PROJECT NUMBER: **1939-001-192**

DATE: **November 6, 2019**

STRUCTURAL CALCULATIONS
for
Race Track Custom
at
APN 179-04-405-011
Henderson, Nevada
Design is per the 2018 IBC
for
Desert Rose Customs LLC
November 6, 2019
Project #: 1939-001-192

01/09/2020

BY: _____

KENT A. BARBER, P.E., S.E.
SR. VICE PRESIDENT

FOR: **L. R. NELSON CONSULTING ENGINEERS, LLC**

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L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-001-192
DATE 11/6/2019

PROJECT: Desert Rose Customs **SHEET** **OF**
SUBJECT: DESIGN CRITERIA **DESIGNED** NES **CHECKED** KAB

1 Structural Design is based on the International Building Code.

2 Basic Wind and Seismic Load Design Criteria: Risk Category II

a. Wind Speed (3 second Gust) = 100 MPH Exposure = C Gcpi= +/-0.18
b. Seismic Design Category = C
Seismic Factors: Ss = 49% S1 = 16% R = 6.5
Equivalent Lateral Force Sds = 42% Sd1 = 16% Ie= 1.30
Seismic base shear, E = Cs x W Site Class = C Cs = 0.06

3 Roof Trusses to be designed by truss manufacturer. **Truss calculations and truss layout must be provided to the engineer of record prior to construction for review and approval.**

4 Lumber:
a. 2x framing shall be Douglas Fir #2 grade, unless noted otherwise.
b. 4x framing shall be Douglas Fir #2 grade, unless noted otherwise.
c. 6x framing shall be Douglas Fir #1 grade, unless noted otherwise.

5 Glue-Laminated Beams: 24F-V4 DF/DF
(24F-V8 DF/DF at cantilever and continuous beams)
Glue-Laminated Beams: 24F-V4 DF/DF
(24F-V8 DF/DF at cantilever and continuous beams)

6 Wood Structural Panels: Plywood or Oriented Strand Board (O.S.B)

7 Anchor Bolts: ASTM F1554-36

8 Connection Hardware Simpson Strong Tie or ICC approved Equal

9 Concrete: Minimum compressive strength of 4000 psi @ 28 days
Cement for all concrete shall be Type V

10 Reinforcing Steel: ASTM A615, Grade 60 or ASTM A706, Grade 60 (welded reinforcing)

11 Foundations are designed for 2000 psf allowable soil bearing pressure.

12 Floor Trusses to be designed by truss manufacturer.
Truss calculations and truss layout must be provided to the engineer of record prior to construction for review and approval.

13 Shear wall materials, nailing, etc, as per attached Shear Wall Schedule.

14 Unless noted otherwise, all rolled steel shapes and plates shall be per ASTM A36 except W shapes shall be ASTM A992.

15 All steel pipe columns shall be per ASTM A53 Grade B. All HSS shapes shall be ASTM A500 Grade C.

16 Welding shall be with E70XX low hydrogen electrodes in an approved fabricating facility.
All field welds shall have constant special inspection.

17 Inspection: As required by governing municipality.

18 Special Inspection: As required per Section 1705 of the IBC.

19 L. R. Nelson Consulting Engineers assumes no liability for the architectural aspects of the provided plans (prepared by others), such as; dimensions, elevations, or sections. All architectural aspects are to be verified by the general contractor.

L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-001-192
DATE 11/6/2019

PROJECT: Desert Rose Customs **SHEET** **OF**
SUBJECT: BUILDING LOADS **DESIGNED** NES **CHECKED** KAB

ROOF

CONCRETE TILE	12.00
15/32" SHEATHING	1.60
ROOF FRAMING/MFR TRUSSES	3.00
INSULATION (R30 BATT)	2.70
1/2" GYPSUM	2.20
M, E & MISC.	0.50
DL	22.00
LL	20.00
SEISMIC SNOW 0.00 SNOW	

ALTERNATE LOAD #1

Load Type

M,E & MISC.
DL 0.00
Description of Load

2ND FLOOR (WHERE OCCURS)

FLOOR COVERING - STONE TILE	12.00
3/4" SHEATHING	2.40
MF'G FLR TRUSSES /FRAMING	2.80
2X4 @ 16" OC FRAMING	1.41
INSULATION (R30 BATT)	2.70
5/8" GYPSUM	2.75
M, E & MISC.	0.94
DL	25.00
LL	40.00

ALTERNATE LOAD #2

Load Type

M,E & MISC.
DL 0.00
Description of Load

EXTERIOR WALLS

3 COAT STUCCO	10.50
2X4 @ 16" OC FRAMING	1.41
WALL INSULATION	1.00
15/32" SHEATHING	1.60
1/2" GYPSUM	2.20
MISC.	2.29
DL	19.00

OVERFILL (WHERE OCCURS)

15/32" SHEATHING	1.60
OVERFILL RAFTERS	3.40
DL	5.00

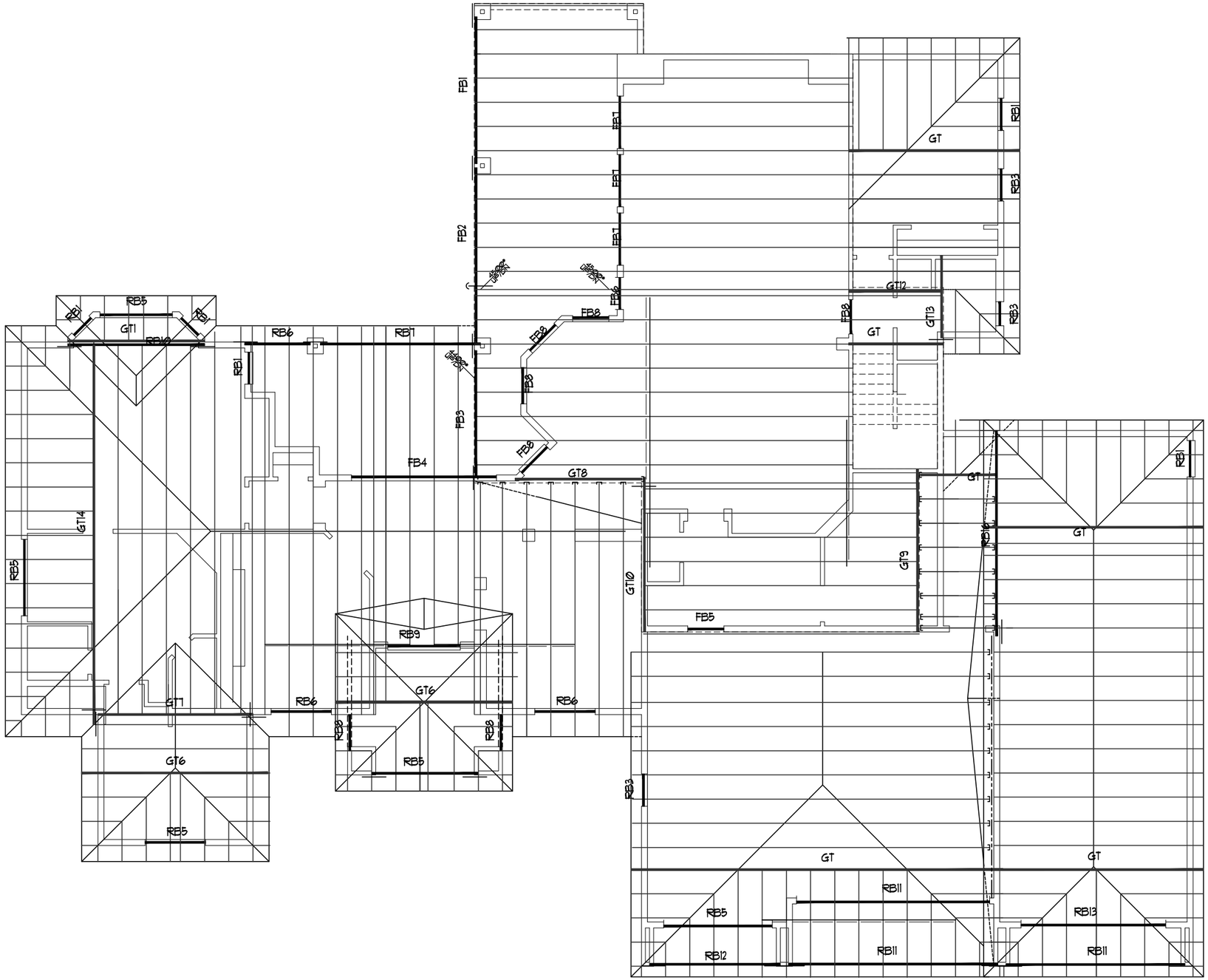
ALT. FLOOR LIVE LOAD

LL

BASE DESIGN VALUES

	DFL (STUD)	DFL #2	DFL #1 (TIMBERS)	24F-V4	24F-V8	LSL (1.5E)	LVL	PSL	
F'_B	700	900	1,350	2,400	2,400	2,250	2,600	2,900	PSI
F'_V	180	180	170	240	240	400	285	290	PSI
F'_C	850	1350	925	1650	1650	1,950	2,510	2,500	PSI
$F'_{C(PERP)}$	625	625	625	650	650	775	750	750	PSI
F'_t	450	575	675	1100	1100	1950	2600	2900	PSI
E	1,400,000	1,600,000	1,600,000	1,800,000	1,800,000	1,500,000	1,900,000	2,000,000	PSI

DEFLECTION CRITERIA: $\Delta_{TLallow}$= L/ 240
$\Delta_{LLallow}$= L/ 360



FLOOR / LOW ROOF FRAMING PLAN

1/4"=1'-0"

L. R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-001-192 DATE 11-11-2019

SHEET ____ OF ____

DESIGNED NES

PROJECT DESERT ROSE CUSTOM

SUBJECT KEY PLAN

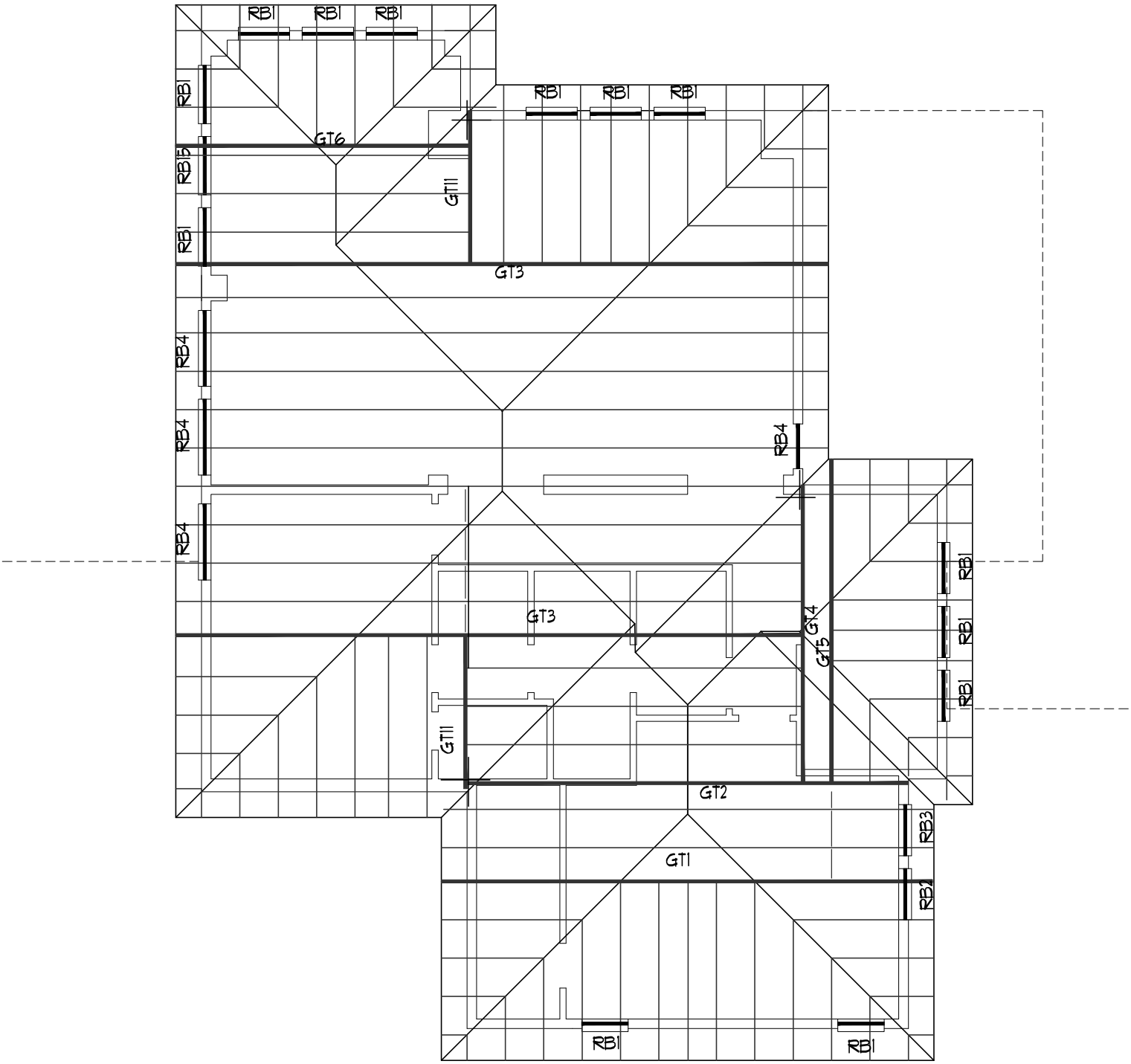
L. R. NELSON CONSULTING ENGINEERS INC.

