STRUCTURAL NOTES

A. BASIS OF DESIGN

1. THE STRUCTURAL DESIGN HAS BEEN PERFORMED IN ACCORDANCE WITH THE 2010 CALIFORNIA BUILDING CODE (C.B.C.)

2. DESIGN LOADS (LIVE LOADS MAY BE REDUCED IN ACCORDANCE WITH THE BUILDING CODE.)

	DL	LL
FLAT ROOF	27 psf	20 psf
PITCHED ROOF (REMODEL AREA)	21 psf	20 psf
PITCHED ROOF (ORIGINAL AREA)	19 psf	20 psf

3. SEISMIC DESIGN DATA

OCCUPANCY CATEGORY	II
SEISMIC IMPORTANCE FACTOR	Ip = 1.0
MAPPED SPECTRAL ACCELERATION	Ss = 256%g
SITE CLASS	D
SITE COEFFICIENT	Fa = 1.0
DESIGN SPECTRAL ACCELERATION	Sds = 170%g
SEISMIC DESIGN CATEGORY	D
ANALYSIS PROCEDURE USED	NONSTRUCTURAL COMPONENTS

4. COMPONENT SEISMIC DESIGN DATA

COMPONENT	ap	Rp
ROOFTOP SOLAR	1.0	2.5

5. WIND DESIGN DATA

BASIC WIND SPEED	V = 85 mph
WIND IMPORTANCE FACTOR	Iw = 1.0
EXPOSURE CATEGORY	B
ENCLOSURE CATEGORY	ENCLOSED
GUST & INTERNAL PRESSURE COEFF	GCp1 = 0.18
DIRECTIONALLY FACTOR	Kd = 0.85
TOPOGRAPHIC FACTOR	K1 = 1.00

B. GENERAL:

1. SPECIFIC NOTES AND DETAILS ON THE STRUCTURAL DRAWINGS SHALL TAKE PRECEDENCE OVER GENERAL NOTES AND TYPICAL DETAILS.

2. WHERE NO DETAILS ARE SHOWN OR NOTED IN ANY PART OF THE WORK THE DETAILS FOR OTHER SIMILAR WORK SHALL APPLY.

3. DETAILS IDENTIFIED AS TYPICAL, SHALL APPLY IN ESTIMATING AND CONSTRUCTION TO EVERY LIKE CONDITION WHETHER OR NOT THE REFERENCE IS REPEATED.

4. THE STRUCTURAL DRAWINGS SHALL NOT BE SCALED. COORDINATE DIMENSIONS WITH MECHANICAL DRAWINGS.

5. STANDARDS REFERENCED ON THE STRUCTURAL DRAWINGS REFER TO THE EDITION APPLICABLE UNDER THE CURRENT BUILDING CODE.

6. THE RESPONSIBILITY FOR THE REVIEW AND COORDINATION OF DRAWINGS AND SPECIFICATIONS PRIOR TO THE START OF RELATED CONSTRUCTION SHALL BEAR ON THE CONTRACTOR. DISCREPANCIES THAT EXIST SHALL BE BROUGHT TO THE ATTENTION OF THE ENGINEER IN A TIMELY MANNER, PRIOR TO START OF RELATED CONSTRUCTION.

7. WORK PERFORMED IN CONFLICT WITH THE STRUCTURAL DRAWINGS OR APPLICABLE BUILDING CODE REQUIREMENTS SHALL BE CORRECTED AT THE EXPENSE OF THE CONTRACTOR.

8. DIMENSIONS, ELEVATIONS, AND SITE CONDITIONS SHALL BE VERIFIEBEFORE STARTING RELATED WORK AND THE ENGINEER NOTIFIED ODISCREPANCIES IN A TIMELY MANNER.

9. MATERIALS AND WORKMANSHIP SHALL CONFORM TO REQUIREMENTS OIAPPLICABLE REGULATIONS AND THE BUILDING CODE AS AMENDED AND ADOPTED BY THE BUILDING OFFICIAL.

C. TEMPORARY WORK AND SITE SAFETY:

1. THE STRUCTURAL DRAWINGS SHOW THE REQUIREMENTS FOR THE COMPLETED STRUCTURE ONLY. TEMPORARY WORKS REQUIRED TO COMPLETE THE CONSTRUCTION PROCESS SHALL BE THE RESPONSIBILITY OF THE CONTRACTOR. THE STRUCTURAL ENGINEER SHALL NOT BE RESPONSIBLE FOR THE DESIGN OR FIELD VERIFICATION OF TEMPORARY AND ANCILLARY WORK.

2. THE RESPONSIBILITY FOR SAFETY IN AND AROUND THE JOBSITE SHALL BEAR ON THE CONTRACTOR. PROPER AND SAFE METHODS OF CONSTRUCTION SHALL BE EMPLOYED AT ALL TIMES INCLUDING THE STABILIZING OF INCOMPLETE STRUCTURES, FORMWORK, SHORING, RESHORING, FALSEWORK, PLATFORMS, SCAFFOLDING, BARRIERS, WALKWAYS, ETC. AND INCLUDING CONTROL OF THE INTENSITY, DURATION AND LOCATION OF CONSTRUCTION LOADS.

D. POST-INSTALLED ADHESIVE ANCHORS:

1. SPECIAL INSPECTION AND TESTING IS REQUIRED IN ACCORDANCE WITH SECTIONS 1704, 1707 AND 1708 OF THE BUILDING CODE AND THE "STATEMENT OF SPECIAL INSPECTIONS" ON THESE CONSTRUCTION DOCUMENTS.

2. ADHESIVE ANCHOR INSTALLERS SHALL BE TRAINED BY A QUALIFIED REPRESENTATIVE OF THE ADHESIVE MANUFACTURER ON THE PROPER PROCEDURES AND TECHNIQUES FOR INSTALLATION.

3. ADHESIVE SHALL BE STORED ON THE JOBSITE IN A COOL, DRY LOCATION IN CONFORMANCE WITH THE MANUFACTURER'S REQUIREMENTS.

4. ADHESIVE ANCHORS FOR INSTALLATION IN SOLID CONCRETE SHALL BE HILTI HIT RE500-SD (ICC #ESR-2322). ALTERNATE PRODUCTS SHALL CARRY AN ICC APPROVAL THAT IS BASED ON TESTING IN ACCORDANCE WITH ICC ACCEPTANCE CRITERIA AC308 AND SHALL BE APPROVED BY THE ENGINEER IN WRITING PRIOR TO DELIVERY TO THE JOBSITE.

5. ADHESIVE ANCHORS FOR INSTALLATION IN CONCRETE MASONRY SHALL BE HILTI HIT HY150 MAX (ICC #ESR-1967). ALTERNATE PRODUCTS SHALL CARRY AN ICC APPROVAL THAT IS BASED ON TESTING IN ACCORDANCE WITH ICC ACCEPTANCE CRITERIA AC58 AND SHALL BE APPROVED BY THE ENGINEER IN WRITING PRIOR TO DELIVERY TO THE JOBSITE.

6. ANCHORS SHALL BE OF THE TYPE, DIAMETER, AND MINIMUM DIMENSIONAL REQUIREMENTS (EMBEDMENT, SPACING AND EDGE DISTANCE) AS INDICATED ON THE DRAWINGS.

7. ANCHORS SHALL BE INSTALLED IN HOLES DRILLED WITH DRILLING EQUIPMENT OF THE TYPE REQUIRED IN THE MANUFACTURER'S PUBLISHED ICC REPORT. HOLES SHALL BE CLEANED IN CONFORMANCE WITH THE ANCHOR MANUFACTURER'S INSTRUCTIONS.

8. WHEN INSTALLING ANCHORS IN EXISTING REINFORCED CONCRETE OR MASONRY, AVOID CUTTING OR DAMAGING THE EXISTING REINFORCING BARS.

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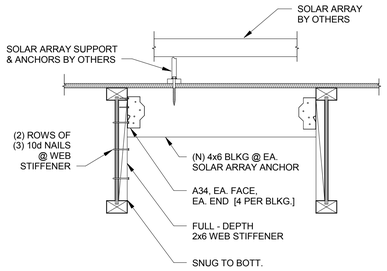
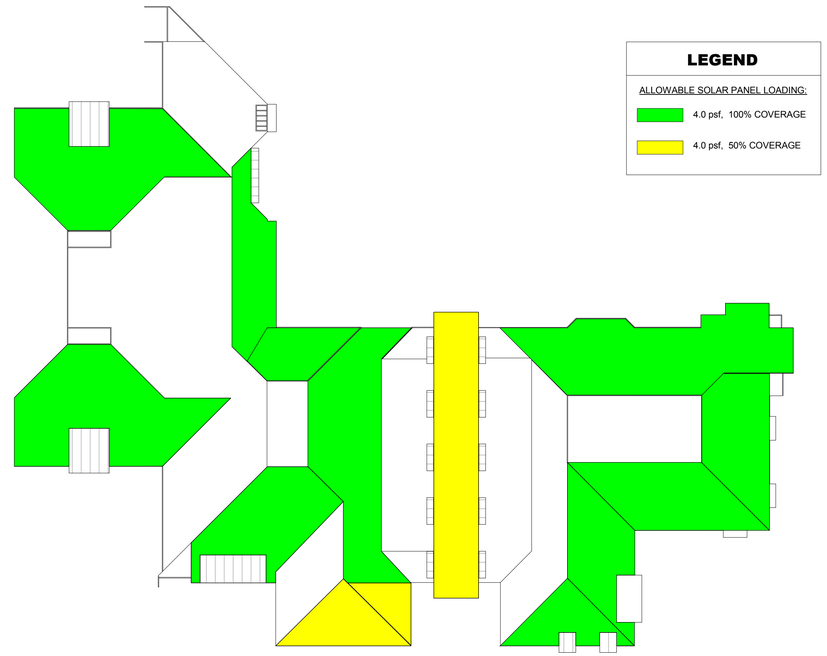
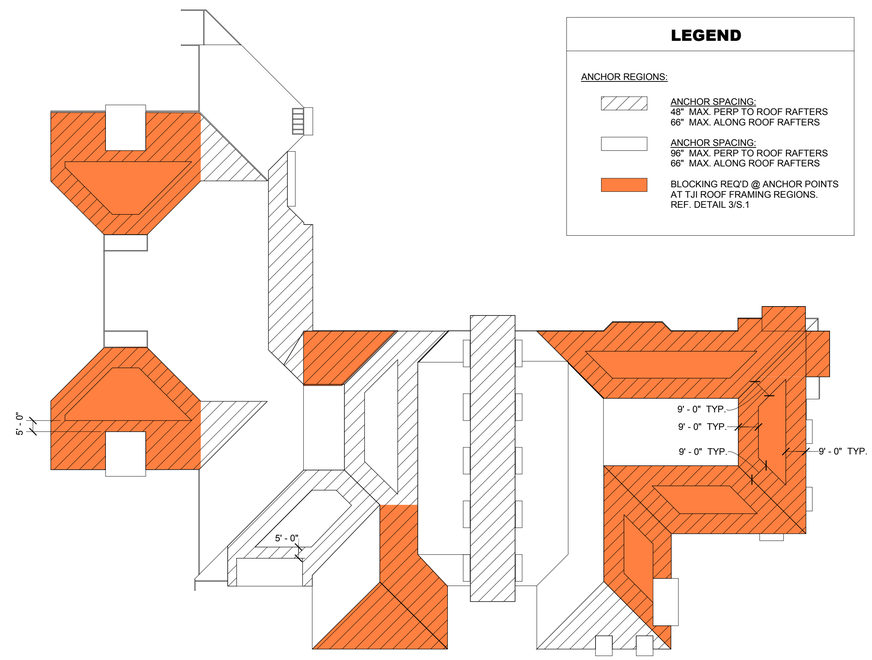


Anchor Detail @ TJI 3
1 1/2" = 1'-0"



Roof Top Solar Arrays - Allowable Regions of Placement 1
1/32" = 1'-0"



Anchor Regions 2
1/32" = 1'-0"

ENGINEER:

TKJ structural engineering
1330 orange avenue
suite 301
coronado, ca 92118
(p) 619.869.6234
(e) tkj@tkjse.com
www.tkjse.com

PROJECT:

Saratoga Community Library: Rooftop Solar Panels

13650 Saratoga Ave.
Saratoga, CA 95070

REVISIONS:

No.	Description	Date

SHEET TITLE:

Structural

NORTH ARROW:

TRUE NORTH

STAMP:

REGISTERED PROFESSIONAL ENGINEER TIMOTHY K. JENDRES S 4845 EXP. 6-30-13 STRUCTURAL STATE OF CALIFORNIA

ISSUE DATE:	12/20/11
DRAWN BY:	BAJ
CHECKED BY:	TKJ

SHEET #:

S.1

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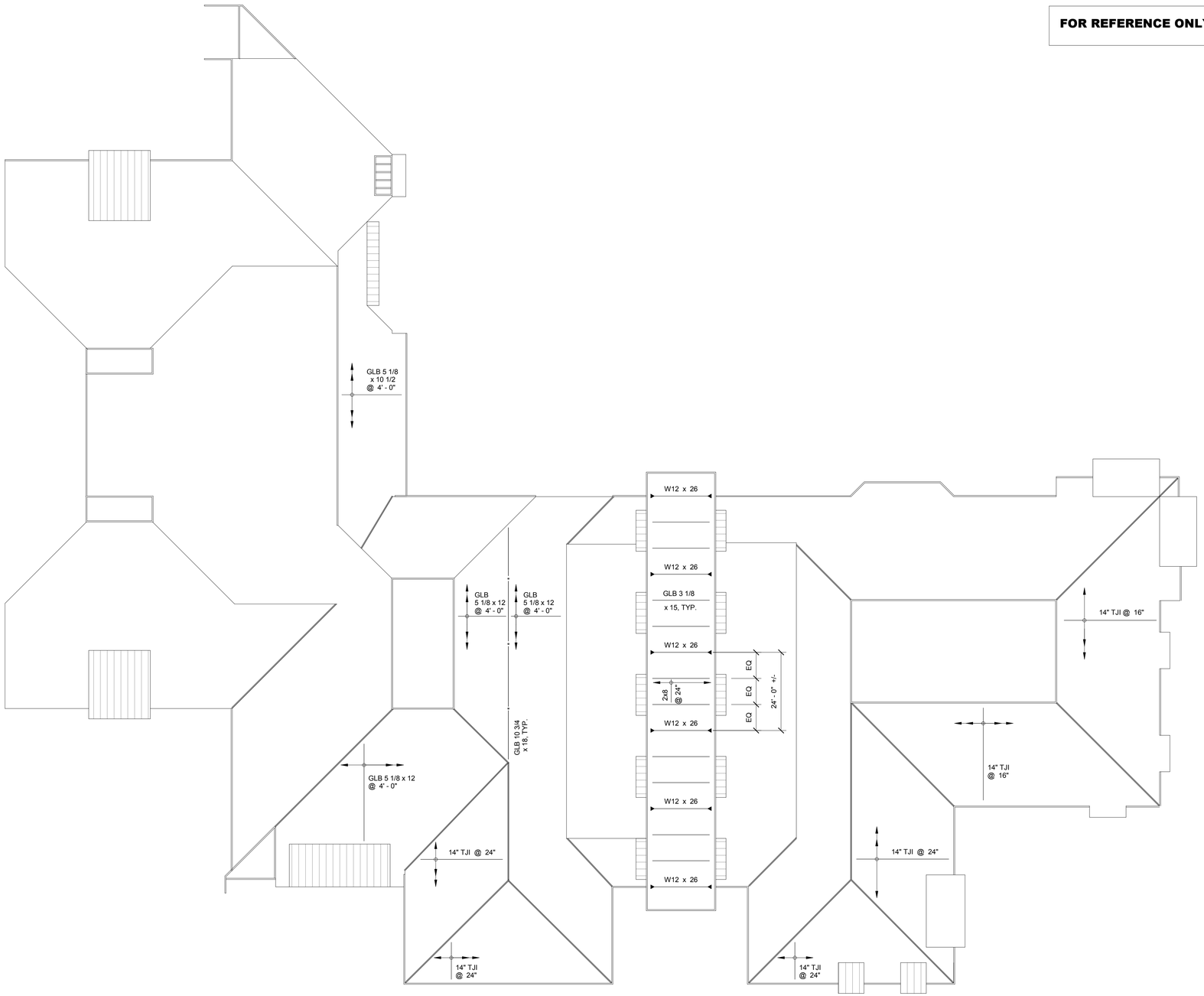

FOR REFERENCE ONLY

Partial Roof Framing Plan
1/16" = 1'-0"
1

ENGINEER:

TKJ structural engineering

1330 orange avenue
suite 301
coronado, ca 92118

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PROJECT:

Saratoga Community Library: Rooftop Solar Panels

13650 Saratoga Ave.
Saratoga, CA 95070

REVISIONS:

No.	Description	Date

SHEET TITLE:

Partial Roof Framing Plan

NORTH ARROW:

TRUE NORTH

STAMP:

REGISTERED PROFESSIONAL ENGINEER
TIMOTHY KYLE JAQUESS
S 4845
EXP 6-30-13
STRUCTURAL
STATE OF CALIFORNIA

ISSUE DATE:	12/20/11
DRAWN BY:	BAJ
CHECKED BY:	TKJ

SHEET #:

S.2

12/20/2011 3:46:30 PM

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STRUCTURAL CALCULATIONS

AND DETAILS

For:

City of Saratoga
Community Library
13650 Saratoga Ave.
Saratoga, CA 95070

Date: December 20, 2011

TKJSE Job No: 11040.01

PROJECT DESCRIPTION:

Client: **City of Saratoga**

Property: **Community Library**
13650 Saratoga Ave.
Saratoga, CA 95070

DESCRIPTION OF WORK:

The project consists solely of designing and evaluating the support and anchorage of the solar panel assemblies shown. The design or anchorage of all other components are the responsibility of others.

TABLE OF CONTENTS

Section	Pages		
Wind Loads	1	to	7
Check Roof Framing	8	to	18
Seismic Analysis for Additional Solar Array Mass	19	to	32

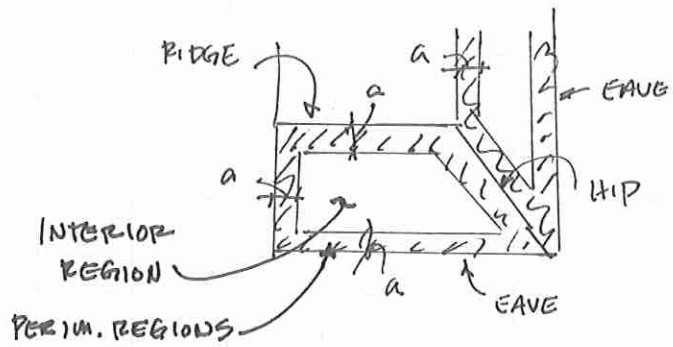
WIND LOADING

DESIGN PARAMETERS:

$V = 85$ mph

EXP. B

$K_{zt} = 1.0$



ROOF PLAN

a:

a min = 10% LEAST HORIZ. DIM : B = 88.75' → 10% B = 8.9' ← governs say 9'-0"
0.4h : h = 26' MAX → 0.4h = 10.4'

ANCHORAGE:

PERIMETER REGION:

$p = 23.5$ psf

$b = 65''$

$s = 48''$

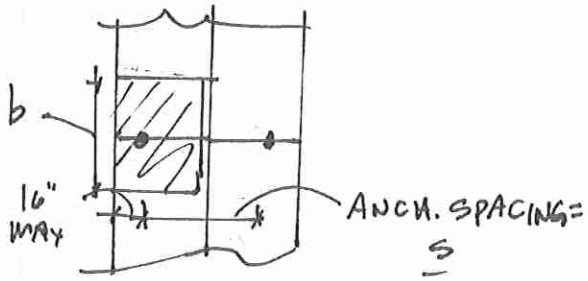
$W^- = 508^{\#} \uparrow$

$W^+ = 85^{\#} \downarrow$

$D = 60^{\#} \downarrow$

$0.6D + W^- = 472^{\#}$

$= 25^{\#} \rightarrow$ (shear)



ROOF SLOPE: 5:12

* 5/16" x 4" LAG w/ 2 1/2" MIN. PEN.

$R_v = 472^{\#}$ $\quad W'_p = \cancel{472^{\#}}\ 758^{\#}$

$R_H = 15^{\#}$ $\quad Z'_p = 188^{\#}$

$R = 472^{\#}$

$Z'_\alpha = 755^{\#} > 472^{\#}$ ∴ ok

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Project: SARATOGA LIBRARY

Project Number: 11040.01 Page: ____ of: ____

Prepared By: TKJ Date: 12-20-11

ANCHORAGE (cont.)

INTERIOR REGION:

$p = 8.5$ psf

$b = 65"$ [PORTRAIT]

$s = 96"$ o.c.

$W^- = 369^{\#} \uparrow$

$W^+ = 143^{\#} \downarrow$

$D = 120^{\#} \downarrow$

$= 50^{\#} \rightarrow$ (shear)

$\left.\right\} 0.6D + W^- = 297^{\#} \uparrow$

5/16" ϕ x 4" LAG w/ 2 1/2" MIN. PEN

$R_V = 297^{\#}$ $\quad W'_p = 758^{\#}$

$R_H = 30^{\#}$ $\quad Z'_p = 188^{\#}$

$R = 299^{\#}$

$\left.\right\} Z'_\alpha = 735^{\#} > 299^{\#} \therefore$ ok

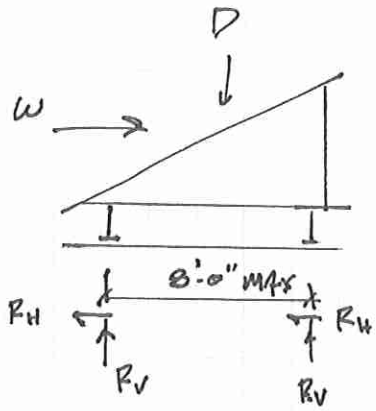
FLAT ROOF ZONES

$R_H = 305^{\#}$/ANCH

$R_V = 89^{\#}$ (UPLIFT)

$= 378^{\#}$ (DOWN)

	D	W
REACTIONS	181#	197#



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Project: SARATOGA LIBRARY

Project Number: 11040.01 Page: ____ of: ____

Prepared By: TKJ Date: 12-20-11

TKJ structural engineering

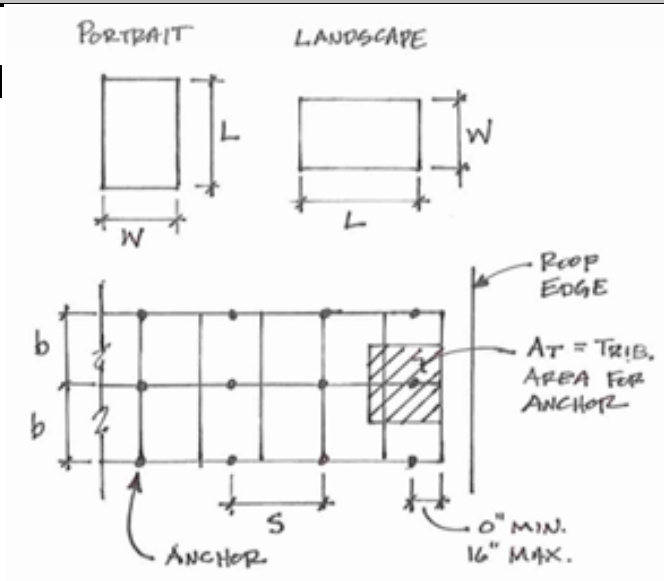
PITCHED ROOF PANEL LOADING

Project: **40"x65", 3.0 psf Solar Panels**
Job No.: **11040.01**
Date: **20-Dec-11**

Orientation & Loads

PANEL DATA

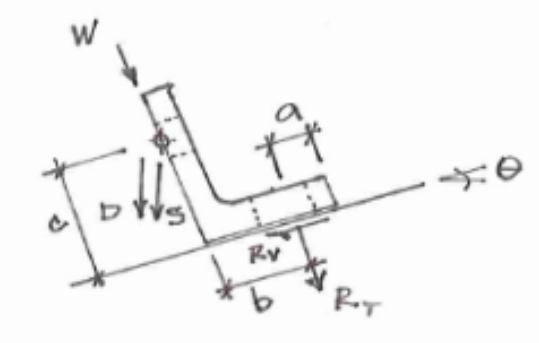
Panel Type =	40"x65", 3.0 psf Solar Panels
Panel Width = W =	40.0 in
Panel Length = L =	65.0 in
Panel Orientation =	Portrait
Tributary Width = b =	65.0 in
Panel Weight =	54.2 #/panel
Panel Weight =	3.00 psf
Anchor Spacing =	96.0 in
Anchor Location =	Interior
Tributary Area = A_t =	43.33 sf
DL per Anchor =	130.0 #



