Coppin State University
Physical Education Complex

Baltimore, MD

Technical Assignment 1

11/26/07

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Structural Option

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Executive Summary:

The purpose of this assignment is to analyze existing conditions and design procedures relating to the structural design of the Coppin State Physical Education Complex.

The Coppin State Physical Education Complex is a state of the art recreation center surrounding the campus's track and soccer field. The building sprawls in several directions at several heights from the hub of the building, the new 2600 seat arena. The building uses several heights ranging from 30' to 60' and a total area of 135,000 sqft. The main structural system is composed of composite steel with a typical 6.25" lightweight concrete slab. 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. Probably the most dramatic features would be the exposed steel trusses supporting the roof of the arena. The building uses IBC 2003 as the main code with references to ASCE 7-05. The building contains 3 expansion joints (see Appendix A), basically subdividing it into 4 separate buildings: Facilities Management, Arena, Physical Education North, and Physical Education South. The analyses performed use these sub-divided buildings rather than the structure as a whole. During my spot checks, several members were sized differently but were generally close to what was specified. The discrepancies were most likely due to the assumptions I've made and the simplicity of the analysis. This report outlines the procedures and analysis I have used but does not claim any errors of any sort made by the design team.

Structural System:

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 are 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 ASTM GR 50.

Lateral Force Resisting System: The building is essentially 3 buildings side by side. A 3" expansion joint on both sides of the arena runs the entire length of the building in the N-S direction that in effect divides the building. The large trusses composed of W14x120 as top and bottom chords and HSS8x8x1/2 diagonal members along with the roofing material of the arena acts as a diaphragm and shifts the wind loads out to the moment frames along the expansion joints. Other smaller trusses composed of W12x53's as top and bottom chords and HSS6x6x1/2 as diagonal members oriented in the E-W direction act in a similar manner on the eastern part over the classrooms, auxiliary gym, and swimming pool areas. Moment frames and vertical trusses composed of W shapes are widespread throughout the building in both directions.

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 XXXX). 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, ACI 318-02, ACI 315-99

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.

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: Main Wind Force Resisting System was used for the analysis of wind loads. The building 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.

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	12.3
15-20	0.62	11.7	-7.5	13.0
20-25	0.67	12.3	-7.5	13.6
25-30	0.70	12.8	-7.5	14.1
30-40	0.76	13.6	-7.5	14.9
40-50	0.81	16.3	-7.5	17.6
50-60	0.85	17.0	-7.5	18.3
Arena		(ht=45ft)		
Completely Enclosed- No Analysis Needed				
Physical Education North		(ht=30ft)		
0-15	0.57	10.5	-8.6	14.0
15-20	0.62	11.2	-8.6	14.6
20-25	0.67	11.7	-8.6	15.2
25-30	0.70	12.2	-8.6	15.7
Physical Education South		(ht=38ft)		
0-15	0.57	10.7	-9.2	14.4
15-20	0.62	11.3	-9.2	15.1
20-25	0.67	11.9	-9.2	15.6
25-30	0.70	12.4	-9.2	16.1
30-40	0.76	13.2	-9.2	16.9

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	12.3
15-20	0.62	11.7	-7.5	13.0
20-25	0.67	12.3	-7.5	13.6
25-30	0.70	12.8	-7.5	14.1
30-40	0.76	13.6	-7.5	14.9
40-50	0.81	16.3	-7.5	17.6
50-60	0.85	17.0	-7.5	18.3
Arena		(ht=45ft)		
0-15	0.57	10.8	-9.6	14.7
15-20	0.62	11.5	-9.6	15.4
20-25	0.67	12.0	-9.6	15.9
25-30	0.70	12.5	-9.6	16.4
30-40	0.76	13.4	-9.6	17.3
40-50	0.81	16.0	-9.6	19.9
Physical Education North		(ht=30ft)		
0-15	0.57	10.8	-2.7	7.9
15-20	0.62	11.5	-2.7	8.6
20-25	0.67	12.0	-2.7	9.2
25-30	0.70	12.5	-2.7	9.7
Physical Education South		(ht=38ft)		
0-15	0.57	10.8	-2.7	7.9
15-20	0.62	11.5	-2.7	8.6
20-25	0.67	12.0	-2.7	9.2
25-30	0.70	12.5	-2.7	9.7
30-40	0.76	13.2	-2.7	10.5

