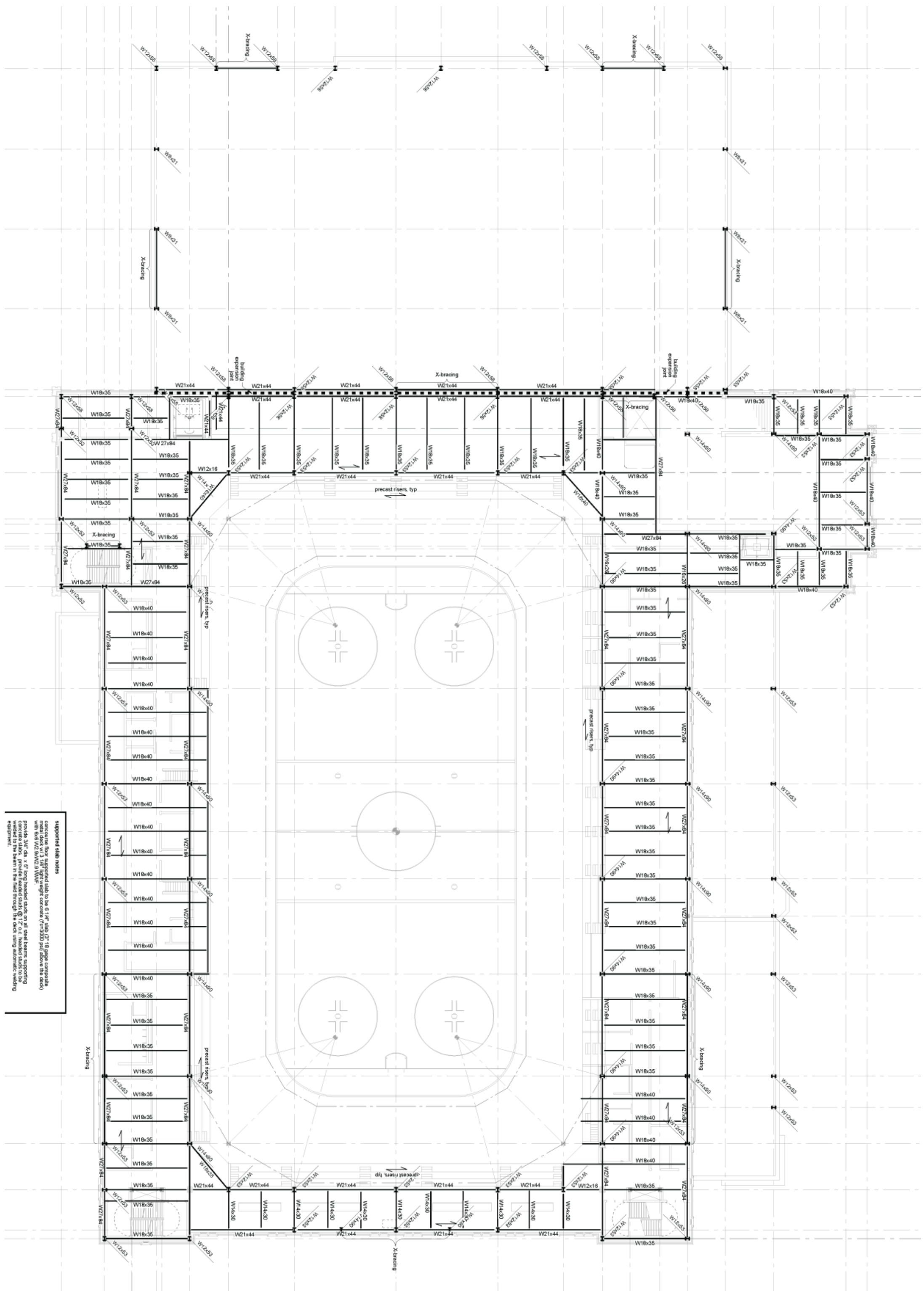
Joist Girder weights between the heavy black and blue lines have 7 1/2 inch bearing depths.
Joist Girder weights to the right of the heavy blue line have 10 inch bearing depths. Check with Vulcraft for material availability.