LOAD SUMMARY

Roof Slope			
Min. Slope	θ_{min}	Max. Slope	θ_{max}
5:12	22.62	5:12	22.62

	θ_{min}= 22.620		θ_{max}= 22.620	
	Horiz. (R_V)	Vertical (R_T)	Horiz. (R_V)	Vertical (R_T)
D	**50.0 lbs**	**120.0 lbs**	**50.0 lbs**	**120.0 lbs**
S	**0 lbs**	**0 lbs**	**0 lbs**	**0 lbs**
W^+	**0 lbs**	**143 lbs**	**0 lbs**	**143 lbs**
W^-	**0 lbs**	**-369 lbs**	**0 lbs**	**-369 lbs**



LOAD COMBINATIONS

			R_H	R_V	R	θ_r	C_D
	LC1a	D	50.0	120	130	67.38	**0.90**
	LC2a	D + S	50.0	120	130	67.38	**1.15**
θ_{min}	LC3a	$D + W^+$	50.0	263	268	79.24	**1.60**
	LC4a	$D + 0.75W^+ + 0.75S$	50.0	227	233	77.60	**1.60**
	LC5a	$0.6D + W^-$	30.0	-297	299	-84.24	**1.60**
	LC1b	D	50.0	120	130	67.38	0.90
	LC2b	D + S	50.0	120	130	67.38	1.15
θ_{max}	LC3b	$D + W^+$	50.0	263	268	79.24	1.60
	LC4b	$D + 0.75W^+ + 0.75S$	50.0	227	233	77.60	1.60
	LC5b	$0.6D + W^-$	30.0	-297	299	-84.24	1.60

TKJ structural engineering

PITCHED ROOF PANEL LOADING

Project: **40"x65", 3.0 psf Solar Panels**
Job No.: **11040.01**
Date: **20-Dec-11**

Wind & Snow Loading

WIND LOADING CRITERIA

Basic Wind Speed = V =	85 mph		
Wind Exposure Category =	B		
Roof Zone =	1		
Roof Slope =	5:12	to	5:12
Tributary Area = A_t =	43.3 sf		

Min. Slope	θ_{min}	Max. Slope	θ_{max}
5:12	22.620	5:12	22.620

$p = q_h[(GC_p) - (GC_{pi})]$, where

$q_h = 0.00256\ K_z\ K_{zt}\ K_d\ V^2\ I$, where

K_z = 0.67

K_{zt} = 1.00 [Assumed]

K_d = 0.85

I = 1.00

q_h = 10.57 psf

GC_{pi} = 0 [Internal pressure does not apply]

	θ_{min}= 22.620				θ_{max}= 22.620			
	Zone 1 GCp^-	**Zone 2 GCp^-**	**Zone 3 GCp^-**	**All Zones GCp^+**	**Zone 1 GCp^-**	**Zone 2 GCp^-**	**Zone 3 GCp^-**	**All Zones GCp^+**
Fig. 6-11C: Gable/Hip Roofs θ<=27°	-0.81	-1.23	-2.04	0.31	-0.81	-1.23	-2.04	0.31
Fig. 6-11D: Gable Roofs 27°<θ								
	-0.81			0.31	-0.81			0.31

	θ_{min}		θ_{max}	
	GC_p^-	GC_p^+	GC_p^-	GC_p^+
p =	-8.5 psf	3.3 psf	-8.5 psf	3.3 psf
W =	**-369 #**	**143 #**	**-369 #**	**143 #**

TKJ structural engineering

Project:	**Community Library**
Job No.:	**11040.01**
Date:	**12/20/11**

ROOFTOP SOLAR RACK ASSEMBLY DESIGN: LOADING

FLAT ROOF PANEL LOADING

PANEL ASSEMBLY DEAD LOAD TAKE-OFF:

Panel Support Frame =	**1.0 psf**
Panels, collectors =	**3.0 psf**
Misc. =	**1.0 psf**
Total =	5.0 psf

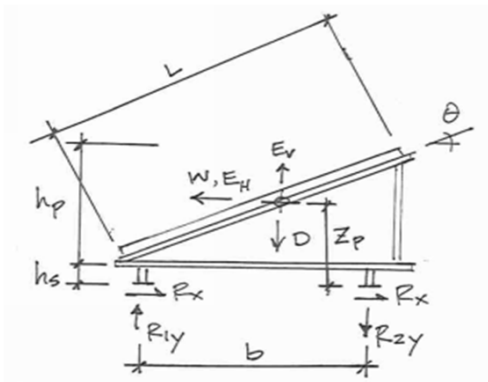
WIND LOADING: ASCE 7-05, SEC. 6.5.15, 6.5.8, TABLE 6-2, TABLE 6-4 & FIG. 6-21

CALCULATIONS:			SOURCES/MESSAGES:
Exposure:	**B**		
h_r (ft) =	**26.00 ft**		[Ht. to roof]
L (ft) =	**10.83 ft**		
θ° =	**22.62**	degrees	
z_g (ft) =	1200.0 ft		[Table 6-2]
α =	7.0		
z (centroid of panel) =	28.1 ft		15' < z < zg

$$q_z = 0.00256 k_z k_{zt} k_d V^2 I$$

[Sec. 6.5.10]

K_z =	0.687		[Table 6-3]
K_{zt} =	**1.0**		[Sec. 6.5.7.2]
K_d =	**0.85**		[Table 6-4]
V (mph) =	**85 mph**		[Fig. 6-1]
I =	**1.0**		[Table 6-1]
q_z (psf) =	10.81 psf		



$$F = q_z G C_f A_f C$$

G =	**0.85**	[6.5.8.1]		
h/D <	**1**	[Fig. 6-21]	[h=h_p, D=B=width of panel array]	
Cross Section =	**Square (wind normal to face)**			
C_f =	1.3			
B (ft) =	**15 ft**	1.9	[A_f < 0.1Bh]	[Sec. 6.5.15.1]
A_f (ft^2) =	62.50 sf			
C (interpolate) =	1.84	1.0	[A_f > Bh]	
F_w (A_f)=	22.0 A_f	[10 psf min.]		

TKJ structural engineering

Project:	**Community Library**
Job No.:	**11040.01**
Date:	**12/20/11**

ROOFTOP SOLAR RACK ASSEMBLY DESIGN: LOADING

SEISMIC LOADING: ASCE 7-05, SEC. 13.3 - SEISMIC DEMANDS ON NONSTRUCTURAL COMPONENTS

FLAT ROOF PANEL LOADING

CALCULATIONS:

$$F_P = \frac{0.4 a_P S_{DS} W_P}{\frac{R_P}{I_P}} (1 + 2\frac{z}{h})$$

SOURCES/MESSAGES:

			[Sec. 13.3.1]
z (ft) =	26.00 ft		[Height at point of attachment]
h (ft) =	26.00 ft		[Average roof height]
a_p =	**1.0**		[Table 13.5-1]
I_p =	**1.0**		[Sec. 13.1.3]
R_p =	**1.5**		[Table 13.5-1]
Zip Code =	**95070**		
S_s (%g) =	**246.0**	%g	[Sec. 11.4.1]
F_a =	**1.0**		[Table 11.4-1]
S_{DS} (%g) =	164.0	%g	[Sec. 11.4.4]
W_p (lbs) =	313 #		
F_p (A_f) =	6.56	A_f	**<-- GOVERNS**
$F_{pmax} = 1.6 S_{DS} I_P W_P$ =	13.12	A_f	
$F_{pmin} = 0.3 S_{DS} I_P W_P$ =	2.46	A_f	
WIND LOADING =	21.97	A_f	**<-- GOVERNS**
SEISMIC LOADING =	6.56	A_f	

TKJ structural engineering

Project:	**Community Library**
Job No.:	**11040.01**
Date:	**12/20/11**

ROOFTOP SOLAR RACK ASSEMBLY DESIGN: ANCHORAGE

SUPPORT REACTIONS:

D + W

0.6D + W

SOURCES/MESSAGES:

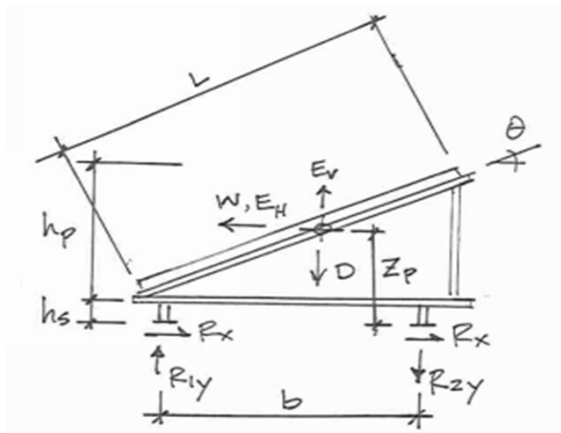
FLAT ROOF PANEL LOADING

ANCHORAGE CONNECTIONS:

CASE 1 - MAX UPLIFT: 0.6D + W

$\Sigma M_L = 0$:

s_a (ft) =	**6.67 ft**	o.c.	[Anchorage support spacing]
h_p (ft) =	4.17 ft		[Height of panel]
h_s (ft)	**0.5 ft**		
A_f (ft^2) = h_p*s_a =	27.78 sf		
F_W (#) =	610.4 #		
z_p (ft) = h_s + h_p/2 =	2.58 ft		
D (#) =	361.1 #		
b (ft) =	**8.0 ft**		



	D	W
R_{2y} (#) =	180.6 #	-197.1 #
0.6D + W (#) =	88.8 # uplift	

$\Sigma F_x = 0$:

R_x (#) = F_w/2 = **305.2 # shear**

CASE 2a - MAX DOWNWARD: D + W

$\Sigma M_R = 0$:

	D	W
R_{1y} (#) =	**180.6 #**	**197.1 #**
D + W (#) =	377.7 # down	

Cheek Framing

Case 1: GLB 5 1/8 x 10 @ 4'-0"

ROOF DL = 19 psf

$w_D = 19 \times 4 = 76$ plf $w_L = 80$ plf

$P_D = 180^{\#}$

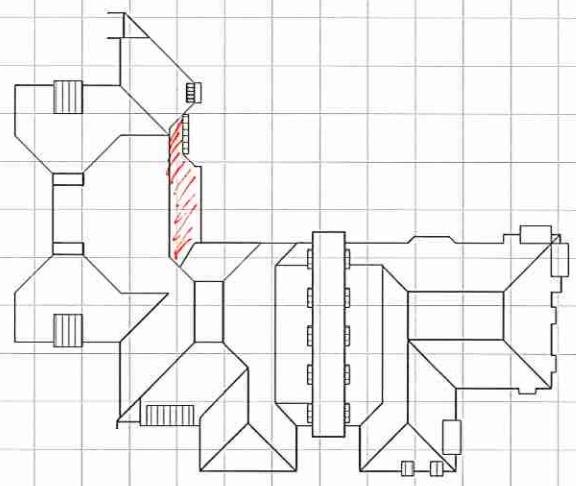
$P_W = 85^{\#} \downarrow$

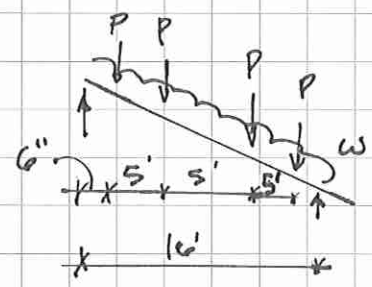
GLB:

$f_v / F'_v = 0.23$ ok ✓

$f_b / F'_b = 0.33$ ok ✓

$\Delta_{TL} = 0.32 = L/600$ ok ✓





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Prepared By: TKJ Date: 12-20-11

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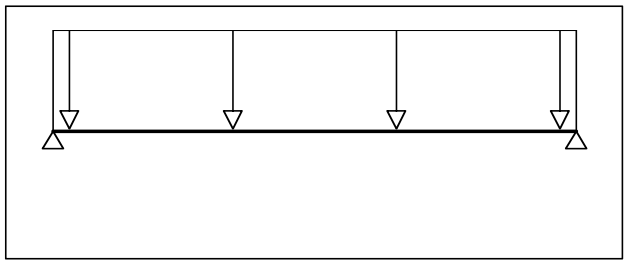
Project: ***Saratoga Comm. Library***
Date: ***12/20/11***

