Coppin State University
Physical Education Complex
Baltimore, MD

Technical Assignment 3

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Executive Summary:

The purpose of this assignment is to analyze the lateral system of the arena in the Coppin State University Physical Education Complex. The Coppin State University Physical Education Complex in itself is a sprawling structure sub-divided by 3 expansion joints into 4 separate sub-buildings; Facilities Management, Arena, Physical Education North and Physical Education South (see Appendix A for a plan of the whole structure). Because of this fact, only the arena will be analyzed in this report. The arena as a sub-building is composed of the one story gym area that has long-span trusses composing the roofing system and the 3 story structure containing locker rooms, concessions and offices directly south of the gym (see Figure 1). The current lateral system for the arena is composed of braced frames, moment frames and roof trusses. This report will outline the current lateral loads and load combinations, provide information on stiffness and load paths, spot check certain critical members, and analyze the current system as a whole with an ETABS model.

Modifications have been made to lateral loads since preliminary investigation and the first technical report. Both wind and seismic forces have been adjusted, and the changes have been noted at the beginning of each section. Overall wind loads have been changed to incorporate internal pressures, so all sub-buildings have been adjusted accordingly. The building weight was looked in at closer detail for the arena, and thus the seismic loads for the arena have changed slightly. Wind base shears and overturning moments were typically greater for all sub-buildings when considering the 1.6 wind factor compared to 1.0 for seismic. It should be noted that the wind analysis was not performed for the E-W direction of the arena since it is completely enclosed. For this reason, the wind load will typically control the design for the N-S direction, but seismic will typically control designs in the E-W direction.

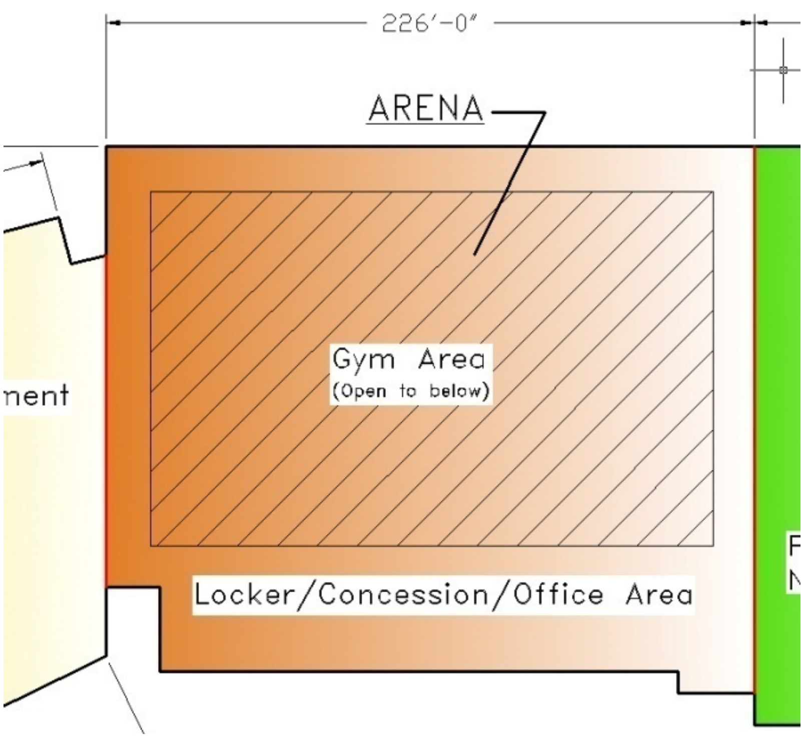


Figure 1 - Arena Spaces

As noted above, the lateral system is composed of braced frames in the E-W direction and moment frames accompanied by the roof trusses in the N-S direction. The lateral system can be seen below in Figure 2. Each lateral member is numbered T1 though T17 for referencing.
*Members T1 through T4 are braced frames.
*Members T5 through T9 are moment frames.
*Members T10 through T17 are roof trusses.
*The space directly under the truss members (T10-T17) is open space from the bottom of the trusses to ground level (No diaphragms).
*Piers extend to the top of floor 1 (15' above ground level) under braced frames T1, T2, and T3 as well as under the W14x257 columns supporting the roof trusses on the north side.
*Floor diaphragms exist only in the locker/concession/office area. For this reason the braced frames on the northern part of the building (T1 and T2) take very little lateral load. The load they do receive is primarily transferred by the roof diaphragm and through torsion. Their primary function is stability. They brace the large W14x257 columns and provide additional redundancy and support.

To analyze the lateral system an ETABS model was created. From the model I was able to consider many load combinations simultaneously. ASCE7-05 was used to create load combinations. The governing combinations were predominately 1.2D +1.6W + L +0.5Lr in the N-S direction and 1.2D +1.0E + L + 0.2S in the E-W direction.

This report will provide an in-depth look into the lateral framing system through member stiffness, load distribution, story shears, drift, and torsion. In addition, spot checks are provided for critical members. It is important to note that this report outlines the procedures and analysis I have used but does not claim any errors of any sort made by the design team.

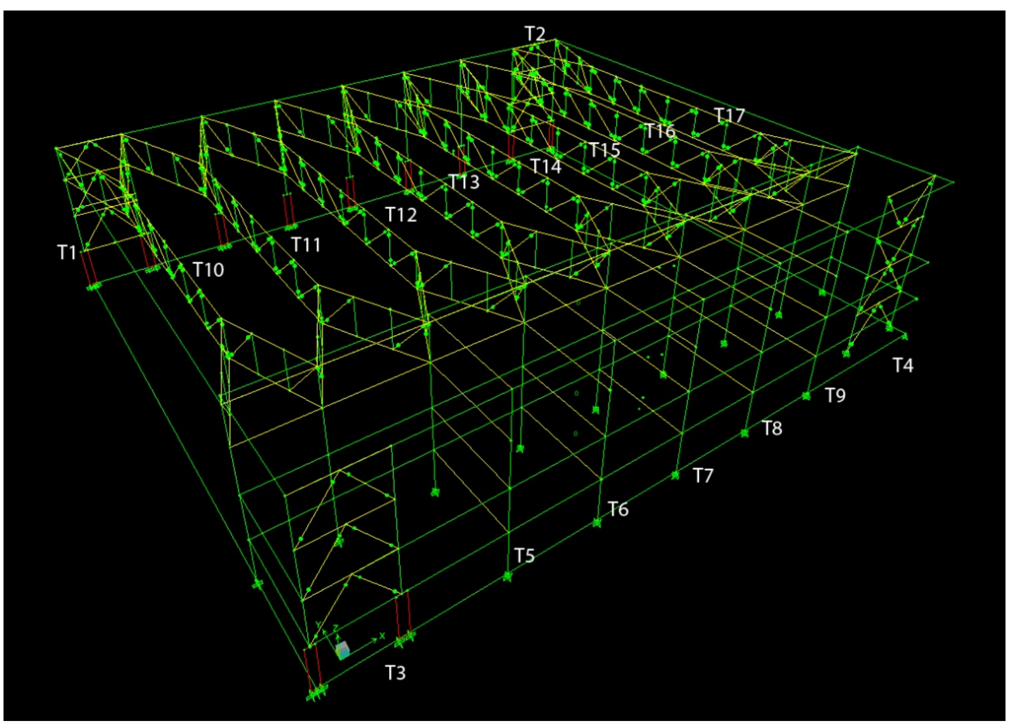


Figure 2 - ETABS Model

Building Description:

Architecture: This state of the art recreation and physical education complex at Coppin State University combines the beauty and sophistication of red brick alongside the sleek appearance that steel and glass construction provides. The building sprawls in several directions at several heights from the hub of the building, the new 2600 seat arena. The arena contains a fully functional basketball court that can be changed to incorporate other sporting events when needed. Probably the most dramatic features would be the exposed steel trusses supporting the roof of the arena. A variety of spaces are all contained within the complex in addition to the arena including an 8-lane swimming pool, racquetball courts, classrooms, and management facilities. The building uses several heights ranging from 30' to 60' which brings an exciting look to the exterior. Alongside the building, tennis courts and other outdoor facilities are being developed, and because the building extends in several directions, greenspaces can be easily incorporated as well. The building actually surrounds a soccer and training field. Canopies stationed around the perimeter also provide a nice gathering spot for young college students. The complex has the potential for bringing new light and rejuvenating the surrounding area.

Foundation: The foundation is comprised of spread footings and slab on grade. The spread footings use strengths of 3000psf, 6000psf and 10000psf allowable bearing pressure depending on loads and geotechnical data. The spread footings around the columns range from 4'x4' to 20'x20'. Typical footings are 12" thick, but various thicker footings exist in areas of especially high load such as under the soccer scoreboard. The typical floor slab is 8" thick concrete slab-on-grade reinforced with 6x6 W2.1x2.1 W.W.F. on waterproofing and 6" compacted granular fill, compacted to at least 95% of the maximum density as defined by the Modified Proctor Test. The concrete used is normal weight and has a minimum compressive strength at 28 days as follows:

Footings: 4000psi
Caisson Caps: 4000psi
Caissons: 4000psi
Walls + Piers: 4000psi
Grade Beams: 4000psi
Slab-On-Grade: 3500psi

The reinforcement bar strength is fy=60ksi for all areas.

Floor System: The floor system of the Coppin State University Physical Education Complex is composed primarily of composite steel beams with a concrete slab, typically 3.25" lightweight concrete on a 3"x20ga. galvanized composite metal deck reinforced with 6x6-W1.4x1.4 W.W.F.. The floor system supporting the SCUP rooms use a 5"x18ga. galvanized composite metal deck reinforced with #4@12" o.c. in direction of deck span and 6x6-W1.4x1.4 W.W.F. All concrete in the superstructure uses an f'c = 4000psi. The beams are typically spaced at 10' intervals (with few exceptions due to vertical openings) to eliminate shoring during construction. Supporting girders are spaced typically at 30'. There is not much conformity of W shape sizing throughout the building due to its odd shape and different loading and spanning conditions.

Columns: The columns of the Coppin State University Physical Education Complex are mostly W shapes. W12's are the most common, but W10's and W14's are also used. Square HSS shapes are also used as columns but rarely. The building uses steel gravity columns as well as moment framed columns. Because the building is only 4 stories maximum, there is only one splice maximum per column line, which generally occurs on level 3. Splicing is specified as 4' above the finished floor which makes the longest column 34'. The lightest W shape used is W10x33 and the heaviest is W14x257. All columns are A992 with minimum yield strength of 50ksi.

Lateral Force Resisting System: The structure is essentially 4 buildings side by side. A 3" expansion joint on both sides of the arena and another midway down the eastern part in effect divides the structure. Large trusses making up the roof structure of the arena are composed of W14x120 as top and bottom chords and HSS8x8x1/2 diagonal members and act as braced frames themselves. These, in conjunction with five moment frames in the area south of the arena resist N-S lateral loads in the arena. Four braced frames located at the four corners of the arena are oriented E-W and resist the E-W lateral loads in the arena area. In Facilities Management moment frames run E-W, and in Physical Education North and Physical Education South moment frames run N-S. Braced frames are oriented in both directions and are widespread throughout the structure. The lateral system for the arena will be examined in detail later in this report with an ETABS model and hand calculations.