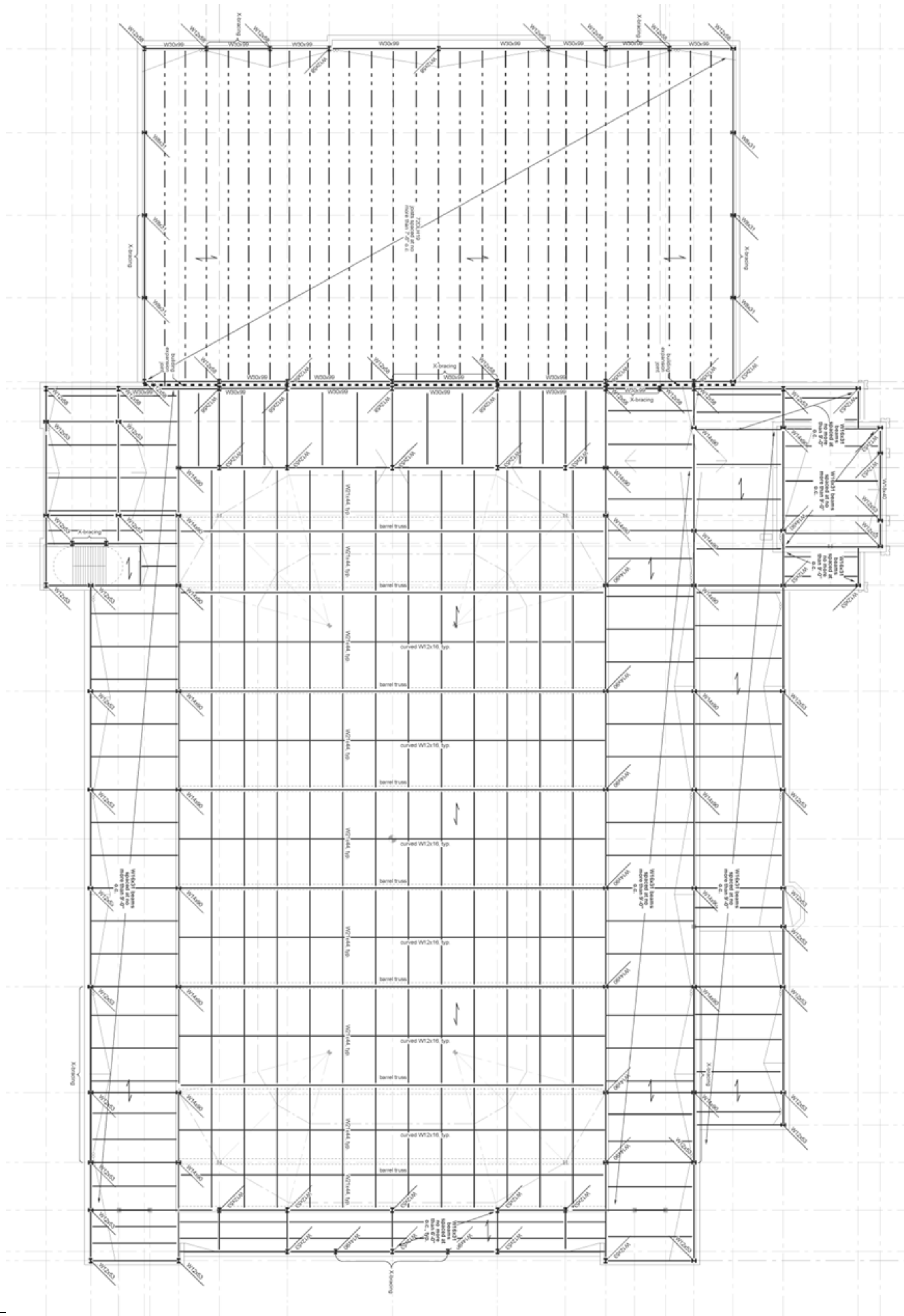
APPENDIX D

TYPICAL FLOOR PLANS

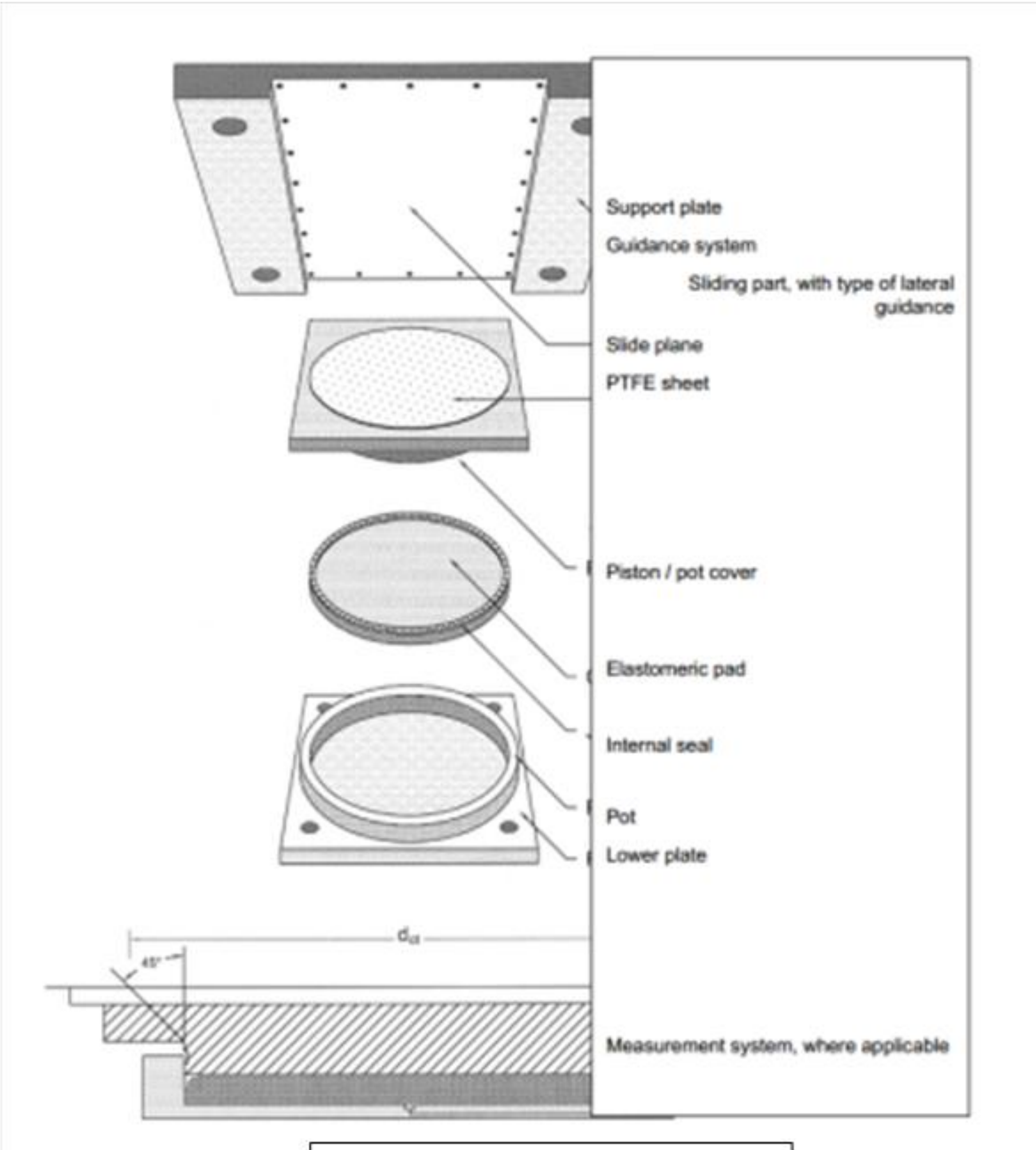








APPENDIX E



Make-Up of a typical pot bearing

APPENDIX F

01 54 Construction Aids

01 54 09 – Protection Equipment

		Crew	Daily Output	Labor-Hours	Unit	Material	2010 Bare Costs Labor	Equipment	Total	Total Incl O&P
01 54 09.60 Safety Nets										
0100	Polypropylene, 6" mesh				S.F.	1.59			1.59	1.75
0200	Small mesh debris nets, 1/4" & 3/4" mesh, stock sizes					.74			.74	.81
0220	Combined 4" mesh and 1/4" mesh, stock sizes					2.05			2.05	2.26
0300	Monthly rental, 4" mesh, stock sizes, 1st month					.50			.50	.55
0320	2nd month rental					.25			.25	.28
0340	Maximum rental/year				↓	1.15			1.15	1.27
01 54 16 – Temporary Hoists										
01 54 16.50 Weekly Forklift Crew										
0010	**WEEKLY FORKLIFT CREW**									
0100	All-terrain forklift, 45' lift, 35' reach, 9000 lb. capacity	A-3P	.20	40	Week		1,650	2,400	4,050	5,100
01 54 19 – Temporary Cranes										
01 54 19.50 Daily Crane Crews										
0010	**DAILY CRANE CREWS** for small jobs, portal to portal									
