Preliminary Structural Calculations
for
Same Polytechnic College

Same, Tanzania

Performed by: EMM
Cal Poly, San Luis Obispo
3/15/2019

Design Description

The design consists of an upper roof that provides shade to reduce solar gains on the lower structure below it. The upper roof is separated from the building beneath to simplify the construction process by eliminating the technical connections that would otherwise occur between the two buildings. This schematic design includes two possible options for the upper roof building material to provide flexibility in cost, since material availability is uncertain. The steel system consists of wide-flange beams and cantilevered pipe columns, while the timber system explores both a Glue Laminated beam system and a truss system with cantilevered columns that eliminate the need for technical moment frame connections. The lower structure design utilizes timber/masonry construction, with an occupiable outdoor roof space shaded by the upper roof.

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SAME, TANZANIA ~ SACRAMENTO, CALIFORNIA

38.55244°N, 121.49506°W

$S_s = 0.688$ $S_1 = 0.296$

$S_{DS} = .573$ $S_{D1} = .357$

USE R = 2.5 (STEEL SPECIAL CANTILEVER COLUMN SYSTEMS)

I = 1.25

$C_s = \frac{S_{DS}}{(R/I_e)} = \frac{0.573}{(2.5/1.25)} = 0.287$

$C_s = \frac{S_{D1}}{T(R/I_e)}$ $T = C_t h_n^x$ [ASCE 12.8-7]

$C_t = .028$

$= \frac{.357}{.356(2.5/1.25)}$ $h_n = 24'$

$x = 0.8$

$= .501$ $T = 0.356$

<u>V = .287W</u> (STEEL SPECIAL CANTILEVER COLUMN SYSTEMS)

USE R = 1.5 (CANTILEVER COLUMN TIMBER FRAME)

$C_s = \frac{0.573}{1.5/1.25} = 0.478$

$C_s = \frac{.357}{.356(1.5/1.25)} = .836$

<u>V = .478W</u> (TIMBER)

USE R = 2 (ORDINARY REINFORCED MASONRY SHEAR WALLS)

$C_s = .573 / {}^{2}/_{1.25} = .358$

$C_s = \frac{.357}{.356(2/1.25)} = .627$

V = .358 W (MASONRY)

BIGGS CARDOSA ASSOCIATES INC.

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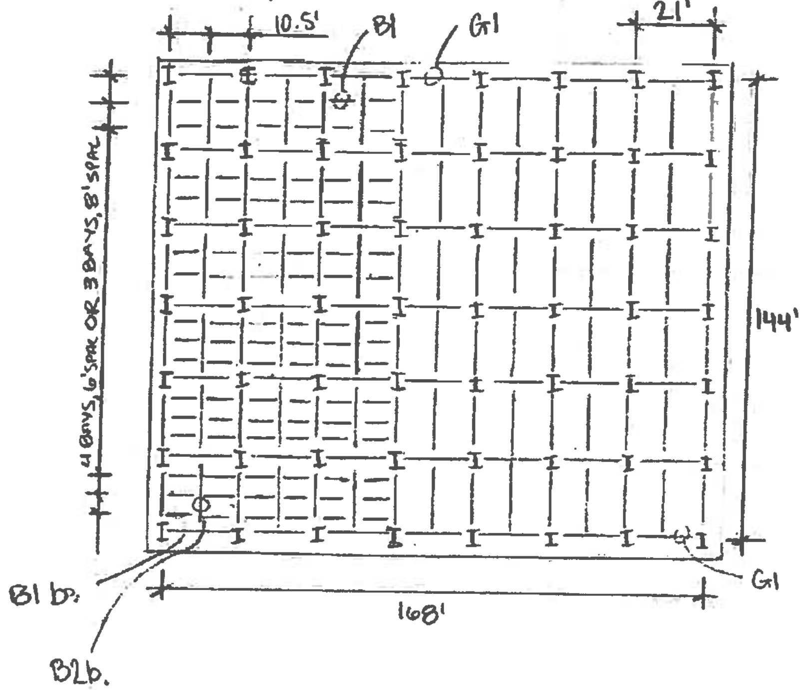
STEEL FRAMING PLAN



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DECKING

$L_o = 20$ PSF

$A_T = 24' \times 24' = 576$ SF

$L_r = L_o R_1 R_2$ $R_1 = 1.2 - .001 A_t = 0.624$

$= 20(.624)(1) = 12.48$

$L_r = 12.48$ psf
DL = 8 psf
} TOTAL ≈ 21 PSF

METAL DECK w/ NO CONC. FILL

USE PLB-36 FORM-LOK (OR SIMILAR), 22 GAGE w/ 3 SPANS @ 8' PER SPAN.

$L/180 = 37$ PSF FOR 1 SPAN @ 8'/SPAN > 21 PSF OK

BIGGS CARDOSA
ASSOCIATES INC.

Project:	Same Polytechnic	
Date:	3/14/2019	
By:	EMM	
	Roof	**psf**
	Asphalt tile	2.7
	Corrugated Steel	2.6
	MEP	2
	Typical Roof Dead Load:	**8**
	Live Load (Reducible):	**20**

Project:	Same Polytechnic			
Date:	3/15/2019			
By:	EMM			
Gravity Demands Steel				
Load Combination:	D	1.2		
	L	1.6		
B1	uniformly distributed load			
Trib width	8	ft		
DL*1.2	9.6	psf		
LL*1.6	32	psf		
distributed load	332.8	plf		
Lbeam	12	ft		
Vmax	1996.8	#		
Mmax	5990.4	#-ft		
B2	uniform distributed and two equal concentrated loads symmetrically placed			
Trib width	12	ft		
DL*1.2	9.6	psf	LL reduction	18.24
LL*1.6	29.184	psf	beam frames in at each side	
Point loads at 8' and 16'	3993.6	#		
distributed load	465.408	plf		
Lbeam	24	ft		
a	8	ft		
b	8	ft		
			reaction at truss	
Vmax	9578.496	#		
Mmax	65458.176	#-ft		
G1/Truss	uniform distributed and concentrated load at center			
Trib width	24	ft		
DL*1.2	9.6	psf	LL reduction	13.92
LL*1.6	22.272	psf		
P @ 12'	9578.496	#		
distributed load	764.928	plf		
Lbeam	21	ft		
Vmax	12820.992	#		
Mmax	92453.76	#-ft		
Column				
P (ASD)	16063.49	#		
Pu (LRFD)	11047.68	#		

Project:	Same Polytechnic					
Date:	3/15/2019					
By:	EMM					

Steel Framing Design

Fy	50	ksi
Esteel	29000000	psi

B1			Seismic - Check Tension/Compression			
span	10.5	ft	Sds	0.573		
L/240	0.525	in	DL	8	psf	
Ireq	6.0	in4	trib width	8	ft	
			D	0.672	k	
Try W8x10			L	1.68	k	
Zx	8.87	in^3	Cs	0.179		
Ix	30.8	in^4	W	193.5	k	
Ag	2.96	in^2				
Mp=FyZx	36.95833333	k-ft	Qe=V=CsW	34.6	k	
Mmax	5.99	k-ft	T=(0.9-0.2Sds)D+rhoQe	35.17	k	
d/c	0.16		C=(1.2+0.2Sds)D+.5L+rhoQe	36.37	k	Demand
			phi*Pn=0.9*Fy*Ag	133.2	k	Capacity

B2			Seismic - Check Tension/Compression			
span	24	ft	Sds	0.573		
L/240	1.2	in	DL	8	psf	
Ireq - dist	99.8	in4	trib width	12	ft	
Ireq - conc.	92.2	in4	D	2.304	k	
Ireq total	192.0	in4	L	3.2832	k	
Try W14x22			Cs	0.179		
Zx	33.2	in^3	W	193.5	k	
Ix	199	in^4				
Ag	6.49	in^2	Qe=V=CsW	34.6	k	
Mp=FyZx	138.3	k-ft	T=(0.9-0.2Sds)D+rhoQe	36.45	k	
Mmax	65.46	k-ft	C=(1.2+0.2Sds)D+.5L+rhoQe	39.31	k	Demand
d/c	0.47		phi*Pn=0.9*Fy*Ag	292.05	k	Capacity