JOB NO. 1939-001-192

DATE 11-11-2019

PROJECT APN# 179-04-405-011 SHEET ____ OF ____

SUBJECT FRAMING KEY PLAN DESIGNED NES CHECKED ____



L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-001-192
DATE 11/6/2019

PROJECT: Desert Rose Customs **SHEET** **OF**
SUBJECT: Beams & Headers **DESIGNED** NES **CHECKED** KAB

GT1

DESCRIPTION: GT1 | Roof Tributary = 5.00 ft | W_{DL} = 167.00 lb/ft | C_D = 1.25 | $C_{V/L}$ = #N/A
SIZE: | Floor Tributary = 0.00 ft | W_{LL} = 100.00 lb/ft | C_t = 1.00 | C_F = #N/A
GRADE: | Wall Tributary = 3.00 ft | P_{DL} = 0.00 lb | I_u = 32.00
LENGTH (ft)= 32.00 | Additional Load = 0.00 plf | P_{LL} = 0.00 lb | Reduction (L_r) %
NOTES: FOR POST REACTION ONLY | $d_{Point\ Load}$ = 0.00 ft | Reduction (L) %

	Actual		Req'd				Source:	L/xxx	Alternate Loads		
S =	#N/A	in^3	#N/A	#N/A	in^3	#N/A	Max LL Deflection = #VALUE!	#VALUE!	360	#1	#2	LL
A =	#N/A	in^2	#N/A	#N/A	in^2	#N/A	Max TL Deflection = #VALUE!	#VALUE!	240			
I =	#N/A	in^4	#N/A	#VALUE!	in^4	#N/A	Addt'l Load to Post	Source	Lx	Ly		
Left Reaction =	4,272	lb	Post:	2X6		DFL#2	O.K.	lb			10.0	2.0 ft
Right Reaction =	4,272	lb	Post:	2X6		DFL#2	O.K.	lb			10.0	2.0 ft

GT2

DESCRIPTION: GT2 | Roof Tributary = 0.00 ft | W_{DL} = 0.00 lb/ft | C_D = 1.25 | $C_{V/L}$ = #N/A
SIZE: | Floor Tributary = 0.00 ft | W_{LL} = 0.00 lb/ft | C_t = 1.00 | C_F = #N/A
GRADE: | Wall Tributary = 0.00 ft | P_{DL} = 0.00 lb | I_u = 23.00
LENGTH (ft)= 23.00 | Additional Load = 0.00 plf | P_{LL} = 0.00 lb | Reduction (L_r) %
NOTES: FOR POST REACTION ONLY (SEE ADDT'L CALC) | $d_{Point\ Load}$ = 0.00 ft | Reduction (L) %

	Actual		Req'd				Source:	L/xxx	Alternate		
S =	#N/A	in^3	#N/A	#N/A	in^3	#N/A	Max LL Deflection = #VALUE!	#VALUE!	360	#1	#2	LL
A =	#N/A	in^2	#N/A	#N/A	in^2	#N/A	Max TL Deflection = #VALUE!	#VALUE!	240			
I =	#N/A	in^4	#N/A	#VALUE!	in^4	#N/A	Addt'l Load to Post	Source	Lx	Ly		
Left Reaction =	2,919	lb	Post:	2X6		DFL#2	O.K.	2,919.45 lb	GT2		10.0	2.0 ft
Right Reaction =	6,952	lb	Post:	(2)2X6		DFL#2	O.K.	6,951.64 lb	GT2		10.0	2.0 ft

GT3

DESCRIPTION: GT3 | Roof Tributary = 5.00 ft | W_{DL} = 167.00 lb/ft | C_D = 1.25 | $C_{V/L}$ = #N/A
SIZE: | Floor Tributary = 0.00 ft | W_{LL} = 100.00 lb/ft | C_t = 1.00 | C_F = #N/A
GRADE: | Wall Tributary = 3.00 ft | P_{DL} = 2,009.13 lb | I_u = 31.33
LENGTH (ft)= 31.33 | Additional Load = 0.00 plf | P_{LL} = 1,339.58 lb | Reduction (L_r) %
NOTES: FOR POST REACTION ONLY | $d_{Point\ Load}$ = 8.00 ft | Reduction (L) %

	Actual		Req'd				Source: GT11	L/xxx	Alternate		
S =	#N/A	in^3	#N/A	#N/A	in^3	#N/A	Max LL Deflection = #VALUE!	#VALUE!	360	#1	#2	LL
A =	#N/A	in^2	#N/A	#N/A	in^2	#N/A	Max TL Deflection = #VALUE!	#VALUE!	240			
I =	#N/A	in^4	#N/A	#VALUE!	in^4	#N/A	Addt'l Load to Post	Source	Lx	Ly		
Left Reaction =	6,676	lb	Post:	2X6		DFL#2	O.K.	lb			10.0	2.0 ft
Right Reaction =	5,038	lb	Post:	2X6		DFL#2	O.K.	lb			10.0	2.0 ft

GT4

DESCRIPTION: GT4 | Roof Tributary = 0.00 ft | W_{DL} = 0.00 lb/ft | C_D = 1.25 | $C_{V/L}$ = #N/A
SIZE: | Floor Tributary = 0.00 ft | W_{LL} = 0.00 lb/ft | C_t = 1.00 | C_F = #N/A
GRADE: | Wall Tributary = 0.00 ft | P_{DL} = 0.00 lb | I_u = 16.00
LENGTH (ft)= 16.00 | Additional Load = 0.00 plf | P_{LL} = 0.00 lb | Reduction (L_r) %
NOTES: FOR POST REACTION ONLY (SEE ADDT'L CALC) | $d_{Point\ Load}$ = 0.00 ft | Reduction (L) %

	Actual		Req'd				Source:	L/xxx	Alternate		
S =	#N/A	in^3	#N/A	#N/A	in^3	#N/A	Max LL Deflection = #VALUE!	#VALUE!	360	#1	#2	LL
A =	#N/A	in^2	#N/A	#N/A	in^2	#N/A	Max TL Deflection = #VALUE!	#VALUE!	240			
I =	#N/A	in^4	#N/A	#VALUE!	in^4	#N/A	Addt'l Load to Post	Source	Lx	Ly		
Left Reaction =	5,375	lb	Post:	2X6		DFL#2	O.K.	5,374.84 lb	GT4		10.0	2.0 ft
Right Reaction =	7,223	lb	Post:	(2)2X6		DFL#2	O.K.	7,222.84 lb	GT4		10.0	2.0 ft

GT5

DESCRIPTION: GT5 | Roof Tributary = 4.00 ft | W_{DL} = 88.00 lb/ft | C_D = 1.25 | $C_{V/L}$ = #N/A
SIZE: | Floor Tributary = 0.00 ft | W_{LL} = 80.00 lb/ft | C_t = 1.00 | C_F = #N/A
GRADE: | Wall Tributary = 0.00 ft | P_{DL} = 0.00 lb | I_u = 16.00
LENGTH (ft)= 16.00 | Additional Load = 0.00 plf | P_{LL} = 0.00 lb | Reduction (L_r) %
NOTES: FOR POST REACTION ONLY | $d_{Point\ Load}$ = 0.00 ft | Reduction (L) %

	Actual		Req'd				Source:	L/xxx	Alternate		
S =	#N/A	in^3	#N/A	#N/A	in^3	#N/A	Max LL Deflection = #VALUE!	#VALUE!	360	#1	#2	LL
A =	#N/A	in^2	#N/A	#N/A	in^2	#N/A	Max TL Deflection = #VALUE!	#VALUE!	240			
I =	#N/A	in^4	#N/A	#VALUE!	in^4	#N/A	Addt'l Load to Post	Source	Lx	Ly		
Left Reaction =	1,344	lb	Post:	2X6		DFL#2	O.K.	lb			10.0	2.0 ft
Right Reaction =	1,344	lb	Post:	2X6		DFL#2	O.K.	lb			10.0	2.0 ft

L. R. NELSON CONSULTING ENGINEERS

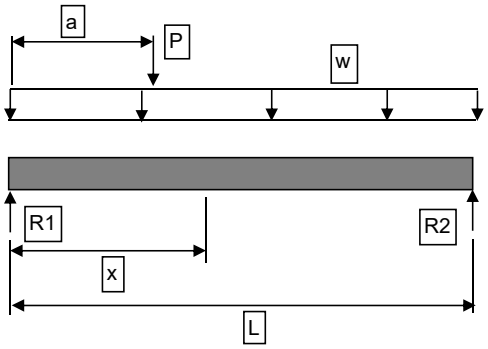
JOB NO. 1939-001-192
DATE:

PROJECT Desert Rose Customs **SHEET** **OF**
SUBJECT BEAMS AND HEADERS **DESIGNED** NES **CHECKED** KAB

(SIMPLY SUPPORTED BEAM)

DESCRIPTION: GT2

SIZE:			C_D =	1.25	
GRADE:			C_M =	1.00	
LENGTH =	23.00	ft	C_t =	1.00	
			C_L =	1.00	
F_b=	#N/A	psi	C_r =	1.00	
F_v=	#N/A	psi	C_c =	1.00	
E=	#N/A	psi	C_f =	1.00	
			C_V =	1.00	
M_{MAX} =	33,616	lb-ft	C_F =	#N/A	
V_{MAX} =	6,952	lb	C_{fu} =	#N/A	
			I_{actual}=	#N/A	
R_1 =	2,919.45	lb			
R_2 =	6,951.64	lb			
S_{req}=	#N/A	IN^3			
A_{req}=	#N/A	IN^2			
$Delta_{LLALLOW.}$ =	0.77	in			
$Delta_{TLALLOW.}$ =	1.15	in			



CHECK:

S_{actual}=	#N/A	in^3	#N/A	#N/A	in^3	**#N/A**
A_{actual}=	#N/A	in^2	#N/A	#N/A	in^2	**#N/A**
Delta LL =	#N/A	in	#N/A	0.77	in	**#N/A**
Delta TL =	#N/A	in	#N/A	1.15	in	**#N/A**

LL => L/? = #N/A

1.5xDL Deflection = **#N/A** inches TL => L/? = #N/A

UNIFORM LOADS

W_{DL}=	30.00	lb/ft	W_{LL}=	0.00	lb/ft
W_{DL}=	0.00	lb/ft	W_{LL}=	0.00	lb/ft
W_{DL}=	0.00	lb/ft	W_{LL}=	0.00	lb/ft
W_{DL}=	0.00	lb/ft	W_{LL}=	0.00	lb/ft
W_{DL}=	0.00	lb/ft	W_{LL}=	0.00	lb/ft

POINT LOADS

			a (ft)				a (ft)	
P_{DL} =	2,815.39	lb @	17.50	P_{LL} =	2,559.45	lb @	17.50	GT4
P_{DL} =	704.00	lb @	19.00	P_{LL} =	640.00	lb @	19.00	GT5
P_{DL} =	0.00	lb @	0.00	P_{LL} =	0.00	lb @	0.00	
P_{DL} =	0.00	lb @	0.00	P_{LL} =	0.00	lb @	0.00	
P_{DL} =	0.00	lb @	0.00	P_{LL} =	0.00	lb @	0.00	

UNIFORM LOADS PARTIALLY DISTRIBUTED

			a (ft)	b (ft)				a (ft)	b (ft)
W_{DL}=	44.00	lb/ft @	0.00	17.50	W_{LL}=	40.00	lb/ft @	0.00	17.50
W_{DL}=	170.50	lb/ft @	17.50	1.50	W_{LL}=	155.00	lb/ft @	17.50	1.50
W_{DL}=	66.00	lb/ft @	19.00	4.00	W_{LL}=	60.00	lb/ft @	19.00	4.00
W_{DL}=	0.00	lb/ft @	0.00	0.00	W_{LL}=	0.00	lb/ft @	0.00	0.00
W_{DL}=	0.00	lb/ft @	0.00	0.00	W_{LL}=	0.00	lb/ft @	0.00	0.00

NOTE: b IS THE DISTANCE FROM THE START TO THE END OF THE UNIFORMLY DISTRIBUTED LOAD.