0100	12-ton truck-mounted hydraulic crane	A-3H	1	8	Day		355	870	1,225	1,500
0200	25-ton	A-3I	1	8			355	1,025	1,380	1,650
0300	40-ton	A-3J	1	8			355	1,250	1,605	1,900
0400	55-ton	A-3K	1	16			660	1,775	2,435	2,950
0500	80-ton	A-3L	1	16			660	2,225	2,885	3,425
0600	100-ton	A-3M	1	16	↓		660	2,425	3,085	3,675
01 54 19.60 Monthly Tower Crane Crew										
0010	**MONTHLY TOWER CRANE CREW**, excludes concrete footing									
0100	Static tower crane, 130' high, 106' jib, 6200 lb. capacity	A-3N	.05	176	Month		7,825	23,100	30,925	37,100

01 54 23 – Temporary Scaffolding and Platforms

6600	Labor only to erect & dismantle				"	43				
6610	Materials only, rent/mo									

01 54 23.75 Scaffolding Specialties

0010	**SCAFFOLDING SPECIALTIES**									
1200	Sidewalk bridge, heavy duty steel posts & beams, including									
1210	parapet protection & waterproofing									
1220	8′ to 10′ wide, 2 posts	3 Carp	15	1.600	L.F.	31.50	66.50		98	137
1230	3 posts	"	10	2.400	"	48.50	99.50		148	208
1500	Sidewalk bridge using tubular steel									