G1			Seismic - Check Tension/Compression			
span	21	ft	Sds	0.573		
L/240	1.05	in	DL	8	psf	
Ireq - dist	109.9	in4	trib width	24	ft	
Ireq - conc.	104.9	in4	D	4.032	k	
Ireq total	214.8	in4	L	4.3848	k	
Try W14x30			Cs	0.179		
Zx	47.3	in^3	W	193.5	k	
Ix	291	in^4				
Ag	8.85	in^2	Qe=V=CsW	34.6	k	
Mp=FyZx	197.1	k-ft	T=(0.9-0.2Sds)D+rhoQe	37.81	k	
Mmax	92.45	k-ft	C=(1.2+0.2Sds)D+.5L+rhoQe	42.14	k	Demand
d/c	0.47		phi*Pn=0.9*Fy*Ag	398.25	k	Capacity

Project:	Same Polytechnic	
Date:	3/15/2019	
By:	EMM	

Column Design		
h	288	in
KL (K=1)	288	in
At	504	ft^2
P	16	k
steel stress	36	ksi
E	29000	ksi
Areq	0.45	in^2
Ireq	19	in^4
Use Pipe 5 Std.		
I	14	in^4
A	4	in^2
phiPn=.9*Fy*Ag	130	k

Footing Design		
Pu	11048	#
Fbearing	2000	psf
Areq	6	ft^2
L1	3	ft
L2	3	ft
Af	9	ft^2
Pipe 5 Std.		
x=y	6	in
c	15	in
d	11	in
h	15	in
cover	4	in
b	65	in
Asmin=.0018bh	1.72	in^2
Use 6-#5 bars		
As	1.86	in^2
Load to each col		
Weight	4032	#
V=Cs*W	1157	k
h	288	in
M	27772	k-ft
e	3	
L/6	6	e<L/6
fb	1637	psf
USE 3' x 3' x 15" deep foundation		

WIND - COMPONENTS & CLADDING

a = .1(144') OR 0.4(24')
= 14.4 OR 9.6 ⇒ USE 14.4

RISK CATEGORY III

V = 115 mph

K_d = 0.85 [ASCE TABLE 26.6-1]

EXPOSURE D [ASCE 26.7]

K_{zt} = 1 [ASCE 26.8]

G = 0.85 (GUST-EFFECT FACTOR) [ASCE 26.9.1 & 26.9.2]

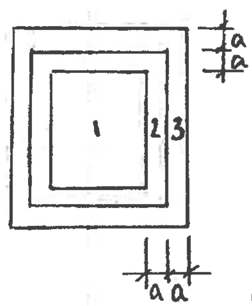
K_z = 1.12 (VELOCITY PRESSURE EXPOSURE COEFFICIENT) [TABLE 30.3-1]

VELOCITY PRESSURE

$q_z = .00256 K_z K_{zt} K_d V^2$ [ASCE 30.3-1]

$= .00256(1.12)(1)(.85)(115) = 32.23$ #/ft²

NET PRESSURE COEFFICIENTS, C_N [ASCE FIGURE 30.8-1]



a = 14.4'

ZONE 3
$C_N = 2.4$ $C_N = -3.3$

ZONE 2
$C_N = 1.8$ $C_N = -1.7$

ZONE 1
$C_N = 1.2$ $C_N = -1.1$

WIND PRESSURE [ASCE 30.8-1]

③ $p = q_h G C_N$
$= 32.23$ #/ft² $\times 0.85 \times (-3.3) = -90.41$ #/ft² (MAX)

② $p = q_h G C_N$
$= 32.23 \times .85 \times 1.8 = 49.31$ #/ft² (MAX)

① $p = q_h G C_N$
$= 32.23 \times .85 \times 1.2 = 32.87$ #/ft² (MAX)

Project:	Same Polytechnic		
Date:	3/15/2019		
By:	EMM		
Wind Loads Steel: Roof Pressures			
G	0.9		(gust effect factor)
qz	32.2	psf	(velocity pressure)
Net Pressure Coefficients			
Zone 3	2.4	-3.3	
Zone 2	1.8	-1.7	
Zone 1	1.2	-1.1	
Wind Pressure (psf)			
Zone 3	65.7	-90.4	
Zone 2	49.3	-46.6	
Zone 1	32.9	-30.1	
D (psf)	8.0		
Lr (psf)	20.0		
1.2D+1.6Lr+0.5W		(Downward Pressure)	
Zone 3	74.5	psf	
Zone 2	66.3	psf	
Zone 1	42.2	psf	
1.2D+1.0W+.5Lr		(Downward Pressure)	
Zone 3	85.3	psf	
Zone 2	68.9	psf	
Zone 1	52.5	psf	
.9D+1.0W		(Uplift)	
Zone 3	-83.2	psf	
Zone 2	-39.4	psf	
Zone 1	-22.9	psf	

Project:	Same Polytechnic					
Date:	3/15/2019					
By:	EMM					

Lateral Check for Wind - Steel

Downward Force

B1 - Zone 3			*B1 - Zone 2*		*B1 - Zone 1*	
trib width	8	ft	8	ft	8	ft
length	12	ft	12	ft	12	ft
wind load	85	psf	69	psf	52	psf
distributed load	683	plf	551	plf	420	plf
deflection	0.357	in	0.288	in	0.219	in
Vmax	4097	#	3308	#	2519	#
Mmax	12290	#-ft	9923	#-ft	7556	#-ft
B2 - Zone 3			*B2 - Zone 2*		*B2 - Zone 1*	
trib width	11	ft	11	ft	11	ft
length	24	ft	24	ft	24	ft
wind load	85	psf	69	psf	52	psf
distributed load	896	plf	724	plf	551	plf
deflection	1.159	in	0.936	in	0.713	in
Vmax	10754	#	8683	#	6612	#
Mmax	64524	#-ft	52097	#-ft	39671	#-ft
G1 - Zone 3			*G1 - Zone 2*		*G1 - Zone 1*	
trib width	24	ft	24	ft	24	ft
length	21	ft	21	ft	21	ft
wind load	85	psf	69	psf	52	psf
distributed load	2048	plf	1654	plf	1259	plf
deflection	0.793	in	0.640	in	0.487	in
Vmax	21508	#	17366	#	13224	#
Mmax	112917	#-ft	91170	#-ft	69424	#-ft

Uplift						
B1 - Zone 3			*B1 - Zone 2*		*B1 - Zone 1*	
trib width	8	ft	8	ft	8	ft
length	12	ft	12	ft	12	ft
wind load	-83	psf	-39	psf	-23	psf
distributed load	-666	plf	-315	plf	-183	plf
deflection	-0.348	in	-0.165	in	-0.096	in
Vmax	-3994	#	-1890	#	-1101	#
Mmax	-11982	#-ft	-5670	#-ft	-3303	#-ft
B2 - Zone 3			*B2 - Zone 2*		*B2 - Zone 1*	
trib width	11	ft	11	ft	11	ft
length	24	ft	24	ft	24	ft
wind load	-83	psf	-39	psf	-23	psf
distributed load	-874	plf	-413	plf	-241	plf
deflection	-1.130	in	-0.535	in	-0.312	in
Vmax	-10484	#	-4961	#	-2890	#
Mmax	-62903	#-ft	-29765	#-ft	-17339	#-ft
G1 - Zone 3			*G1 - Zone 2*		*G1 - Zone 1*	
trib width	24	ft	24	ft	24	ft
length	21	ft	21	ft	21	ft
wind load	-83	psf	-39	psf	-23	psf
distributed load	-1997	plf	-945	plf	-550	plf
deflection	-0.773	in	-0.366	in	-0.213	in
Vmax	-20968	#	-9922	#	-5780	#
Mmax	-110080	#-ft	-52090	#-ft	-30343	#-ft