Beam Design

Beam Label: **Case 1: GLB 5 1/8 x 10 @ 4'-0"**



Geometry

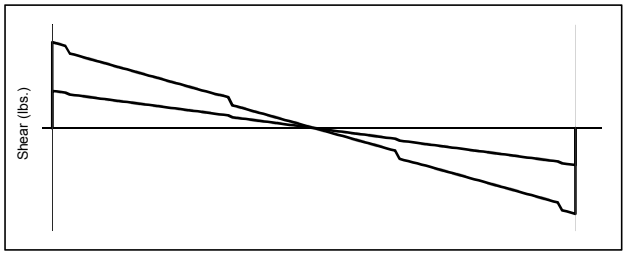
Left Overhang= 0.00 ft
Center Span= 16.00 ft Overall Length= 16.00 ft
Right Overhang= 0.00 ft

Loads

	DL	LL	$\rho E_h/1.4$ or W	Location From Left
Point Load 1	120 #	0 #	85 #	0.50 ft
Point Load 2	120 #	0 #	85 #	5.50 ft
Point Load 3	120 #	0 #	85 #	10.50 ft
Point Load 4	120 #	0 #	85 #	15.50 ft

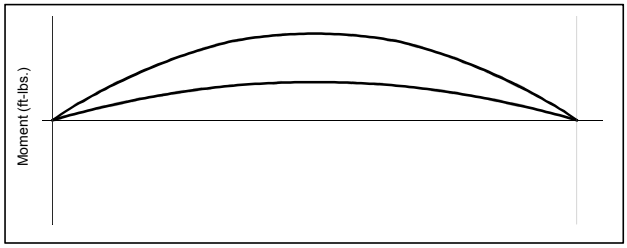
	DL (plf)		LL (plf)		$\rho E_h/1.4$ or W (plf)		Location From Left	
	Start	End	Start	End	Start	End	Start	End
Beam Self Wt.	13.1	13.1	0	0	0	0	0.00 ft	16.00 ft
Dist Load 1	76	76	80	80	0	0	0.00 ft	16.00 ft
Dist Load 2	0	0	0	0	0	0	0.00 ft	0.00 ft
Dist Load 3	0	0	0	0	0	0	0.00 ft	0.00 ft
Dist Load 4	0	0	0	0	0	0	0.00 ft	0.00 ft

Ω= 1.0 [1.0 for Wind loading or if not req'd per ASCE 7-05 12.4.3.1]



Load Combinations

D + L 0.9D +/- (ρE/1.4)
D + (W or ρE/1.4) D + 0.75[L + (W or ρE/1.4)]

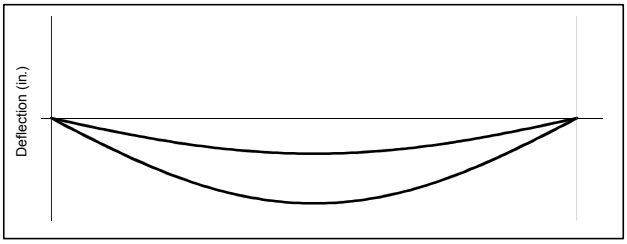


Deflection Criteria

Δ_{LL}<= L / 360 Δ_{KcrD+L}<= L / 240 Kcr= 1.0

Beam Member Design

Section		Grade	
5.125"x10.5" GLB		24F-V4	
b=	5.13 in	F_v=	190 psi
d=	10.50 in	F_b=	2400 psi
S_x=	94.17 in3	E=	1800 ksi
I_x=	494.40 in4		



Shear Design Equation

$F'_v = F_v(C_D)(C_M)(C_t)(C_i)(C_F)$
f_v= VQ / Ib [=3V / 2bd for rectangular sections]

Flexural Design Equation

$F'_b = F_b(C_D)(C_M)(C_t)(C_L)(C_F)(C_V)(C_{fu})(C_i)(C_r)(C_c)$
f_b= M / S_x

Modification Factors

C_D=	1.00	Load Duration Factor [NDS 05 Table 2.3.2]
C_M=	1.00	Wet Service Factor [NDS 05 Table 4A, 4D, 5A, 5B]
C_t=	1.00	Temperature Factor [NDS 05 Table 2.3.3]
C_L=	1.00	Beam Stability Factor [NDS 05 3.3.3]
l_u=	24 in	Effective Unbraced Length [NDS 05 Table 3.3.3]
R_b=	3.10	
C_F=	1.00	Size Factor [NDS 05 4.3.6]
C_V=	1.00	Volume Factor [GLB's only. NDS 05 5.3.6]
C_{fu}=	1.00	Flat Use Factor [NDS 05 Table 4A, 4D, 5A, 5B]
C_i=	1.00	Incising Factor [NDS 05 Table 4.3.8]
C_r=	1.00	Repetitive Member Factor [=1.15 if s<= 24" o.c.]
C_c=	1.00	Curvature Factor [GLB's only. See NDS 05 5.3.8]

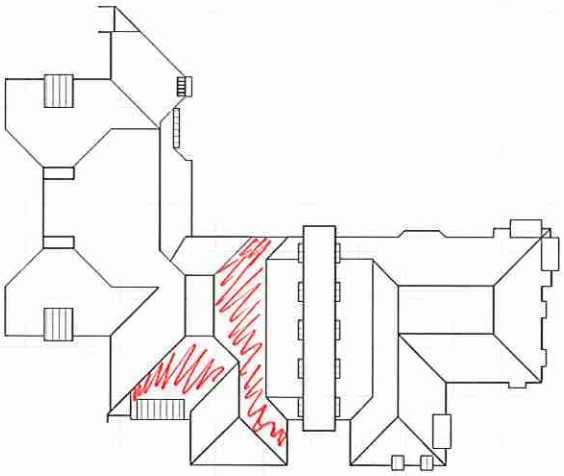
Analysis Summary

	DL	LL	E or W	Combinations
V_{max}=	953 #	640 #	170 #	1593 #
V_{min}=	-953 #	-640 #	-170 #	-1593 #
M_{max}=	3570 ft-lbs	2560 ft-lbs	510 ft-lbs	6130 ft-lbs
M_{min}=	0 ft-lbs	0 ft-lbs	0 ft-lbs	0 ft-lbs
R_{Lmax}=	953 #	640 #	170 #	1593 #
R_{Lmin}=				687 #
R_{Rmax}=	953 #	640 #	170 #	1593 #
R_{Rmin}=				687 #

Design Summary

f_v= 44 psi F'_v= 190 psi		D/C =	0.23	**OK**
f_b= 781 psi F'_b= 2396 psi		D/C =	0.33	**OK**
Δ_{LL}= 0.00 in	0.13 in =L / 1449	0.00 in		**OK**
Δ_{KcrD+L}= 0.00 in	0.32 in =L / 603	0.00 in		**OK**

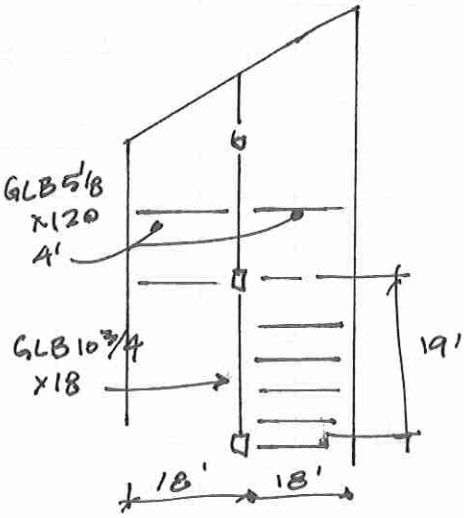
Check Framing:



CASE 2a: GLB 5 1/8 x 12 @ 4'-0"

$L = 18'$
$w_D = 96$ plf
$w_L = 80$ plf
$P_D = 120^{\#}$
$P_W = 85^{\#} \downarrow$

GLB:
$f_v/F'_v = 0.23$ ok
$f_b/F'_b = 0.32$ ok
$\Delta_{TL} = 0.35" = L/620$ ok



Case 2b: GLB 10 3/4 x 18

$L = 19'$
$P_D = 1059^{\#} \times 2 = 2120^{\#}$
$P_L = 720^{\#} \times 2 = 1440^{\#}$
$P_W = 170^{\#} \times 2 = 340^{\#}$

GLB:
$f_v/F'_v = 0.39$ ok
$f_b/F'_b = 0.35$ ok
$\Delta_{TL} = 0.24" = L/945$ ok

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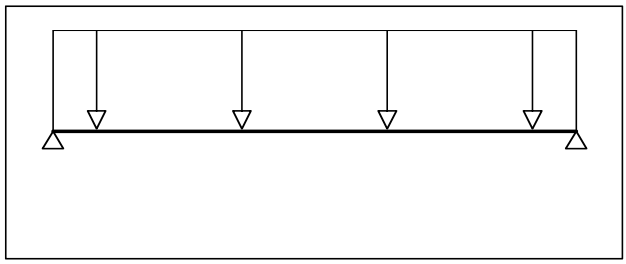
Project: ***Saratoga Comm. Library***
Date: ***12/20/11***

Beam Design

Beam Label: **Case 2a: GLB 5 1/8 x 12 @ 4'-0"**



Geometry

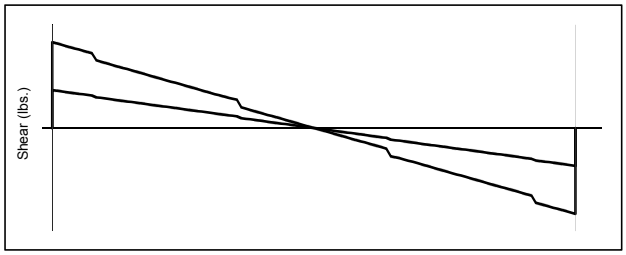
Left Overhang= 0.00 ft
Center Span= 18.00 ft Overall Length= 18.00 ft
Right Overhang= 0.00 ft

Loads

	DL	LL	$\rho E_h/1.4$ or W	Location From Left
Point Load 1	120 #	0 #	85 #	1.50 ft
Point Load 2	120 #	0 #	85 #	6.50 ft
Point Load 3	120 #	0 #	85 #	11.50 ft
Point Load 4	120 #	0 #	85 #	16.50 ft

	DL (plf)		LL (plf)		$\rho E_h/1.4$ or W (plf)		Location From Left	
	Start	End	Start	End	Start	End	Start	End
Beam Self Wt.	14.9	14.9	0	0	0	0	0.00 ft	18.00 ft
Dist Load 1	76	76	80	80	0	0	0.00 ft	18.00 ft
Dist Load 2	0	0	0	0	0	0	0.00 ft	0.00 ft
Dist Load 3	0	0	0	0	0	0	0.00 ft	0.00 ft
Dist Load 4	0	0	0	0	0	0	0.00 ft	0.00 ft

Ω= 1.0 [1.0 for Wind loading or if not req'd per ASCE 7-05 12.4.3.1]

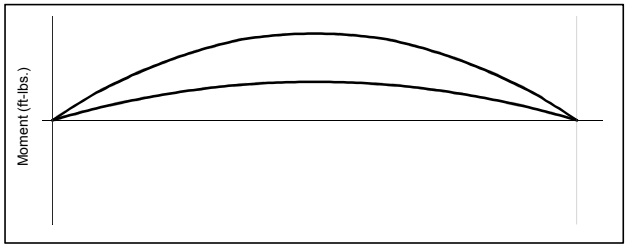


Load Combinations

D + L 0.9D +/- (ρE/1.4)
D + (W or ρE/1.4) D + 0.75[L + (W or ρE/1.4)]

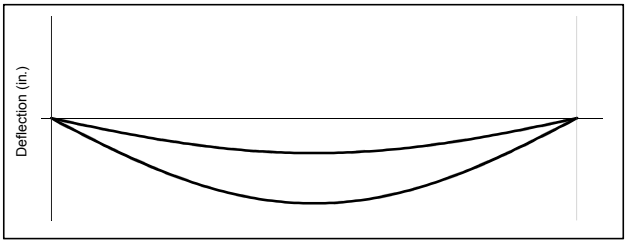
Deflection Criteria

Δ_{LL}<= L / 360 Δ_{KcrD+L}<= L / 240 Kcr= 1.0



Beam Member Design

Section		Grade	
5.125"x12" GLB		24F-V4	
b=	5.13 in	F_v=	190 psi
d=	12.00 in	F_b=	2400 psi
S_x=	123.00 in3	E=	1800 ksi
I_x=	738.00 in4		



Shear Design Equation

$F'_v = F_v(C_D)(C_M)(C_t)(C_i)(C_F)$
f_v= VQ / Ib [=3V / 2bd for rectangular sections]