L. R. NELSON CONSULTING ENGINEERS

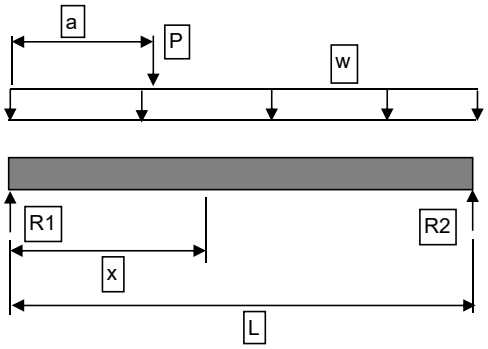
JOB NO. 1939-001-192
DATE:

PROJECT Desert Rose Customs **SHEET** **OF**
SUBJECT BEAMS AND HEADERS **DESIGNED** NES **CHECKED** KAB

(SIMPLY SUPPORTED BEAM)

DESCRIPTION: GT4

SIZE:			C_D =	1.25	
GRADE:			C_M =	1.00	
LENGTH =	16.00	ft	C_t =	1.00	
			C_L =	1.00	
F_b=	#N/A	psi	C_r =	1.00	
F_v=	#N/A	psi	C_c =	1.00	
E=	#N/A	psi	C_f =	1.00	
			C_V =	1.00	
M_{MAX} =	35,271	lb-ft	C_F =	#N/A	
V_{MAX} =	7,223	lb	C_{fu} =	#N/A	
			I_{actual}=	#N/A	
R_1 =	5,374.84	lb			
R_2 =	7,222.84	lb			
S_{req}=	#N/A	IN^3			
A_{req}=	#N/A	IN^2			
$Delta_{LLALLOW.}$ =	0.53	in			
$Delta_{TLALLOW.}$ =	0.80	in			



CHECK:

S_{actual}=	#N/A	in^3	#N/A	#N/A	in^3	**#N/A**
A_{actual}=	#N/A	in^2	#N/A	#N/A	in^2	**#N/A**
Delta LL =	#N/A	in	#N/A	0.53	in	**#N/A**
Delta TL =	#N/A	in	#N/A	0.80	in	**#N/A**

LL => L/? = #N/A

1.5xDL Deflection = **#N/A** inches TL => L/? = #N/A

UNIFORM LOADS

W_{DL}=	16.50	lb/ft	W_{LL}=	15.00	lb/ft
W_{DL}=	0.00	lb/ft	W_{LL}=	0.00	lb/ft
W_{DL}=	0.00	lb/ft	W_{LL}=	0.00	lb/ft
W_{DL}=	0.00	lb/ft	W_{LL}=	0.00	lb/ft
W_{DL}=	0.00	lb/ft	W_{LL}=	0.00	lb/ft

POINT LOADS

			a (ft)				a (ft)	
P_{DL} =	3,129.11	lb @	8.00	P_{LL} =	1,908.56	lb @	8.00	GT3
P_{DL} =	0.00	lb @	0.00	P_{LL} =	0.00	lb @	0.00	
P_{DL} =	0.00	lb @	0.00	P_{LL} =	0.00	lb @	0.00	
P_{DL} =	0.00	lb @	0.00	P_{LL} =	0.00	lb @	0.00	
P_{DL} =	0.00	lb @	0.00	P_{LL} =	0.00	lb @	0.00	

UNIFORM LOADS PARTIALLY DISTRIBUTED

			a (ft)	b (ft)				a (ft)	b (ft)
W_{DL}=	110.00	lb/ft @	0.00	8.00	W_{LL}=	100.00	lb/ft @	0.00	8.00
W_{DL}=	352.00	lb/ft @	8.00	8.00	W_{LL}=	320.00	lb/ft @	8.00	8.00
W_{DL}=	0.00	lb/ft @	0.00	0.00	W_{LL}=	0.00	lb/ft @	0.00	0.00
W_{DL}=	0.00	lb/ft @	0.00	0.00	W_{LL}=	0.00	lb/ft @	0.00	0.00
W_{DL}=	0.00	lb/ft @	0.00	0.00	W_{LL}=	0.00	lb/ft @	0.00	0.00

NOTE: b IS THE DISTANCE FROM THE START TO THE END OF THE UNIFORMLY DISTRIBUTED LOAD.

L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-001-192
DATE 11/6/2019

PROJECT: Desert Rose Customs **SHEET** ____ **OF** ____
SUBJECT: Beams & Headers **DESIGNED** NES **CHECKED** KAB

DESCRIPTION:		Roof Tributary =	0.00	ft	W_{DL}=	0.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	1.00
SIZE:	2X6	Floor Tributary =	0.00	ft	W_{LL}=	0.00	lb/ft	C_t =	1.00	C_F =	1.30
GRADE:	DFL#2	Wall Tributary =	0.00	ft	P_{DL} =	0.00	lb	I_u =	2.00		
LENGTH (ft)=	1.00	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%	
NOTES:					$d_{Point\ Load}$ =	0.00	ft	Reduction (L)		%	
	Actual		Req'd		Source:			L/xxx	Alternate Loads		

S =	7.56	in^3	>	0.00	in^3	O.K.	Max LL Deflection =	0.00 in =	N/A	360	#1	#2	LL
A =	8.25	in^2	>	0.00	in^2	O.K.	Max TL Deflection =	0.00 in =	N/A	240			
I =	20.80	in^4	>	0.00	in^4	O.K.		Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	0	lb	Post:	2X6	DFL#2	O.K.			lb		10.0	2.0	ft
Right Reaction =	0	lb	Post:	2X6	DFL#2	O.K.			lb		10.0	2.0	ft

DESCRIPTION:	GT6	Roof Tributary =	5.00	ft	W_{DL}=	167.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	#N/A
SIZE:		Floor Tributary =	0.00	ft	W_{LL}=	100.00	lb/ft	C_t =	1.00	C_F =	#N/A
GRADE:		Wall Tributary =	3.00	ft	P_{DL} =	0.00	lb	I_u =	14.00		
LENGTH (ft)=	14.00	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%	
NOTES:	FOR POST REACTION ONLY				$d_{Point\ Load}$ =	0.00	ft	Reduction (L)		%	
	Actual		Req'd		Source:			L/xxx	Alternate		

S =	#N/A	in^3	#N/A	#N/A	in^3	#N/A	Max LL Deflection =	#VALUE!	#VALUE!	360	#1	#2	LL
A =	#N/A	in^2	#N/A	#N/A	in^2	#N/A	Max TL Deflection =	#VALUE!	#VALUE!	240			
I =	#N/A	in^4	#N/A	#VALUE!	in^4	#N/A		Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	1,869	lb	Post:	2X6	DFL#2	O.K.			lb		10.0	2.0	ft
Right Reaction =	1,869	lb	Post:	2X6	DFL#2	O.K.			lb		10.0	2.0	ft

DESCRIPTION:	GT7	Roof Tributary =	17.00	ft	W_{DL}=	431.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	#N/A
SIZE:		Floor Tributary =	0.00	ft	W_{LL}=	340.00	lb/ft	C_t =	1.00	C_F =	#N/A
GRADE:		Wall Tributary =	3.00	ft	P_{DL} =	0.00	lb	I_u =	13.00		
LENGTH (ft)=	13.00	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%	
NOTES:	FOR POST REACTION ONLY				$d_{Point\ Load}$ =	0.00	ft	Reduction (L)		%	
	Actual		Req'd		Source:			L/xxx	Alternate		

S =	#N/A	in^3	#N/A	#N/A	in^3	#N/A	Max LL Deflection =	#VALUE!	#VALUE!	360	#1	#2	LL
A =	#N/A	in^2	#N/A	#N/A	in^2	#N/A	Max TL Deflection =	#VALUE!	#VALUE!	240			
I =	#N/A	in^4	#N/A	#VALUE!	in^4	#N/A		Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	5,012	lb	Post:	2X6	DFL#2	O.K.			lb		10.0	2.0	ft
Right Reaction =	5,012	lb	Post:	2X6	DFL#2	O.K.			lb		10.0	2.0	ft

DESCRIPTION:	GT8	Roof Tributary =	15.00	ft	W_{DL}=	646.01	lb/ft	C_D =	1.25	$C_{V/L}$ =	#N/A
SIZE:		Floor Tributary =	2.00	ft	W_{LL}=	380.00	lb/ft	C_t =	1.00	C_F =	#N/A
GRADE:		Wall Tributary =	14.00	ft	P_{DL} =	0.00	lb	I_u =	2.00		
LENGTH (ft)=	14.00	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%	
NOTES:	FOR POST REACTION ONLY				$d_{Point\ Load}$ =	0.00	ft	Reduction (L)		%	
	Actual		Req'd		Source:			L/xxx	Alternate		

S =	#N/A	in^3	#N/A	#N/A	in^3	#N/A	Max LL Deflection =	#VALUE!	#VALUE!	360	#1	#2	LL
A =	#N/A	in^2	#N/A	#N/A	in^2	#N/A	Max TL Deflection =	#VALUE!	#VALUE!	240			
I =	#N/A	in^4	#N/A	#VALUE!	in^4	#N/A		Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	7,182	lb	Post:	(2)2X6	DFL#2	O.K.			lb		10.0	2.0	ft
Right Reaction =	7,182	lb	Post:	(2)2X6	DFL#2	O.K.			lb		10.0	2.0	ft

DESCRIPTION:	GT9	Roof Tributary =	0.00	ft	W_{DL}=	0.00	lb/ft	C_D =	1.00	$C_{V/L}$ =	1.00
SIZE:	5-1/8X18	Floor Tributary =	0.00	ft	W_{LL}=	0.00	lb/ft	C_t =	1.00	C_F =	1.00
GRADE:	24F-V4	Wall Tributary =	0.00	ft	P_{DL} =	0.00	lb	I_u =	2.00		
LENGTH (ft)=	13.50	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%	
NOTES:	FOR POST REACTION ONLY (SEE ADDT'L CALC)				$d_{Point\ Load}$ =	0.00	ft	Reduction (L)		%	
	Actual		Req'd		Source:			L/xxx	Alternate		

S =	276.75	in^3	>	0.00	in^3	O.K.	Max LL Deflection =	0.00 in =	N/A	360	#1	#2	LL
A =	92.25	in^2	>	0.00	in^2	O.K.	Max TL Deflection =	0.00 in =	N/A	240			
I =	2490.75	in^4	>	0.00	in^4	O.K.		Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	11,653	lb	Post:	(2)2X6	DFL#2	O.K.		11,653.06	lb	GT9	10.0	2.0	ft
Right Reaction =	19,889	lb	Post:	6X6	DFL#1 (TIMBERS)	O.K.		19,889.50	lb	GT9	10.0	2.0	ft

NEVADA - 6765 West Russell Road, Suite 200 - Las Vegas, NV 89118 - (702) 798-7978 - Fax (702) 451-2296
UTAH - 51 West 9000 South - Sandy, UT 84070 - (801) 565-9340 - Fax (801) 565-9340

L. R. NELSON CONSULTING ENGINEERS

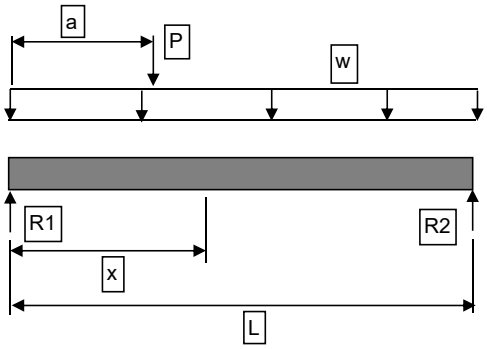
JOB NO. 1939-001-192
DATE:

PROJECT Desert Rose Customs **SHEET** **OF**
SUBJECT BEAMS AND HEADERS **DESIGNED** NES **CHECKED** KAB

(SIMPLY SUPPORTED BEAM)

DESCRIPTION: GT9

SIZE:	**5-1/8X18**		C_D =	1.00	
GRADE:	24F-V4		C_M =	1.00	
LENGTH =	13.50	ft	C_t =	1.00	
			C_L =	1.00	
F_b=	2,400	psi	C_r =	1.00	
F_v=	240	psi	C_c =	1.00	
E=	1,800,000	psi	C_f =	1.00	
			C_V =	1.00	
M_{MAX} =	49,549	lb-ft	C_F =	1.00	
V_{MAX} =	19,889	lb	C_{fu} =	1.00	
			I_{actual}=	2,490.75	
R_1 =	11,653.06	lb			
R_2 =	19,889.50	lb			
S_{req}=	247.75	IN^3			
A_{req}=	113.53	IN^2			
$Delta_{LLALLOW.}$ =	0.45	in			
$Delta_{TLALLOW.}$ =	0.68	in			



CHECK:

S_{actual}=	276.75	in^3	>	247.75	in^3	**O.K.**
A_{actual}=	92.25	in^2	<	113.53	in^2	**N.G.**
Delta LL =	0.15	in	<	0.45	in	**O.K.**
Delta TL =	0.35	in	<	0.68	in	**O.K.**

1.5xDL Deflection = **0.29** inches

LL => L/? = 1063
TL => L/? = 469

UNIFORM LOADS

W_{DL}=	287.50	lb/ft	W_{LL}=	460.00	lb/ft	11.5 FLOOR
W_{DL}=	71.50	lb/ft	W_{LL}=	65.00	lb/ft	3.25 ROOF
W_{DL}=	266.00	lb/ft	W_{LL}=	0.00	lb/ft	14 WALL
W_{DL}=	0.00	lb/ft	W_{LL}=	0.00	lb/ft	
W_{DL}=	0.00	lb/ft	W_{LL}=	0.00	lb/ft	

POINT LOADS

			a (ft)				a (ft)	
P_{DL} =	2,672.04	lb @	7.83	P_{LL} =	1,600.00	lb @	7.83	GT1
P_{DL} =	3,641.33	lb @	13.00	P_{LL} =	3,310.30	lb @	13.00	GT2
P_{DL} =	0.00	lb @	0.00	P_{LL} =	0.00	lb @	0.00	
P_{DL} =	0.00	lb @	0.00	P_{LL} =	0.00	lb @	0.00	
P_{DL} =	0.00	lb @	0.00	P_{LL} =	0.00	lb @	0.00	

UNIFORM LOADS PARTIALLY DISTRIBUTED

			a (ft)	b (ft)				a (ft)	b (ft)
W_{DL}=	110.00	lb/ft @	0.00	7.67	W_{LL}=	100.00	lb/ft @	0.00	7.67
W_{DL}=	286.00	lb/ft @	7.67	6.00	W_{LL}=	260.00	lb/ft @	7.67	6.00
W_{DL}=	0.00	lb/ft @	0.00	0.00	W_{LL}=	0.00	lb/ft @	0.00	0.00
W_{DL}=	0.00	lb/ft @	0.00	0.00	W_{LL}=	0.00	lb/ft @	0.00	0.00
W_{DL}=	0.00	lb/ft @	0.00	0.00	W_{LL}=	0.00	lb/ft @	0.00	0.00

NOTE: b IS THE DISTANCE FROM THE START TO THE END OF THE UNIFORMLY DISTRIBUTED LOAD.