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

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

Wind Load Base Shears and Overturning Moments		
	Total Base Shear(kips):	Total Overturning Moment(ft-kip):
Facilities Management		
E-W	136	3638
N-S	184	5984
Arena		
E-W	N/A	N/A
N-S	174	4133
Physical Education North		
E-W	189	2900
N-S	62	569
Physical Education South		
E-W	107	2111
N-S	40	803

Seismic Loads: 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 separated into 4 separate building for the analysis. For a complete description of seismic design criteria see Appendix C.

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	Roof	30		164398.6	1	230	6900
			SUM	**164398.6**	**1**	**230**	**6900**
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**

*T=0.723 sec == k=1.1

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

Seismic Base Shear and Overturning Moments		
Building	Base Shear	Overturning Moment
Facilities Management	338k	11808'k
Arena	230k	6900'k
Physical Education North	127k	3538'k
Physical Education South	99k	2208'k

Summary of Lateral Loads:

It is shown that seismic lateral loads control over wind lateral loads for the larger areas of the Coppin State Physical Education Complex (Facilities Management and the Arena). The building is not very high (only 60' maximum height) but it is heavy and expansive, and because seismic base shear depends on weight, as expected seismic controls. However, in the lighter areas (physical education north and south), wind loads control. This is due to less weight above the ground level. Most of my design shears agreed with the designer within 10%. The only variance would be with Physical Education South. This could be due to a load I have not accounted for or the designer could have used a more conservative view on the length of the individual buildings. Since the Coppin State University Physical Education Complex is really 4 building designed together, the designer could have used a different length when calculating wind loads. I will investigate this discrepancy in later reports.

Special Loads:

Retaining Walls:

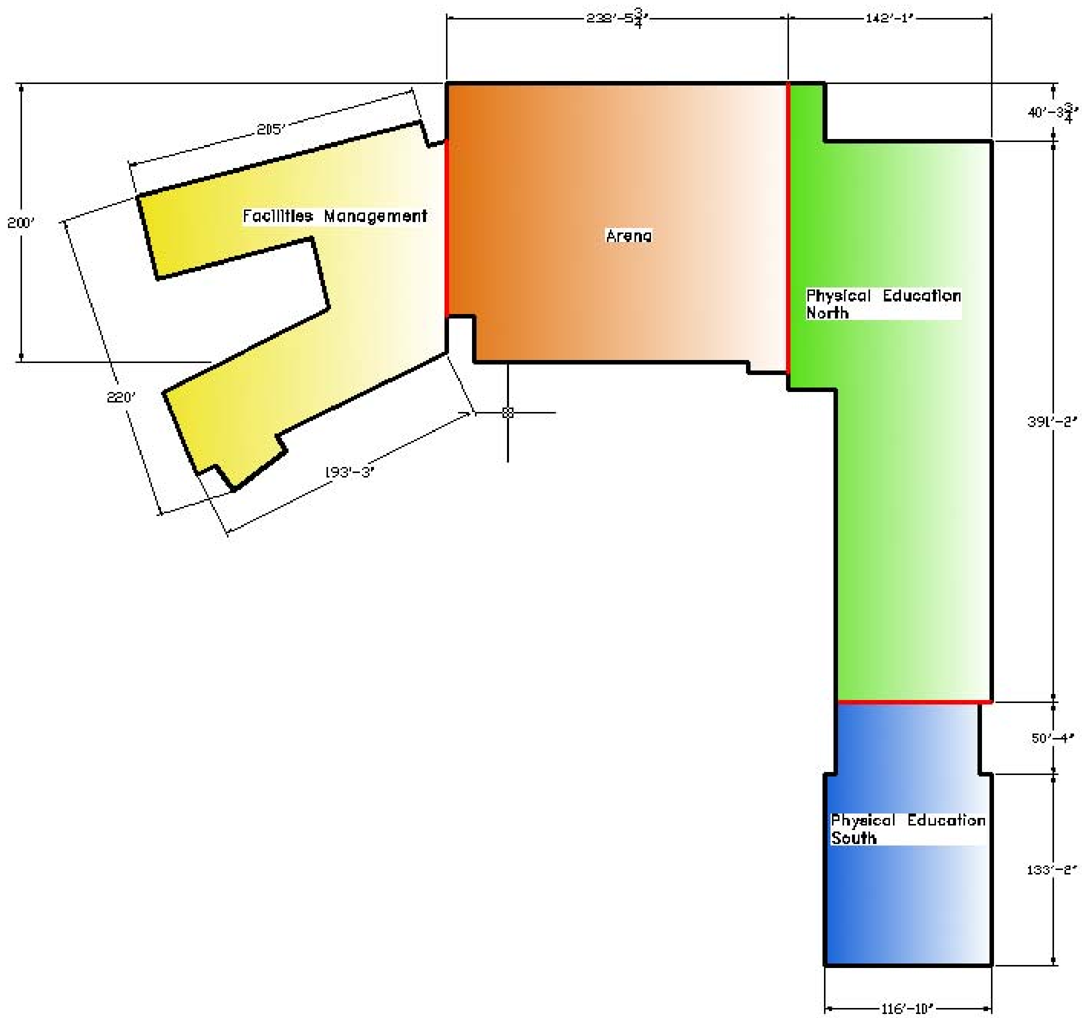
Equivalent at rest earth pressure...54pcf
Equivalent passive earth pressure..330pcf
Bulk Density..125pcf
At rest horizontal surcharge..0.42 x vertical surcharge
Active horizontal surcharge...0.38 x vertical surcharge
Friction..0.36