Arena Trusses: The Coppin State University Physical Education Complex makes use of several trusses supporting the roof structure of the arena. The span of these trusses is 166'6". As noted before, W14x120's make up the top and bottom chords and HSS8x8x1/2's make up the diagonal members. The depth of the trusses is 10'7". The trusses do not span the 166'6" continuously, but rather the adjacent trusses meet about 45' from each end forming a triangle section (see Appendix E for visual clarification). The trusses are generally flat with a small slope for water runoff. Special connections are required at the midspan and intersection of the end triangle pieces.

Codes:

Building Code: International Building Code (IBC), 2003 edition

Steel Design: American Institute of Steel Construction LRFD (AISC) 9th Edition
AWSD1.1 Rev. 5

Concrete Design: ACI 301-99 (Building Design), ACI 318-02 (Building Code Requirements for Structural Concrete), ACI 315-99 (Details and Detailing of Concrete Reinforcement)

Loads:

Dead and Live Loads: The building uses several floor systems. The most common is the standard floor, but the SCUP area (area supporting the cooling towers), and mechanical rooms have a larger load. Other areas such as the canopy and the roof areas take a smaller load. These loads are outlined in the following table. The arena only uses standard loads and roof loads.

Table 1

Dead and Live Loads:					
Dead Load Description	Standard Floor	SCUP	Roof	Canopy	Mech. Floor
Concrete Slab	51	79			51
Metal Deck	2	2	2	2	2
M/E/C/L	7	10	16	6	7
Membrane			1.5	1.5	1.5
Roofing			3	3	3
Insulation			2.5	2.5	2.5
Total DL:	60	91	25	15	67
Live Load:	100	300	30	30	55

* Does Not Include Weight of Steel Members
*Live Load Reduction Taken Into Account

Lateral Loads:

Wind Loads: Since Technical Report 1, I have adjusted the wind load criteria to include internal pressure. The *P total* columns have changed significantly and so have base shears and overturning moments. Main Wind Force Resisting System was used for the analysis of wind loads. The structure was subdivided according to the 3 expansion joints into 4 sub-buildings: Facilities Management, Arena, Physical Education North, and Physical Education South (see Appendix A).The two tables below outline the wind loads per each sub-divided building. For a complete of wind design criteria see Appendix B.

Table 2

MWFRS: E-W direction				
Facilities Management		(ht=60ft)		
Height	**Kz**	**P ww**	**P lw**	**P total (psf)**
0-15	0.57	11.0	-7.5	18.5
15-20	0.62	11.7	-7.5	19.2
20-25	0.67	12.3	-7.5	19.8
25-30	0.70	12.8	-7.5	20.3
30-40	0.76	13.6	-7.5	21.1
40-50	0.81	16.3	-7.5	23.8
50-60	0.85	17.0	-7.5	24.5
Arena		(ht=60ft)		
Completely Enclosed- No Analysis Needed				
Physical Education North		(ht=30ft)		
0-15	0.57	10.5	-8.6	19.1
15-20	0.62	11.2	-8.6	19.8
20-25	0.67	11.7	-8.6	20.3
25-30	0.70	12.2	-8.6	20.8
Physical Education South		(ht=38ft)		
0-15	0.57	10.7	-9.2	19.9
15-20	0.62	11.3	-9.2	20.5
20-25	0.67	11.9	-9.2	21.1
25-30	0.70	12.4	-9.2	21.6
30-40	0.76	13.2	-9.2	22.4

Table 3

MWFRS: N-S direction				
Facilities Management		(ht=60ft)		
Height	**Kz**	**P ww**	**P lw**	**P total (psf)**
0-15	0.57	11.0	-7.5	18.5
15-20	0.62	11.7	-7.5	19.2
20-25	0.67	12.3	-7.5	19.8
25-30	0.70	12.8	-7.5	20.3
30-40	0.76	13.6	-7.5	21.1
40-50	0.81	16.3	-7.5	23.8
50-60	0.85	17.0	-7.5	24.5
Arena		(ht=60ft)		
0-15	0.57	11.0	-10.5	**21.5**
15-20	0.62	11.7	-10.5	**22.2**
20-25	0.67	12.4	-10.5	**22.9**
25-30	0.70	12.8	-10.5	**23.3**
30-40	0.76	13.6	-10.5	**24.1**
40-50	0.81	16.3	-10.5	**26.8**
50-60	0.85	17.0	-10.5	**27.5**
Physical Education North		(ht=30ft)		
0-15	0.57	10.8	-2.7	13.5
15-20	0.62	11.5	-2.7	14.2
20-25	0.67	12.0	-2.7	14.7
25-30	0.70	12.5	-2.7	15.2
Physical Education South		(ht=38ft)		
0-15	0.57	10.8	-2.7	13.5
15-20	0.62	11.5	-2.7	14.2
20-25	0.67	12.0	-2.7	14.7
25-30	0.70	12.5	-2.7	15.2
30-40	0.76	13.2	-2.7	15.9

Table 4

MWFRS: Uplift at Roof (psf) All Cases			
D (ft) from windward edge			
0	to	30	-13.09
30	to	60	-13.09
60	to	120	-8.38
	>	120	-6.02

A summary of wind base shears and overturning moments is shown in the following table. The Arena values are bolded because I will be using these values in this report. The other sub-buildings will not be examined in this report but are presented here for comparison purposes.

Table 5

Wind Load Base Shears and Overturning Moments		
	Total Base Shear(kips):	Total Overturning Moment(ft-kip):
Facilities Management		
E-W	255	9132
N-S	332	13544
Arena		
E-W	N/A	N/A
N-S	**292**	**10381**
Physical Education North		
E-W	270	6846
N-S	88	1952
Physical Education South		
E-W	141	4681
N-S	59	3244

Seismic Loads: The arena was looked at in more detail and seismic loads were adjusted since Technical Report 1. The overall base shear and overturning moments have also been adjusted. Loads are based on Seismic Use Group II in Site Class D, Seismic Design Category B and Basic Seismic Force Resisting System of Structural Steel Not Specifically Detailed for Seismic Resistance. Equivalent Lateral Force Method was used for the analysis. The building was again divided into 4 separate building for the analysis. The arena values have been bolded because I will be exploring the arena further in this report. The period from analysis with mass input (Tb) on ETABS was found to be 1.0256 seconds (see printout in Appendix D), and the period found with Equivalent Lateral Force procedure (Cu*Ta) was found to be a conservative 0.723 seconds. The period (T) used was based on Cu*Ta which controlled over Tb. For a complete description of seismic design criteria and arena weights see Appendix C.

Table 6

Seismic Base Shear and Moment Calculations							
Building	Level	Height(ft.)		h^kWx	Cvx	Fx	∑Fxh
Facilities Management	2	15		55003.82	0.250813	84.7748	1271.62
	3	30		45778.67	0.208747	70.5565	2116.7
	4	45		71509.48	0.326078	110.214	4959.64
	Roof N	30		19179.83	0.087459	29.561	886.829
	Roof S	60		27830.25	0.126904	42.8935	2573.61
			SUM	219302	1	338	11808.4
Arena	2	15		17521.77	0.08841	17.6819	265
	3	30		39708.57	0.200357	40.07147	1202.144
	Roof N	60		113851	0.574458	114.8915	6893.49
	Roof S	60		27107.39	0.136776	27.35512	1641.307
			SUM	**164398.6**	**1**	**200**	**10002.17**
Physical Education North	2	15		11169.89	0.142849	18.1418	272.126
	Roof	30		67024.02	0.857151	108.858	3265.75
			SUM	78193.91	1	127	3537.87
Physical Education South	2	15		22890.4	0.513244	50.8112	762.168
	Roof	30		21709.04	0.486756	48.1888	1445.66
			SUM	44599.44	1	99	2207.83

For a summary of seismic base shears and overturning moments of:

Table 7

Seismic Base Shear and Overturning Moments		
Building	Base Shear	Overturning Moment
Facilities Management	338k	11808'k
Arena	**200k**	**10002'k**
Physical Education North	127k	3538'k
Physical Education South	99k	2208'k

Summary of Lateral Loads:

These results show that wind loads control for most situations. Wind forces will typically be multiplied by a 1.6 factor and seismic by a 1.0 factor under load combinations, so the design forces will be typically 60% higher than the values presented in the previous tables. The arena will be explored for the duration of this report. Since the arena is completely enclosed in the E-W direction by the other surrounding buildings, seismic forces will typically control in the E-W direction, while wind forces will typically control in the N-S direction. An ETABS model will explore the arena lateral system further.

Load Combinations:

Load combinations were taken from ASCE 7-05. The lateral loads were broken down into N-S and E-W directions. These loading combinations were input into ETABS for ease of finding the controlling case for any situation. The N-S direction corresponds to direction Y and the E-W direction corresponds to direction X in ETABS. The load combinations are numbered so as to reference them by number later in the report. They are as follows:

1) 1.4D
2) 1.2D + 1.6L + 0.5Lr
3) 1.2D + 1.6Lr + L
4) 1.2D + 1.6W + L + 0.5Lr
5) 1.2D + 1.0Ex + L + 0.2S
6) 1.2D + 10.Ey + L + 0.2S
7) 0.9D + 1.6W
8) 0.9D + 1.0Ex
9) 0.9D +1.0Ey

ETABS Model:

An ETABS model was developed for the lateral system of the arena of the Coppin State University Physical Education Complex. A copy of Figure 2 is presented below for reference. Included in the model were the four braced frames making up the E-W lateral system, the five moment frames with the roof trusses making up the N-S lateral system, and the floor diaphragms. The gym area is open space between the ground floor and the roof, but the locker/concession/office area has 2 floors above grade. These floors are composed of composite steel with a typical 3.25" topping on a 3" deck, thus were modeled in ETABS as rigid diaphragms. The roofing system is a 4-1/2" metal deck without a topping. ASCE 7-05 12.3.1.2 allows this to be modeled as flexible, so it was modeled as such. All connections were assumed to be to be rigid except the diagonal members making up the braced frames and roof trusses. These diagonal members were modeled as pinned connections, so they were released of end fixity. The piers were modeled as area elements with a maximum mesh size of 24"x24". Area elements and line elements sometimes have trouble acting together in ETABS, so I changed them to line elements to double check. The shear values and periods I found varied very slightly, so the area elements were permitted for the analysis.

The lateral load inputs were based on hand calculated values and were input at the center of mass of each diaphragm. These calculated values can be found in Appendix B and Appendix C for wind and seismic respectively. To find the period of the structure, masses were input for each floor based on the weight of building materials. The controlling period (Tb) was found to be 1.0256 seconds, see Appendix D. Loads were then input to find controlling cases. Different load combinations controlled, depending on what was analyzed. The report notes what load combination controls for each element/frame/system analyzed.