L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-001-192
DATE 11/6/2019

PROJECT: Desert Rose Customs **SHEET** **OF**
SUBJECT: Beams & Headers **DESIGNED** NES **CHECKED** KAB

DESCRIPTION: GT10 | Roof Trib = 0.00 ft | W_{DL} = 0.00 lb/ft | C_D = 1.25 | $C_{V/L}$ = #N/A
SIZE: | Floor Trib = 0.00 ft | W_{LL} = 0.00 lb/ft | C_t = 1.00 | C_F = #N/A
GRADE: | Wall Trib = 0.00 ft | P_{DL} = 0.00 lb | I_u = 31.00
LENGTH (ft)= 31.00 | Additional Load = 0.00 plf | P_{LL} = 0.00 lb | Reduction (L_r) %
NOTES: SEE ADDT'L CALC | $d_{Point\ Load}$ = 0.00 ft | Reduction (L) %

	Actual		Req'd				Source:	L/xxx	Alternate Loads		
S =	#N/A	in³	#N/A	#N/A	in³	#N/A	Max LL Deflection = #VALUE!	360	#1	#2	LL
A =	#N/A	in²	#N/A	#N/A	in²	#N/A	Max TL Deflection = #VALUE!	240			
I =	#N/A	in⁴	#N/A	#VALUE!	in⁴	#N/A	Addt'l Load to Post	Source	Lx	Ly	
Left Reaction =	3,760 lb	Post:	2X6	DFL#2	O.K.		3759.5 lb	GT10	10.0	2.0	ft
Right Reaction =	19,593 lb	Post:	(3)2X6	DFL#2	O.K.		19592.5 lb	GT10	10.0	2.0	ft

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DESCRIPTION: GT11 | Roof Trib = 10.00 ft | W_{DL} = 277.00 lb/ft | C_D = 1.25 | $C_{V/L}$ = #N/A
SIZE: | Floor Trib = 0.00 ft | W_{LL} = 200.00 lb/ft | C_t = 1.00 | C_F = #N/A
GRADE: | Wall Trib = 3.00 ft | P_{DL} = 1,169.02 lb | I_u = 2.00
LENGTH (ft)= 8.00 | Additional Load = 0.00 plf | P_{LL} = 700.00 lb | Reduction (L_r) %
NOTES: FOR POST REACTION ONLY | $d_{Point\ Load}$ = 6.17 ft | Reduction (L) %

	Actual		Req'd				Source: GT6	L/xxx	Alternate		
S =	#N/A	in³	#N/A	#N/A	in³	#N/A	Max LL Deflection = #VALUE!	360	#1	#2	LL
A =	#N/A	in²	#N/A	#N/A	in²	#N/A	Max TL Deflection = #VALUE!	240			
I =	#N/A	in⁴	#N/A	#VALUE!	in⁴	#N/A	Addt'l Load to Post	Source	Lx	Ly	
Left Reaction =	2,336 lb	Post:	HGR	DFL#2	-		lb		10.0	2.0	ft
Right Reaction =	3,349 lb	Post:	(2)2X6	DFL#2	O.K.		lb		10.0	2.0	ft

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DESCRIPTION: GT14 | Roof Trib = 4.00 ft | W_{DL} = 126.00 lb/ft | C_D = 1.25 | $C_{V/L}$ = #N/A
SIZE: | Floor Trib = 0.00 ft | W_{LL} = 80.00 lb/ft | C_t = 1.00 | C_F = #N/A
GRADE: | Wall Trib = 2.00 ft | P_{DL} = 0.00 lb | I_u = 31.00
LENGTH (ft)= 31.00 | Additional Load = 0.00 plf | P_{LL} = 0.00 lb | Reduction (L_r) %
NOTES: FOR POST REACTION ONLY | $d_{Point\ Load}$ = 0.00 ft | Reduction (L) %

	Actual		Req'd				Source:	L/xxx	Alternate		
S =	#N/A	in³	#N/A	#N/A	in³	#N/A	Max LL Deflection = #VALUE!	360	#1	#2	LL
A =	#N/A	in²	#N/A	#N/A	in²	#N/A	Max TL Deflection = #VALUE!	240			
I =	#N/A	in⁴	#N/A	#VALUE!	in⁴	#N/A	Addt'l Load to Post	Source	Lx	Ly	
Left Reaction =	3,193 lb	Post:	(2)2X6	DFL#2	O.K.		lb		10.0	2.0	ft
Right Reaction =	3,193 lb	Post:	(2)2X6	DFL#2	O.K.		lb		10.0	2.0	ft

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DESCRIPTION: GT12 | Roof Trib = 5.50 ft | W_{DL} = 412.01 lb/ft | C_D = 1.25 | $C_{V/L}$ = #N/A
SIZE: | Floor Trib = 1.00 ft | W_{LL} = 150.00 lb/ft | C_t = 1.00 | C_F = #N/A
GRADE: | Wall Trib = 14.00 ft | P_{DL} = 704.00 lb | I_u = 8.00
LENGTH (ft)= 8.00 | Additional Load = 0.00 plf | P_{LL} = 640.00 lb | Reduction (L_r) %
NOTES: FOR POST REACTION ONLY | $d_{Point\ Load}$ = 1.50 ft | Reduction (L) %

	Actual		Req'd				Source: GT5	L/xxx	Alternate		
S =	#N/A	in³	#N/A	#N/A	in³	#N/A	Max LL Deflection = #VALUE!	360	#1	#2	LL
A =	#N/A	in²	#N/A	#N/A	in²	#N/A	Max TL Deflection = #VALUE!	240			
I =	#N/A	in⁴	#N/A	#VALUE!	in⁴	#N/A	Addt'l Load to Post	Source	Lx	Ly	
Left Reaction =	10,563 lb	Post:	4X10	DFL#2	O.K.		7,223 lb	GT4	10.0	2.0	ft
Right Reaction =	2,500 lb	Post:	2X6	DFL#2	O.K.		lb		10.0	2.0	ft

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DESCRIPTION: | Roof Trib = 0.00 ft | W_{DL} = 0.00 lb/ft | C_D = 1.00 | $C_{V/L}$ = #N/A
SIZE: | Floor Trib = 0.00 ft | W_{LL} = 0.00 lb/ft | C_t = 1.00 | C_F = #N/A
GRADE: | Wall Trib = 0.00 ft | P_{DL} = 0.00 lb | I_u = 1.00
LENGTH (ft)= 1.00 | Additional Load = 0.00 plf | P_{LL} = 0.00 lb | Reduction (L_r) %
NOTES: FOR POST REACTION ONLY | $d_{Point\ Load}$ = 0.50 ft | Reduction (L) %

	Actual		Req'd				Source:	L/xxx	Alternate		
S =	#N/A	in³	#N/A	#N/A	in³	#N/A	Max LL Deflection = #VALUE!	360	#1	#2	LL
A =	#N/A	in²	#N/A	#N/A	in²	#N/A	Max TL Deflection = #VALUE!	240			
I =	#N/A	in⁴	#N/A	#VALUE!	in⁴	#N/A	Addt'l Load to Post	Source	Lx	Ly	
Left Reaction =	0 lb	Post:	2X6	DFL#2	O.K.		lb		10.0	2.0	ft
Right Reaction =	0 lb	Post:	2X6	DFL#2	O.K.		lb		10.0	2.0	ft

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Lic. # : KW-06000484 L. R. NELSON LV

DESCRIPTION: GT10 - For post reaction only

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-16

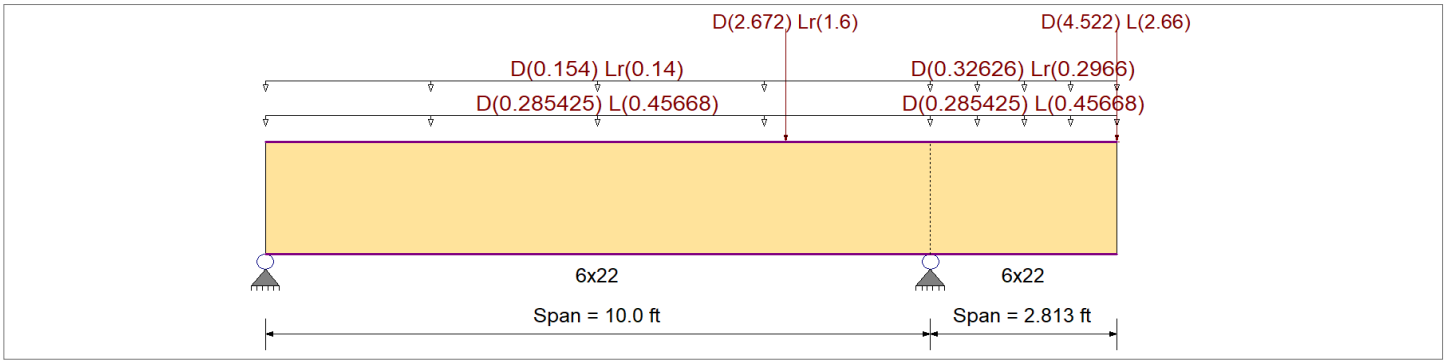
Wood Species : Douglas Fir-Larch
Wood Grade : No.2

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	750.0 psi
Fb -	750.0 psi
Fc - Prll	700.0 psi
Fc - Perp	625.0 psi
Fv	170.0 psi
Ft	475.0 psi

E : Modulus of Elasticity

Ebend- xx	1,300.0 ksi
Eminbend - xx	470.0 ksi
Density	31.210 pcf



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Load for Span Number 1
- Uniform Load : D = 0.0250, L = 0.040 ksf, Tributary Width = 11.417 ft, (Floor Load)
- Uniform Load : D = 0.0220, Lr = 0.020 ksf, Tributary Width = 7.0 ft, (Roof Load)
- Point Load : D = 2.672, Lr = 1.60 k @ 7.833 ft, (GT1)

Load for Span Number 2
- Uniform Load : D = 0.0250, L = 0.040 ksf, Tributary Width = 11.417 ft, (Floor Load)
- Uniform Load : D = 0.0220, Lr = 0.020 ksf, Tributary Width = 14.830 ft, (Roof Load)
- Point Load : D = 4.522, L = 2.660 k @ 2.813 ft, (GT8)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.922 : 1	Maximum Shear Stress Ratio	=	0.617 : 1
Section used for this span		6x22	Section used for this span		6x22
	=	691.71 psi		=	104.94 psi
	=	750.00 psi		=	170.00 psi
Load Combination		+D+L+H, LL Comb Run (LL)	Load Combination		+D+L+H, LL Comb Run (*L)
Location of maximum on span	=	10.000 ft	Location of maximum on span	=	10.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.032 in	Ratio =	2096 >=360
Max Upward Transient Deflection	-0.016 in	Ratio =	4322 >=360
Max Downward Total Deflection	0.059 in	Ratio =	1150 >=180
Max Upward Total Deflection	-0.020 in	Ratio =	5979 >=180

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
+D+H													0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.635	0.427	0.90	0.937	1.00	1.00	1.00	1.00	1.00	15.14	428.69	675.00	5.15	65.29	153.00
Length = 2.813 ft	2	0.635	0.427	0.90	0.937	1.00	1.00	1.00	1.00	1.00	15.14	428.69	675.00	5.15	65.29	153.00
+D+L+H, LL Comb Run (*L)					0.937	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.922	0.617	1.00	0.937	1.00	1.00	1.00	1.00	1.00	24.42	691.71	750.00	8.27	104.94	170.00
Length = 2.813 ft	2	0.922	0.617	1.00	0.937	1.00	1.00	1.00	1.00	1.00	24.42	691.71	750.00	8.27	104.94	170.00

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

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Lic. # : KW-06000484 — L. R. NELSON LV