Spot Checks:

Spot Checks can be found in Appendix D. Several members were analyzed including the floor system, composite beam and girder, typical column, typical truss member over the arena, and a typical moment frame. Most of the members I sized were similar to the designer's, however in certain instances my members were smaller. This could be due to several factors which I have outlined per each section. Considering the assumptions I have made, along with the straightforwardness of my calculations, it can be seen that most, if not all of the original designs seem valid. Further insight will occur in later reports for specific members.

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

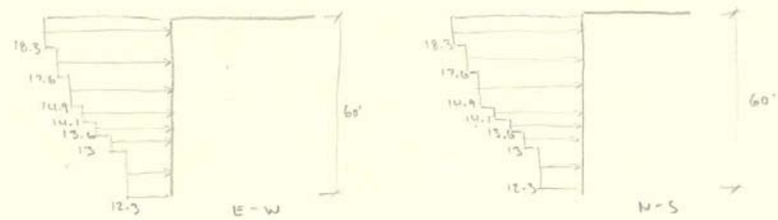
Building Category		II
3 second gust speed V		90 mph
Importance factor Iw		1.15
Building mean roof height H	*	60 ft.
Roof slope Theta		0 to 10 degrees
Exposure Category		B
Topography Factor, Kzt		1
Velocity pressure exposure coefficient at mean roof height, Kh		0.85
Velocity pressure at mean roof height, 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

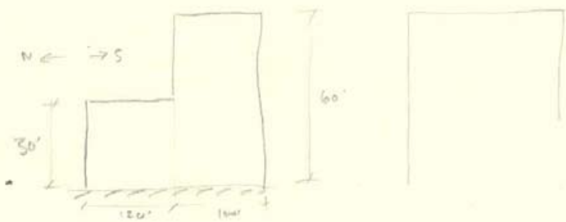
*Varies Between Buildings, Shown for N-S Wind on Facilities Management