Flexural Design Equation

$F'_b = F_b(C_D)(C_M)(C_t)(C_L)(C_F)(C_V)(C_{fu})(C_i)(C_r)(C_c)$
f_b= M / S_x

Modification Factors

C_D=	1.00	Load Duration Factor [NDS 05 Table 2.3.2]
C_M=	1.00	Wet Service Factor [NDS 05 Table 4A, 4D, 5A, 5B]
C_t=	1.00	Temperature Factor [NDS 05 Table 2.3.3]
C_L=	1.00	Beam Stability Factor [NDS 05 3.3.3]
l_u=	24 in	Effective Unbraced Length [NDS 05 Table 3.3.3]
R_b=	3.31	
C_F=	1.00	Size Factor [NDS 05 4.3.6]
C_V=	1.00	Volume Factor [GLB's only. NDS 05 5.3.6]
C_{fu}=	1.00	Flat Use Factor [NDS 05 Table 4A, 4D, 5A, 5B]
C_i=	1.00	Incising Factor [NDS 05 Table 4.3.8]
C_r=	1.00	Repetitive Member Factor [=1.15 if s<= 24" o.c.]
C_c=	1.00	Curvature Factor [GLB's only. See NDS 05 5.3.8]

Analysis Summary

	DL	LL	E or W	Combinations
V_{max}=	1059 #	720 #	170 #	1779 #
V_{min}=	-1059 #	-720 #	-170 #	-1779 #
M_{max}=	4643 ft-lbs	3240 ft-lbs	680 ft-lbs	7883 ft-lbs
M_{min}=	0 ft-lbs	0 ft-lbs	0 ft-lbs	0 ft-lbs
R_{Lmax}=	1059 #	720 #	170 #	1779 #
R_{Lmin}=				783 #
R_{Rmax}=	1059 #	720 #	170 #	1779 #
R_{Rmin}=				783 #

Design Summary

f_v= 43 psi F'_v= 190 psi		D/C =	0.23	**OK**
f_b= 769 psi F'_b= 2396 psi		D/C =	0.32	**OK**
Δ_{LL}= 0.00 in	0.14 in =L / 1519	0.00 in		**OK**
Δ_{KcrD+L}= 0.00 in	0.35 in =L / 622	0.00 in		**OK**

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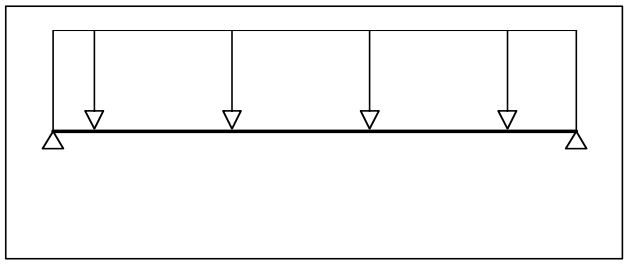
Project: ***Saratoga Comm. Library***
Date: ***12/20/11***

Beam Design

Beam Label: **Case 2b: GLB 10 3/4 x 18 Girder**



Geometry

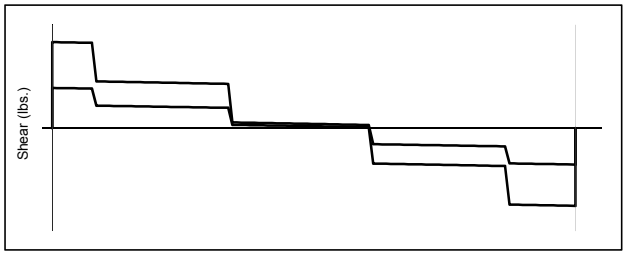
Left Overhang= 0.00 ft
Center Span= 19.00 ft Overall Length= 19.00 ft
Right Overhang= 0.00 ft

Loads

	DL	LL	$\rho E_h/1.4$ or W	Location From Left
Point Load 1	2120 #	1440 #	340 #	1.50 ft
Point Load 2	2120 #	1440 #	340 #	6.50 ft
Point Load 3	2120 #	1440 #	340 #	11.50 ft
Point Load 4	2120 #	1440 #	340 #	16.50 ft

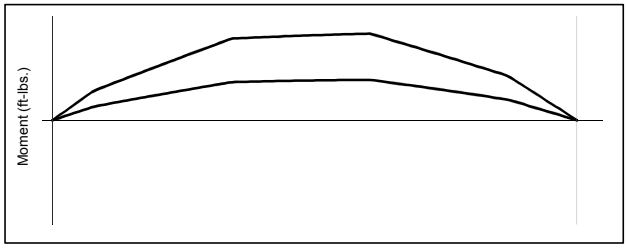
	DL (plf)		LL (plf)		$\rho E_h/1.4$ or W (plf)		Location From Left	
	Start	End	Start	End	Start	End	Start	End
Beam Self Wt.	47.0	47.0	0	0	0	0	0.00 ft	19.00 ft
Dist Load 1	0	0	0	0	0	0	0.00 ft	19.00 ft
Dist Load 2	0	0	0	0	0	0	0.00 ft	0.00 ft
Dist Load 3	0	0	0	0	0	0	0.00 ft	0.00 ft
Dist Load 4	0	0	0	0	0	0	0.00 ft	0.00 ft

Ω= 1.0 [1.0 for Wind loading or if not req'd per ASCE 7-05 12.4.3.1]



Load Combinations

D + L 0.9D +/- (ρE/1.4)
D + (W or ρE/1.4) D + 0.75[L + (W or ρE/1.4)]

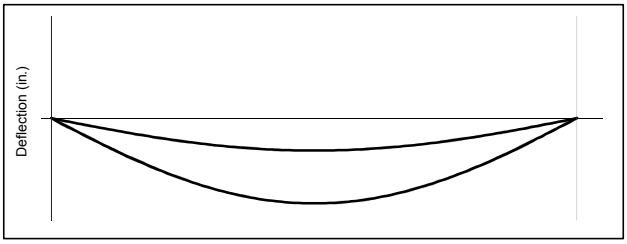


Deflection Criteria

Δ_{LL}<= L / 360 Δ_{KcrD+L}<= L / 240 Kcr= 1.0

Beam Member Design

Section		Grade	
10.75"x18" GLB		24F-V4	
b=	10.75 in	F_v=	190 psi
d=	18.00 in	F_b=	2400 psi
S_x=	580.50 in3	E=	1800 ksi
I_x=	5224.50 in4		



Shear Design Equation

$F'_v = F_v(C_D)(C_M)(C_t)(C_i)(C_F)$
f_v= VQ / Ib [=3V / 2bd for rectangular sections]

Flexural Design Equation

$F'_b = F_b(C_D)(C_M)(C_t)(C_L)(C_F)(C_V)(C_{fu})(C_i)(C_r)(C_c)$
f_b= M / S_x

Modification Factors

C_D=	1.00	Load Duration Factor [NDS 05 Table 2.3.2]
C_M=	1.00	Wet Service Factor [NDS 05 Table 4A, 4D, 5A, 5B]
C_t=	1.00	Temperature Factor [NDS 05 Table 2.3.3]
C_L=	1.00	Beam Stability Factor [NDS 05 3.3.3]
l_u=	24 in	Effective Unbraced Length [NDS 05 Table 3.3.3]
R_b=	1.93	
C_F=	0.96	Size Factor [NDS 05 4.3.6]
C_V=	0.90	Volume Factor [GLB's only. NDS 05 5.3.6]
C_{fu}=	1.00	Flat Use Factor [NDS 05 Table 4A, 4D, 5A, 5B]
C_i=	1.00	Incising Factor [NDS 05 Table 4.3.8]
C_r=	1.00	Repetitive Member Factor [=1.15 if s<= 24" o.c.]
C_c=	1.00	Curvature Factor [GLB's only. See NDS 05 5.3.8]

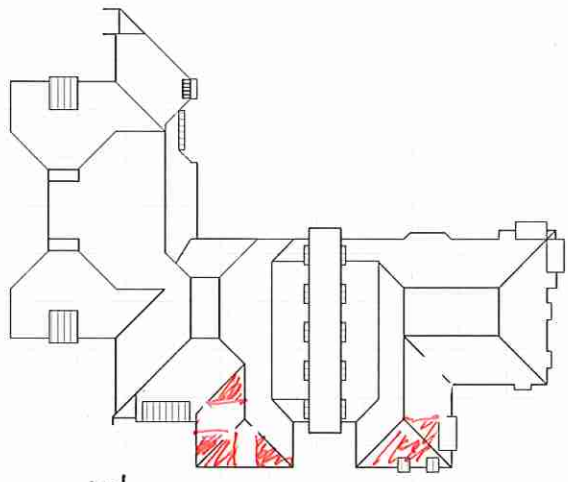
Analysis Summary

	DL	LL	E or W	Combinations
V_{max}=	4910 #	3032 #	716 #	7942 #
V_{min}=	-4464 #	-2728 #	-644 #	-7192 #
M_{max}=	21554 ft-lbs	13263 ft-lbs	3131 ft-lbs	34817 ft-lbs
M_{min}=	0 ft-lbs	0 ft-lbs	0 ft-lbs	0 ft-lbs
R_{Lmax}=	4910 #	3032 #	716 #	7942 #
R_{Lmin}=				3703 #
R_{Rmax}=	4464 #	2728 #	644 #	7192 #
R_{Rmin}=				3373 #

Design Summary

f_v= 62 psi F'_v= 182 psi		D/C =	0.34	**OK**
f_b= 720 psi F'_b= 2065 psi		D/C =	0.35	**OK**
Δ_{LL}= 0.00 in	0.09 in =L / 2487	0.00 in		**OK**
Δ_{KcrD+L}= 0.00 in	0.24 in =L / 945	0.00 in		**OK**

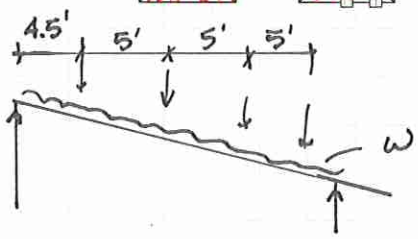
Check Framing



Case 3: 14" TJI @ 24"

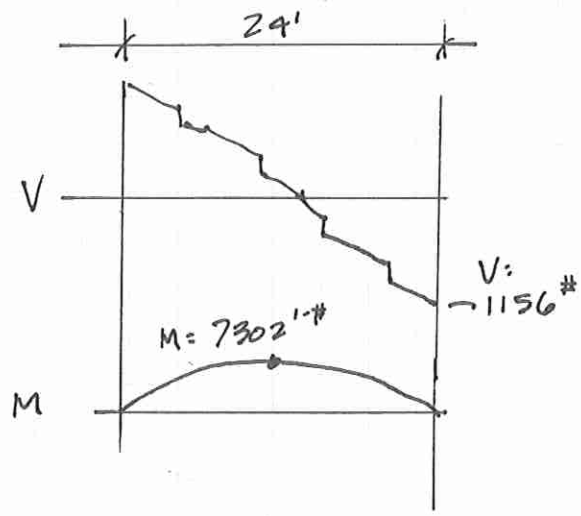
DL: 19 psf → w_d = 38 plf
LL: 20 psf → w_l = 40 plf

14" TJI 165 @ 24":
M_a = 8030'-#
V_a = 2125#



PANEL LOADS:
P_D = 120#
P_W = 85# ↓

M = 7302'-# < 8030'-# ∴ ok ✓
V = 1156# < 2125# ∴ ok ✓



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Check Framing:

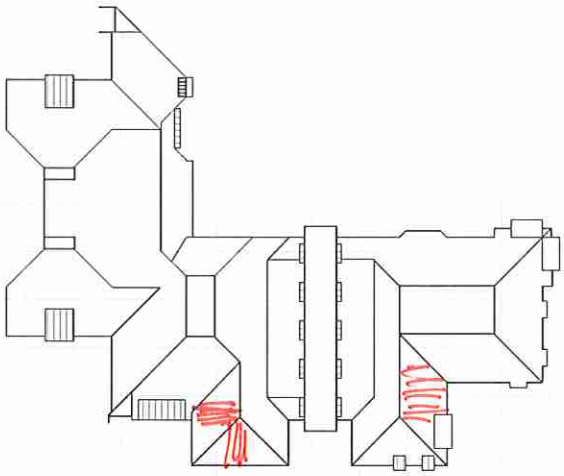
Case 4: 14" TJI @ 16"

L = 29'-6"

[SEE Case 3 for Loading]