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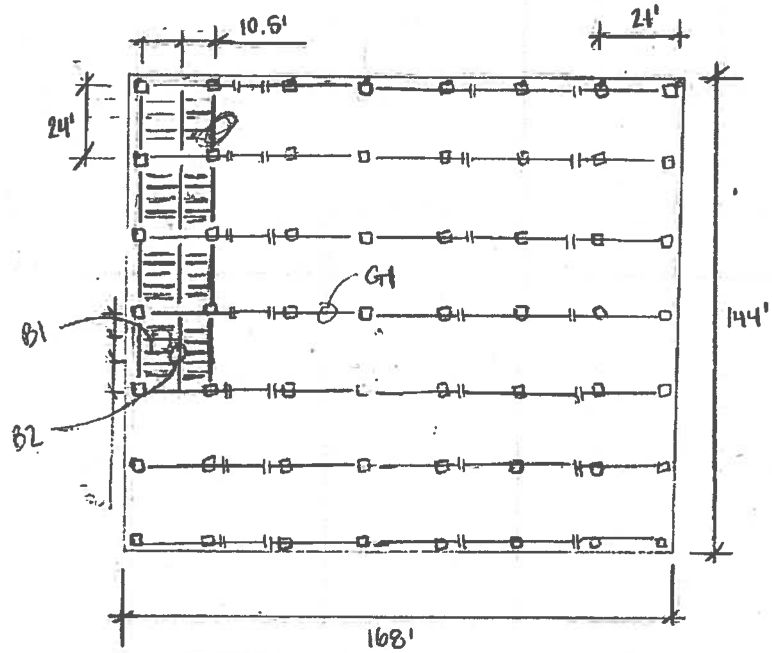
CHKD. BY ________ DATE ________ ________ JOB NO. ________

TIMBER FRAMING PLAN



Project:	Same Polytechnic			
Date:	3/15/2019			
By:	EMM			
Gravity Demands - Timber				
B1	uniformly distributed load			
Trib width	4	ft		
DL	8	psf		
LL	20	psf		
distributed load, w	112	plf		
Lbeam	10.5	ft		
Vmax=wL/2	588	#		
Mmax=wL^2/8	1543.5	#-ft		
B2	uniformly distributed load			
Trib width	12	ft		
DL	8	psf		
LL	18.24	psf	LL reduction	18.24
distributed load, w	314.88	plf		
Lbeam	24	ft		
Vmax=wL/2	3778.56	#	reaction at truss	
Mmax=wL^2/8	22671.36	#-ft		
G1/Truss	concentrated load			
Trib width	24	ft		
DL	8	psf		
LL	13.92	psf	LL reduction	13.92
P @ 10.5'	11047.68	#		
Lbeam	21	ft		
Vmax=wL/2	5523.84	#		
Mmax=wL^2/8	58000.32	#-ft		
Column				
P=At*(DL+LL)	11047.68	#		

Project:	Same Polytechnic
Date:	3/15/2019
By:	EMM

Timber Framing Design

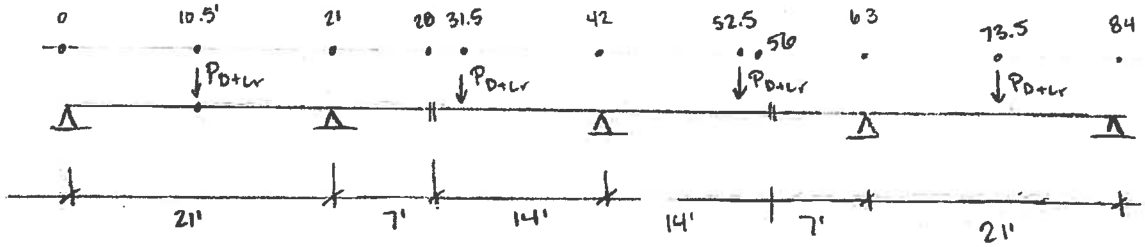
B1				
Vmax	588.0	#		
Mmax	1543.5	#-ft		
Check Deflection - Solve for Iallow				
Lbeam	126.0	in		
distributed load wD+L	9.3	#/in		
Iallow=5wl^4/384E*defl	134.5	in^4		
Try 2x8				
I (2x8)	47.6	in^4		
wL	6.7	#/in		
Lallow=l/360	0.4	in		
Lactual	0.6	in	Lallow>Lactual	OK
D+Lallowable=l/240	0.5	in		
D+Lactual	0.8	in	allow>actual	OK
Check Bending				
Cd	*1.3*	*Cf*	*1.2*	
Cm	*1.0*	*Cfu*	*1*	
Ct	*1.0*	*Ci*	*1*	
Cl	*1.0*	*Cr*	*1.15*	
S	13.1	in^3		
Fb'=Fb*Cd*Cf*Cr	2018.4	psi		
fb=M/S	1409.6	psi	Fb'>fb	OK
Check Shear				
fv=1.5(V/bd)	81.1	psi		
Fv'=Fv*Cd	953.3	psi	Fv'>fv	OK

Check Tension/Compression				
Sds	0.6			
DL	8.0	psf		
Lr	18.2	psf		
trib width	4.0	ft		
D	0.3	k		
L	0.8	k		
Cs	0.5			
W	193.5			
Qe=V=CsW	92.5			
Demand:				
T=(1+.14Sds)D+(.7*rho*Qe)	65.1	k	demand	
C=(1.0+.105Sds)D+(.525*rho*Qe)	48.9	k	demand	
C=(.6-.14Sds)D+(.7*rho*Qe)	64.9	k	demand	
b (2x8)	1.5	in		
d (2x8)	7.3	in		
(le/d)y	0.0			
(le/d)x	17.4			
Fc	661.6	psi		
Cd	1.6			
CM	1.0			
Ct	1.0			
Cf (2x10)	1.1			
Cp	1.0			
E'min=Emin*Cm*Ct	339273049.6			
Fc*=Fc*Cd*Cm*Ct*Cf	1164.4			
FcE	16046807.5			
1+(FcE/Fc*)/2c	8613.9			
(FcE/Fc*)/c	17226.5			
F'c	1164.4	psi		
Capacity: C=F'c*A	12.7	k	<demand	N.G.
USE 2x12				
A	19.9	in^2		
Capacity: C=F'c*A	23.1	k	>demand	OK

B2				
Vmax	3778.6	#		
Mmax	22671.4	#-ft		
Check Deflection - Solve for Iallow				
Lbeam	288.0	in		
distributed load wD+L	26.2	#/in		
D+Lallowable=l/240	1.2	in		
(5wl^4/384E*defl)	4516.2	in^4		
Iallow	4516.2	in^4		
Try 3.125x19.5 GLB				
I (3.125x19.5)	1931.0	in^4		
wL	18.2	#/in		
Lallow=L/360	0.8	in		
Lactual-dist	0.8	in	allow>actual	OK
D+Lallowable=l/240	1.2	in		
D+Lactual-allow	1.2	in		
			allow>actual	OK
Check Bending				
Cd	*1.3*	*Cf*	*0.94*	
Cm	*1.0*	*Cfu*	*1*	
Ct	*1.0*	*Ci*	*1*	
Cl	*1.0*	*Cr*	*1.15*	
S	198.0	in^3		
Fb'=Fb*Cd*Cf*Cr	1576.5	psi		
fb=M/S	1374.0	in^3	Fb'>fb	OK
Check Shear				
fv=1.5(V/bd)	27.7	psi		
Fv'=Fv*Cd	207.2	psi	Fv'>fv	OK