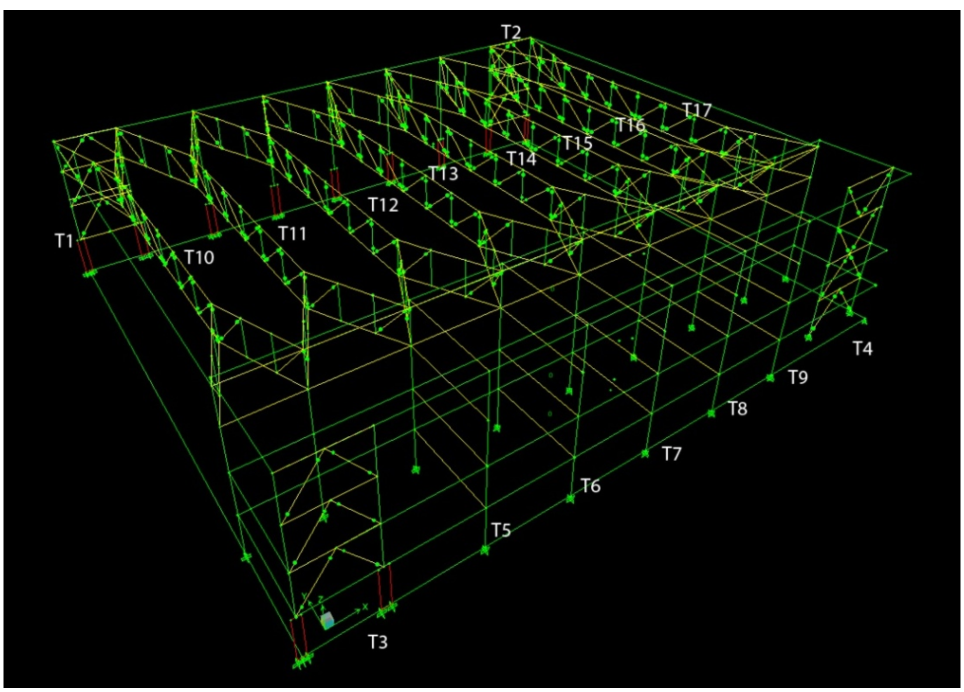


Figure 2 – ETABS Model

Stiffness

Realizing that the lateral elements are very similar to one another and located approximately evenly spaced from the center of the building, the stiffness of each member should be very close. It should be noted up front that just because the stiffness might be approximately the same for each member, this does not directly correlate to the amount of load each member will take. This discrepancy is due to the building having 2 above grade floors that only span the bottom 53' of the 200' structure, causing an uneven load distribution. This will be discussed further in the load distribution section below.

An approximate method was developed to show the relative stiffness's. Realizing that stiffness varies inversely with deflection, a good approximation for the stiffness would be the inverse of the deflection of each member. To find the relative stiffness for each lateral element (braced frames, moment frames, and roof trusses), I placed lateral loads on the building diaphragms in each direction and measured the relative deflection of for each member. This procedure was done on ETABS. For the Y-direction (N-S), I used the service wind load already generated for the load combinations. For the X-direction (E-W), since no wind load was generated on ETABS, I created a 1 kip load placed on the center of the diaphragm. The deflections were measured for each level of each lateral element, then inverted and converted to a percentage of the total. This percentage closely approximates the relative stiffness for each lateral element at each floor. As expected, the relative stiffness's are very close to each other. The results can be seen in the table below, which is separated by each direction and each floor.

Table 8

Relative Stiffness in E-W and N-S directions									
X-Direction (E-W)					Y-Direction (N-S)				
Level 1	Truss	Def(in)	1/Def	Rel. Stiffness (%)	Level 1	Truss	Def(in)	1/Def	Rel. Stiffness (%)
	1	0.087	11.49	25.09		5	0.278	3.60	20.34
	2	0.081	12.35	26.94		6	0.281	3.56	20.13
	3	0.09	11.11	24.25		7	0.283	3.53	19.98
	4	0.092	10.87	23.72		8	0.285	3.51	19.84
	SUM	0.35	45.82	100.00		9	0.287	3.48	19.70
						SUM	1.414	17.68	100.00
Level 2	1	0.155	6.45	28.22	Level 2	5	0.853	1.17	20.29
	2	0.17	5.88	25.73		6	0.859	1.16	20.15
	3	0.189	5.29	23.14		7	0.866	1.15	19.99
	4	0.191	5.24	22.90		8	0.872	1.15	19.85
	SUM	0.705	22.86	100.00		9	0.878	1.14	19.72
						SUM	4.328	5.78	100.00
Roof	1	0.28	3.57	26.33	Roof	5	1.237	0.81	7.74
	2	0.28	3.57	26.33		6	1.241	0.81	7.71
	3	0.312	3.21	23.63		7	1.246	0.80	7.68
	4	0.311	3.22	23.71		8	1.251	0.80	7.65
	SUM	1.183	13.56	100.00		9	1.255	0.80	7.63
						10	1.228	0.81	7.79
						11	1.232	0.81	7.77
						12	1.236	0.81	7.74
						13	1.241	0.81	7.71
						14	1.245	0.80	7.69
						15	1.25	0.80	7.66
						16	1.254	0.80	7.63
						17	1.259	0.79	7.60
						SUM	16.175	10.45	100.00

Load Distribution

As mentioned above, the loads will not be evenly distributed by stiffness for most cases. The reason for this is because the arena contains both the gym area which is open space from the ground level to the roof and the locker/concession/office area which contains 2 floors above the ground level. Figure 1 on page 3 shows the location of these areas.

The reason this is important is because when lateral loads are applied, they are applied to the center of mass, which does not lie in center of the building but somewhere inside the floor diaphragm for the locker/concession/office area that is significantly south of the building center. The center of mass has been calculated for floor 2 taking into account the exterior wall weight and an approximations for the weight and framing for the interior catwalks above the gym area as(106.75, 12.405) in X-Y coordinates. The center of the building is (106.75, 96.67). This shows that the center of mass is 84.27' more south than the center of the building (see Appendix F for calculations for the center of mass). Braced frames T1 and T2 are not directly attached to this diaphragm and only receive the lateral load transferred by the flexible diaphragm on the roof level and through torsion effects. For this reason, the shears in these braced frames are significantly lower than their counterparts on the south side (T3 and T4). The torsion effects on these frames will be examined under the torsion section.

Shown below in Table 9 is the shear measured for the different lateral elements per floor. They were measured with the controlling load cases for each direction and the load cases are shown in the tables. Again, they have been separated into X (E-W) and Y (N-S) directions. The relative load was then calculated and given as a percentage for the total load received by the lateral elements that played a role in that particular direction. This is a good approximation for the relative load each element receives. The reason that the 5 moment frames (T5-T9) take a percentage of shear on the ground floor is because these frames are directly linked to the roof trusses that have triangular end sections that help relieve some lateral load in the X direction. See Appendix E for a floorplan of the roof for a visual clarification of these triangular pieces. Additionally, Appendix E shows the shears taken per element per floor. The total base shear for the E-W direction should be 200 kips (200 *1.0 factor), and the 9 elements listed take 189.7 kips, which leaves roughly 5% left for the other columns that are not part of the lateral system in the E-W direction. The total base shear in the N-S direction should be 465.4 kips (292 *1.6 factor), and the 5 elements listed take 424.7 kips, which leaves roughly 10% for the columns not part of the N-S lateral system.

Table 9

LOAD PATH/LOAD DISTRIBUTION							
X-Direction (E-W) (Controlling LC = Combination 5)				Y-Direction (N-S) (Controlling LC = Combination 4)			
Level 1	Truss	Shear(kips)	Relative Load (%)	Level 1	Truss	Shear(kips)	Relative Load (%)
	1	6.1	3.22		5	83.8	19.73
	2	6	3.16		6	84.4	19.87
	3	74.1	39.06		7	84.9	19.99
	4	45.5	23.99		8	85.5	20.13
	5	11.6	6.11		9	86.1	20.27
	6	11.6	6.11		SUM	424.7	100.00
	7	11.6	6.11				
	8	11.6	6.11				
	9	11.6	6.11				
	SUM	189.7	100.00				
Level 2	1	6.1	3.90	Level 2	5	27.7	19.52
	2	6	3.83		6	28	19.73
	3	80	51.12		7	28.4	20.01
	4	64.4	41.15		8	28.7	20.23
	SUM	156.5	100.00		9	29.1	20.51
					SUM	141.9	100.00
Roof	1	5.4	5.13	Roof	5	6.8	3.89
	2	5.3	5.03		6	6.9	3.94
	3	60	56.98		7	6.9	3.94
	4	34.6	32.86		8	6.9	3.94
	SUM	105.3	100.00		9	6.8	3.89
					10	15.2	8.69
					11	16.3	9.31
					12	22	12.57
					13	21.5	12.29
					14	21.2	12.11
					15	21.2	12.11
					16	15.5	8.86
					17	7.8	4.46
					SUM	175	100.00

Drift:

Drift is an important feature in structure, but more important and harder to deal with in high-rises. Since the Coppin State University Physical Education Complex is only 60' maximum, drift will likely not control. Nevertheless, drift was looked at through the aid of the ETABS model. No special conditions exist, so the lateral deflection was limited to L/400 for wind loads. This would be Δ= 60*12/400 = 1.8" for the Coppin State University Physical Education Complex. Following requirements of ACSE 7-05 section 12.12 the allowable drift per floor for building occupancy III is 0.02h in seismic situations which is Δ=14.4" at the top floor. The reason for the difference in deflections between wind and seismic is that when designing for seismic controlled cases, the building is designed to yield, whereas in wind design the building is designed to remain in place.

Deflections were measured in ETABS at the center of mass for each floor as required by ASCE 7-05 section 12.8.6. Since this is a serviceability issue, only lateral service loads were used. The following results were found and the controlling cases are summarized in the following table. The seismic deflections (Δ_{xe} were multiplied by C_d/I to find Δ_x following Equation 12.8-15 in ASCE 7-05 then compared with allowable deflections. The torsional amplification factor, Ax, is not required for this building because it is in Seismic Design Category B. As is evident by the deflections seen in Table 10, all areas meet code requirement. By inspection they also meet story drift requirements.

Table 10

Deflections based on Service Loads							
Story	Load Combo(X)	Δxe (in.)	Δx	Story Drift(X)	Load Combo(Y)	Δy(in.)	Story Drift(Y)
Roof	Seismic X	0.3916	**0.93984**	0.7188	Wind Y(North)	**1.2469**	0.6626
2	Seismic X	0.1372	**0.32928**	0.22104	Wind Y(North)	**0.8668**	0.5843
1	Seismic X	0.0451	**0.10824**	0.10824	Wind Y(North)	**0.2825**	0.2825

Cd=3 for "Steel systems not specicifically detailed for seismic resistance"
I=1.25 for Bldg. Cat. III

Torsion:

Torsion was played a large role in the Coppin State University Physical Education Complex. Most of the building weight and diaphragms are located on the southern 53' (see Figure 1 on Page 3). For this reason torsion played a large role when evaluating E-W lateral forces. Torsion analysis was performed for both building directions, however.

First the story shears and overturning moments were found using the ETABS model. The 5% accidental torsional moment (Mtax) was found using the provisions of ACSE 7-05 section 12.8.4.1 and 12.8.4.2 and added to the overturning moment (Mt). There was no need to use the torsional amplification factor, Ax, because the building is assigned Seismic Design Category B. The controlling load cases, story shear, Mt, Mtx, and Mt, total are summarized below for both directions in Table 11. The results clearly show there is a higher torsional moment in the E-W direction (X). This was expected with the center of mass being located far from the center of the building in the N-S direction (Y).