M = 8350'-#
M_A = 8030'-#
} D/C = 1.04 ⇒ close - say ok

V = 1032#
V_A = 2125#
} V < V_A ∴ ok



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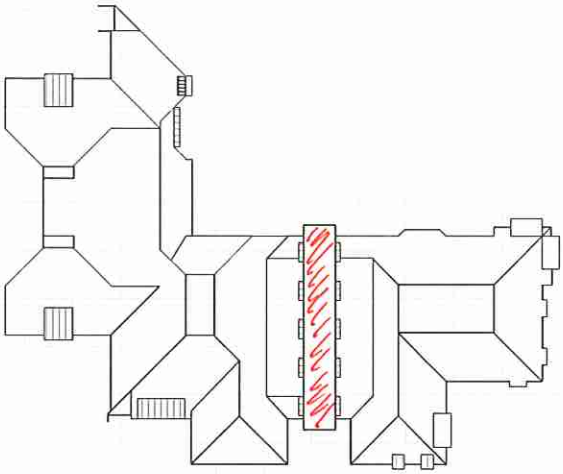
CHECK FRAMING:

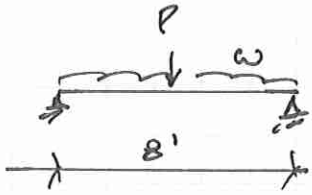


Case 5a: 2x8 @ 24"

L = 8'-0"
DL = 27 psf
LL = 20 psf

FLAT ROOF ZONE

R_H = 305#
R_V = 181# (D)
197# (W)

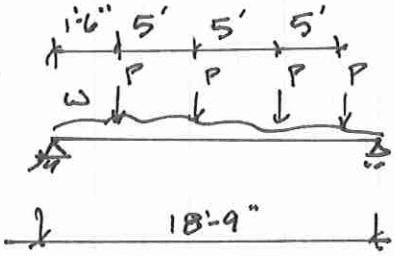


2x8 DF #1:

f_v/F'_v = 0.24 ok✓
f_b/F'_b = 0.40 ok✓
Δ_{TL} = 0.13" = L/740 ok✓

Case 5b: GLB 3 1/8 x 15 @ 8'-0"

L = 18'-9"
w_d = 125 plf x 2 = 250 plf
w_L = 80 plf x 2 = 160 plf
w_w = 160 plf
P_D = 181#
P_W = 197#



GLB 3 1/8 x 15:

f_v/F'_v = 0.46 ok✓
f_b/F'_b = 0.54 ok✓
Δ_{TL} = .81 = L/280 ok✓

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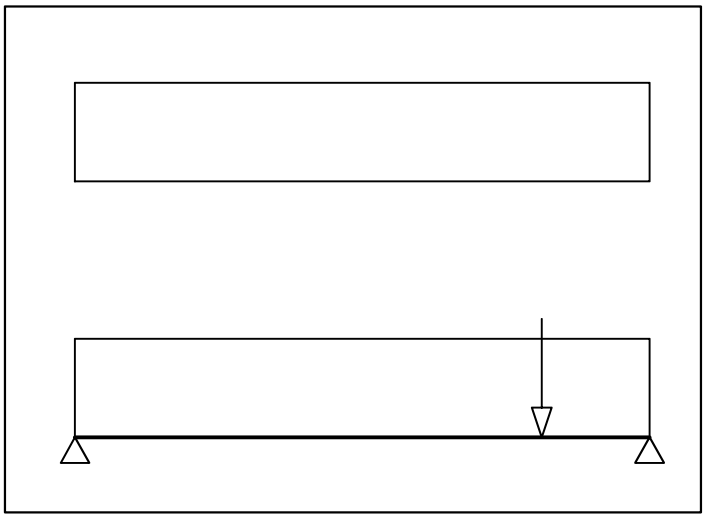
Date: ***12/20/11***

Joist Design

Joist Label: **Case 5a: 2x8 @ 24"**



Geometry

Left Overhang=	0.00 ft	True Length=	0.00 ft
Center Span=	8.00 ft	True Length=	8.00 ft
Right Overhang=	0.00 ft	True Length=	0.00 ft
Joist Spacing=	24" o.c.	Overall Length=	8.00 ft
Roof Pitch=	0:12		

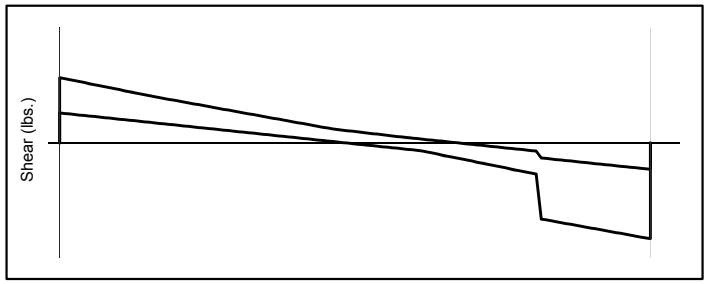
Loads

	DL	LL	$\rho E_h/1.4$ or W	Location From Left
Point Load 1	181 #	0 #	197 #	6.50 ft
Point Load 2	0 #	0 #	0 #	11.50 ft
Point Load 3	0 #	0 #	0 #	16.50 ft
Point Load 4	0 #	0 #	0 #	21.50 ft

	DL (psf)		LL (psf)		$\rho E_h/1.4$ or W (psf)		Location From Left	
	Start	End	Start	End	Start	End	Start	End
Dist Load 1	27	27	20	20	0	0	0.00 ft	8.00 ft
Dist Load 2	0	0	0	0	0	0	0.00 ft	0.00 ft
Dist Load 3	0	0	0	0	0	0	0.00 ft	0.00 ft
Dist Load 4	0	0	0	0	0	0	0.00 ft	0.00 ft

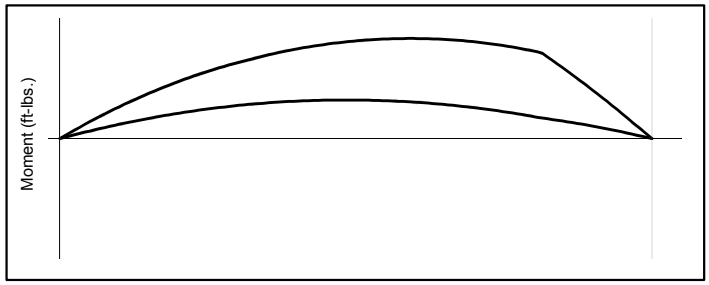
	DL (plf)		LL (plf)		$\rho E_h/1.4$ or W (plf)		Location From Left	
	Start	End	Start	End	Start	End	Start	End
Dist Load 1	54	54	40	40	0	0	0.00 ft	8.00 ft
Dist Load 2	0	0	0	0	0	0	0.00 ft	0.00 ft
Dist Load 3	0	0	0	0	0	0	0.00 ft	0.00 ft
Dist Load 4	0	0	0	0	0	0	0.00 ft	0.00 ft

Ω= 1.0 [1.0 for Wind loading or if not req'd per ASCE 7-05 12.4.3.1]



Load Combinations

D + L — 0.9D +/- (ρE/1.4)
D + (W or ρE/1.4) — D + 0.75[L + (W or ρE/1.4)]



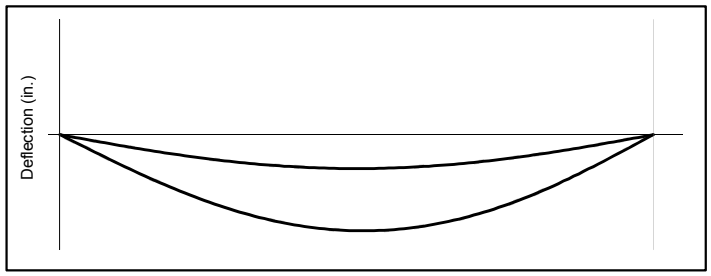
Deflection Criteria

Δ_{LL}<= L / 360 Δ_{KcrD+L}<= L / 240 Kcr= 1.0

Joist Member Design

Section	Grade
2x8 Sawn	DF#1

b=	1.50 in	F_v=	180 psi
d=	7.25 in	F_b=	1000 psi
S_x=	13.14 in3	E=	1700 ksi
I_x=	47.63 in4		



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Joist Design

Shear Design Equation

$F'_v = F_v(C_D)(C_M)(C_t)(C_i)(C_F)$

$f_v = VQ / Ib$ [=3V / 2bd for rectangular sections]

Flexural Design Equation

$F'_b = F_b(C_D)(C_M)(C_t)(C_L)(C_F)(C_V)(C_{fu})(C_i)(C_r)(C_c)$

$f_b = M / S_x$

Modification Factors

C_D=	1.60	Load Duration Factor [NDS 05 Table 2.3.2]
C_M=	1.00	Wet Service Factor [NDS 05 Table 4A, 4D, 5A, 5B]
C_t=	1.00	Temperature Factor [NDS 05 Table 2.3.3]
C_L=	0.96	Beam Stability Factor [NDS 05 3.3.3]
l_u=	48 in	Effective Unbraced Length [NDS 05 Table 3.3.3]
R_b=	12.44	
C_F=	1.20	Size Factor [NDS 05 4.3.6]
C_V=	1.00	Volume Factor [GLB's only. NDS 05 5.3.6]
C_{fu}=	1.00	Flat Use Factor [NDS 05 Table 4A, 4D, 5A, 5B]
C_i=	1.00	Incising Factor [NDS 05 Table 4.3.8]
C_r=	1.15	Repetitive Member Factor [=1.15 if s<= 24" o.c.]
C_c=	1.00	Curvature Factor [GLB's only. See NDS 05 5.3.8]

Analysis Summary

	DL	LL	E or W	Combinations
V_{max}=	250 #	160 #	37 #	410 #
V_{min}=	-363 #	-160 #	-160 #	-603 #
M_{max}=	578 ft-lbs	320 ft-lbs	238 ft-lbs	941 ft-lbs
M_{min}=	0 ft-lbs	0 ft-lbs	0 ft-lbs	0 ft-lbs
R_{Lmax}=	250 #	160 #	37 #	410 #
R_{Lmin}=				188 #
R_{Rmax}=	363 #	160 #	160 #	603 #
R_{Rmin}=				167 #

Design Summary

f_v= 83 psi F'_v= 346 psi			D/C = 0.24	**OK**
f_b= 859 psi F'_b= 2124 psi			D/C = 0.40	**OK**
Δ_{LL}=	0.00 in	0.046 in =L / 2109	0.00 in	**OK**
Δ_{TL}=	0.0000 in	0.1292 in =L / 743	0.00 in	**OK**

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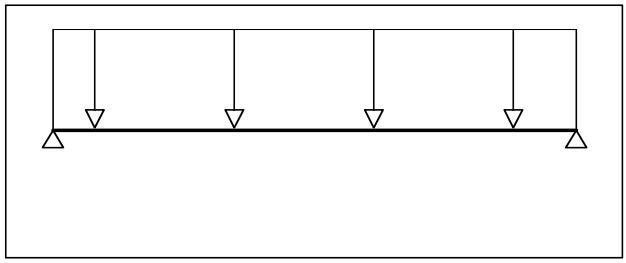
Project: ***Saratoga Comm. Library***
Date: ***12/20/11***

Beam Design

Beam Label: **Case 5b: GLB 3 1/8 x 15 @ 8'-0"**



Geometry

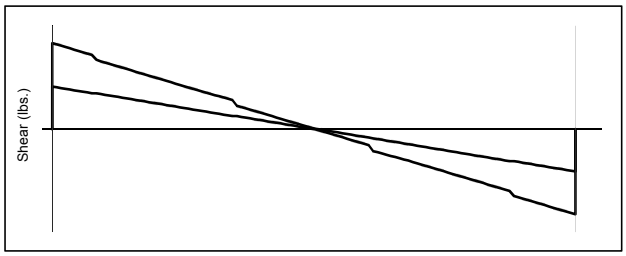
Left Overhang= 0.00 ft
Center Span= 18.75 ft Overall Length= 18.75 ft
Right Overhang= 0.00 ft

Loads

	DL	LL	$\rho E_h/1.4$ or W	Location From Left
Point Load 1	181 #	0 #	197 #	1.50 ft
Point Load 2	181 #	0 #	197 #	6.50 ft
Point Load 3	181 #	0 #	197 #	11.50 ft
Point Load 4	181 #	0 #	197 #	16.50 ft