Check Tension/Compression				
Sds	0.6			
DL	8.0	psf		
Lr	18.2	psf		
trib width	12.0	ft		
D	2.3	k		
L	5.3	k		
Cs	0.5			
W	193.5			
Qe=V=CsW	92.5			
Demand:				
T=(1+.14Sds)D+(.7*rho*Qe)	67.2	k	demand	
C=(1.0+.105Sds)D+(.525*rho*Qe)	51.9	k	demand	
D=(.6-.14Sds)D+(.7*rho*Qe)	66.9	k	demand	
b (3.125x19.5)	3.1	in		
d (3.125x19.5)	19.5	in		
(le/d)y	0.0			
(le/d)x	14.8			
Fc	661.6			
Cd	1.6			
CM	1.0			
Ct	1.0			
Cp	1.0			
E'min=Emin*Cm*Ct	339273049.6			
Fc*=Fc*Cd*Cm*Ct	1058.5			
FcE	1278513.8	psi		
1+(FcE/Fc*)/2c	671.6			
(FcE/Fc*)/c	1342.0			
F'c	3419.6	psi		
A (3.125x19.5)	60.9	in^2		
Capacity: C=F'c*A	208.4	k	>demand	OK

GIRDER DESIGN



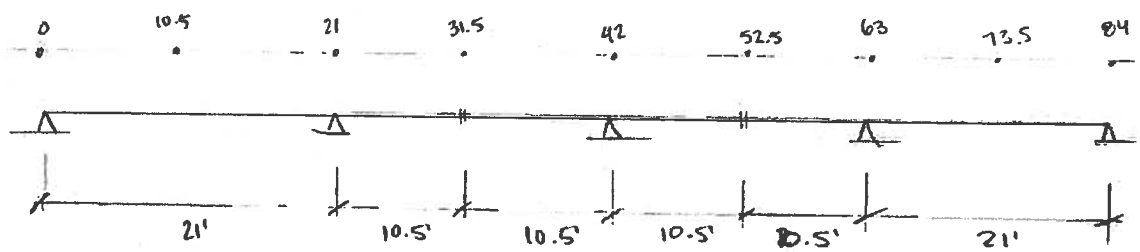
LOADING *NEGLECT SELF-WT.

$D = 8\text{ PSF}(12')(10.5') = 1.1\text{ K}$

$L_r = 20\text{ PSF}(12')(10.5') = 2.52\text{ K}$

LRFD: $1.2(1.1) + 1.6(2.52) = 5.35\text{ K}$

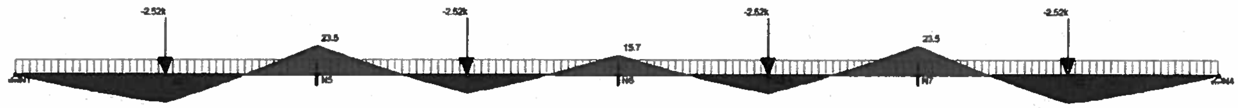
$w_{SELF} = 8\text{ PSF}(12') = 96\text{ PLF}$



USE 31.5', 21', 31.5' GIRDERS

Project:	Same Polytechnic
Date:	3/15/201
By:	EMM

Glulam Framing Design



Envelope Maximum Member Section Forces

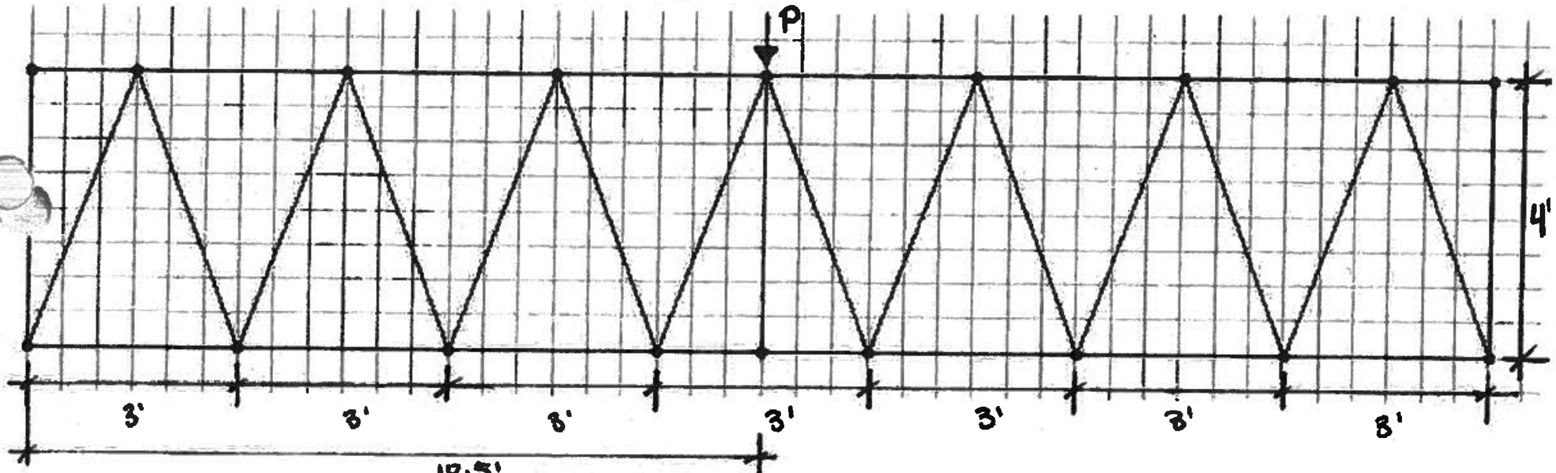
Sections | Maximums | End Reactions

	Member		Axial[k]	Loc[ft]	LC	Shear[k]	Loc[ft]	LC	Moment[k-ft]	Loc[ft]	LC
1	[illegible]	max	0	0	1	4.217	21.328	1	23.458	21	1
2		min	0	0	1	-5.003	21	1	-22.719	10.5	1
3	M2	max	0	0	1	3.492	10.719	1	15.713	10.5	1
4		min	0	0	1	-3.517	10.5	1	-14.863	0	1
5	M3	max	0	0	1	5.003	10.5	1	23.458	10.5	1
6		min	0	0	1	-4.217	10.172	1	-22.719	21	1

Project:	Same Polytechnic		
Date:	3/15/201		
By:	EMM		
Glulam Framing Design			
31.5' - 21' - 31.5'			
31.5' span			
24F-1.8E			
M+ max	282	k-in	
M- max	272	k-in	
Positive Bending			
Lateral stability			
F'b=F'bx+=			
Fbx+*Cd*Cm*Ct*Cl			
Lu	0		
Volume Effect			
F'b=F'bx+=			
Fbx+*Cd*Cm*Ct*Cv	2700	psi	
Cd	1		
Cm	1		
Ct	1		
Cv	1		
Fbx+	2400	psi	
Reqd S	104	in^3	
Try 3.125x19.5 GLB			
Sx (3.125x19.5)	198	in^3	
Verify Cv			
L	30	ft	
d	20	in	
b	3	in	
Kl	1		
Cv	1		
F'b=F'bx-=			
Fbx+*Cd*Cm*Ct*Cv	2898	psi	
S*F'bx-	574	#*in	OK
fb=M/S	1424	psi	demand
F'b=F'bx+=	2898	psi	capacity
Negative Bending			
M- max	467	k-in	
Lu	72	in	
Lu/d	4	<7	
therefore Le=2.06Lu	148	in	
sqrt((le*d/b^2))	17		
Fbx-	1450	psi	
coefficients for beam stability factor Cl			
E'ymin*Cm*Ct	950000	psi	
FbE	3849		
Fbx*=Fbx*Cd*Cm*Ct	1813		
FbE/Fbx*	2		
1+FbE/Fbx*/1.9	2		
FbE/Fbx*/.95	2		
beam stability factor			
Cl	1		
L=6+4	10	ft	
d	20	in	
b	3	in	
Kl	1		
Cv	1	>1.0	
Use Cv =	1		
F'b=F'bx-=	1741	psi	capacity
fb=M/S	1376	psi	demand
3.125x19.5 GLB OK for bending			