Table 11

Story Shears and Overturning Moments						
Story	X-Combinaton	Vx	Width(x)	Mtx	Mtax	Mt,tot x
4/Roof	5/1.0 Factor	**142.25**	226	66277	1607.425	**67884.4**
2	5/1.0 Factor	**40.07**	226	117847	452.791	**118299.8**
1	5/1.0 Factor	**17.68**	226	167613	199.784	**167812.8**
Story	Y-Combination	Vy	Width(y)	Mty	Mtay	Mt,tot y
4/Roof	4/1.6 Factor	**147.8**	200	45453	1478	**46931**
2	4/1.6 Factor	**198.4**	200	47432	1984	**49416**
1	4/1.6 Factor	**119.2**	200	28885	1192	**30077**

Torsional shear was looked at next. The equation $V_i = V_{tot} e d_i R_i / J$ was used where V_i represents the torsional shear, V_{tot} the story shear, *e* the distance from the center of mass to the center of rigidity, d_i the distance from element I to the center of rigidity, and J the torsional moment of inertia. Calculations for the torsional shear is presented for level 2 in both E-W (X) and N-S (Y) directions and are presented below in Table 12. The direct shear was found using the relative load percentages that were obtained in earlier shear calculations. The torsional shear, however, was calculated with relative stiffness percentages that were obtained from deflection calculations earlier. The total shear is presented for worst case loading conditions. For additional information on calculations for the V_i equation see Appendix F.

Table 12

Level 2 Total Shear Calculations							
Y	Frame	Rel. Stiffness(%)	Rel. Load (%)	Dist from C.R	Direct Shear	Torsional Shear	Total Shear
	5	20.29	19.52	57.9	38.73	15.39	**54.12**
	6	20.15	19.73	27.89	39.14	7.36	**46.51**
	7	19.99	20.01	2.1	39.70	0.55	**40.25**
	8	19.85	20.23	32.1	40.14	8.35	**48.48**
	9	19.72	20.51	62.1	40.69	16.04	**56.74**
				J= 1802.843	198.40		**246.10**
X	Frame	Rel. Stiffness(%)	Rel. Load (%)	Dist from C.R	Direct Shear	Torsional Shear	Total Shear
	1	28.22	3.9	195.36	1.56	1.28	**2.84**
	2	25.73	3.83	195.36	1.53	1.16	**2.70**
	3	23.14	51.12	4.31	20.48	0.02	**20.46**
	4	22.9	41.15	1.39	16.49	0.01	**16.48**
					40.07		**42.48**
				J= 20595.04			

Overturning Moments:

Overturning moments were calculated by hand for the controlling load cases for wind and seismic lateral loads. These were calculated as service loads and are as follows:

Wind (controlling load in the N-S direction): 10381.5'k
Seismic (controlling load in E-W direction): 10002.2'k

The calculations can be found in the load sections on pages 10 and 11. The overturning moments are very close for service loads, however when multiplied by respective controlling load factors, 1.0 for seismic and 1.6 for wind, the wind overturning moment becomes 16610.4'k while the seismic overturning moment stays at 10002.2'k. This shows that the north-south direction is more susceptible to overturning than the east-west direction. The east-west direction will have additional resistance to the overturning moment because the arena is completely surrounded on both the east and west sides by other buildings. These buildings, which happen to be *facilities management* on the west and *physical education north* on the east, will almost lock the arena into place when lateral loads arrive. However, these buildings will also suffer from the same seismic loads and could possibly transfer the load past the expansion joint to the arena if the force becomes large enough.

Spot Checks:

Spot Checks can be found in Appendix G. The lateral elements analyzed include a diagonal brace and the column supporting the brace for braced frame T2 and a spot check of the roof truss chords. Also included is the hand calculated truss chords from Technical Report 1. Along with the calculations, ETABS prints are provided to show the member being analyzed.

To analyze the lateral elements, the ETABS model was used again. Line loads were input for dead loads, live loads, and snow loads (where relevant) onto the horizontal elements for floors 1, 2 and the roof to find forces in the chords. These loads were based on approximate tributary widths. Vertical point loads were placed on the columns at floors 1, 2 and the roof based on approximate tributary areas to find the column forces.

All of the spot checks showed the designed members to be adequate. Some of the spot checks showed that smaller members could be used, however, the difference was not excessive. This difference most likely stems from the fact that only a lateral model was developed rather than a full gravity and lateral model. The designer could have also used additional safety measures when specifying members, or loading conditions could have been modified since the initial design.

Conclusions:

Through the analysis of the current lateral system for the arena of the Coppin University Physical Education Complex, several conclusions can be drawn.

Wind loads typically control designs in the N-S direction, while seismic loads typically control designs in the E-W direction. The seismic load controls the E-W direction designs because the arena is enclosed by *facilities management* on the west and *physical education north* on the east, which will block the wind. The controlling load combinations are primarily 1.2D + 1.6W + L + $0.5L_w$ in the N-S direction and 1.2D + 1.0Ex + L +0.2S in the E-W direction. Base shear and overturning moments were controlled with these load combinations, however some spot checks used a different load combination.

Lateral load paths were shown not to be completely evenly distributed for either direction. The load path for the N-S direction distributes direct shear very evenly amongst its members, however, the torsional effects create more shear on the outer members. This effect is present on all levels. The lateral load path is not evenly distributed among the four braced frames taking the E-W lateral load either. As mentioned before, the reason for this is because the arena contains both the gym area (which is open space from the ground level to the roof) and the locker/concession/office area (which contains 2 floors above the ground level). The lateral load will only directly affect the members attached to the diaphragm, so members T1 and T2 only receive lateral load from carried over load from the roof diaphragm and torsional effects. The braced frames are most likely there for redundancy and stability of the W14x257 exterior columns.

The building is more susceptible to drift issues in the N-S direction. This is due to the wind loads that act in that direction, which are typically higher than the seismic loads acting in the E-W direction. However drift, as expected, met code and did not control designs for this structure.

Spot checks showed all checked members to be adequately designed. Some of the spot checks showed that smaller members could be used; however, the difference between member sizes was not excessive. This difference most likely stems from the fact that only a lateral model was developed rather than a full gravity and lateral model.

Appendix A

General Floorplan:

*Expansion joints shown in red

Appendix B

Wind Load Information:

All Information is based obtained using the basis of ASCE7-05

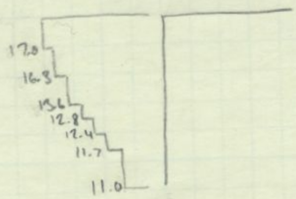
Building Category		II
3 second gust speed V		90 mph
Importance factor Iw		1.15
Building mean roof height H	*	60 ft.
Roof slope Theta		0 to 10 degrees
Exposure Category		B
Topography Factor, Kzt		1
Velocity pressure exposure coefficient at mean roof height, Kh		0.85
Velocity pressure at mean roof height, qh (psf)		17.31
Gust Effect Factor, G		0.85
External pressure coefficient Windward wall (Cpww)		0.8
External pressure coefficient Leeward wall (Cplw)		-0.3
External pressure coefficient Sidewall (Cpsw)		-0.7
Building length parallel to wind L	*	220
Building length normal to wind B	*	205
Roof Area (B*L)	*	45100sqft.
Roof Uplift Reduction Factor	*	0.8
H/L =	*	0.27
Internal Pressure Coefficients for Buildings, +/- GCpi		0.18

*Varies Between Buildings, Shown for N-S Wind on Facilities Management

Wind Pressures Shown In Report.
-Tabulated using an excel spreadsheet
-Leeward pressures calculated using full buildings lengths
-Total pressures subtract out internal pressure ($2*q_h*GC_{pi}$)

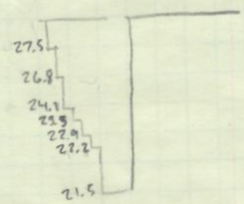
Wind Loads Arena

Windward (psf)



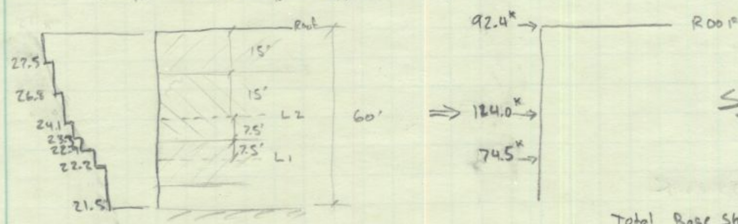
Leeward (psf)

Total (psf)



Wind-Loads:

Arena (re-calculated since tech 1)



South - North
℄ span - 226

SERVICE LOADS

Total Base Shear = 291.6^k

Total Overturning Moment = 10381.5'^k

(if only considering arena roof)

Snow Load: Ground SL = 25 psf

Snow Exposure Factor = B

Snow Importance Factor = 1.10

Thermal Factor = 1.0

Flat Roof Snow Load = 22 psf

Appendix C

Seismic Load Information:

Seismic Analysis: Equivilent Lateral Force Method

Seismic Use Group: II   Occupancy Category III

Seismic Importance Factor: 1.25

Mapped Spectral Response Accelerations

$S_s = 0.191g$ $S_{MS} = 1.6(0.191g) = 0.3056g$

$S_1 = 0.064g$ $S_{M1} = 2.4(0.064g) = 0.1536g$

Site Class D

Design Spectral Response Coefficients

$S_{DS} = 0.204g$

$S_{D1} = 0.102g$

Seismic Design Category B

Basic Seismic Force Resisting System - Structural Steel Not Specifically Detailed For Seismic Resistance

Seismic Response Coefficient

$C_s = 0.059$

Response Modification Factor

$R = 3.0$

$C_u = 1.7$ $T_L = 8$

$T_a = C_t h_n^x$

assume $h_n = 30'$ (conservative)

$C_t = 0.028$ $x = 0.8$

$T_a = 0.028(30)^{.8} = 0.425$

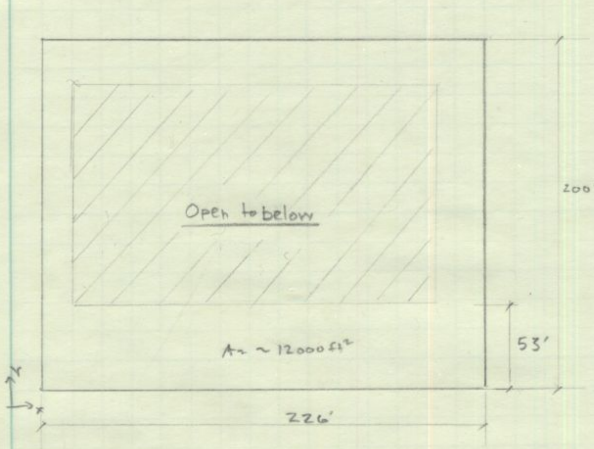
$T = C_u T_a = 1.7(0.425) = 0.723$

$$C_s = \text{MIN} \begin{cases} S_{DS}/(R \cdot I) = 0.085 \\ S_{D1}/[T \cdot R/I] = \underline{0.059} \text{ controls} \\ S_{D1} \cdot T_L/[T^2 \cdot R/I] = 0.65 \end{cases}$$

$C_s = 0.059$ (compare w/ 0.064 specified in dwgs)

Weight of Arena (by floors) - recalculated since tech 1

Height ~ 60'



A_{tot} ~ 46000 ft²

Truss wt = 410^k (from tech 1)

Floors: Flr 2 (@ 15') - std floor

$12000(60+10) + 15(15)(226) = \underline{891^k}$

ext. partition

Flr 3 (@ 30') - std floor $= 12000(70) + 15(30)(226) = \underline{942^k}$

Roof: $46000(25) + 410^k = \underline{1560^k}$

Roof over arena ~ 1260^k, Roof over other ~ 300^k

Total ~ 3393^k

Total Base Shear $= 3393 \times 0.59 = \underline{200^k}$

Mass / Area:

Flr 2 $891^k/(53 \times 226) = \underline{74.4\ lb/ft^2} / 32.2 / 12^3 = \underline{1.337E-6}$ kip-in

Flr 3 $942^k/(53 \times 226) = \underline{78.6\ lb/ft^2} / 32.2 / 12^3 = \underline{1.413E-6}$ kip-in

Roof over arena $1560/(147 \times 226) = \underline{46.9\ lb/ft^2} / 32.2 / 12^3 = \underline{8.439E-7}$ kip-in

Roof other $300/(53 \times 226) = \underline{25.0\ lb/ft^2} / 32.2 / 12^3 = \underline{4.501E-7}$ kip-in

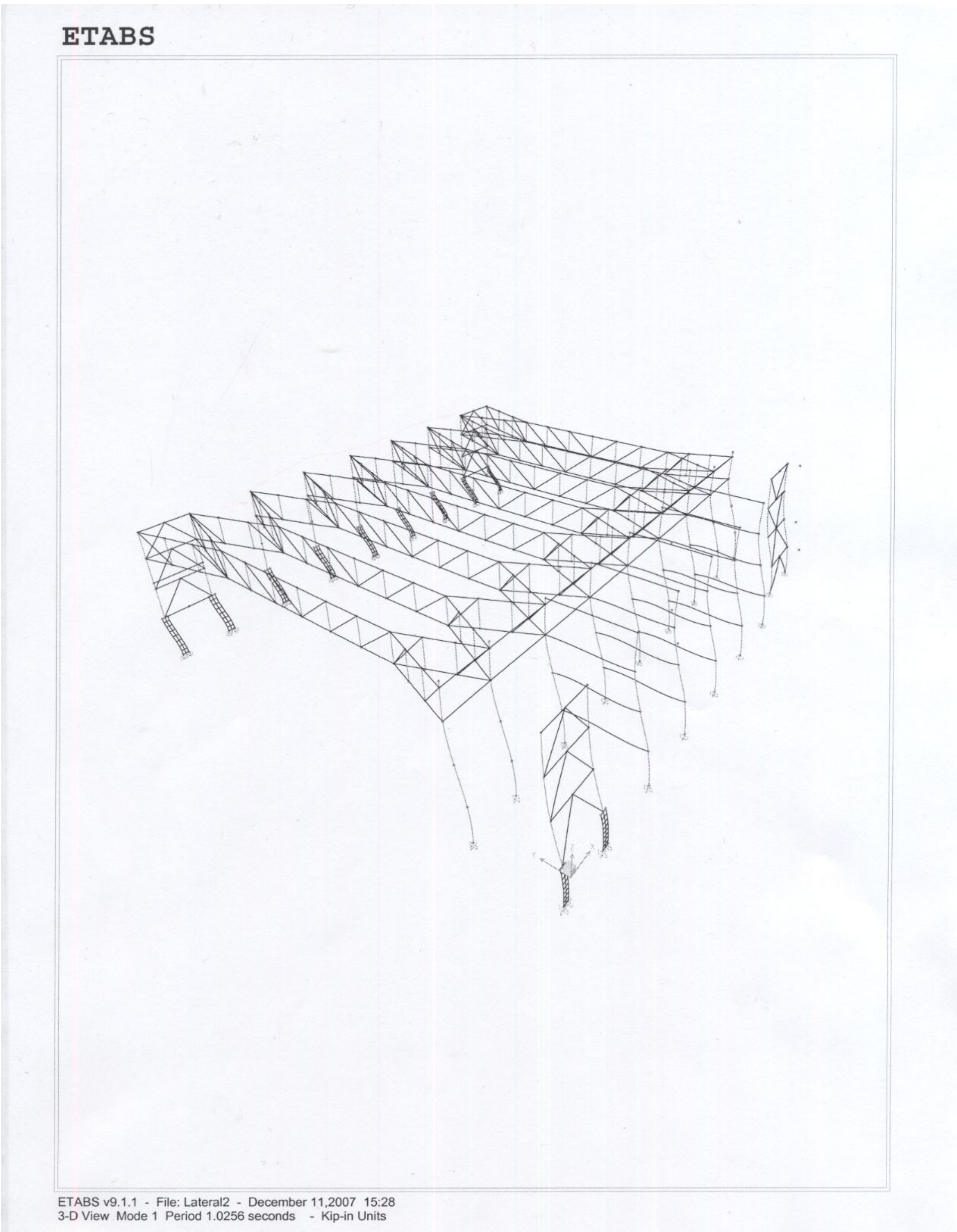
Seismic Base Shear and Moment Calculations									
Building	Level	Height(ft.)	W(kip)		h^kWx	Cvx	V(kip)	Fx	∑Fxh
	2	15	2797		55003.82	0.250813		84.7748	1271.62
Facilities	3	30	1086		45778.67	0.208747		70.5565	2116.7
Management	4	45	1086		71509.48	0.326078		110.214	4959.64
	Roof N	30	455		19179.83	0.087459		29.561	886.829
	Roof S	60	308		27830.25	0.126904		42.8935	2573.61
				SUM	**219302**	**1**	**338**	**338**	**11808.4**
Arena	2	15	891		17521.78	0.088409		**17.6819**	265
	3	30	942		39708.57	0.200357		**40.0714**	1202.144
	Roof N	60	1260		113851.02	0.574457		**114.8915**	6893.49
	Roof S	60	300		27107.39	0.136775		**27.35512**	1641.307
				SUM	**164398.6**	**1**	**200**	**200**	**10002.17**
Physical Education North	2	15	568		11169.89	0.142849		18.1418	272.126
	Roof	30	1590		67024.02	0.857151		108.858	3265.75
				SUM	**78193.91**	**1**	**127**	**127**	**3537.87**
Physical Education South	2	15	1164		22890.4	0.513244		50.8112	762.168
	Roof	30	515		21709.04	0.486756		48.1888	1445.66
				SUM	**44599.44**	**1**	**99**	**99**	**2207.83**

T=0.723 sec == k=1.1

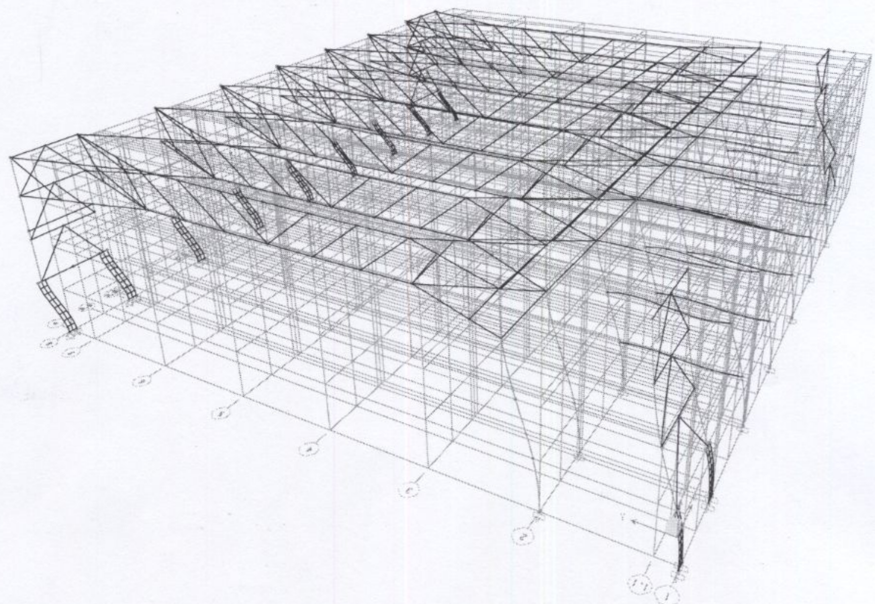
Appendix D

Periods of Vibration:

Shown are the 3 periods of vibration. The first mode shows a period of 1.0256 and is shown below without gridlines to see directional bending better. The other modes show gridlines to show relative movement better. The first mode is shown both ways.

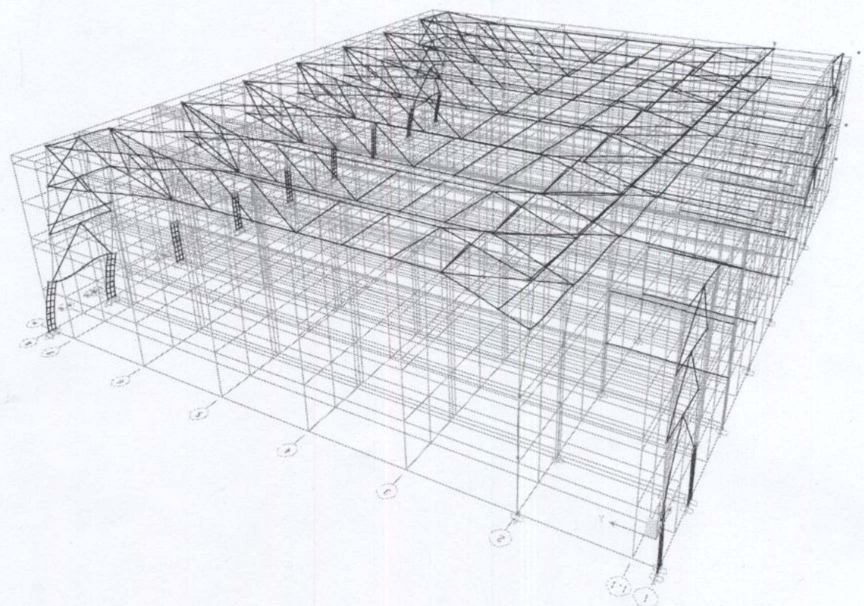


ETABS



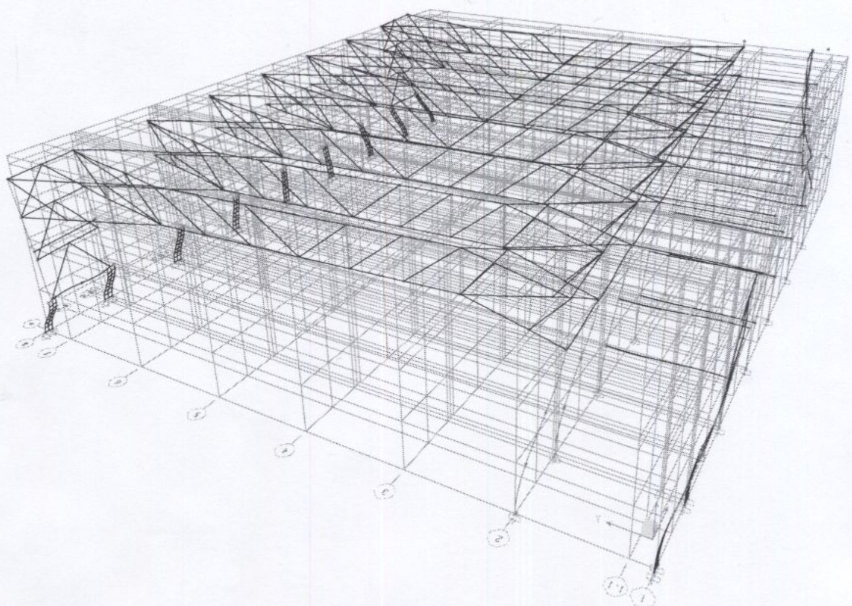
ETABS v9.1.1 - File: Lateral2 - December 11,2007 15:25
3-D View Mode 1 Period 1.0256 seconds - Kip-in Units