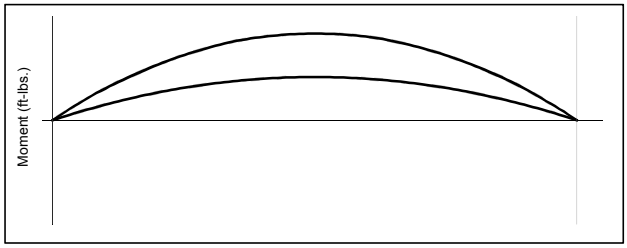
	DL (plf)		LL (plf)		$\rho E_h/1.4$ or W (plf)		Location From Left	
	Start	End	Start	End	Start	End	Start	End
Beam Self Wt.	11.4	11.4	0	0	0	0	0.00 ft	18.75 ft
Dist Load 1	250	250	160	160	0	0	0.00 ft	18.75 ft
Dist Load 2	0	0	0	0	0	0	0.00 ft	0.00 ft
Dist Load 3	0	0	0	0	0	0	0.00 ft	0.00 ft
Dist Load 4	0	0	0	0	0	0	0.00 ft	0.00 ft

Ω= 1.0 [1.0 for Wind loading or if not req'd per ASCE 7-05 12.4.3.1]



Load Combinations

D + L 0.9D +/- (ρE/1.4)
D + (W or ρE/1.4) D + 0.75[L + (W or ρE/1.4)]



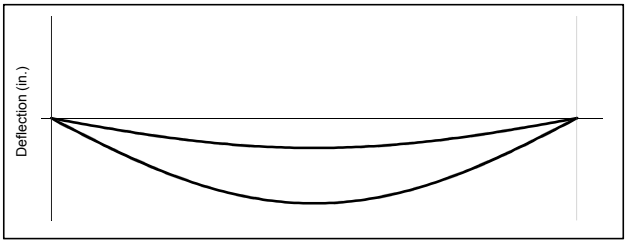
Deflection Criteria

Δ_{LL}<= L / 360 Δ_{KcrD+L}<= L / 240 Kcr= 1.0

Beam Member Design

Section	Grade
3.125"x15" GLB	24F-V4

b=	3.13 in	F_v=	190 psi
d=	15.00 in	F_b=	2400 psi
S_x=	117.19 in3	E=	1800 ksi
I_x=	878.91 in4		



Shear Design Equation

$F'_v = F_v(C_D)(C_M)(C_t)(C_i)(C_F)$
f_v= VQ / Ib [=3V / 2bd for rectangular sections]

Flexural Design Equation

$F'_b = F_b(C_D)(C_M)(C_t)(C_L)(C_F)(C_V)(C_{fu})(C_i)(C_r)(C_c)$
f_b= M / S_x

Modification Factors

C_D=	1.60	Load Duration Factor [NDS 05 Table 2.3.2]
C_M=	1.00	Wet Service Factor [NDS 05 Table 4A, 4D, 5A, 5B]
C_t=	1.00	Temperature Factor [NDS 05 Table 2.3.3]
C_L=	0.99	Beam Stability Factor [NDS 05 3.3.3]
l_u=	24 in	Effective Unbraced Length [NDS 05 Table 3.3.3]
R_b=	6.07	
C_F=	1.00	Size Factor [NDS 05 4.3.6]
C_V=	1.00	Volume Factor [GLB's only. NDS 05 5.3.6]
C_{fu}=	1.00	Flat Use Factor [NDS 05 Table 4A, 4D, 5A, 5B]
C_i=	1.00	Incising Factor [NDS 05 Table 4.3.8]
C_r=	1.00	Repetitive Member Factor [=1.15 if s<= 24" o.c.]
C_c=	1.00	Curvature Factor [GLB's only. See NDS 05 5.3.8]

Analysis Summary

	DL	LL	E or W	Combinations
V_{max}=	2827 #	1500 #	410 #	4327 #
V_{min}=	-2798 #	-1500 #	-378 #	-4298 #
M_{max}=	13071 ft-lbs	7031 ft-lbs	1757 ft-lbs	20102 ft-lbs
M_{min}=	0 ft-lbs	0 ft-lbs	0 ft-lbs	0 ft-lbs
R_{Lmax}=	2827 #	1500 #	410 #	4327 #
R_{Lmin}=				2135 #
R_{Rmax}=	2798 #	1500 #	378 #	4298 #
R_{Rmin}=				2140 #

Design Summary

f_v= 138 psi F'_v= 304 psi		D/C =	0.46	**OK**
f_b= 2058 psi F'_b= 3799 psi		D/C =	0.54	**OK**
Δ_{LL}= 0.00 in	0.28 in =L / 800	0.00 in		**OK**
Δ_{KcrD+L}= 0.00 in	0.81 in =L / 279	0.00 in		**OK**

1997 UBC

Design Base Shear:

$$V = \frac{C_v I}{RT} W = .922 W$$

$$V_{max} = \frac{2.5 C_a I}{R} W = .26 W \quad \rightarrow \text{Governs}$$

$$V_{min} = \max \left| \begin{array}{l} \frac{.8 Z N_v I}{R} W = .096 W \\ .11 C_a I W = .063 W \end{array} \right. \quad \rightarrow V_{min} = .096 W$$

$Z = .4$

$I = 1$

$R = 5.5$

Soil Type = S_D

$C_v = (.64) N_v$ Table 16-T $N_v = 1.6$

$C_v = 1.024$

$C_a = (.44)(N_a)$ Table 16-S $N_a = 1.3$

$C_a = .572$

$T = C_t (h_n)^{3/4} = .202$ sec

$C_t = .02$

$h_n = 21' \ 10''$

ASD $\rightarrow .7E$

$.7 V_{max} = .182 W$

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2010 ASCE-7

From Design Maps:

$S_S = 2.558g$

$S_1 = 1.027g$

$S_{M_S} = 2.558g$

$S_{M_1} = 1.540g$

$S_{DS} = 1.705g$

$S_{D1} = 1.027g$

$I = 1$

$R = 6.5$

$V = C_S W$ so $V = .262W$

$C_S = \frac{S_{DS}}{\left(\frac{R}{I}\right)} = .262$ ← Governs

$C_{S_{max}} = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} = .782$

$C_{S_{min}} = \max \begin{cases} .044 S_{DS} I_e \geq .01 = .075 \\ \frac{.5 S_1}{\left(\frac{R}{I}\right)} = .079 \end{cases}$

$T = C_t h_n^x = .202$ sec

$C_t = .02$

$h_n = 21' \ 10''$

$x = .75$

ASD → .7E

$.7V = .183W$ ⇒ ∴ 1997 CBC & 2010 CBC ARE EQUIVALENT FOR SEISMIC LOADING

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Areas Available for panels

$S_1 = 1671\ ft^2$
$S_2 = 1839\ ft^2$
$S_3 = 1376\ ft^2$
$S_4 = 809\ ft^2$
$S_5 = 3351\ ft^2$
$S_6 = 980\ ft^2$
$S_7 = 2219\ ft^2$
$S_8 = 3221\ ft^2$
$S_9 = 1381\ ft^2$

$F_1 = 2445\ ft^2$
$F_2 = 742\ ft^2$

Assume Panel (wts/ft²) & Area

$W_{panels} = 3\ psf$
$W_{anch+misc} = 1\ psf$ } $W_{sloped} = 4\ psf$
$W_{Flat} = 5\ psf$
$A = 18\ ft^2/panel$ (40" x 65" in plan)

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Seismic: Roof Panel Layout

✓ N-S Seismic:

Grid T:

Tributary Areas

$A_{pitched} = 3209\ ft^2$

$A_{wall} = (8\ ft)[30.5 + 60'\ 7'' + 37'\ 9'' + (2)(6'\ 5'') + (2)(4'\ 5'') + 30'\ 4''] = 1446\ ft^2$

$W_{pitched} = 67389\ lb$

$W_{wall} = 8676\ lb$

$\Big\rangle\ W_T = 76065\ lb$

Max Additional Loadings

@T: $10\%\ W_T = 7606\ lb$

@Q: $10\%\ W_Q = 17432\ lb$

@ 7.9, 8.5, 9.2: $10\%\ W_{7.9,\ 8.5,\ 9.2} = 22957\ lb$

@ 9.7: $10\%\ W_{9.7_C} = 7010\ lb$

✓ Grid Q:

Tributary Areas

$A_{pitched} = 5495\ ft^2$

$A_{wall} = (8'\ 5'')[(2)(30') + (2)(30'\ 3'') + 52'] = 1452\ ft^2$

$A_{flat} = 1860\ ft^2$

$W_{flat} = 50220\ lb$

$W_{pitched} = 115395\ lb$

$W_{wall} = 8712\ lb$

$\Big\rangle\ W_Q = 174327\ lb$

E-W Seismic:

✓ Grid 7.9, 8.5, 9.2:

$A_{pitched} = 5575\ ft^2$

$A_{flat} = 3759\ ft^2$

$A_{wall} = (8'\ 5'')[(58)(2) + (3)(34)] = 1834\ ft^2$

$W_{pitched} = 110755\ lb$

$W_{flat} = 101493\ lb$

$W_{wall} = 11044\ lb$

$\Big\rangle\ W_{7.9,\ 8.5,\ 9.2} = 229572\ lb$

✓ Grid 9.7: @ Building C

$A_{pitched} = 2941\ ft^2$

$A_{wall} = (8'\ 5'')[91 + 74'] = 1389\ ft^2$

$W_{pitched} = 61761\ lb$

$W_{wall} = 8334\ lb$

$\Big\rangle\ W_{9.7_C} = 70095\ lb$

Seismic: Roof Panel Layout

N-S Seismic:

✓ Grid N:

$A_{pitched} = 4516\ ft^2$

$A_{flat} = 2156\ ft^2$

$A_{wall} = (8'\ 5'')[258'] = 2172\ ft^2$

$W_{pitched} = 94836\ lb$

$W_{flat} = 58212\ lb$

$W_{wall} = 13032\ lb$

$\Big\rangle\ W_N = 166080\ lb$

✓ Grid K:

$A_{pitched} = 1595\ ft^2$

$A_{flat} = 2820\ ft^2$

$A_{wall} = (8'\ 5'')[238'] = 2004\ ft^2$

$W_{pitched} = 33495\ lb$

$W_{flat} = 76140\ lb$

$W_{wall} = 12024\ lb$

$\Big\rangle\ W_K = 121659$

E-W Seismic:

✓ Grid 11: @ Building C

$A_{pitched} = 1902\ ft^2$

$A_{wall} = (8'\ 5'')(119') = 1002\ ft^2$

$W_{pitched} = 39942\ lb$

$W_{wall} = 6012\ lb$

$\Big\rangle\ W_{11_C} = 45954\ lb$

Max additional Loading

@ 11: $10\%\ W_{11} = 4595\ lb$

@ K: $10\%\ W_K = 12166\ lb$

@ N: $10\%\ W_N = 16608\ lb$

Seismic: Roof Panel Layout

N-S seismic:

✓ Grid G:

$A_{pitched} = 4515\ ft^2$

$A_{flat} = 251\ ft^2$

$A_{wall} = (8.42')[107.5'] = 905\ ft^2$

$W_{pitched} = 75\%\ A_{pitched}\ W_{Exist} + 25\%\ A_{pitched}\ W_{new} = 64339 + 23703 = 88042\ lb$

$W_{flat} = 6777\ lb$

$W_{wall} = 5430\ lb$

$W_G = 100249\ lb$

✓ Grid F:

$A_{pitched} = 3142 + 298 = 3440\ ft^2 + (2190\ ft^2) = 5630\ ft^2$

$A_{flat} = 426\ ft^2$

$A_{wall} = (106)(8.42) = 888\ ft^2 + (8.42)(133) = 2008\ ft^2$

$W_{pitched} = 65956\ lb + 41610\ lb$

$W_{flat} = 11502\ lb + 6720\ lb$

$W_{wall} = 5328\ lb +$

$W_F = 131116\ lb$

✓ Grid D:

$A_{pitched} = 3937 + 7562 = 11499\ ft^2$

$A_{wall} = (8.42)(227) = 1911\ ft^2$

$W_{pitched} = 233605\ lb$

$W_{wall} = 11466\ lb$

$W_D = 245071\ lb$

Max additional Load

@G: $.1 W_G = 10025\ lb$

@F: $.1 W_F = 13112\ lb$

@D: $.1 W_D = 24507\ lb$

@A: $.1 W_A = 10166\ lb$

✓ Grid A:

$A_{pitched} = 4230\ ft^2$

$A_{wall} = (8.42)[254] = 2139\ ft^2$

$W_{pitched} = 88830\ lb$

$W_{wall} = 12834\ lb$

$W_A = 101664\ lb$

Seismic: Roof Panel Layout

E-W Seismic

Grid 11: @ Building B

$A_{pitched} = 1051\ ft^2$

$A_{wall} = (8.42)(95) = 800\ ft^2$

$W_{pitched} = 22071\ lb$

$W_{wall} = 4800\ lb$

$\Big\rangle\ W_{11_B} = 26871\ lb$

Grid 10:

$A_{pitched} = 1009 + 1898 = 2907\ ft^2$

$A_{wall} = (8.42)(111) = 935\ ft^2$

$W_{pitched} = 59029\ lb$

$W_{wall} = 5610\ lb$

$\Big\rangle\ W_{10} = 64639\ lb$

Grid 9.7: @ Building B

$A_{pitched} = 2260\ ft^2$

$A_{wall} = (8.42)(106) = 892\ ft^2$

$A_{flat} = 388\ ft^2$

$W_{pitched} = 42940\ lb$

$W_{flat} = 10476\ lb$

$W_{wall} = 5352\ lb$

$\Big\rangle\ W_{9.7_B} = 58768\ lb$

Grid 9:

$A_{pitched} = 2148 + 3710 = 5858\ ft^2$

$A_{wall} = (8.42)(244) = 2054\ ft^2$

$A_{flat} = 1331\ ft^2$

$W_{pitched} = 115598\ lb$

$W_{wall} = 12324\ lb$

$W_{flat} = 35937\ lb$

$\Big\rangle\ W_9 = 163859\ lb$

Max additional Load

@ 11_B: $.1 W_{11_B} = 2687\ lb$

@ 10: $.1 W_{10} = 6464\ lb$

@ 9.7_B: $.1 W_{9.7_B} = 5877\ lb$

@ 9: $.1 W_9 = 16359\ lb$

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Seismic : Roof Panel Layout

Grid 6:

$A_{pitched} = 4141 + 2443 = 6583\ ft^2$

$A_{flat} = 549\ ft^2$

$A_{wall} = (8.42)(195) = 1642\ ft^2$

$W_{pitched} = 133378\ lb$

$W_{flat} = 14823\ lb$

$W_{wall} = 9852\ lb$

$> W_6 = 158053$

Max additional Load

@ 6 : .1 W_6 = 15805 lb

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Panel Area Limitations

S_1:

S_1 Area = 1671 ft^2

$10\% W_T$ = 7606 lb

W_{sloped} = 4 psf

Panel Area = $\frac{10\% W_T}{W_{sloped}}$ = 1901 ft^2

$(\#\text{ of Panels})_{max}$ = 92 $(\#\text{ of Panels})_{Possible}$ = 69

S_2:

S_2 Area = 1839 ft^2

$10\% W_{9.7_E}$ = 7010 lb

Panel Area = 1752 ft^2

# of Panels = 97 $(\#\text{ of Panels})_{Possible}$ = 77

S_3:

S_3 Area = 1376 ft^2

$10\% W_Q$ = 17432 lb

Panel Area = 4358 ft^2

# of Panels = 76 $(\#\text{ of Panels})_{Possible}$ = 54

S_4:

S_4 Area = 809 ft^2

$10\% W_{11}$ = 4595 lb

Panel Area = 1149 ft^2

# of Panels = 44 $(\#\text{ of Panels})_{Possible}$ = 30

S_5:

S_5 Area = 3351 ft^2

$10\% W_K$ = 12166 lb

$10\% W_G$ = 10025 lb

$10\% W_6$ = 15805 lb

K: Panel Area = $\frac{10\% W_K}{W_{sloped}}$ = 3041 ft^2

G: Panel Area = $\frac{10\% W_G}{W_{sloped}}$ = 2506 ft^2 ← Govern

6: Panel Area = 3951 ft^2

# of Panels = 139

$(\#\text{ of Panels})_{Possible}$ = 124

S_6:

S_6 Area = 980 ft^2

$10\% W_{11_B}$ = 2687 lb

Panel Area = 671 ft^2

# of panels = 32 $(\#\text{ of panels})_{Possible}$ = 36

Governs

S_7:

S_7 Area = 2219 ft^2

$10\% W_F$ = 13112 lb

$10\% W_D$ = 24507 lb

$10\% W_{10}$ = 6464 lb

$10\% W_{9.7_B}$ = 5877 lb

F: Panel Area = 3278 ft^2

D: Panel Area = 6126 ft^2

10: Panel Area = 1616 ft^2

9.7_B: Panel Area = 1469 ft^2 → Govern

# of Panels = 81 $(\#\text{ of Panels})_{Possible}$ = 64

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Panel Area Limitations

S_8:

S_8 Area = 3221 ft^2

10% W_A = 10166 lb — A: Panel Area = 2541 ft^2 ← Govern

10% W_D = 24507 lb — D: Panel Area = 6126 ft^2

10% W_9 = 16359 lb — 9: Panel Area = 4089 ft^2

# of Panels = 141 — $(\#\text{ of panels})_{possible}$ = 118

S_9:

S_9 Area = 1381 ft^2

10% W_6 = 15805 lb — 6: Panel Area = 3951 ft^2

10% W_F = 13112 lb — F: Panel Area = 3278 ft^2 → Govern

# of panels = 76 — $(\#\text{ of panels})_{posible}$ = 47

F_1:

F_1 Area = 2445 ft^2

10% W_N = 16608 lb — N: Panel Area = 3321 ft^2

10% W_K = 12166 lb — K: Panel Area = 2433 ft^2 → Govern

10% $W_{7.9, 8.5, 9.2}$ = 22957 lb — 7.9, 8.5, 9.2: Panel Area = 5100 ft^2

W_{Flat} = 5 psf

# of Panels = 135 — $(\#\text{ of Panels})_{Posible}$ = 100

F_2:

F_2 Area = 742 ft^2

10% W_G = 10025 lb — G: Panel Area = 2005 ft^2 → Govern

10% W_F = 13112 lb — F: Panel Area = 2622 ft^2

10% W_9 = 16359 — 9: Panel Area = 3271 ft^2

# of Panels = 41 — $(\#\text{ of panels})_{Posible}$ = 10

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Check: Roof Panel Layout

Grid T:

$A_{pitched} = 3209\ ft^2$

$A_{Panel} = (69+20)(18) = 1602\ ft^2$

$A_{wall} = 1446\ ft^2$

$W_{pitched} = 67389\ lb$

$W_{wall} = 8676\ lb$

$W_{Panel} = 6408\ lb$

$\Rightarrow W_T = 76065\ lb \Rightarrow 10\%\ W_T = 7606\ lb$

$W_{Panel} < 10\%\ W_T \therefore OK$

Grid Q:

$A_{pitched} = 22242\ ft^2$

$A_{wall} = 1452\ ft^2$

$A_{flat} = 1860\ ft^2$

$A_{Panel} = (54+72)(18) = 2268\ ft^2$

$W_{pitched} = 115395\ lb$

$W_{flat} = 50220\ lb$

$W_{wall} = 8712\ lb$

$W_{Panel} = 9072\ lb$

$\Rightarrow W_Q = 174327\ lb \Rightarrow 10\%\ W_Q = 17432\ lb$

$W_{Panel} < 10\%\ W_Q \therefore OK$

Grid N:

$A_{pitched} = 4516\ ft^2$

$A_{flat} = 2156\ ft^2$

$A_{wall} = 2172\ ft^2$

$A_{Panel} = (50+17)(18) = 1206\ ft^2$

$W_{pitched} = 94836\ lb$

$W_{flat} = 58212\ lb$

$W_{wall} = 13032\ lb$

$W_{Panel} = 5724\ lb$

$\Rightarrow W_N = 166080\ lb \Rightarrow 10\%\ W_N = 16608\ lb$

$W_{Panel} < 10\%\ W_N \therefore OK$

Check: Roof Panel Layout

Grid 9.7: @ Building C

$A_{pitched} = 2941\ ft^2$

$A_{wall} = 1389\ ft^2$

$A_{Panel} = (75)(18) = 1350\ ft^2$

$W_{pitched} = 61761\ lb$

$W_{wall} = 8334\ lb$

$W_{Panel} = 5400\ lb$

$\Rightarrow W_{9.7_C} = 70095\ lb \Rightarrow 10\%\ W_{9.7_C} = 7009\ lb$

$W_{Panel} < 10\%\ W_{9.7_C}$ ∴ ok

Grid 11: @ Building C

$A_{pitched} = 1902\ ft^2$

$A_{wall} = 1002\ ft^2$

$A_{Panel} = (39)(18) = 702\ ft^2$

$W_{pitched} = 39942\ lb$

$W_{wall} = 6012\ lb$

$W_{Panel} = 2808\ lb$

$\Rightarrow W_{11_C} = 45954\ lb \Rightarrow 10\%\ W_{11_C} = 4595\ lb$

$W_{Panel} < 10\%\ W_{11_C}$ ∴ ok

Grid 7.9, 8.5, 9.2:

$A_{pitched} = 5575\ ft^2$

$A_{wall} = 1834\ ft^2$

$A_{flat} = 3759\ ft^2$

$A_{Panel} = (99+46)18 = 2610\ ft^2$

$W_{pitched} = 117075\ lb$

$W_{wall} = 11044\ lb$

$W_{flat} = 101493\ lb$

$W_{Panel} = 11268\ lb$

$\Rightarrow W_{7.9,\,8.5,\,9.2} = 229612\ lb \Rightarrow 10\%\ W_{7.9,\,8.5,\,9.2} = 22961\ lb$

$W_{Panel} < 10\%\ W_{7.9,\,8.5,\,9.2}$ ∴ ok

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Check: Roof Panel Layout

Grid K:

$A_{pitched} = 1595\ ft^2$

$A_{flat} = 2820\ ft^2$

$A_{wall} = 2004\ ft^2$

$A_{panel} = (50+100)18 = 2700\ ft^2$

$W_{pitched} = 33495\ lb$

$W_{flat} = 76140\ lb$

$W_{wall} = 12024\ lb$

$W_{panel} = 11700\ lb$

$W_K = 121659\ lb \Rightarrow 10\% W_K = 12165\ lb$

$W_{panel} < 10\% W_K \therefore OK$

Grid G:

$A_{pitched} = 4515\ ft^2$

$A_{flat} = 251\ ft^2$

$A_{wall} = 905\ ft^2$

$A_{panel} = (93+13)(18) = 1908\ ft^2$

$W_{pitched} = 88042\ lb$

$W_{flat} = 6777\ lb$

$W_{wall} = 5430\ lb$

$W_{panel} = 7866\ lb$

$W_G = 100249\ lb \Rightarrow 10\% W_G = 10024\ lb$

$W_{panel} < 10\% W_G \therefore OK$

Grid F:

$A_{pitched} = 5630\ ft^2$

$A_{flat} = 426\ ft^2$

$A_{wall} = 2008\ ft^2$

$A_{panel} = (47+44+2)(18) = 1674\ ft^2$

$W_{pitched} = 107566\ lb$

$W_{flat} = 11502\ lb$

$W_{wall} = 12048\ lb$

$W_{panel} = 6732\ lb$

$W_F = 131116 \Rightarrow 10\% W_F = 13111\ lb$

$W_{panel} < 10\% W_F \therefore OK$

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Grid D:

$A_{Panel} = (77)(18) = 1386\ ft^2$

$W_{Panel} = 5544\ lb$

$W_{Panel} < 10\% W_D \therefore OK$

$10\% W_D = 24507\ lb$

Grid A:

$A_{Panel} = (57)(18) = 1026\ ft^2$

$W_{Panel} = 4104\ lb$

$W_{Panel} < 10\% W_A \therefore OK$

$10\% W_A = 10166\ lb$

Grid 11_B:

$A_{Panel} = (36+10)(18) = 828\ ft^2$

$W_{Panel} = 3312\ lb$

$W_{Panel} > 10\% W_{11_B} \therefore$ Not Good

$10\% W_{11_B} = 2687\ lb$

Grid 10:

$A_{Panel} = (32)(18) = 576\ ft^2 + (11)(18) = 774\ ft^2$

$W_{Panel} = 2304\ lb + 792\ lb = 3096\ lb$

$W_{Panel} < 10\% W_{10} \therefore OK$

$10\% W_{10} = 6464\ lb$

Grid 9.7_B:

$A_{Panel} = (50)(18) = 900\ ft^2$

$W_{Panel} = 3600\ lb$

$W_{Panel} < 10\% W_{9.7_B} \therefore OK$

$10\% W_{9.7_B} = 5877\ lb$

Grid 9:

$A_{Panel} = (165+13)(18) = 3204\ ft^2$

$W_{Panel} = 13050\ lb$

$W_{Panel} < 10\% W_9 \therefore OK$

$10\% W_9 = 16359\ lb$

Grid 6:

$A_{Panel} = (89+2)(18) = 1638\ ft^2$

$W_{Panel} = 6588\ lb$

$W_{Panel} < 10\% W_6 \therefore OK$

$10\% W_6 = 15805\ lb$