DESCRIPTION: GT10 - For post reaction only

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
+D+L+H, LL Comb Run (L*)					0.937	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.572	0.484	1.00	0.937	1.00	1.00	1.00	1.00	1.00	15.14	428.69	750.00	6.49	82.27	170.00
Length = 2.813 ft	2	0.572	0.484	1.00	0.937	1.00	1.00	1.00	1.00	1.00	15.14	428.69	750.00	5.15	82.27	170.00
+D+L+H, LL Comb Run (LL)					0.937	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.922	0.617	1.00	0.937	1.00	1.00	1.00	1.00	1.00	24.42	691.71	750.00	8.27	104.94	170.00
Length = 2.813 ft	2	0.922	0.617	1.00	0.937	1.00	1.00	1.00	1.00	1.00	24.42	691.71	750.00	8.27	104.94	170.00
+D+Lr+H, LL Comb Run (*L)					0.937	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.493	0.325	1.25	0.937	1.00	1.00	1.00	1.00	1.00	16.31	461.91	937.50	5.45	69.13	212.50
Length = 2.813 ft	2	0.493	0.325	1.25	0.937	1.00	1.00	1.00	1.00	1.00	16.31	461.91	937.50	5.45	69.13	212.50
+D+Lr+H, LL Comb Run (L*)					0.937	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.457	0.401	1.25	0.937	1.00	1.00	1.00	1.00	1.00	15.14	428.69	937.50	6.72	85.26	212.50
Length = 2.813 ft	2	0.457	0.401	1.25	0.937	1.00	1.00	1.00	1.00	1.00	15.14	428.69	937.50	5.15	85.26	212.50
+D+Lr+H, LL Comb Run (LL)					0.937	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.493	0.408	1.25	0.937	1.00	1.00	1.00	1.00	1.00	16.31	461.91	937.50	6.84	86.75	212.50
Length = 2.813 ft	2	0.493	0.408	1.25	0.937	1.00	1.00	1.00	1.00	1.00	16.31	461.91	937.50	5.45	86.75	212.50
+D+0.750Lr+0.750L+H, LL Comb R					0.937	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.694	0.461	1.25	0.937	1.00	1.00	1.00	1.00	1.00	22.98	650.87	937.50	7.72	97.91	212.50
Length = 2.813 ft	2	0.694	0.461	1.25	0.937	1.00	1.00	1.00	1.00	1.00	22.98	650.87	937.50	7.72	97.91	212.50
+D+0.750Lr+0.750L+H, LL Comb R					0.937	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.457	0.441	1.25	0.937	1.00	1.00	1.00	1.00	1.00	15.14	428.69	937.50	7.40	93.82	212.50
Length = 2.813 ft	2	0.457	0.441	1.25	0.937	1.00	1.00	1.00	1.00	1.00	15.14	428.69	937.50	5.15	93.82	212.50
+D+0.750Lr+0.750L+H, LL Comb R					0.937	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.694	0.488	1.25	0.937	1.00	1.00	1.00	1.00	1.00	22.98	650.87	937.50	8.18	103.77	212.50
Length = 2.813 ft	2	0.694	0.488	1.25	0.937	1.00	1.00	1.00	1.00	1.00	22.98	650.87	937.50	7.72	103.77	212.50
+D+0.750L+0.750S+H, LL Comb Ru					0.937	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.726	0.486	1.15	0.937	1.00	1.00	1.00	1.00	1.00	22.10	625.96	862.50	7.49	95.03	195.50
Length = 2.813 ft	2	0.726	0.486	1.15	0.937	1.00	1.00	1.00	1.00	1.00	22.10	625.96	862.50	7.49	95.03	195.50
+D+0.750L+0.750S+H, LL Comb Ru					0.937	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.497	0.397	1.15	0.937	1.00	1.00	1.00	1.00	1.00	15.14	428.69	862.50	6.12	77.61	195.50
Length = 2.813 ft	2	0.497	0.397	1.15	0.937	1.00	1.00	1.00	1.00	1.00	15.14	428.69	862.50	5.15	77.61	195.50
+D+0.750L+0.750S+H, LL Comb Ru					0.937	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.726	0.486	1.15	0.937	1.00	1.00	1.00	1.00	1.00	22.10	625.96	862.50	7.49	95.03	195.50
Length = 2.813 ft	2	0.726	0.486	1.15	0.937	1.00	1.00	1.00	1.00	1.00	22.10	625.96	862.50	7.49	95.03	195.50
+D+0.750Lr+0.750L+0.450W+H, LL					0.937	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.542	0.360	1.60	0.937	1.00	1.00	1.00	1.00	1.00	22.98	650.87	1200.00	7.72	97.91	272.00
Length = 2.813 ft	2	0.542	0.360	1.60	0.937	1.00	1.00	1.00	1.00	1.00	22.98	650.87	1200.00	7.72	97.91	272.00
+D+0.750Lr+0.750L+0.450W+H, LL					0.937	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.357	0.345	1.60	0.937	1.00	1.00	1.00	1.00	1.00	15.14	428.69	1200.00	7.40	93.82	272.00
Length = 2.813 ft	2	0.357	0.345	1.60	0.937	1.00	1.00	1.00	1.00	1.00	15.14	428.69	1200.00	5.15	93.82	272.00
+D+0.750Lr+0.750L+0.450W+H, LL					0.937	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.542	0.382	1.60	0.937	1.00	1.00	1.00	1.00	1.00	22.98	650.87	1200.00	8.18	103.77	272.00
Length = 2.813 ft	2	0.542	0.382	1.60	0.937	1.00	1.00	1.00	1.00	1.00	22.98	650.87	1200.00	7.72	103.77	272.00
+D+0.750L+0.750S+0.450W+H, LL					0.937	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.522	0.349	1.60	0.937	1.00	1.00	1.00	1.00	1.00	22.10	625.96	1200.00	7.49	95.03	272.00
Length = 2.813 ft	2	0.522	0.349	1.60	0.937	1.00	1.00	1.00	1.00	1.00	22.10	625.96	1200.00	7.49	95.03	272.00
+D+0.750L+0.750S+0.450W+H, LL					0.937	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.357	0.285	1.60	0.937	1.00	1.00	1.00	1.00	1.00	15.14	428.69	1200.00	6.12	77.61	272.00
Length = 2.813 ft	2	0.357	0.285	1.60	0.937	1.00	1.00	1.00	1.00	1.00	15.14	428.69	1200.00	5.15	77.61	272.00
+D+0.750L+0.750S+0.450W+H, LL					0.937	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.522	0.349	1.60	0.937	1.00	1.00	1.00	1.00	1.00	22.10	625.96	1200.00	7.49	95.03	272.00
Length = 2.813 ft	2	0.522	0.349	1.60	0.937	1.00	1.00	1.00	1.00	1.00	22.10	625.96	1200.00	7.49	95.03	272.00
+D+0.750L+0.750S+0.5250E+H, LL					0.937	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.522	0.349	1.60	0.937	1.00	1.00	1.00	1.00	1.00	22.10	625.96	1200.00	7.49	95.03	272.00
Length = 2.813 ft	2	0.522	0.349	1.60	0.937	1.00	1.00	1.00	1.00	1.00	22.10	625.96	1200.00	7.49	95.03	272.00
+D+0.750L+0.750S+0.5250E+H, LL					0.937	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.357	0.285	1.60	0.937	1.00	1.00	1.00	1.00	1.00	15.14	428.69	1200.00	6.12	77.61	272.00
Length = 2.813 ft	2	0.357	0.285	1.60	0.937	1.00	1.00	1.00	1.00	1.00	15.14	428.69	1200.00	5.15	77.61	272.00
+D+0.750L+0.750S+0.5250E+H, LL					0.937	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.522	0.349	1.60	0.937	1.00	1.00	1.00	1.00	1.00	22.10	625.96	1200.00	7.49	95.03	272.00
Length = 2.813 ft	2	0.522	0.349	1.60	0.937	1.00	1.00	1.00	1.00	1.00	22.10	625.96	1200.00	7.49	95.03	272.00

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Lic. # : KW-06000484 L. R. NELSON LV

DESCRIPTION: GT10 - For post reaction only

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750Lr+0.750L+0.450W+H, LL C	1	0.0209	4.581	+D+L+H, LL Comb Run (*L)	-0.0201	6.536
+D+L+H, LL Comb Run (*L)	2	0.0586	2.813		0.0000	6.536

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	3.760	19.592	
Overall MINimum	1.355	7.157	
+D+H	1.262	12.046	
+D+L+H, LL Comb Run (*L)	0.334	16.919	
+D+L+H, LL Comb Run (L*)	3.546	14.330	
+D+L+H, LL Comb Run (LL)	2.617	19.203	
+D+Lr+H, LL Comb Run (*L)	1.145	12.998	
+D+Lr+H, LL Comb Run (L*)	2.309	14.000	
+D+Lr+H, LL Comb Run (LL)	2.192	14.951	
+D+0.750Lr+0.750L+H, LL Comb Run (*	0.478	16.415	
+D+0.750Lr+0.750L+H, LL Comb Run (L	3.760	15.224	
+D+0.750Lr+0.750L+H, LL Comb Run (L	2.975	19.592	
+D+0.750L+0.750S+H, LL Comb Run (*L	0.566	15.701	
+D+0.750L+0.750S+H, LL Comb Run (L*	2.975	13.759	
+D+0.750L+0.750S+H, LL Comb Run (LL	2.278	17.414	
+D+0.750Lr+0.750L+0.450W+H, LL Comb	0.478	16.415	
+D+0.750Lr+0.750L+0.450W+H, LL Comb	3.760	15.224	
+D+0.750Lr+0.750L+0.450W+H, LL Comb	2.975	19.592	
+D+0.750L+0.750S+0.450W+H, LL Comb	0.566	15.701	
+D+0.750L+0.750S+0.450W+H, LL Comb	2.975	13.759	
+D+0.750L+0.750S+0.450W+H, LL Comb	2.278	17.414	
D Only	1.262	12.046	
Lr Only, LL Comb Run (*L)	-0.117	0.951	
Lr Only, LL Comb Run (L*)	1.047	1.953	
Lr Only, LL Comb Run (LL)	0.929	2.905	
L Only, LL Comb Run (*L)	-0.929	4.873	
L Only, LL Comb Run (L*)	2.283	2.283	
L Only, LL Comb Run (LL)	1.355	7.157	

L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-001-192
DATE 11/6/2019

PROJECT:	**Desert Rose Customs**	**SHEET**		**OF**	
SUBJECT:	**Beams & Headers**	**DESIGNED**	**NES**	**CHECKED**	**KAB**

DESCRIPTION:	RB1	Roof Tributary =	5.00	ft	W_{DL}=	167.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	1.00
SIZE:	6X4	Floor Tributary =	0.00	ft	W_{LL}=	100.00	lb/ft	C_t =	1.00	C_F =	1.00
GRADE:	DFL#2	Wall Tributary =	3.00	ft	P_{DL} =	0.00	lb	I_u =	5.50		
LENGTH (ft)=	5.50	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%	
NOTES:					$d_{Point Load}$ =	0.00	ft	Reduction (L)		%	
					Source:						

	Actual			Req'd						L/xxx	Alternate Loads			
S =	11.23	in^3	>	10.26	in^3	O.K.	Max LL Deflection =	0.07 in =	L / 1008	360	#1	#2	LL	
A =	19.25	in^2	>	4.38	in^2	O.K.	Max TL Deflection =	0.17 in =	L / 377	240				
I =	19.65	in^4	>	12.49	in^4	O.K.			Addt'l Load to Post	Source		Lx	Ly	
Left Reaction =	7,410	lb	Post:	2X6			DFL#2	O.K.	6,676 lb	GT3		8.0	2.0	ft
Right Reaction =	734	lb	Post:	2X6			DFL#2	O.K.	lb			8.0	2.0	ft

DESCRIPTION:	RB2	Roof Tributary =	13.00	ft	W_{DL}=	343.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	1.00
SIZE:	6X6	Floor Tributary =	0.00	ft	W_{LL}=	260.00	lb/ft	C_t =	1.00	C_F =	1.00
GRADE:	DFL#1 (TIMBERS)	Wall Tributary =	3.00	ft	P_{DL} =	2,672.04	lb	I_u =	3.17		
LENGTH (ft)=	3.17	Additional Load =	0.00	plf	P_{LL} =	1,600.00	lb	Reduction (L_r)		%	
NOTES:					$d_{Point Load}$ =	2.25	ft	Reduction (L)		%	
					Source:	GT1					

	Actual			Req'd						L/xxx	Alternate			
S =	27.73	in^3	>	24.18	in^3	O.K.	Max LL Deflection =	0.02 in =	L / 2295	360	#1	#2	LL	
A =	30.25	in^2	>	26.20	in^2	O.K.	Max TL Deflection =	0.04 in =	L / 894	240				
I =	76.26	in^4	>	20.48	in^4	O.K.			Addt'l Load to Post	Source		Lx	Ly	
Left Reaction =	2,196	lb	Post:	2X6			DFL#2	O.K.	lb			8.0	2.0	ft
Right Reaction =	3,988	lb	Post:	2X6			DFL#2	O.K.	lb			8.0	2.0	ft