ETABS



ETABS v9.1.1 - File: Lateral2 - December 11,2007 15:25
3-D View Mode 2 Period 0.5889 seconds - Kip-in Units

ETABS

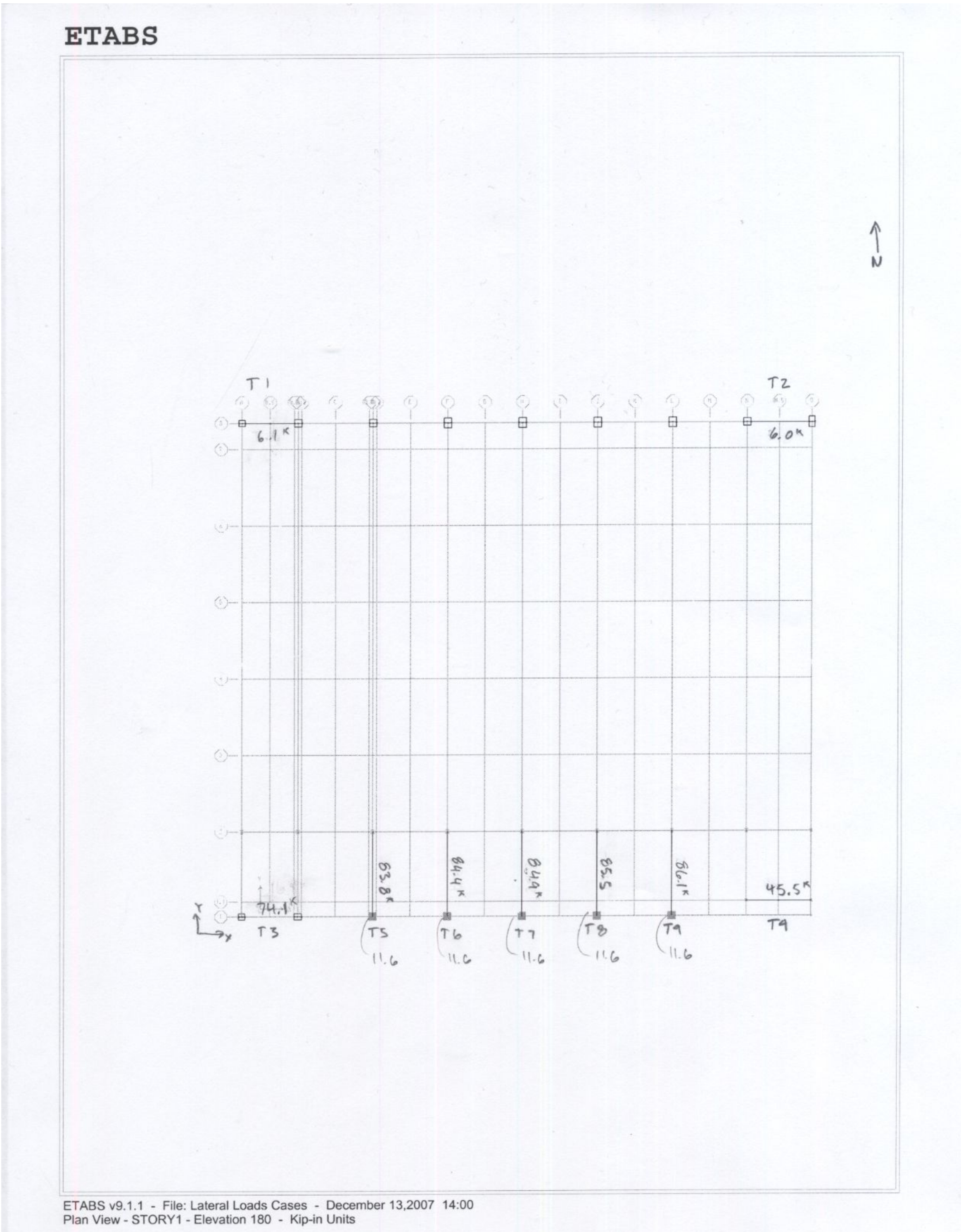


ETABS v9.1.1 - File: Lateral2 - December 11,2007 15:25
3-D View Mode 3 Period 0.5064 seconds - Kip-in Units

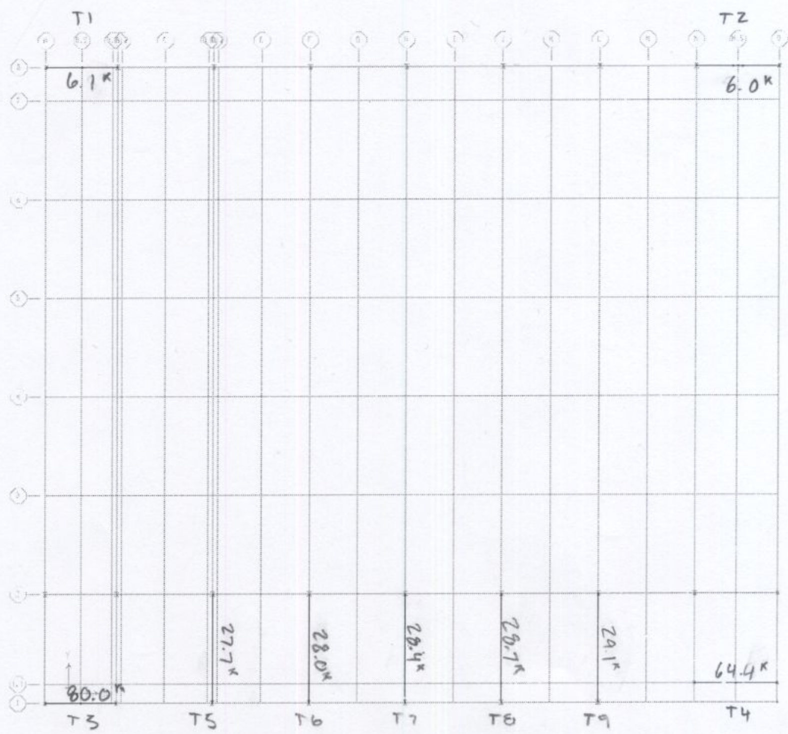
Appendix E

Shear Diagrams:

*Shown per floor with member labels

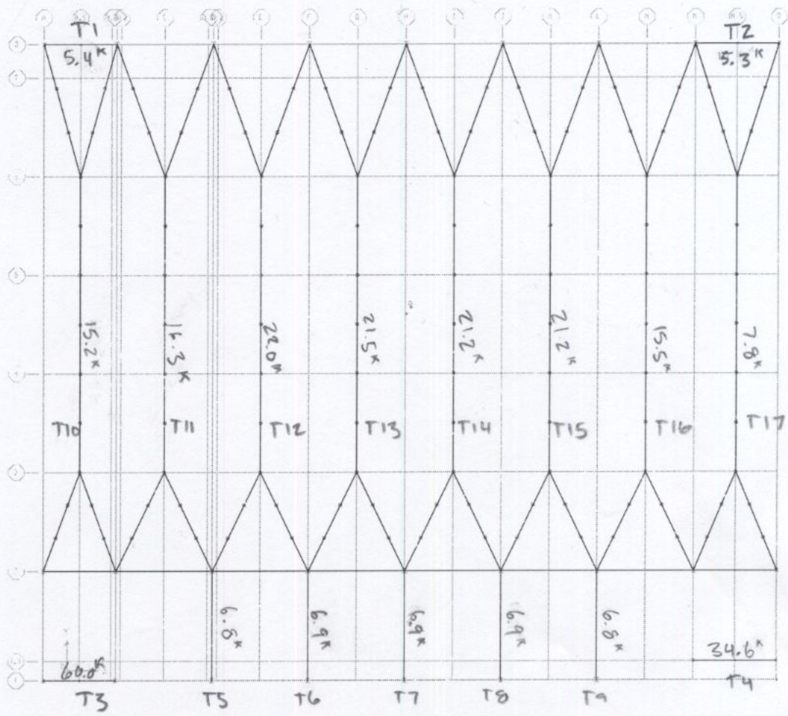


ETABS



ETABS v9.1.1 - File: Lateral Loads Cases - December 13,2007 14:02
Plan View - STORY2 - Elevation 360 - Kip-in Units

ETABS



ETABS v9.1.1 - File: Lateral Loads Cases - December 13,2007 14:04
Plan View - STORY4 - Elevation 720 - Kip-in Units

Appendix F

Center of Mass / Vi Information:

Floor 2 Centerof Mass/ Center of Rigidity/ e Calcs											
Coordinates	C.R	C.M.	T1	T2	T3	T4	T5	T6	T7	T8	T9
X	102.899	106.75	3.75	208	3.75	208	45	75	105	135	165
Y	1.14	12.405	196.5	196.5	-3.17	2.83	17.09	17.09	17.09	17.09	17.09
dist/e=		11.90506	195.36	195.36	-4.31	1.69	-57.899	-27.899	2.101	32.101	62.101

Level 2 J Calc				
Y	Frame	Rel. Stiffness(%)	Dist from C.R	j calc
	5	20.29	57.9	680.204
	6	20.15	27.89	156.7372
	7	19.99	2.1	0.881559
	8	19.85	32.1	204.5364
	9	19.72	62.1	760.4841
			J=	**1802.843**
X	Frame	Rel. Stiffness(%)	Dist from C.R	j calc
	1	28.22	195.36	10770.31
	2	25.73	195.36	9819.99
	3	23.14	4.31	4.30
	4	22.9	1.39	0.44
			J=	**20595.04**

Appendix G

<u>Spot Checks:</u>

Spot Checks

Load Imputs for Spot Checks

T4: trib. length = 20'   DL = 60 psf   LL = 100 psf   Standard
DL = 25 psf   LL = 30 psf   Roof

lineal loads: DL = 60 psf × 20' = 1.2 KLF = .1 K/in
LL = 100 psf × 20' = 2 KLF = .166 K/in   Standard
DL = 25 psf × 20' = .5 KLF = 41.67 #/in
LL = 30 psf × 20' = .6 KLF = 50 #/in   Roof

Apply Loads on ETABS For Spot Checks

Column Loads: trib area = 30' × 20' = 600 ft²

Axial Loads   DL = 60 × 600 = 36 K
LL = 100 × 600 = 60 K   Standard

DL = 25 × 600 = 15 K
LL = 30 × 600 = 18 K   Roof

Spot Checks T4: – controlling load case is COMB5 – 1.2D + 1.0Ev + L + 0.2S for braces
* Bracing Member highlighted – see axial force printout for COMB5
Axial Load = 100.04 K
Moment = 14.39 K-in = 1.2 'K
Shear = 0.24 K