1510	scaffold frames, including planking	3 Carp	45	.533	L.F.	5.60	22		27.60	40
1600	For 2 uses per month, deduct from all above					50%				
1700	For 1 use every 2 months, add to all above					100%				
1900	Catwalks, 20″ wide, no guardrails, 7′ span, buy				Ea.	145			145	160
2000	10′ span, buy					203			203	223
3720	Putlog, standard, 8′ span, with hangers, buy					75			75	82.50
3730	Rent per month					10			10	11
3750	12′ span, buy					113			113	124
3755	Rent per month					15			15	16.50
3760	Trussed type, 16′ span, buy					260			260	286
3770	Rent per month					20			20	22
3790	22′ span, buy					310			310	345
3795	Rent per month					30			30	33
3800	Rolling ladders with handrails, 30″ wide, buy, 2 step					237			237	261
4000	7 step					725			725	800
4050	10 step					1,000			1,000	1,100
4100	Rolling towers, buy, 5′ wide, 7′ long, 10′ high					1,350			1,350	1,500
4200	For 5′ high added sections, to buy, add					207			207	228
4300	Complete incl. wheels, railings, outriggers,									
4350	21′ high, to buy				Ea.	2,250			2,250	2,475
4400	Rent/month = 5% of purchase cost				"	113			113	124