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 Base Shears

Facilities Management 220 x 205





E - W

Base Shear = $(12.3 \times 15' + 13 \times 5' + 13.6 \times 5' + 14.1 \times 5')(120') + [12.3(15') + 13(5') + 13.6(5) + 14.1(5) + 14.9(10)$
$+ 17.6(10) + 18.3(10)](100') = 136^k$

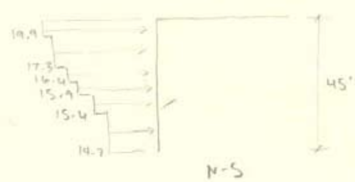
Overturning Moment = $3638'^k$

N - S

Base Shear = 184^k

Overturning Moment = $5984'^k$

Arena 238.5'

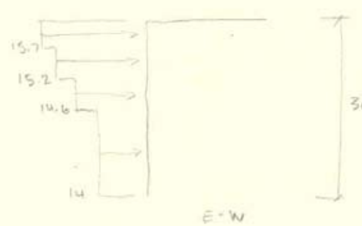


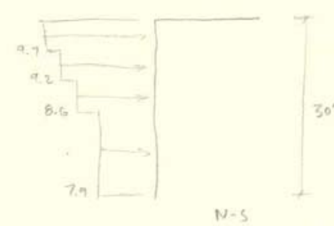
N - S

Base Shear = 174^k

Overturning Moment = $4153'^k$

Physical Education North (ht=30') 142' x 431.5'





E-W

Base Shear = 189^k

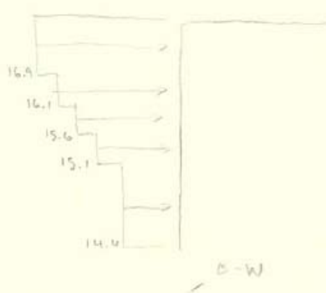
Overturning Moment = 2900'^k

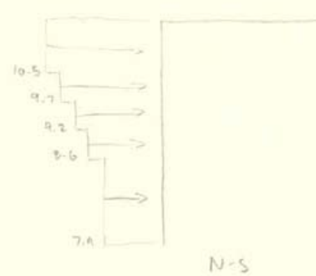
N-S

Base Shear = 62^k

Overturning Moment = 569'^k

Physical Education South (ht=38') 117' x 185.5'





E-W

Base Shear = 107^k

Overturning Moment = 2111'^k

N-S

Base Shear = 40^k

Overturning Moment = 803

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)

$C_t = 0.028$ $x = 0.8$

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

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

$S_{DS}/(R \cdot I) = 0.085$

$C_s =$ MIN $S_{D1}/[T \cdot R/I] = \underline{0.059}$ controls

$S_{D1} \cdot T_L/[T^2 \cdot R/I] = 0.65$

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

Areas

Facilities Management:

Area	height
Level 2 SCUP: A = 11200 sqft	15'
Level 2 Standard: A = 19300 sqft.	15'
Level 3 Standard: A = 12300 sqft.	30'
Level 4 Standard: A = 12300 sqft.	45'
Roof North: A = 18200 sqft.	30'
Roof South: A = 12300 sqft	60'

Arena

Roof Area = 46000 sqft.	60'

Physical Education North

Level 2 Mechanical: A: 5000 sqft	15'
Roof: A = 49500 sqft.	30'

Physical Education South

Level 2 Standard: A = 15600 sqft.	15'
Roof: A = 20600 sqft.	30'

Weights

Exterior Walls ⇒ 15 psf
Partitions ⇒ 10psf
Snow Load ⇒ 25psf
Cooling Tower Weights ⇒ $15^K \rightarrow 45^K$

Truss wts.
$T1: 120(2) + 48.72\sqrt{2} = 309\ ^{\#}/ft\ (166') = 51^K$
$51^K \times 8\ trusses = 410^K$
$T4: 53(2) + 35.11\sqrt{2} = 156\ ^{\#}/ft \times 104' = 16^K$

Facilities Management : ext. perimeter ≅ 1000ft

$W_{L2} = 11200(91+10) + 19300(60+10) + 15(1000)(15) + 3(30^K) = 2797^K$
$W_{L3} = 12300(60+10) + 15(1000)(15) = 1086^K$
$W_{L4} = 12300(60+10) + 15(1000)(15) = 1086^K$
$W_{RN} = 18206(25) = 455^K$
$W_{RS} = 12300(25) = 308^K$

Arena: ext perimeter ≅ 600ft
$W_R = 46000(60+10) + 410^K + 15(30)(600) = 3900^K$

Physical Education North ext. perimeter ≅ 815ft.