Check Tension/Compression			
Sds	0.6		
DL	8.0	psf	
Lr	13.9	psf	
trib width	24.0	ft	
D	4.6	k	
L	7.0	k	
Cs	0.5		
W	193.5		
Qe=V=CsW	92.5		
Demand:			
T=(1+.14Sds)D+(.7*rho*Qe)	69.7	k	demand
C=(1.0+.105Sds)D+(.525*rho*Qe)	53.5	k	demand
C=(.6-.14Sds)D+(.7*rho*Qe)	67.2	k	demand
b (3.125x19.5)	3.1	in	
d (3.125x19.5)	19.5	in	
(le/d)y	0.0		
(le/d)x	18.5		
Fc	2137.5		
Cd	1.6		
CM	1.0		
Ct	1.0		
Cp	1.0		
E'min=Emin*Cm*Ct	765400.0		
Fc*=Fc*Cd*Cm*Ct	3419.9		
FcE	34079.4	psi	
1+(FcE/Fc*)/2c	6.1		
(FcE/Fc*)/c	11.1		
F'c	3382.6	psi	
A (3.125x19.5)	60.9	in^2	
Capacity: C=F'c*A	206.1	k	>demand
Ft	2040.6		
F't	3265.0		
T=F't*A	199.0	k	>demand

21' span			
24F-1.8E			
M+ max	188	k-in	
Positive Bending			
Lateral stability			
F'b=F'bx+=			
Fbx+*Cd*Cm*Ct*Cl			
Lu	0		
Volume Effect			
F'b=F'bx+=			
Fbx+*Cd*Cm*Ct*Cv	2700	psi	
Cd	1		
Cm	1		
Ct	1		
Cv	1		
Fbx+	2400	psi	
Reqd S	70	in^3	
Try 3.125x19.5 GLB			
Sx (3.125x19.5)	104	in^3	OK
USE 3.125x19.5 due to beam framing			

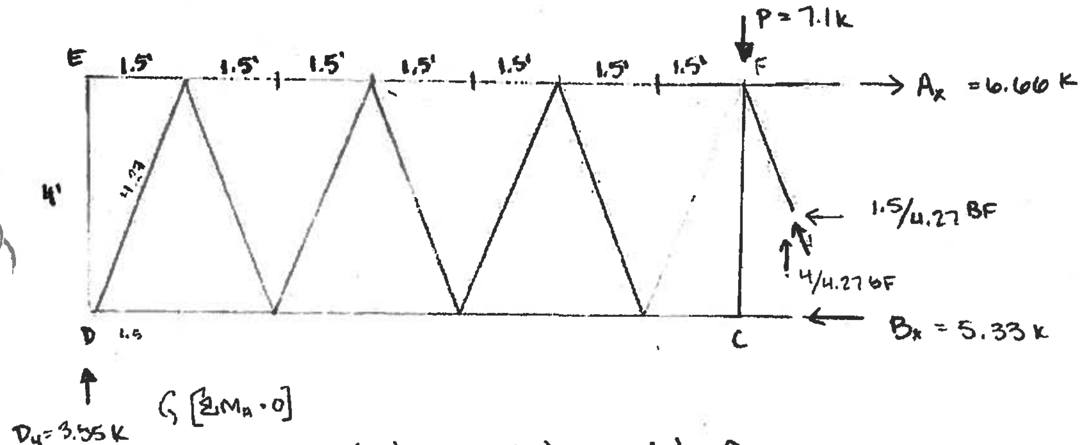


TRUSS DESIGN

D = 8 PSF L_r = 20 PSF

$A_T = 24' \times 10.5' = 252\ SF$

$P = (8 + 20\ PSF) \times 252\ SF = 7.1\ k$



$[\Sigma M_A = 0]$

$-3.55k(12') + 7.1k(3') + B_x(4') = 0$

$B_x = 5.33\ k$

$[\Sigma F_y = 0]$

$+3.55k - 7.1k - \frac{4}{4.27}BF = 0$

$-\frac{4}{4.27}BF = 3.55k$

$BF = -3.79 = 3.79k\ T$

$[\Sigma F_x = 0]$

$\frac{1.5}{4.27}(-3.79k) - 5.33k + A_x = 0$

$A_x = 6.66\ k$

Project:	Same Polytechnic		
Date:	3/15/2019		
By:	EMM		
Truss Design			
Tension force	6.66	k	
Ft	266	psi	
Cd	1.25		
Cm	1		
Ct	1		
Cf	1.5		
Ci	1		
F't	499	psi	
Req'd An=T/F't	13.35	in^2	
Req'd Ag=An-Ah	14.56	in^2	
Try 4x6			
A	19.25	in^2	OK
b	3.5	in	
h	5.5	in	
d=1/2 truss height	24	in	
Compression force	5.33	k	
Fc	662	psi	
Cd	1.25		
Cm	1		
Ct	1		
Cf	1.5		
Ci	1		
Cp	1		
F'c	1240.5		
Req'd An=C/F't	4.30		
Req'd Ag=An-Ah	5.52		
4x6			
A	19.25	in^2	OK
Check deflection			
I=(bh^3/12)+Ad^2	11137	in^3	
P	4000	#	
L	288	in	
E	433727	psi	
defl	0.4	in	
L/240	1.2	in	OK

Project:	Same Polytechnic		
Date:	3/15/2019		
By:	EMM		
Wind Loads Timber			
G	0.85		(gust effect factor)
qz	32.23	psf	(velocity pressure)
Net Pressure Coefficients			
Zone 3	2.4	-3.3	
Zone 2	1.8	-1.7	
Zone 1	1.2	-1.1	
Wind Pressure (psf)		*ASD	0.6
Zone 3	65.7	-90.4	
Zone 2	49.3	-46.6	
Zone 1	32.9	-30.1	
D (psf)	8		
Lr (psf)	20		
D+.6W			
Zone 3	47.4	psf	39.44952
Zone 2	37.6	psf	
Zone 1	27.7	psf	
D+.75L+.75Lr+.75(.6W)			
Zone 3	52.6	psf	
Zone 2	45.2	psf	
Zone 1	37.8	psf	
.6D+.6W			
Zone 3	-49.4	psf	
Zone 2	-23.1	psf	
Zone 1	-13.3	psf	

Project:	Same Polytechnic
Date:	3/15/2019
By:	EMM

Lateral Check for Wind - Timber

Downward Force

B1 - Zone 3			*B1 - Zone 2*		*B1 - Zone 1*	
trib width	4.00	ft	4.00	ft	4.0	ft
length	10.5	ft	10.5	ft	10.5	ft
wind load	47.4	psf	37.6	psf	27.7	psf
distributed load	189.8	plf	150.3	plf	110.9	plf
Vmax	996.4	#	789.3	#	582.2	#
Mmax	2615.7	#-ft	2072.0	#-ft	1528.3	#-ft

B2 - Zone 3			*B2 - Zone 2*		*B2 - Zone 1*	
trib width	12.00	ft	12.00	ft	12.0	ft
length	24.0	ft	24.0	ft	24.0	ft
wind load	47.4	psf	37.6	psf	27.7	psf
distributed load	569.4	plf	451.0	plf	332.7	plf
Vmax	6832.7	#	5412.5	#	3992.4	#
Mmax	40996.4	#-ft	32475.3	#-ft	23954.2	#-ft

G1 - Zone 3			*G1 - Zone 2*		*G1 - Zone 1*	
trib width	24.00	ft	24.00	ft	24.0	ft
length	21.0	ft	21.0	ft	21.0	ft
wind load	47.4	psf	37.6	psf	27.7	psf
distributed load	1138.8	plf	902.1	plf	665.4	plf
Vmax	11957.3	#	9472.0	#	6986.6	#
Mmax	62775.7	#-ft	49727.8	#-ft	36679.9	#-ft