DESCRIPTION:	RB3	Roof Tributary =	16.00	ft	W_{DL}=	352.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	1.00
SIZE:	6X4	Floor Tributary =	0.00	ft	W_{LL}=	320.00	lb/ft	C_t =	1.00	C_F =	1.00
GRADE:	DFL#2	Wall Tributary =	0.00	ft	P_{DL} =	0.00	lb	I_u =	3.50		
LENGTH (ft)=	3.50	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%	
NOTES:					$d_{Point Load}$ =	0.00	ft	Reduction (L)		%	
					Source:						

	Actual			Req'd						L/xxx	Alternate			
S =	11.23	in^3	>	10.45	in^3	O.K.	Max LL Deflection =	0.03 in =	L / 1222	360	#1	#2	LL	
A =	19.25	in^2	>	6.53	in^2	O.K.	Max TL Deflection =	0.07 in =	L / 582	240				
I =	19.65	in^4	>	8.10	in^4	O.K.			Addt'l Load to Post	Source		Lx	Ly	
Left Reaction =	1,176	lb	Post:	2X6			DFL#2	O.K.	lb			8.0	2.0	ft
Right Reaction =	1,176	lb	Post:	2X6			DFL#2	O.K.	lb			8.0	2.0	ft

DESCRIPTION:	RB4	Roof Tributary =	16.00	ft	W_{DL}=	409.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	1.00
SIZE:	6X6	Floor Tributary =	0.00	ft	W_{LL}=	320.00	lb/ft	C_t =	1.00	C_F =	1.00
GRADE:	DFL#1 (TIMBERS)	Wall Tributary =	3.00	ft	P_{DL} =	0.00	lb	I_u =	4.50		
LENGTH (ft)=	4.50	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%	
NOTES:					$d_{Point Load}$ =	0.00	ft	Reduction (L)		%	
					Source:						

	Actual			Req'd						L/xxx	Alternate			
S =	27.73	in^3	>	13.12	in^3	O.K.	Max LL Deflection =	0.02 in =	L / 2232	360	#1	#2	LL	
A =	30.25	in^2	>	9.22	in^2	O.K.	Max TL Deflection =	0.06 in =	L / 980	240				
I =	76.26	in^4	>	18.68	in^4	O.K.			Addt'l Load to Post	Source		Lx	Ly	
Left Reaction =	1,640	lb	Post:	2X6			DFL#2	O.K.	lb			8.0	2.0	ft
Right Reaction =	1,640	lb	Post:	2X6			DFL#2	O.K.	lb			8.0	2.0	ft

DESCRIPTION:	RB5	Roof Tributary =	5.00	ft	W_{DL}=	167.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	1.00
SIZE:	6X6	Floor Tributary =	0.00	ft	W_{LL}=	100.00	lb/ft	C_t =	1.00	C_F =	1.00
GRADE:	DFL#1 (TIMBERS)	Wall Tributary =	3.00	ft	P_{DL} =	0.00	lb	I_u =	9.50		
LENGTH (ft)=	9.50	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%	
NOTES:					$d_{Point Load}$ =	0.00	ft	Reduction (L)		%	
					Source:						

	Actual			Req'd						L/xxx	Alternate			
S =	27.73	in^3	>	21.42	in^3	O.K.	Max LL Deflection =	0.15 in =	L / 759	360	#1	#2	LL	
A =	30.25	in^2	>	8.09	in^2	O.K.	Max TL Deflection =	0.40 in =	L / 284	240				
I =	76.26	in^4	>	64.38	in^4	O.K.			Addt'l Load to Post	Source		Lx	Ly	
Left Reaction =	1,268	lb	Post:	2X6			DFL#2	O.K.	lb			8.0	2.0	ft
Right Reaction =	1,268	lb	Post:	2X6			DFL#2	O.K.	lb			8.0	2.0	ft

L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-001-192
DATE 11/6/2019

PROJECT: Desert Rose Customs **SHEET** ______ **OF** ______
SUBJECT: Beams & Headers **DESIGNED** NES **CHECKED** KAB

DESCRIPTION:	RB6	Roof Tributary =	16.00	ft	W_{DL}=	428.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	1.00	
SIZE:	6X6	Floor Tributary =	0.00	ft	W_{LL}=	320.00	lb/ft	C_t =	1.00	C_F =	1.00	
GRADE:	DFL#1 (TIMBERS)	Wall Tributary =	4.00	ft	P_{DL} =	0.00	lb	I_u =	6.00			
LENGTH (ft)=	6.00	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%		
NOTES:					$d_{Point\ Load}$ =	0.61	ft	Reduction (L)		%		
	Actual		Req'd			Source:	GT10		L/xxx	Alternate Loads		
S =	27.73 in^3	>	23.94 in^3	O.K.	Max LL Deflection =	0.08 in =	L / 941		360	#1	#2	LL
A =	30.25 in^2	>	13.42 in^2	O.K.	Max TL Deflection =	0.18 in =	L / 403		240			
I =	76.26 in^4	>	45.44 in^4	O.K.		Addt'l Load to Post			Source	Lx	Ly	
Left Reaction =	2,244 lb	Post:	2X6	DFL#2	O.K.		lb			10.0	2.0	ft
Right Reaction =	2,244 lb	Post:	2X6	DFL#2	O.K.		lb			10.0	2.0	ft

DESCRIPTION:	RB7	Roof Tributary =	7.00	ft	W_{DL}=	230.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	0.99	
SIZE:	6X10	Floor Tributary =	0.00	ft	W_{LL}=	140.00	lb/ft	C_t =	1.00	C_F =	1.00	
GRADE:	DFL#1 (TIMBERS)	Wall Tributary =	4.00	ft	P_{DL} =	0.00	lb	I_u =	13.50			
LENGTH (ft)=	13.50	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%		
NOTES:					$d_{Point\ Load}$ =	0.00	ft	Reduction (L)		%		
	Actual		Req'd			Source:			L/xxx	Alternate		
S =	82.73 in^3	>	60.78 in^3	O.K.	Max LL Deflection =	0.17 in =	L / 974		360	#1	#2	LL
A =	52.25 in^2	>	15.56 in^2	O.K.	Max TL Deflection =	0.44 in =	L / 368		240			
I =	392.96 in^4	>	256.03 in^4	O.K.		Addt'l Load to Post			Source	Lx	Ly	
Left Reaction =	2,498 lb	Post:	2X6	DFL#2	O.K.		lb			10.0	2.0	ft
Right Reaction =	2,498 lb	Post:	2X6	DFL#2	O.K.		lb			10.0	2.0	ft

DESCRIPTION:	RB8	Roof Tributary =	5.00	ft	W_{DL}=	167.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	1.00	
SIZE:	6X6	Floor Tributary =	0.00	ft	W_{LL}=	100.00	lb/ft	C_t =	1.00	C_F =	1.00	
GRADE:	DFL#1 (TIMBERS)	Wall Tributary =	3.00	ft	P_{DL} =	1,169.02	lb	I_u =	4.00			
LENGTH (ft)=	4.00	Additional Load =	0.00	plf	P_{LL} =	700.00	lb	Reduction (L_r)		%		
NOTES:					$d_{Point\ Load}$ =	2.25	ft	Reduction (L)		%		
	Actual		Req'd			Source:	GT6		L/xxx	Alternate		
S =	27.73 in^3	>	16.77 in^3	O.K.	Max LL Deflection =	0.02 in =	L / 2718		360	#1	#2	LL
A =	30.25 in^2	>	10.33 in^2	O.K.	Max TL Deflection =	0.05 in =	L / 1018		240			
I =	76.26 in^4	>	17.98 in^4	O.K.		Addt'l Load to Post			Source	Lx	Ly	
Left Reaction =	1,352 lb	Post:	2X6	DFL#2	O.K.		lb			16.0	2.0	ft
Right Reaction =	1,585 lb	Post:	2X6	DFL#2	O.K.		lb			16.0	2.0	ft

DESCRIPTION:	RB9	Roof Tributary =	9.00	ft	W_{DL}=	388.01	lb/ft	C_D =	1.25	$C_{V/L}$ =	1.00	
SIZE:	6X6	Floor Tributary =	0.00	ft	W_{LL}=	180.00	lb/ft	C_t =	1.00	C_F =	1.00	
GRADE:	DFL#1 (TIMBERS)	Wall Tributary =	10.00	ft	P_{DL} =	0.00	lb	I_u =	6.50			
LENGTH (ft)=	6.50	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%		
NOTES:					$d_{Point\ Load}$ =	0.00	ft	Reduction (L)		%		
	Actual		Req'd			Source:			L/xxx	Alternate		
S =	27.73 in^3	>	21.33 in^3	O.K.	Max LL Deflection =	0.06 in =	L / 1316		360	#1	#2	LL
A =	30.25 in^2	>	11.19 in^2	O.K.	Max TL Deflection =	0.19 in =	L / 417		240			
I =	76.26 in^4	>	43.87 in^4	O.K.		Addt'l Load to Post			Source	Lx	Ly	
Left Reaction =	1,846 lb	Post:	2X6	DFL#2	O.K.		lb			12.0	2.0	ft
Right Reaction =	1,846 lb	Post:	2X6	DFL#2	O.K.		lb			12.0	2.0	ft

DESCRIPTION:	RB10	Roof Tributary =	15.00	ft	W_{DL}=	387.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	0.98	
SIZE:	5-1/8X12	Floor Tributary =	0.00	ft	W_{LL}=	300.00	lb/ft	C_t =	1.00	C_F =	1.00	
GRADE:	24F-V4	Wall Tributary =	3.00	ft	P_{DL} =	1,672.54	lb	I_u =	11.00			
LENGTH (ft)=	11.00	Additional Load =	0.00	plf	P_{LL} =	1,520.49	lb	Reduction (L_r)		%		
NOTES:					$d_{Point\ Load}$ =	4.00	ft	Reduction (L)		%		
	Actual		Req'd			Source:	GT14		L/xxx	Alternate		
S =	123.00 in^3	>	72.51 in^3	O.K.	Max LL Deflection =	0.12 in =	L / 1066		360	#1	#2	LL
A =	61.50 in^2	>	25.62 in^2	O.K.	Max TL Deflection =	0.27 in =	L / 481		240			
I =	738.00 in^4	>	367.86 in^4	O.K.		Addt'l Load to Post			Source	Lx	Ly	
Left Reaction =	5,810 lb	Post:	(2)2X6	DFL#2	O.K.		lb			10.0	2.0	ft
Right Reaction =	4,940 lb	Post:	2X6	DFL#2	O.K.		lb			10.0	2.0	ft

NEVADA - 6765 West Russell Road, Suite 200 - Las Vegas, NV 89118 - (702) 798-7978 - Fax (702) 451-2296
UTAH - 51 West 9000 South - Sandy, UT 84070 - (801) 565-9340 - Fax (801) 565-9340

L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-001-192
DATE 11/6/2019

PROJECT: Desert Rose Customs **SHEET** **OF**
SUBJECT: Beams & Headers **DESIGNED** NES **CHECKED** KAB

RB11

DESCRIPTION: RB11 | Roof Trib. = 5.00 ft | W_{DL} = 262.01 lb/ft | C_D = 1.25 | $C_{V/L}$ = 0.97
SIZE: 6X12 | Floor Trib. = 0.00 ft | W_{LL} = 100.00 lb/ft | C_t = 1.00 | C_F = 1.00
GRADE: DFL#1 (TIMBERS) | Wall Trib. = 8.00 ft | P_{DL} = 0.00 lb | l_u = 18.00
LENGTH (ft)= 18.00 | Additional Load = 0.00 plf | P_{LL} = 0.00 lb | Reduction (L_r) %
NOTES: | $d_{Point\ Load}$ = 0.00 ft | Reduction (L) %

	Actual		Req'd			Source:	L/xxx	Alternate Loads		
S =	121.23 in^3	>	107.03 in^3	O.K.	Max LL Deflection = 0.21 in =	L / 1020	360	#1	#2	LL
A =	63.25 in^2	>	20.55 in^2	O.K.	Max TL Deflection = 0.77 in =	L / 282	240			
I =	697.07 in^4	>	593.78 in^4	O.K.		Addt'l Load to Post	Source	Lx	Ly	
Left Reaction =	3,258 lb	Post:	(2)2X6	DFL#2	O.K.	lb		12.0	2.0	ft
Right Reaction =	3,258 lb	Post:	(2)2X6	DFL#2	O.K.	lb		12.0	2.0	ft

RB12

DESCRIPTION: RB12 | Roof Trib. = 5.00 ft | W_{DL} = 186.00 lb/ft | C_D = 1.25 | $C_{V/L}$ = 0.99
SIZE: 6X8 | Floor Trib. = 0.00 ft | W_{LL} = 100.00 lb/ft | C_t = 1.00 | C_F = 1.00
GRADE: DFL#1 (TIMBERS) | Wall Trib. = 4.00 ft | P_{DL} = 0.00 lb | l_u = 11.00
LENGTH (ft)= 11.00 | Additional Load = 0.00 plf | P_{LL} = 0.00 lb | Reduction (L_r) %
NOTES: | $d_{Point\ Load}$ = 0.00 ft | Reduction (L) %