$W_{L2} = 5000(67+10) + 15(815)(15) = 568^K$
$W_R = 49500(25) + 16^K(22\ trusses) = 1590^K$

Physical Education South ext. perimeter = 320ft.

$W_{L2} = 15600(60+10) + 15(320)(15) = 1164^K$
$W_R = 20600(25) = 515^K$

Total Base Shear ⇒ $V = C_s W = 0.059\,W$

Facilities Management
$V = 338^K$

Arena
$V = 230^K$

Physical Education North
$V = 127^K$

Physical Education South
$V = 99^K$

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	Roof	30	3900		164398.6	1		230	6900
				SUM	**164398.6**	**1**	**230**	**230**	**6900**
Physical									
Education	2	15	568		11169.89	0.142849		18.1418	272.126
North	Roof	30	1590		67024.02	0.857151		108.858	3265.75
				SUM	**78193.91**	**1**	**127**	**127**	**3537.87**
Physical									
Education	2	15	1164		22890.4	0.513244		50.8112	762.168
South	Roof	30	515		21709.04	0.486756		48.1888	1445.66
				SUM	**44599.44**	**1**	**99**	**99**	**2207.83**

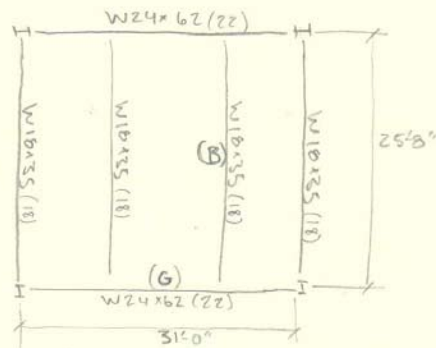
T=0.723 sec == k=1.1

Appendix D

Spot Checks:

Spot Check

Section B typ bay



3¼" LW conc on 3"x18GA Galv.
Composite Metal Deck
#4 @ 12" in span direction
6x6 - W1.4x1.4 W.W.F. Drape

Loads : Standard Floor (psf)

Conc. slab	51
Metal Deck	2
M/E/C/L	7

DL = 60 psf

LL = 100 No LL reduction for stadium occupancy

Floor System Check : Using USD Design Manual & Catalogue

Span 10' 4"

Max unshored length for 20GA 3" Lok-floor = 11.43' > 10.33' (3 spans)

→ will support 225# LL > 100 ok

20GA 3" Lok Floor compare w/ designer 18GA 3" Lok-Floor

Designer chose a thicker floor system possibly due to progressive loads, lateral loads, or unforseen loads

Composite Beam Check

$w_u = 1.2(60) + 1.6(100) = 232\ psf$

$M_u = \dfrac{.232\,(25\,{}^{8}/_{12})^2\,(10.33)}{8} = 197'^{k}$

Assume $a = 1"$ $Y2 = 6.25 - \frac{1}{2} = 5.75$

try W14×22 $\phi M_n = 212'^{k}$ @ PNA 6 $b_{eff} = 10.33'$ or $\dfrac{25\,{}^{8}/_{12}}{4} = 77"$

$\Sigma Q_n = 119$ $\dfrac{\Sigma Q_n}{b_{eff}\ .85 f'c} = \dfrac{119}{77(.85 \times 3.5)} \Rightarrow a = .52" < 1"$ ok

Using 3/4" dia stud

$Q_n = 0.5\left(\pi \frac{(.75)^2}{4}\right)\sqrt{3.5 \times 110^{1.5}\sqrt{3.5}} = 19.2$