Upward Force

B1 - Zone 3			*B1 - Zone 2*		*B1 - Zone 1*	
trib width	4.00	ft	4.00	ft	4.00	ft
length	10.5	ft	10.5	ft	10.5	ft
wind load	-49.4	psf	-23.1	psf	-13.3	psf
distributed load	-197.8	plf	-92.6	plf	-53.1	plf
Vmax	-1038.3	#	-486.0	#	-278.9	#
Mmax	-2725.6	#-ft	-1275.8	#-ft	-732.1	#-ft

B2 - Zone 3			*B2 - Zone 2*		*B2 - Zone 1*	
trib width	12.00	ft	12.00	ft	12.00	ft
length	24.0	ft	24.0	ft	24.0	ft
wind load	-49.4	psf	-23.1	psf	-13.3	psf
distributed load	-593.3	plf	-277.7	plf	-159.4	plf
Vmax	-7119.8	#	-3332.7	#	-1912.5	#
Mmax	-42718.8	#-ft	-19995.9	#-ft	-11474.8	#-ft

G1 - Zone 3			*G1 - Zone 2*		*G1 - Zone 1*	
trib width	24.00	ft	24.00	ft	24.00	ft
length	21.0	ft	21.0	ft	21.0	ft
wind load	-49.4	psf	-23.1	psf	-13.3	psf
distributed load	-1186.6	plf	-555.4	plf	-318.7	plf
Vmax	-12459.7	#	-5832.1	#	-3346.8	#
Mmax	-65413.2	#-ft	-30618.7	#-ft	-17570.8	#-ft

Project:	Same Polytechnic				
Date:	3/15/2019				
By:	EMM				
Check Members for Wind					
Fy	50	ksi			
Etimber	1040944	psi			
B1					
I (2x8)	47.6	in^4			
L	10.5	ft			
S	13.1	in^3			
Downward Force					
L/240	0.5	in			
wdist1	189.8	plf			
.42*wdist	79.72	plf			
Ix	39.89	in^4	<allow OK		
Check Bending					
fb=M/S=	2388.725836	psi			
F'b	2018.410961	psi	F'b>fb OK		
Uplift					
wdist	-197.8	plf			
.42*wdist	-83.0643912	plf			
Ix	-41.56884245	in^4	<allow OK		
Check Bending					
fb=M/S=	-2489.087065	psi			
F'b	2018.410961	psi	F'b>fb OK	d/c	-1.23
B2					
I (3.125x19.5)	1931.0	in^4			
L	24	ft			
S	198.0	in^3			
Downward Force					
L/240	1.2	in			
wdist1	569.4	plf			
.42*wdist	239.15	plf			
Ix	1429.16	in^4	<allow OK		
Check Bending					
Zone 1					
fb=M/S=	2484.629411	psi			
F'b	1576.480704	psi	F'b<fb NG		
TRY GLB 3.125x22.5					
I	2966	in^4			
S	263.7	in^3			
fb=M/S=	1865.592049	psi	F'b>fb OK	d/c	1.18
Uplift					
wdist	-593.3	plf			
.42*wdist	-249.19	plf			
Ix	-1489.206811	in^4	<allow OK		
Check Bending					
fb=M/S=	-1943.974051	psi			
F'b	1576.480704	psi	F'b<fb NG		
Truss					
I=(bh^3/12)+Ad^	11137	in^3			
L	288	in			
Ix	0.172	in^4	<allow OK		

Project:	Same Polytechnic	
Date:	3/15/2019	
By:	EMM	

Column Design				
h	288.0	in		
KL=	144.0	in	(Braced at half column height)	
At	504.0	ft^2		
P=At*(D+Lr)	11.0	k/column	(see Gravity Demands)	
Try 6x6 Post				
b	5.5			
d	5.5			
Fc	2137.5	psi		
Cf	1.1			
Emin	765400.0	psi		
A	30.3	in^2		
le/d=KL/d	26.2			
E'min	765400.0	psi		
Cd	1.3			
Cm	1.0			
Ct	1.0			
CT	1.0			
Ci	1.0			
FcE	917.8	psi		
Fc*	2671.8			
FcE/Fc*	0.3			
1+FcE/Fc*/2c	0.8			
Cp	0.3			
F'c	924.7	psi		
P=F'c*A	28.0	k	Allowable	
P=	11.0	k/column	demand	OK
Try 8x8 Post				
b	7.5			
d	7.5			
Fc	2137.5	psi		
Cf	1.1			
Emin	765400.0	psi		
A	56.3	in^2		
le/d=KL/d	3.7			
E'min	765400.0	psi		
Cd	1.3			
Cm	1.0			
Ct	1.0			
CT	1.0			
Ci	1.0			
FcE	45229.7	psi		
Fc*	2671.8			
FcE/Fc*	16.9			
1+FcE/Fc*/2c	11.2			
Cp	1.0			
F'c	2903.0	psi		
P=F'c*A	163.3	k	Allowable	
P=	11.0	k/column	demand	OK
Use 6x6 post				

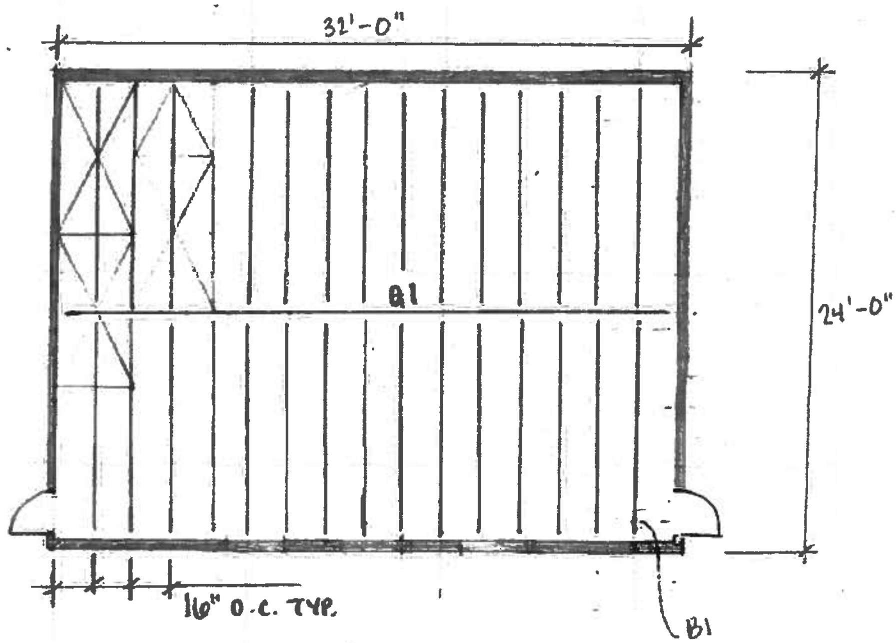
Check Wind			
Trib width	24.0	ft	
Wind pressure	29.6	psf	
Ww	710.1	plf	
M	7362227.2	#-in	
S (6x6)	27.7	in^3	
P	11047.7	k	
A	30.3	in^2	
Cd	1.6	(Wind/Earthquake)	
fc=P/A	365.2	psi	
F'c	924.7	psi	
fb=M/S	265784.4	psi	
F'b	1872.1	psi	
Bending + Compression	142.4	>1.0	N.G.
M	1840556.8	#-in	braced at 1/2 height (12')
S (8x8)	70.3	in^3	
P	11047.7	k	
A	56.3	in^2	
Cd	1.6	(Wind/Earthquake)	
fc	196.4		
fb	26181.5		
F'c	924.7		
F'b	1872.1		
Bending + Compression	14.2	>1.0	N.G.