	Actual		Req'd			Source:	L/xxx	Alternate		
S =	51.56 in^3	>	31.02 in^3	O.K.	Max LL Deflection = 0.11 in =	L / 1240	360	#1	#2	LL
A =	41.25 in^2	>	9.84 in^2	O.K.	Max TL Deflection = 0.30 in =	L / 433	240			
I =	193.36 in^4	>	107.06 in^4	O.K.		Addt'l Load to Post	Source	Lx	Ly	
Left Reaction =	1,573 lb	Post:	2X6	DFL#2	O.K.	lb		12.0	2.0	ft
Right Reaction =	1,573 lb	Post:	2X6	DFL#2	O.K.	lb		12.0	2.0	ft

RB13

DESCRIPTION: RB13 | Roof Trib. = 5.00 ft | W_{DL} = 262.01 lb/ft | C_D = 1.25 | $C_{V/L}$ = 0.99
SIZE: 6X10 | Floor Trib. = 0.00 ft | W_{LL} = 100.00 lb/ft | C_t = 1.00 | C_F = 1.00
GRADE: DFL#1 (TIMBERS) | Wall Trib. = 8.00 ft | P_{DL} = 0.00 lb | l_u = 12.50
LENGTH (ft)= 12.50 | Additional Load = 0.00 plf | P_{LL} = 0.00 lb | Reduction (L_r) %
NOTES: | $d_{Point\ Load}$ = 0.00 ft | Reduction (L) %

	Actual		Req'd			Source:	L/xxx	Alternate		
S =	82.73 in^3	>	50.93 in^3	O.K.	Max LL Deflection = 0.09 in =	L / 1717	360	#1	#2	LL
A =	52.25 in^2	>	13.95 in^2	O.K.	Max TL Deflection = 0.32 in =	L / 474	240			
I =	392.96 in^4	>	198.86 in^4	O.K.		Addt'l Load to Post	Source	Lx	Ly	
Left Reaction =	2,263 lb	Post:	2X6	DFL#2	O.K.	lb		8.0	2.0	ft
Right Reaction =	2,263 lb	Post:	2X6	DFL#2	O.K.	lb		8.0	2.0	ft

RB14

DESCRIPTION: RB14 | Roof Trib. = 7.00 ft | W_{DL} = 230.00 lb/ft | C_D = 1.25 | $C_{V/L}$ = 1.00
SIZE: 4X6 | Floor Trib. = 0.00 ft | W_{LL} = 140.00 lb/ft | C_t = 1.00 | C_F = 1.30
GRADE: DFL#2 | Wall Trib. = 4.00 ft | P_{DL} = 0.00 lb | l_u = 3.50
LENGTH (ft)= 3.50 | Additional Load = 0.00 plf | P_{LL} = 0.00 lb | Reduction (L_r) %
NOTES: | $d_{Point\ Load}$ = 0.00 ft | Reduction (L) %

	Actual		Req'd			Source:	L/xxx	Alternate		
S =	17.65 in^3	>	4.65 in^3	O.K.	Max LL Deflection = 0.01 in =	L / 6899	360	#1	#2	LL
A =	19.25 in^2	>	3.19 in^2	O.K.	Max TL Deflection = 0.02 in =	L / 2610	240			
I =	48.53 in^4	>	4.46 in^4	O.K.		Addt'l Load to Post	Source	Lx	Ly	
Left Reaction =	648 lb	Post:	2X6	DFL#2	O.K.	lb		8.0	2.0	ft
Right Reaction =	648 lb	Post:	2X6	DFL#2	O.K.	lb		8.0	2.0	ft

RB15

DESCRIPTION: RB15 | Roof Trib. = 5.00 ft | W_{DL} = 167.00 lb/ft | C_D = 1.25 | $C_{V/L}$ = 1.00
SIZE: 6X8 | Floor Trib. = 0.00 ft | W_{LL} = 100.00 lb/ft | C_t = 1.00 | C_F = 1.00
GRADE: DFL#2 | Wall Trib. = 3.00 ft | P_{DL} = 1,169.02 lb | l_u = 5.50
LENGTH (ft)= 5.50 | Additional Load = 0.00 plf | P_{LL} = 700.00 lb | Reduction (L_r) %
NOTES: | $d_{Point\ Load}$ = 2.50 ft | Reduction (L) %

	Actual		Req'd			Source: GT6	L/xxx	Alternate		
S =	51.56 in^3	>	37.84 in^3	O.K.	Max LL Deflection = 0.02 in =	L / 3291	360	#1	#2	LL
A =	41.25 in^2	>	10.58 in^2	O.K.	Max TL Deflection = 0.05 in =	L / 1233	240			
I =	193.36 in^4	>	37.65 in^4	O.K.		Addt'l Load to Post	Source	Lx	Ly	
Left Reaction =	1,754 lb	Post:	2X6	DFL#2	O.K.	lb		8.0	2.0	ft
Right Reaction =	1,584 lb	Post:	2X6	DFL#2	O.K.	lb		8.0	2.0	ft

L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-001-192
DATE 11/6/2019

PROJECT: **Desert Rose Customs** **SHEET** ______ **OF** ______
SUBJECT: **Beams & Headers** **DESIGNED** **NES** **CHECKED** **KAB**

DESCRIPTION:	RB16	Roof Tributary =	0.00	ft	W_{DL}=	0.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	0.94
SIZE:	5-1/8X15	Floor Tributary =	0.00	ft	W_{LL}=	0.00	lb/ft	C_t =	1.00	C_F =	1.00
GRADE:	24F-V4	Wall Tributary =	0.00	ft	P_{DL} =	0.00	lb	I_u =	17.00		
LENGTH (ft)=	17.00	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%	
NOTES:	SEE ADDT"L CALC				$d_{Point Load}$ =	0.00	ft	Reduction (L)		%	
					Source:			L/xxx	Alternate Loads		

	Actual			Req'd										
S =	192.19	in^3	>	0.00	in^3	O.K.	Max LL Deflection =	0.00 in =	N/A		360	#1	#2	LL
A =	76.88	in^2	>	0.00	in^2	O.K.	Max TL Deflection =	0.00 in =	N/A		240			
I =	1441.41	in^4	>	0.00	in^4	O.K.		Addt'l Load to Post			Source	Lx	Ly	
Left Reaction =	6,258	lb	Post:	(2)2X6		DFL#2	O.K.	6,257.54	lb		RB16	10.0	2.0	ft
Right Reaction =	7,218	lb	Post:	(2)2X6		DFL#2	O.K.	7,217.92	lb		RB16	10.0	2.0	ft

DESCRIPTION:		Roof Tributary =	0.00	ft	W_{DL}=	0.00	lb/ft	C_D =	1.00	$C_{V/L}$ =	1.00
SIZE:	4X6	Floor Tributary =	0.00	ft	W_{LL}=	0.00	lb/ft	C_t =	1.00	C_F =	1.30
GRADE:	DFL#2	Wall Tributary =	0.00	ft	P_{DL} =	0.00	lb	I_u =	1.00		
LENGTH (ft)=	1.00	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%	
NOTES:					$d_{Point Load}$ =	0.00	ft	Reduction (L)		%	
					Source:			L/xxx	Alternate		

	Actual			Req'd										
S =	17.65	in^3	>	0.00	in^3	O.K.	Max LL Deflection =	0.00 in =	N/A		360	#1	#2	LL
A =	19.25	in^2	>	0.00	in^2	O.K.	Max TL Deflection =	0.00 in =	N/A		240			
I =	48.53	in^4	>	0.00	in^4	O.K.		Addt'l Load to Post			Source	Lx	Ly	
Left Reaction =	0	lb	Post:	2X6		DFL#2	O.K.		lb			8.0	2.0	ft
Right Reaction =	0	lb	Post:	2X6		DFL#2	O.K.		lb			8.0	2.0	ft

DESCRIPTION:		Roof Tributary =	0.00	ft	W_{DL}=	0.00	lb/ft	C_D =	1.00	$C_{V/L}$ =	1.00
SIZE:	4X6	Floor Tributary =	0.00	ft	W_{LL}=	0.00	lb/ft	C_t =	1.00	C_F =	1.30
GRADE:	DFL#2	Wall Tributary =	0.00	ft	P_{DL} =	0.00	lb	I_u =	1.00		
LENGTH (ft)=	1.00	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%	
NOTES:					$d_{Point Load}$ =	0.00	ft	Reduction (L)		%	
					Source:			L/xxx	Alternate		

	Actual			Req'd										
S =	17.65	in^3	>	0.00	in^3	O.K.	Max LL Deflection =	0.00 in =	N/A		360	#1	#2	LL
A =	19.25	in^2	>	0.00	in^2	O.K.	Max TL Deflection =	0.00 in =	N/A		240			
I =	48.53	in^4	>	0.00	in^4	O.K.		Addt'l Load to Post			Source	Lx	Ly	
Left Reaction =	0	lb	Post:	2X6		DFL#2	O.K.		lb			8.0	2.0	ft
Right Reaction =	0	lb	Post:	2X6		DFL#2	O.K.		lb			8.0	2.0	ft

DESCRIPTION:		Roof Tributary =	0.00	ft	W_{DL}=	0.00	lb/ft	C_D =	1.00	$C_{V/L}$ =	1.00
SIZE:	4X6	Floor Tributary =	0.00	ft	W_{LL}=	0.00	lb/ft	C_t =	1.00	C_F =	1.30
GRADE:	DFL#2	Wall Tributary =	0.00	ft	P_{DL} =	0.00	lb	I_u =	1.00		
LENGTH (ft)=	1.00	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%	
NOTES:					$d_{Point Load}$ =	0.50	ft	Reduction (L)		%	
					Source:			L/xxx	Alternate		

	Actual			Req'd										
S =	17.65	in^3	>	0.00	in^3	O.K.	Max LL Deflection =	0.00 in =	N/A		360	#1	#2	LL
A =	19.25	in^2	>	0.00	in^2	O.K.	Max TL Deflection =	0.00 in =	N/A		240			
I =	48.53	in^4	>	0.00	in^4	O.K.		Addt'l Load to Post			Source	Lx	Ly	
Left Reaction =	0	lb	Post:	2X6		DFL#2	O.K.		lb			8.0	2.0	ft
Right Reaction =	0	lb	Post:	2X6		DFL#2	O.K.		lb			8.0	2.0	ft

DESCRIPTION:		Roof Tributary =	0.00	ft	W_{DL}=	0.00	lb/ft	C_D =	1.00	$C_{V/L}$ =	1.00
SIZE:	4X6	Floor Tributary =	0.00	ft	W_{LL}=	0.00	lb/ft	C_t =	1.00	C_F =	1.30
GRADE:	DFL#2	Wall Tributary =	0.00	ft	P_{DL} =	0.00	lb	I_u =	1.00		
LENGTH (ft)=	1.00	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%	
NOTES:					$d_{Point Load}$ =	0.50	ft	Reduction (L)		%	
					Source:			L/xxx	Alternate		

	Actual			Req'd										
S =	17.65	in^3	>	0.00	in^3	O.K.	Max LL Deflection =	0.00 in =	N/A		360	#1	#2	LL
A =	19.25	in^2	>	0.00	in^2	O.K.	Max TL Deflection =	0.00 in =	N/A		240			
I =	48.53	in^4	>	0.00	in^4	O.K.		Addt'l Load to Post			Source	Lx	Ly	
Left Reaction =	0	lb	Post:	2X6		DFL#2	O.K.		lb			8.0	2.0	ft
Right Reaction =	0	lb	Post:	2X6		DFL#2	O.K.		lb			8.0	2.0	ft

L. R. NELSON CONSULTING ENGINEERS

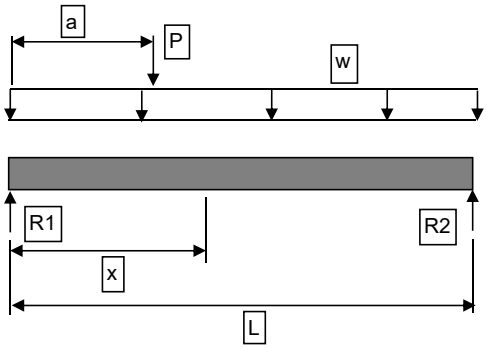
JOB NO. 1939-001-192
DATE:

PROJECT Desert Rose Customs **SHEET** **OF**
SUBJECT BEAMS AND HEADERS **DESIGNED** NES **CHECKED** KAB

(SIMPLY SUPPORTED BEAM)

DESCRIPTION: RB16

SIZE:	**5-1/8X15**		C_D =	1.25	
GRADE:	24F-V4		C_M =	1.00	
LENGTH =	17.00	ft	C_t =	1.00	
			C_L =	1.00	
F_b=	3,000	psi	C_r =	1.00	
F_v=	300	psi	C_c =	1.00	
E=	1,800,000	psi	C_f =	1.00	
			C_V =	1.00	
M_{MAX} =	37,123	lb-ft	C_F =	1.00	
V_{MAX} =	7,218	lb	C_{fu} =	1.00	
			I_{actual}=	1,441.41	
R_1 =	6,257.54	lb			
R_2 =	7,217.92	lb			
S_{req}=	148.49	IN^3			
A_{req}=	35.23	IN^2			
$Delta_{LLALLOW.}$ =	0.57	in			
$Delta_{TLALLOW.}$ =	0.85	in			