$\dfrac{119}{19.2}(2) = 14$ studs

	ΣQ_n	ϕM_n	# studs	wt of stl	Equiv wt of studs	Total Equiv. wt.
W12×19	174	198	20	488	200	688
W14×22	119	212	14	565	140	705
W16×26	96	248	10	667	100	767
W18×35	129	367	14	898	140	1038

W12×19 is economically the best buy, but for probable deflection complications I will continue the analysis w/ W14×22 beams

check construction loads : $w_{conc} = \dfrac{12\,(3.25 + 1.5)}{144}(150) = 59.4\ psf = DL$

$LL = 20\ psf$

$w = 1.2D + 1.6L = 103\ psf$

$M_u = \dfrac{.103\,(10.33)\,(25.66)^2}{8} = 87.9'^{k} < \phi M_{n_{W14\times22\ bare}} = 125'^{k}$ ok

Construction DL (W14x22)

$$\frac{5(.0594 \times 10.33)(25.66)^4(1728)}{384(29000)(199)} = 1.03 > L/360 = \frac{25.66(12)}{360} = .856''$$

↑ Shoring or camber required

Try 18x35 to check if camber/shoring would be required

$$\frac{5(.0594 \times 10.33)(25.66)^4(1728)}{384(29000)(510)} = 0.404'' < 0.856''$$

check LL Δ $\qquad I_{LR} = 575 in^4$

$$\frac{5(.1 \times 10.33)(25.66)^4(1728)}{384(29000)(575)} = 0.604'' < L/360 = 0.856'' \text{ ok}$$

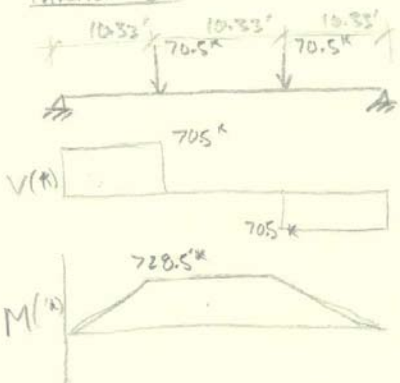
check LL deflection for W14x22

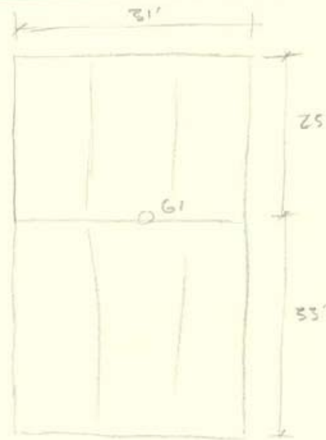
$$\frac{5(.1 \times 10.33)(25.66)^4(1728)}{384(29000)(466)} = 0.746'' < 0.856'' \text{ ok}$$

Conclusions: The W14x22 is acceptable as long as it is cambered or the contractor shores it under construction. The designer used a W18x35 most likely to avoid the use of shoring or camber or due to other unknown or unspecified loads. Design is valid

Spot Check of Girder

interior G1





Point Load on G = 30.75K + 39.75 = 70.5K

	ΣQ_n	ϕM_n	# studs	wt. of stl	Equiv. wt. of studs	Total Equiv wt.
W24x62	228	827	24	1922	240	2162
W21x55	292	734	32	1705	320	2025
W18x55	454	731	48	1705	480	2185

choose W21x55 as most economical

$b_{eff} = 7.75' = 93"$

$a = \frac{734}{.85(93)(3.5)} = 2.65 > 1"$ assume $\frac{a}{2} = 1.5"$

$Y2 = 6.25 - 1.5 = 4.75"$

	ΣQ_n	ϕM_n	# studs	wt. stl	equiv wt of studs	Total Equiv wt.
W24x62	228	810	24	1922	240	2162
W21x55	381	752	40	1705	400	2105
W18x55	573	741	60	1705	600	2305