Axial Force is the obvious controlling state for this member

Using AISC Steel Manual 13th Edition

W8×31 @ 21.2' = 131 K > 100 K ok

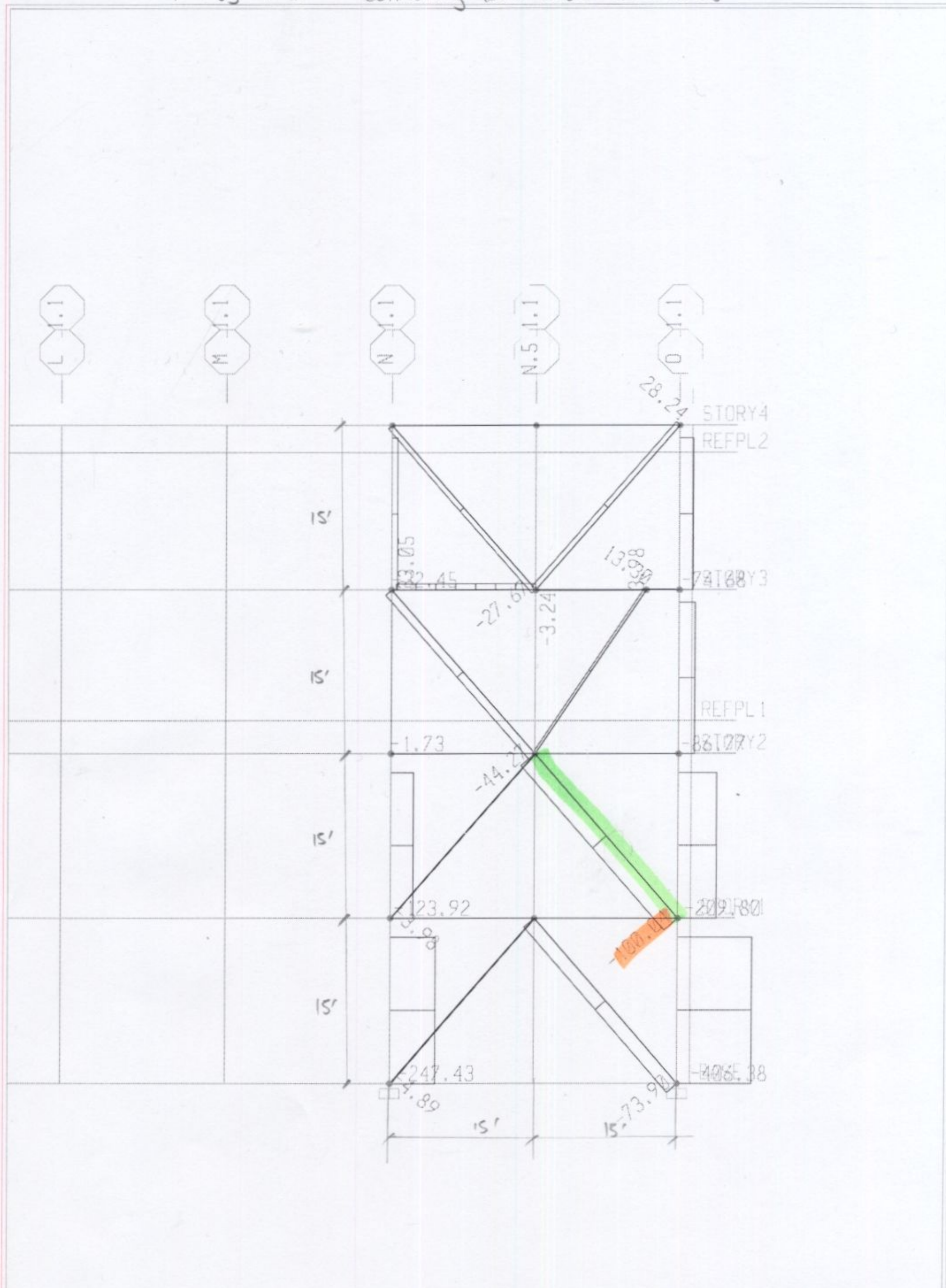
Moment & Shear are ok   Designer chose W8×31 ✓

Spot Check T4
* Column – controlling load case is COMB2 – 1.2D + 1.6L + 0.5Lr for axial load
– see axial force printout for COMB2
Axial Load = 432.9 K @ 15' unbraced length
Moment = 2.56 K-in
Shear = .01 K
> small w/ this load combo.
– Additional P caused by moment is negligible b/c moment is so small

W10×49 ⇒ ϕP_n = 450 K > 432.9 K @ 15'

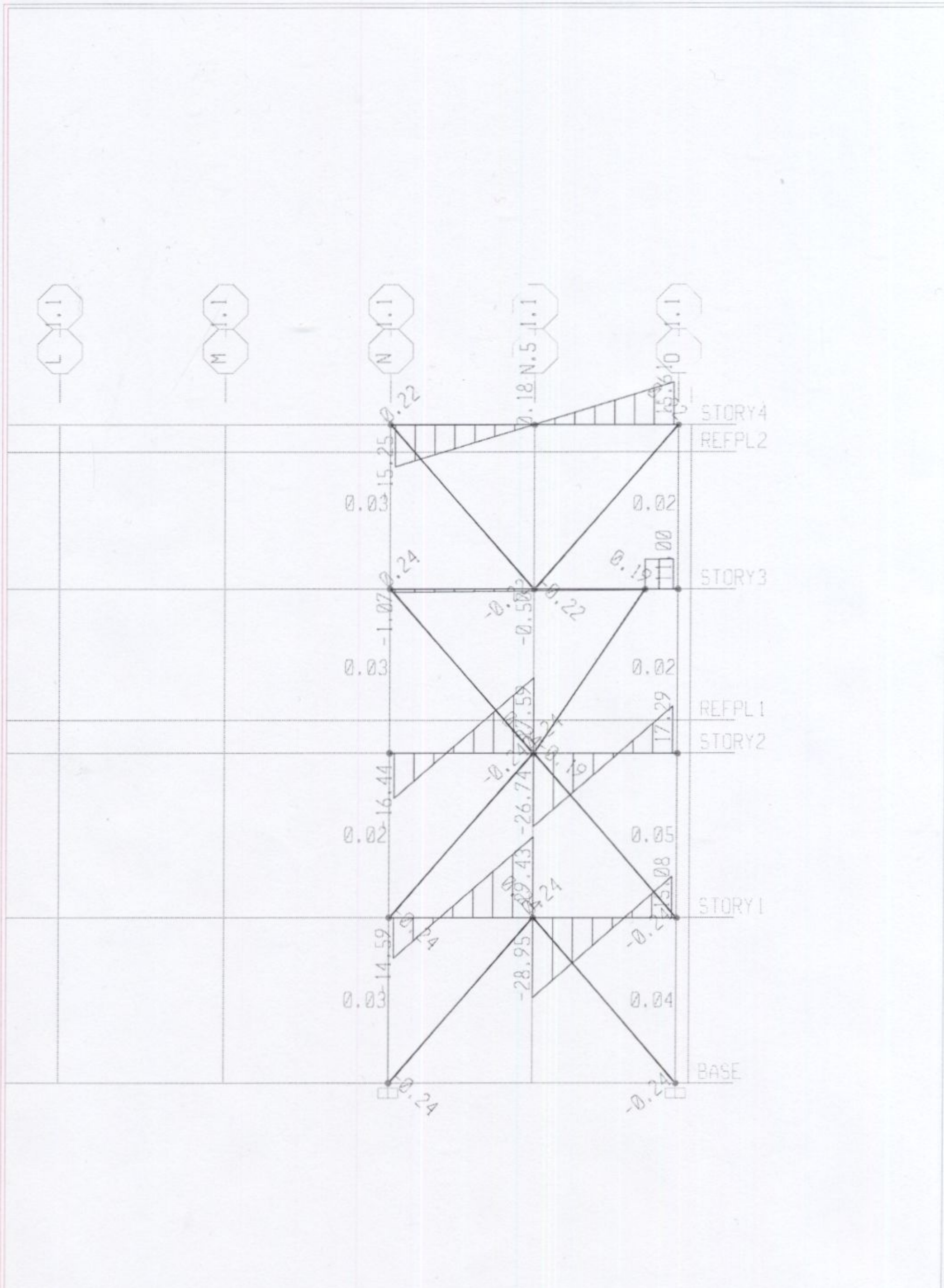
Designer chose W10×60 ϕP_n = 555 K probably for additional safety factor or eccentric loads or special design considerations
Design valid ✓

ETABS Truss T4 - controlling load case (comb5)



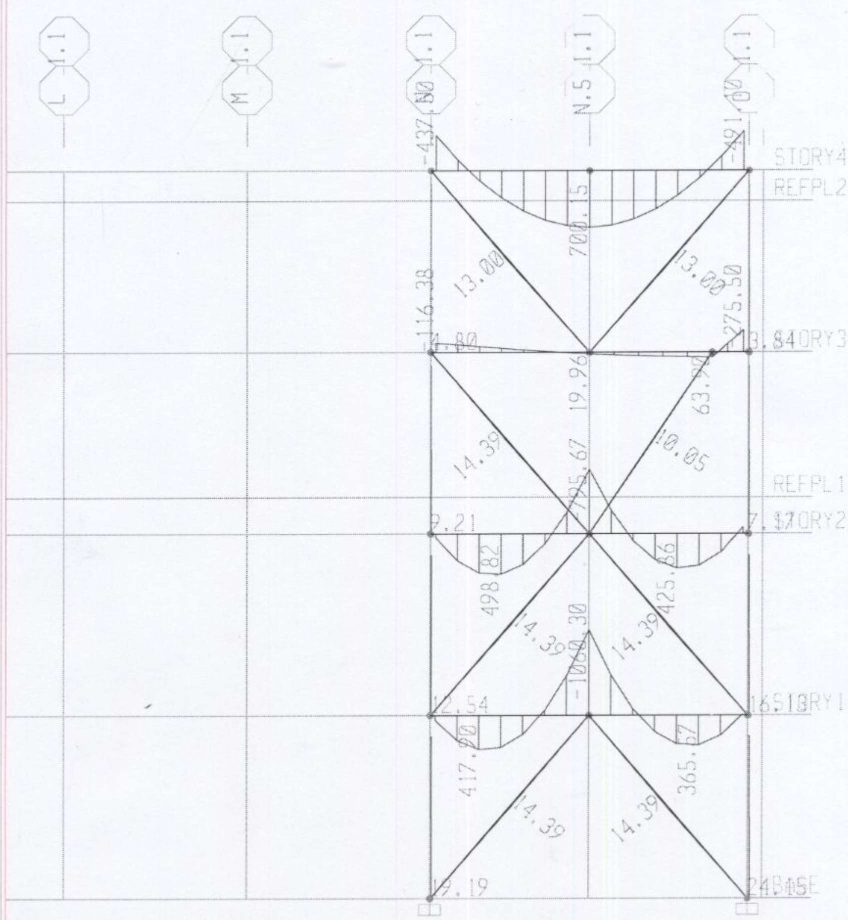
ETABS v9.1.1 - File: Lateral Loads Cases T4 - December 14,2007 17:53
Elevation View - 1.1 Axial Force Diagram (COMB5) - Kip-in Units