Project:	Same Polytechnic	
Date:	3/15/2019	
By:	EMM	
Footing Design - Timber		
Pu	11.05	k
Pdl	4.03	#
Pll	7.02	#
1.2Pdl+1.6Pll	4.85	k
Fbearing	2	ksf
Areq	6	ft^2
L1	4	ft
L2	4	ft
Af	16	ft^2
8x8 post		
x=y	0.625	ft
c	1.6875	ft
d	2.04	in
h	6	in
cover	4	in
b	48.00	in
As	0.52	in^2
Use 2 #4 bars ea. way		
As	0.4	in^2
Load to each col		
Weight	4032	#
.7V=.7*Cs*W	1349.1072	#
h	24	ft
M	32378.5728	#-ft
e	2.9	ft
L/6	8.0	e<L/6
L/3	1.333333333	
x	-0.9	
fb	0.92064	psf
USE 3' x 3' x 14" deep foundation		

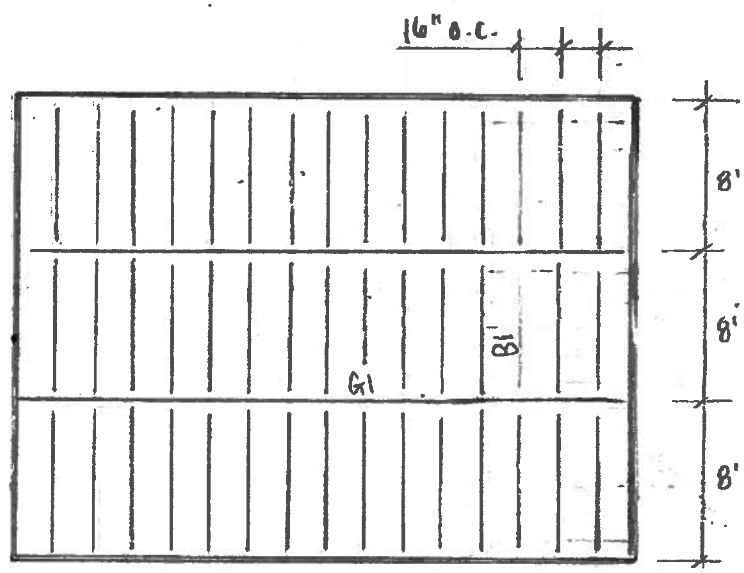
BY ________ DATE ________ SUBJECT ________ SHEET NO. M-1 OF ________

CHKD. BY ________ DATE ________ ________ JOB NO. ________

SUBSTRUCTURE FRAMING PLAN I.



SUBSTRUCTURE FRAMING PLAN II.

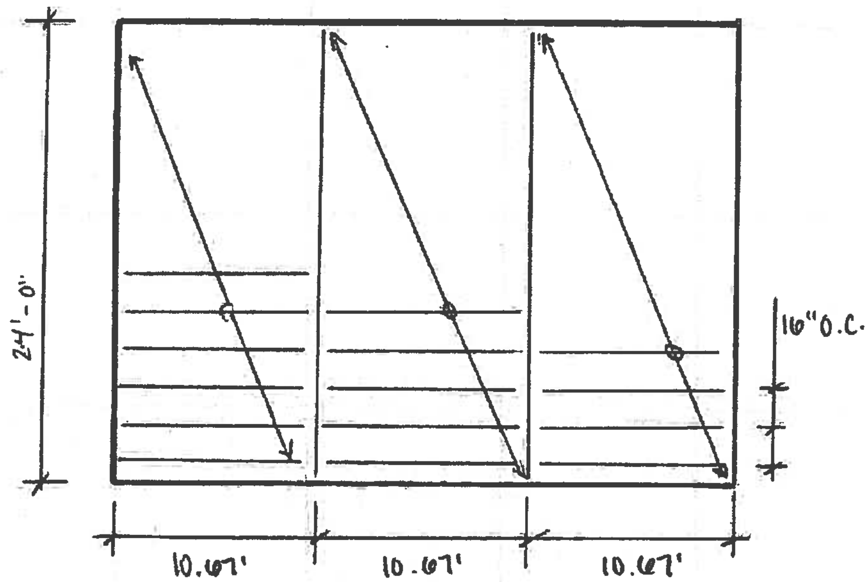


BIGGS CARDOSA ASSOCIATES INC.

BY ______ DATE ______ SUBJECT ______ SHEET NO. M-2 OF ______

CHKD. BY ______ DATE ______ ______ JOB NO. ______

SUBSTRUCTURE III FRAMING



Project:	Same Polytechnic	
Date:	3/15/2019	
By:	EMM	

Load Take-Off		
	Roof	psf
	3/8" Ply	1.1
	2x joists	2.6
	Girder	0.9
	Misc.	2
	MEP	2
	Typical Roof Dead Load:	9
	Live Load (Reducible):	50

Project:	Same Polytechnic			
Date:	3/15/2019			
By:	EMM			
Gravity Demands				
B1	uniformly distributed load			
Trib width	1.66	ft		
DL	9	psf		
LL	50	psf	no reduction	
distributed load	97.94	plf		
Lbeam	10.67	ft		
Vmax	522.5099	#		
Mmax	1393.795158	#-ft		
G1	uniformly distributed load			
Trib width	10.67	ft		
DL	9	psf		
LL	47.196	psf	LL reduction	47.196
distributed load, w	599.61132	plf		
Lbeam	24	ft		
Vmax	7195.33584	#		
Mmax	3597.66792	#-ft		

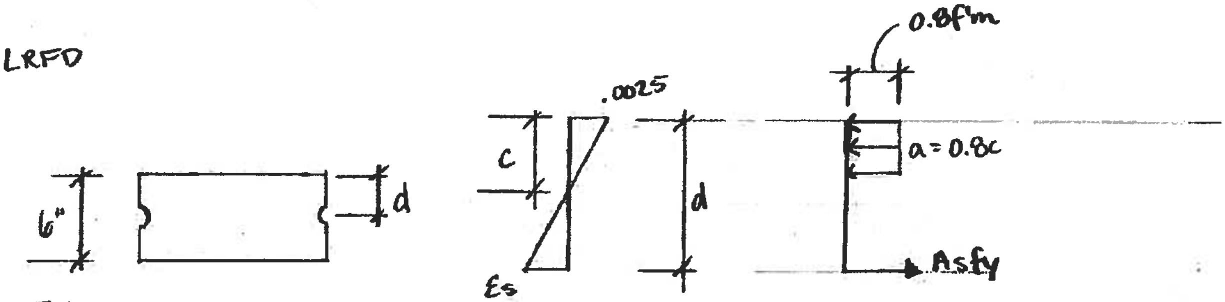
Project:	Same Polytechnic				
Date:	3/15/2019				
By:	EMM				
B1					
Vmax	522.5099	#			
Mmax	1393.795158	#-ft			
Check Deflection - Solve for Iallow					
Lbeam	128.04	in			
distributed load w	8.2	#/in			
Iallow=5wl^4/384	51.4	in^4			
Try 2x8					
I (2x8)	47.63	in^4			
wL	6.9	#/in			
Lallow=l/360	0.3556666667	in			
Lactual	0.49	in	Lallow>Lactual	OK	
D+Lallowable=l/2	0.53	in			
D+Lactual	0.58	in	allow>actual	OK	
Check Bending					
Cd	*1.25*	*Cf*	*1.2*		
Cm	*1*	*Cfu*	*1*		
Ct	*1*	*Ci*	*1*		
Cl	*1*	*Cr*	*1.15*		
S	13.14	in^3			
Fb'=Fb*Cd*Cf*Ci	2018	psi			
fb=M/S	1273	psi	Fb'>fb	OK	
Check Shear					
fv=1.5(V/bd)	72	psi			
Fv'=Fv*Cd	953	psi	Fv'>fv	OK	