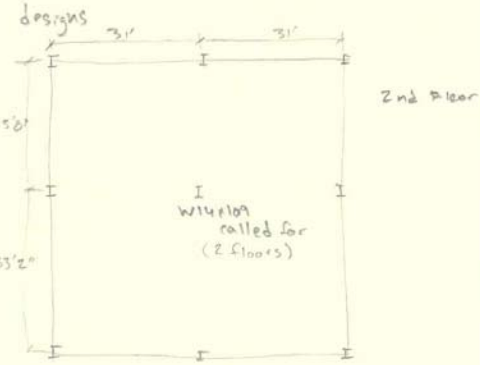
W21x55 most economical

* designer chose W24x62 probably to limit deflection of member

Conclusion: Design seems valid

Spot Check - column

LL reductions : Applicable for column designs



$L = L_o\left(.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right)$

$A_T = 31' \times \left(\frac{25'8" + 33'2"}{2}\right) = 911.9\ ft^2$

2 Floors above

$A_T = 911.9 \times 2 = 1823.8$

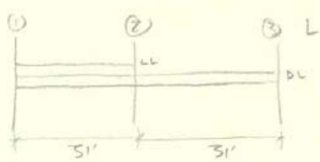
$.25 + \frac{15}{\sqrt{4(1823.8)}} = .43$

$L = .43(100) = 43psf$

Load Combination $1.2D + 0.5R_L + 1.6L$

Roof LL : $30(911.9) = 27.4^K$
Roof DL : $25(911.9) = 22.8^K$
Floor LL : $43(911.9) = 39.2^K$
Floor DL : $60(911.9) = 54.7^K$

$P_u = 1.2(54.7 + 22.8) + .5(27.4) + 1.6(39.2) = \underline{169.4}^K$



L: $1.6(39.2) + 1.2(54.7) = 128.4\ psf \times \left(\frac{25'8"}{2} + \frac{33'2"}{2}\right)$
$= 3.78^K/ft$

$FEM_{12} = \frac{3.78(31)^2}{12} = 302.7'^K$

R: $1.2(54.7) = 65.6\ psf \times \left(\frac{25'8"}{2} + \frac{33'2"}{2}\right) = 1.93^K/ft$

$FEM_{23} = \frac{1.93(31)^2}{12} = 154.6'^K$

$FEM_{12} - FEM_{23} = 148.1'^K$

. Assume ½ ΔFEM goes to column

$M_{col} = \frac{148}{2} = 74'^{k}$

$P_u = 169.4^{k}$

$P_{eff} = P_u + m M_{uy}$ $m = \frac{24}{d} = \frac{24}{14} = 1.71$

$P_{eff} = 169.4 + 1.71(74) = 296.3^{k}$

$L = 15'$

$KL = 15$

W14x48 acceptable

Conclusions:
The designer used a continuous column from the base through to level 2 for a total length of 30' maintain continuity and save in connections and contracting costs. The larger columns could also be due to lateral forces which have not been accounted for at this point. The designer could have used unreduced live loads for an added degree of safety.

I will check the design @ level 1

mM_{uy} still is $1.71(74)$

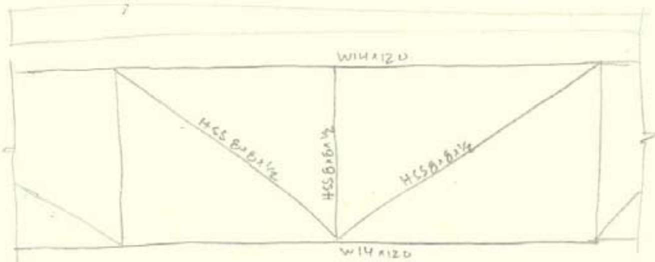
$P_u = 1.2(54.7(2)+22.8) + .5(27.4) + 1.6(39.2(2)) = 297.8$