ETABS



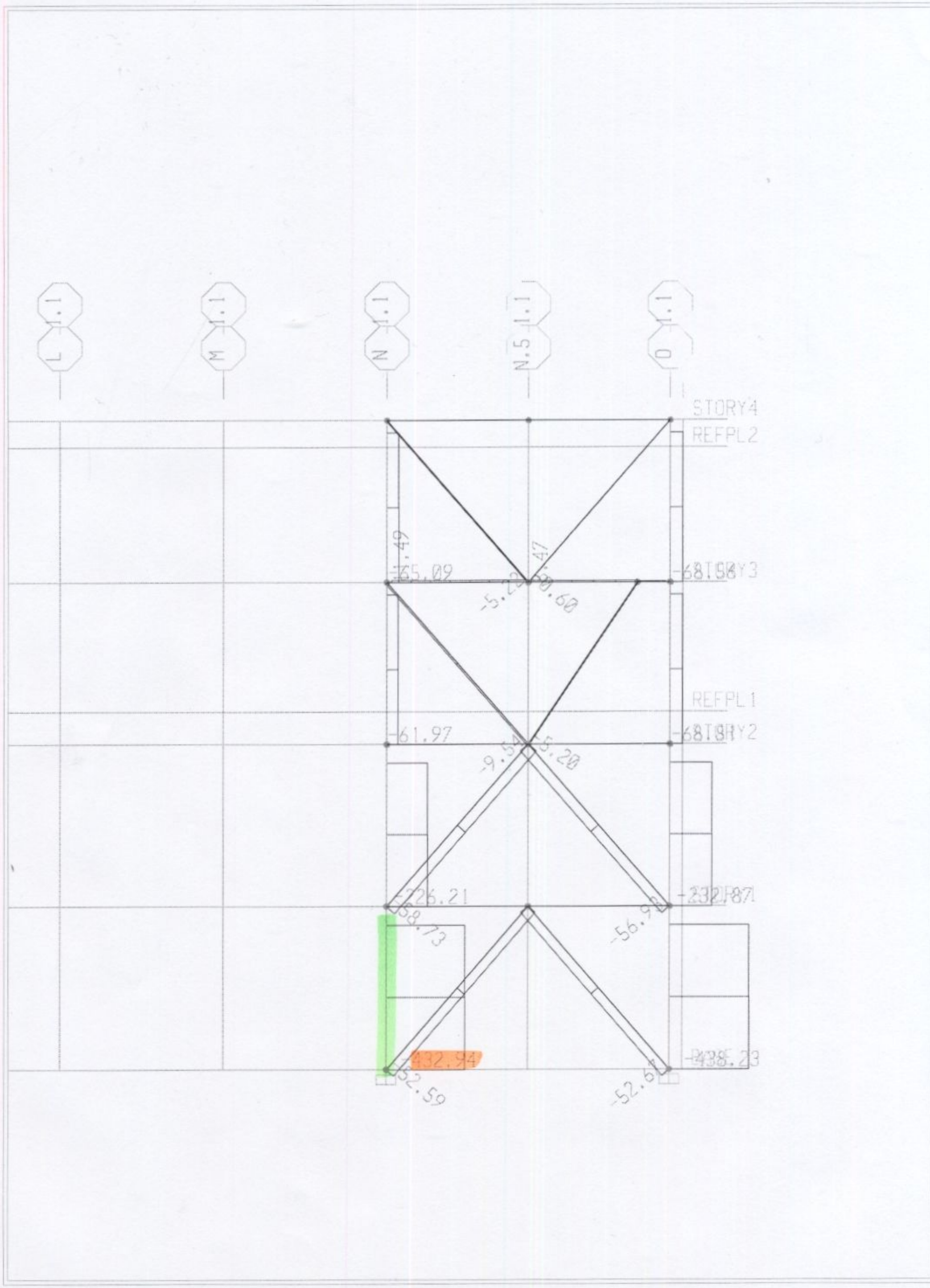
ETABS v9.1.1 - File: Lateral Loads Cases T4 - December 14,2007 17:44
Elevation View - 1.1 Shear Force 2-2 Diagram (COMB5) - Kip-in Units

ETABS



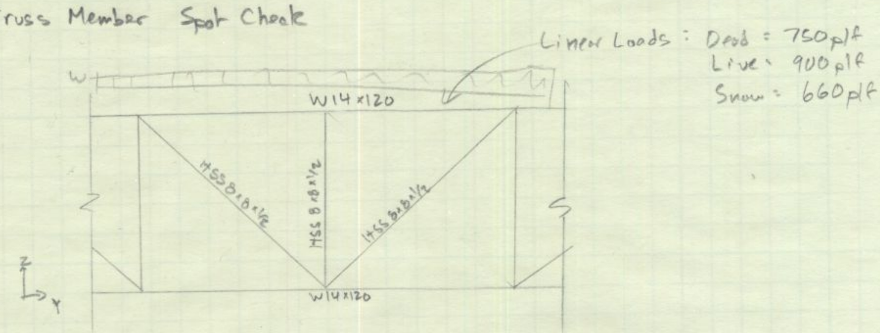
ETABS v9.1.1 - File: Lateral Loads Cases T4 - December 14,2007 17:44
Elevation View - 1.1 Moment 3-3 Diagram (COMB5) - Kip-in Units

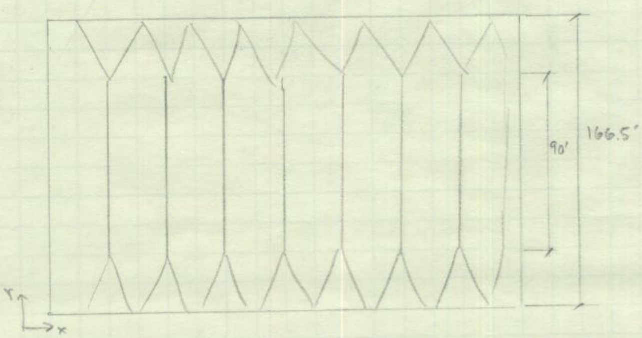
ETABS



ETABS v9.1.1 - File: Lateral Loads Cases T4 - December 14,2007 17:51
Elevation View - 1.1 Axial Force Diagram (COMB2) - Kip-in Units

Truss Member Spot Check





Section cuts on ETABS yield the following: (see printout

Unbraced length = 15'

T_{max} = 621.0^k (comb3 - 1.2D + 1.6L_r + L) on top chord

ΔY_{max} = 1.62" ΔZ_{max} = 4.60"

W14x74 ϕP_n = 667^k @ 15' > 621'

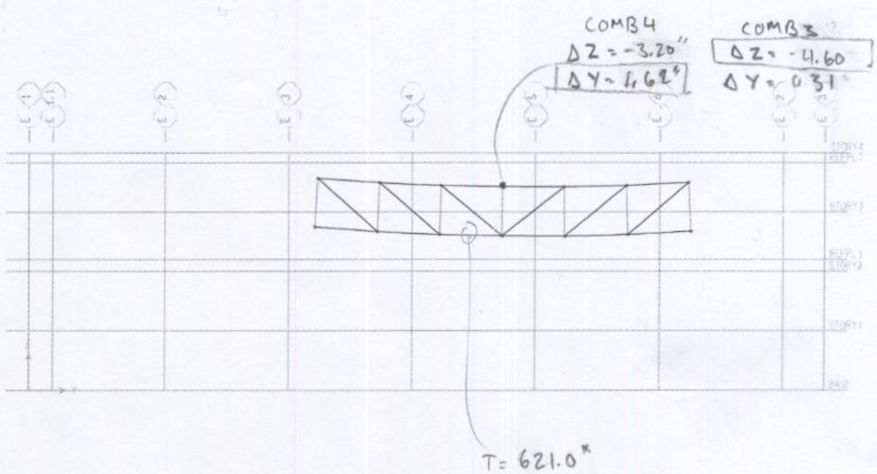
Use W14x74 for strength

Designer chose W14x120 ← most likely to limit Δ.

ΔZ = 4.60" = L/434 with W14x120 + HSS 8x8x1/2 assembly. Deflection is a key issue because a large deflection on the roof would cause ponding when it rains or snows and would create a larger Dead Load. The designer probably limited the deflection to L/400 which has merit with long-span truss designs.

Design valid ✓

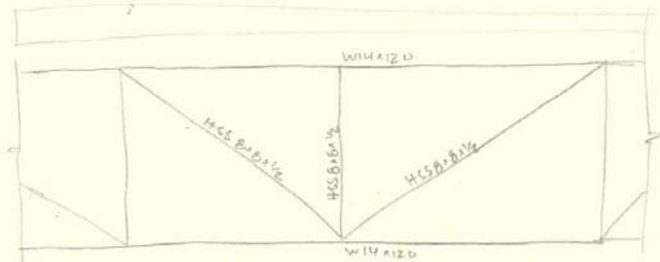
ETABS



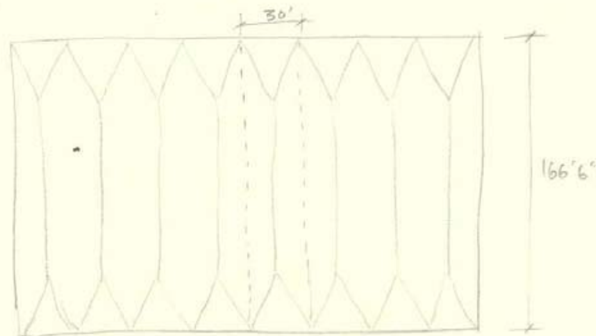
ETABS v9.1.1 - File: Lateral Loads Cases spot checks - December 14,2007 19:35
Elevation View - E Deformed Shape (COMB4) - Kip-in Units

Spot Check from Technical Report 1:

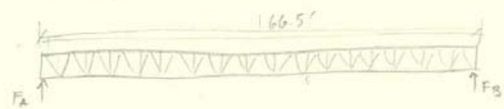
Spot Check of Truss Members



Detail



trib width = 30'



Loads

SL = 25psf

DL = 28.1psf

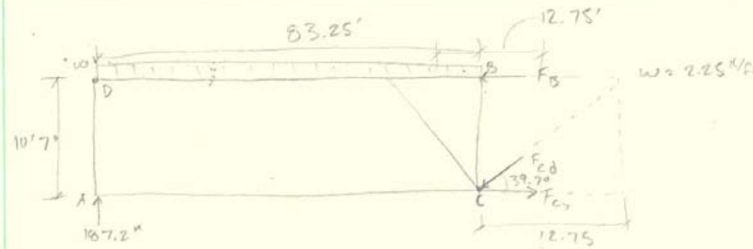
DL: 4½" Deck ⇒ $\frac{4\frac{1}{2}"}{2}\left(\frac{150\text{ pcf}}{12"/\text{ft}}\right) = 28.1$ psf

Load Combination

1.2D + 1.6S

$w = 1.2(25) + 1.6(28.1) = 74.96$ psf $\times$ 30' trib = 2249 plf = 2.25 KLF

$F_A = F_B = \frac{wl}{2} = 187.2^k$

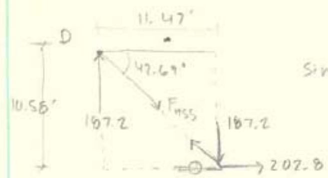


$\Sigma M_c = 0 \quad 187.2(83.25) = F_{B_x}(10.583) + 83.25(2.25)\left(\frac{83.25}{2}\right)$

$F_{B_x} = 736^k$

$187.2 = F_{cd}\cos 39.69° + 83.25(2.25)$

$F_{cd} =$



$\sin 42.69 = \frac{187.2}{F_{HSS}} \quad F_{HSS} = 276^k$

Size HSS members for 276^k(T) & Top members for 736^k (c)

HSS: HSS 6x6x1/2 $\phi_t P_n = 318^k > 276^k$ ok

Designer chose HSS8x8x1/2 probably assuming a greater load or using a greater safety factor, but close to my solution

My design also does not include lateral forces or the bending caused by roof loads

Design valid

Top chords: Using $KL = 12.75$

W14x74 $\phi P_n = 742^k > 736^k$ acceptable

Designer used W14x120 probably using lateral loads and incorporating bending from roof loads

Design valid