G1				
Vmax	7195.33584	#		
Mmax	3597.66792	#-ft		
Check Deflection - Solve for Iallow				
Lbeam	288.0	in		
distributed load wD+L	49.97	#/in		
D+Lallowable=l/240	1.2	in		
(5wl^4/384E*defl)	3583.3	in^4		
Iallow	3583.3	in^4		
Try 3.125x19.5 GLB				
I (3.125x19.5)	1931	in^4		
wL	42.0	#/in		
Lallow=L/360	0.8	in		
Lactual-dist	1.87	in	allow>actual	OK
D+Lallowable=l/240	1.20	in		
D+Lactual-allow	2.23	in	allow>actual	OK
Check Bending				
Cd	*1.25*	*Cf*	*0.95*	
Cm	*1*	*Cfu*	*1*	
Ct	*1*	*Ci*	*1*	
Cl	*1*	*Cr*	*1.15*	
S	198	in^3		
Fb'=Fb*Cd*Cf*Cr	2043	psi		
fb=M/S	218	psi	Fb'>fb	OK
Check Shear				
fv=1.5(V/bd)	53	psi		
Fv'=Fv*Cd	953	psi	Fv'>fv	OK

Designer:	EMM		
Date:	3/15/2019		
Project:	Submittal #4		
Footing Design - Masonry			
Dimensions:			
Masonry Weight	110	pcf	
Concrete Weight	150	pcf	
Depth	1	ft	
Width	2	ft	
Length	24	ft	
wall length	24	ft	
wall thickness	0.5	ft	
wall height	10.5	ft	
S.O.G. thickness	0.333	ft	
Wall Self Wt	13860	#	
Footing Wt	7200	#	
Total Wt	21.06	k	
Loading (LRFD):			
Moment	0.310	k-ft	(Seismic Demand from Masonry Design)
Pu	19	k	
Mu	0.31	k-ft	
Eccentricity:			
e=M/P	0.016	ft	
L/6	4.00	ft	
	(Inside Kern)		
P/A	0.395	ksf	
S=bh^2/6	16.0	ft^3	
M/S	0.019	ksf	
qu-max	0.414	ksf	
qu-min	0.376	ksf	
qallow	2	ksf	d/c 0.21
Asmin=.0018bh	0.5184	in^2	
Use 3-#5 bars			
As	0.93	in^2	

USE 2' WIDE, 12" DEEP CONTINUOUS FOUNDATION AT MASONRY WALLS

LRFD



CAPACITY

T = C

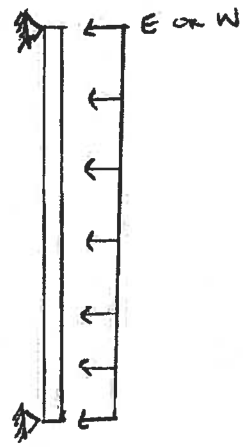
$A_s f_y = .80 f'_m b a$

$a = \frac{A_s f_y}{.8 f'_m b} = \frac{0.11 in^2 \cdot 60 ksi}{0.8(310 psi)(18")} = 1.47"$

$c = a/.8 = 1.85"$

$\varepsilon_s = \varepsilon_{cu}\left(\frac{d-c}{c}\right) = .002\left(\frac{3-1.85}{1.85}\right) = .0012$ OK

$M_n = T(d - a/2) = 6.6 k \left(3 - \frac{1.47}{2}\right) = 14.95$ k·in = 1.25 k·ft



DEMAND

$F_p = 0.4 S_{DS} I_e W_p$ $S_{DS} = 0.573$ $I_e = 1.25$

$W_p = 110 PCF \times 6"/12 = 55 PSF$

$= 0.4(.573)(1.25)(55 PSF) = 15.76 PSF$

$F_{PMIN} = 0.1 W_p = 5.5 PSF$ (INFILL HT.)

$M_u = \frac{F_p l^2}{8} = \frac{15.76 PSF (10.5')^2}{8} = 217$ #·ft × 1.43 = 310 #·ft (LRFD)

$A_{s,req} = \frac{M_u}{\phi f_y (d - a/2)} = \frac{310 \# \cdot ft}{0.9(60,000)(.95d)} = .002 in^2/ft \times 3' = .00067 in^2/3'$

#3 SPACED @ 36" O.C. = .00067

WIND CHECK

WIND LOADS → ENCLOSED, FLAT ROOF

$K_{zt} = 1.0$ $\lambda = 1.21$

$P_{S30} = 12.7 PSF$ (ZONED - WALLS)

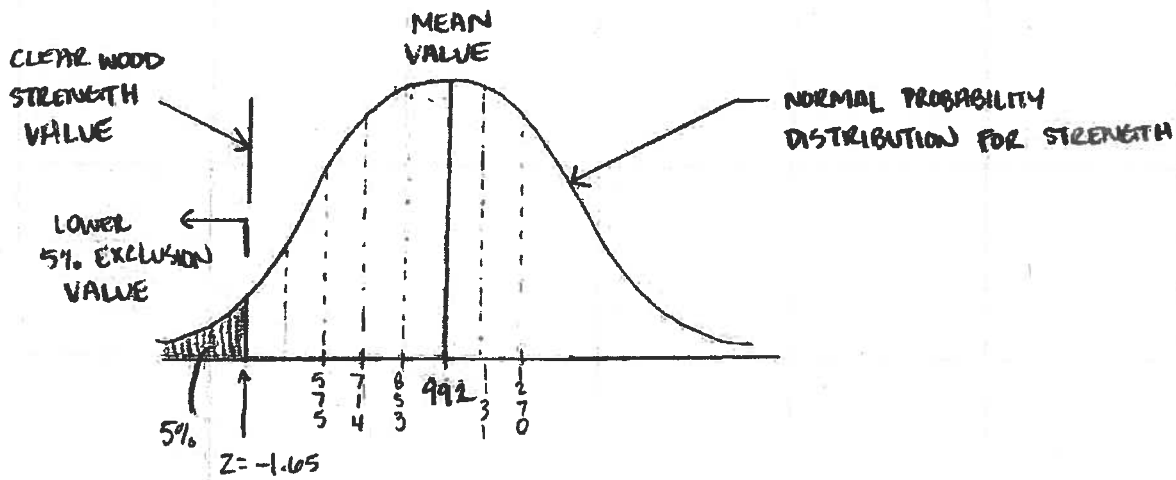
$P_S = K_{zt} \times \lambda \times P_{S30} = 15.4 PSF$

ASD → 0.6W = 0.6(15.4 PSF) = 9.24 PSF < 15.76 PSF

SEISMIC GOVERNS

BY EMM DATE SUBJECT SHEET NO. A-1 OF

CHKD. BY DATE JOB NO.



Avg. Compression strength = 992 psi

Std. Deviation = 139 psi

Z-score$_{.05}$ = −1.65 (.0495) STD. DEV. BELOW MEAN

992 − 1.65(139) = 762.65

5% PERCENTILE = 762.65 psi

BIGGS CARDOSA ASSOCIATES INC.

Conclusion

To provide the most cost-effective and constructable solution, a timber truss system or steel framing should be utilized. Building trusses would use more labor, which may be more cost effective due to low labor costs in the area. Smaller members would be used for the truss compared to the hinged girders, making the materials cheaper.

If Glu-Lam beams are available in the area, it may be more efficient to use this option, however, it should be considered that it is easier to replace a truss system than a Glu-lam beam, since a truss system can be built by hand while a Glu-lam beam would have to be ordered from a nearby city.

Steel framing may be more expensive due to local availability, and shipping costs may increase the price. Additionally, it may be difficult to find a skilled welder for the steel connections. However, steel would have the longest lifespan of the materials, so less maintenance would be needed.

For the full structural design, a few more aspects should be considered. The preliminary design does not include openings for the courtyard spaces. It also does not consider openings in the masonry substructures. Connections should also be further studied based on availability of hardware in the field.