$P_{eff} = 297.8 + 1.71(74) = 424.3$

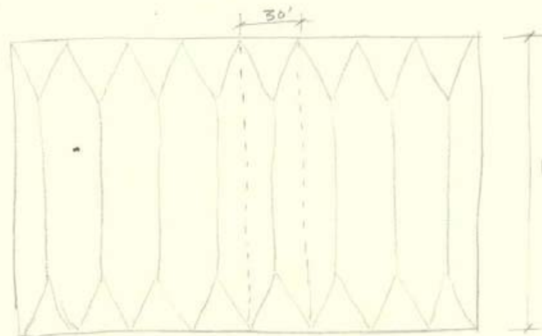
$L = 15'$
$KL = 15$

W14x61 acceptable < W14x109 called for, so most likely a higher live load was used or lateral forces controlled

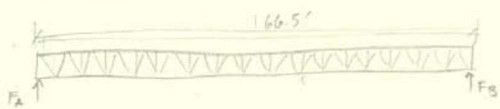
Spot Check of Truss Members



Detail



trib width = 30'



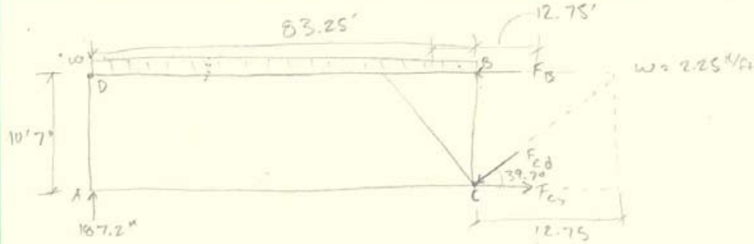
Loads
SL = 25psf
DL = 28.1psf

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

Load Combination
1.2D + 1.6S

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

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

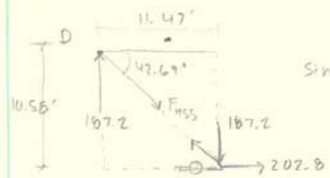


$\Sigma M_c = 0 \qquad 187.2(83.25) = F_{B_y}(10.583) + 83.25(2.25)\left(\frac{83.25}{2}\right)$

$F_{B_y} = 736^k$

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

$F_{cd} =$



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

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

HSS: HSS 6×6×1/2 $\phi_t P_n = 318^k > 276^k$ ok

Designer chose HSS 8×8×1/2 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

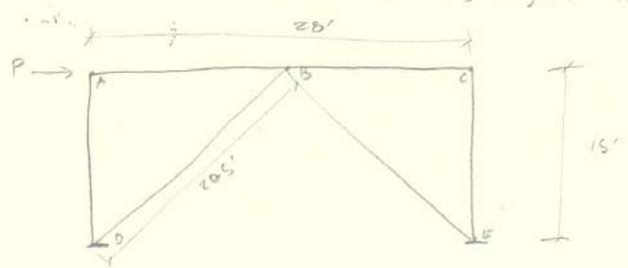
W14×74 $\phi P_n = 742^k > 736^k$ acceptable

Designer used W14×120 probably using lateral loads and incorporating bending from roof loads

Design valid

Lateral Spot Check

Facilities Management end moment frame (N-S)



All W members

Wind:

$$P = \left[14.1\,psf \times \left(\frac{10'11" + 10'11" + 30'6"}{2}\right) \times \left(\frac{30' + 15'}{2}\right)\right] 1.6 = 102.5^k$$

dist. to next moment frame

ave ht.

Seismic:

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$ ← ASCE 7-05 12.8.3

C_{vx} @ level 2 Facilities Management = 0.251 ← see Seismic Base Shear & Moment Calc. Spreadsheet

$P = F_x = 84.8^k\ (1.0)$ ← see spreadsheet

⇒ Wind controls

$102.5 = (F_{BD} + F_{BE}) \cos 47°$

$F_{BD} = F_{BE} = 75.1^k$ T + C

Diagonal Member

$KL = 20.5'$

W8×31 $\phi P_n = 140^k > 75.1^k$ ok

use W8×31 as diagonal member

Top member subjected to floor loads also ⇒ will not size

Designer chose W8×31 which agrees with my solution

Design valid