CHECK:

S_{actual}=	192.19	in^3	>	148.49	in^3	**O.K.**
A_{actual}=	76.88	in^2	>	35.23	in^2	**O.K.**
Delta LL =	0.29	in	<	0.57	in	**O.K.**
Delta TL =	0.68	in	<	0.85	in	**O.K.**

1.5xDL Deflection = **0.58** inches

LL => L/? = 692
TL => L/? = 299

UNIFORM LOADS

W_{DL}=	71.96	lb/ft	W_{LL}=	65.42	lb/ft
W_{DL}=	0.00	lb/ft	W_{LL}=	0.00	lb/ft
W_{DL}=	0.00	lb/ft	W_{LL}=	0.00	lb/ft
W_{DL}=	0.00	lb/ft	W_{LL}=	0.00	lb/ft
W_{DL}=	0.00	lb/ft	W_{LL}=	0.00	lb/ft

POINT LOADS

			a (ft)				a (ft)	
P_{DL} =	2,672.04	lb @	9.00	P_{LL} =	1,600.00	lb @	9.00	GT1
P_{DL} =	1,309.55	lb @	13.33	P_{LL} =	1,190.50	lb @	13.33	GT12
P_{DL} =	0.00	lb @	0.00	P_{LL} =	0.00	lb @	0.00	
P_{DL} =	0.00	lb @	0.00	P_{LL} =	0.00	lb @	0.00	
P_{DL} =	0.00	lb @	0.00	P_{LL} =	0.00	lb @	0.00	

UNIFORM LOADS PARTIALLY DISTRIBUTED

			a (ft)	b (ft)				a (ft)	b (ft)
W_{DL}=	176.00	lb/ft @	0.00	9.00	W_{LL}=	160.00	lb/ft @	0.00	9.00
W_{DL}=	88.00	lb/ft @	9.00	8.00	W_{LL}=	80.00	lb/ft @	9.00	8.00
W_{DL}=	0.00	lb/ft @	0.00	0.00	W_{LL}=	0.00	lb/ft @	0.00	0.00
W_{DL}=	0.00	lb/ft @	0.00	0.00	W_{LL}=	0.00	lb/ft @	0.00	0.00
W_{DL}=	0.00	lb/ft @	0.00	0.00	W_{LL}=	0.00	lb/ft @	0.00	0.00

NOTE: b IS THE DISTANCE FROM THE START TO THE END OF THE UNIFORMLY DISTRIBUTED LOAD.

L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-001-192
DATE 11/6/2019

PROJECT: Desert Rose Customs **SHEET** **OF**
SUBJECT: Beams & Headers **DESIGNED** NES **CHECKED** KAB

FB1

DESCRIPTION: FB1 | Roof Trib. = 5.00 ft | W_{DL} = 526.02 lb/ft | C_D = 1.25 | $C_{V/L}$ = 1.00
SIZE: 5-1/8X13-1/2 | Floor Trib. = 6.00 ft | W_{LL} = 340.00 lb/ft | C_t = 1.00 | C_F = 1.00
GRADE: 24F-V4 | Wall Trib. = 14.00 ft | P_{DL} = 4,112.19 lb | I_u = 2.00
LENGTH (ft)= 13.50 | Additional Load = 0.00 plf | P_{LL} = 2,564.03 lb | Reduction (L_r) %
NOTES: | $d_{Point\ Load}$ = 1.00 ft | Reduction (L) %
Source: GT3 | L/xxx | Alternate Loads

	Actual			Req'd								
S =	155.67	in^3	>	93.17	in^3	O.K.	Max LL Deflection =	0.16 in =	L / 1007	360	#1	#2	LL
A =	69.19	in^2	>	55.27	in^2	O.K.	Max TL Deflection =	0.41 in =	L / 394	240			
I =	1050.79	in^4	>	640.45	in^4	O.K.							

						Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	12,027	lb	Post:	(2)2X6	DFL#2	O.K.		lb		10.5	2.0	ft
Right Reaction =	6,340	lb	Post:	2X6	DFL#2	O.K.		lb		10.5	2.0	ft

FB2

DESCRIPTION: FB2 | Roof Trib. = 16.00 ft | W_{DL} = 768.02 lb/ft | C_D = 1.25 | $C_{V/L}$ = 1.00
SIZE: 5-1/8X13-1/2 | Floor Trib. = 6.00 ft | W_{LL} = 560.00 lb/ft | C_t = 1.00 | C_F = 1.00
GRADE: 24F-V4 | Wall Trib. = 14.00 ft | P_{DL} = 0.00 lb | I_u = 2.00
LENGTH (ft)= 14.25 | Additional Load = 0.00 plf | P_{LL} = 0.00 lb | Reduction (L_r) %
NOTES: | $d_{Point\ Load}$ = 0.00 ft | Reduction (L) %
Source: | L/xxx | Alternate

	Actual			Req'd								
S =	155.67	in^3	>	135.33	in^3	O.K.	Max LL Deflection =	0.27 in =	L / 623	360	#1	#2	LL
A =	69.19	in^2	>	39.84	in^2	O.K.	Max TL Deflection =	0.65 in =	L / 263	240			
I =	1050.79	in^4	>	960.70	in^4	O.K.							

						Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	9,462	lb	Post:	(2)2X6	DFL#2	O.K.		lb		10.5	2.0	ft
Right Reaction =	9,462	lb	Post:	(2)2X6	DFL#2	O.K.		lb		10.5	2.0	ft

FB3

DESCRIPTION: FB3 | Roof Trib. = 5.00 ft | W_{DL} = 526.02 lb/ft | C_D = 1.25 | $C_{V/L}$ = 1.00
SIZE: 5-1/8X12 | Floor Trib. = 6.00 ft | W_{LL} = 340.00 lb/ft | C_t = 1.00 | C_F = 1.00
GRADE: 24F-V4 | Wall Trib. = 14.00 ft | P_{DL} = 4,112.19 lb | I_u = 2.00
LENGTH (ft)= 11.00 | Additional Load = 0.00 plf | P_{LL} = 2,564.03 lb | Reduction (L_r) %
NOTES: | $d_{Point\ Load}$ = 7.67 ft | Reduction (L) %
Source: GT3 | L/xxx | Alternate

	Actual			Req'd								
S =	123.00	in^3	>	106.63	in^3	O.K.	Max LL Deflection =	0.16 in =	L / 833	360	#1	#2	LL
A =	61.50	in^2	>	42.75	in^2	O.K.	Max TL Deflection =	0.41 in =	L / 324	240			
I =	738.00	in^4	>	547.42	in^4	O.K.							

						Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	6,786	lb	Post:	2X6	DFL#2	O.K.		lb		8.0	2.0	ft
Right Reaction =	9,416	lb	Post:	2X6	DFL#2	O.K.		lb		8.0	2.0	ft

FB4

DESCRIPTION: FB4 | Roof Trib. = 16.25 ft | W_{DL} = 471.50 lb/ft | C_D = 1.25 | $C_{V/L}$ = 0.98
SIZE: 5-1/8X12 | Floor Trib. = 0.00 ft | W_{LL} = 325.00 lb/ft | C_t = 1.00 | C_F = 1.00
GRADE: 24F-V4 | Wall Trib. = 6.00 ft | P_{DL} = 4,139.08 lb | I_u = 12.50
LENGTH (ft)= 12.50 | Additional Load = 0.00 plf | P_{LL} = 2,646.90 lb | Reduction (L_r) %
NOTES: | $d_{Point\ Load}$ = 10.50 ft | Reduction (L) %
Source: FB3 | L/xxx | Alternate

	Actual			Req'd								
S =	123.00	in^3	>	94.67	in^3	O.K.	Max LL Deflection =	0.20 in =	L / 751	360	#1	#2	LL
A =	61.50	in^2	>	49.41	in^2	O.K.	Max TL Deflection =	0.50 in =	L / 302	240			
I =	738.00	in^4	>	586.91	in^4	O.K.							

						Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	6,064	lb	Post:	2X6	DFL#2	O.K.		lb		10.0	2.0	ft
Right Reaction =	10,678	lb	Post:	(2)2X6	DFL#2	O.K.		lb		10.0	2.0	ft

FB5

DESCRIPTION: FB5 | Roof Trib. = 5.00 ft | W_{DL} = 426.01 lb/ft | C_D = 1.00 | $C_{V/L}$ = 1.00
SIZE: 6X6 | Floor Trib. = 2.00 ft | W_{LL} = 180.00 lb/ft | C_t = 1.00 | C_F = 1.00
GRADE: DFL#1 (TIMBERS) | Wall Trib. = 14.00 ft | P_{DL} = 0.00 lb | I_u = 3.50
LENGTH (ft)= 3.50 | Additional Load = 0.00 plf | P_{LL} = 0.00 lb | Reduction (L_r) %
NOTES: | $d_{Point\ Load}$ = 0.50 ft | Reduction (L) %
Source: | L/xxx | Alternate

	Actual			Req'd								
S =	27.73	in^3	>	8.25	in^3	O.K.	Max LL Deflection =	0.00 in =	L / 8432	360	#1	#2	LL
A =	30.25	in^2	>	6.91	in^2	O.K.	Max TL Deflection =	0.02 in =	L / 2504	240			
I =	76.26	in^4	>	7.31	in^4	O.K.							

						Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	1,061	lb	Post:	2X6	DFL#2	O.K.		lb		8.0	2.0	ft
Right Reaction =	1,061	lb	Post:	2X6	DFL#2	O.K.		lb		8.0	2.0	ft

L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-001-192
DATE: 11/06/19

PROJECT Desert Rose Customs **SHEET** **OF**
SUBJECT BEAMS AND HEADERS **DESIGNED** NES **CHECKED** KAB

Span , ft 12.50 MEMBER: FB4 SEISMIC AND WIND CHECK
UNIFORM LOADS, plf (continuous across entire member)
w1DL 471.50 Continuous lateral support (1= yes, 0= no)? 1 Yes
w1LL 325.00 Stresses adjusted to ASD design (1= yes, 0= no)? 1 Yes

POINT LOADS, lbs Distance to Point Loads (from left end of span), ft

P1 D	4139.08	a1	10.33 ft	FB3	P3 D	0.00	a3	1.00 ft
P1 LL	2646.90				P3 LL	0.00		
P2 D	0.00	a2	1.00 ft		P4 D	0.00	a4	1.00 ft
P2 LL	0.00				P4 LL	0.00		
PE1	1303.59	a E1	10.33 ft		PW1	1099.18	a W1	10.33 ft
PE2	0.00	a E2	0.00 ft		PW2	0.00	a W2	0.00 ft
PE3	0.00	a E3	0.00 ft		PW3	0.00	a W3	0.00 ft
PE4	0.00	a E4	0.00 ft		PW4	0.00	a W4	0.00 ft

(Note: seismic loads (PE) should be at ASD level, eg E/1.4,program adjusts PE loads to Strength levels)

LINEAR LOADS, plf

qa DL	0.00	a q	1.00 ft	CD =	1.60
qa LL	0.00	b q	1.00 ft	Fasd =	1.2
qb DL	0.00			Ω =	2.5
qb LL	0.00			SDS =	0.422
				f =	0.75
		1.0+0.14SDS =	1.059	1.0+0.105SDS =	1.044
				0.6-0.14SDS =	0.541

ANALYSIS RESULTS

CASE 1, (1+0.105SDS)D + 0.75(LL+Lr) + 0.525ΩE
CASE 2, (0.6-0.14SDS)D - 0.7ΩE
CASE 3, D +0.75(LL + Lr + (W or 0.7E))
CASE 4, 0.6D - (W or 0.7E)
CASE 5, (1+0.14SDS)D + 0.7ΩE

	MAXIMUM	CASE	MINIMUM	CASE
V_a =	5804 LB	1	1307 LB	2
V_b =	11521 LB	1	643 LB	2
RL =	6119 LB	1	1416 LB	2
RR =	11835 LB	1	752 LB	2

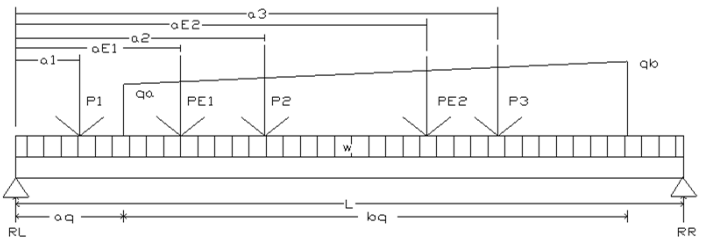
MEMBER DESIGN FORCES

NEGATIVE MOMENT	0 LB-FT
POSITIVE MOMENT	25415 LB-FT
MAXIMUM SHEAR	11521 LB

MEMBER DESIGN

		A in^2	S in^3	I in^4
Member Type	24F-V4			
Member Section	5-1/8X12	61.50	123.00	738.00

Allowable Stress	psi	adjustment factor CD(CL)..(Cx)(Fasd)	Adjusted Stress psi	Member Stresses psi	
Fv	240.00	1.92	F`v = 461 psi	fv =	281 psi OKAY
Fb	2400.00	1.92	F`b = 4608 psi	pos fb =	2479 psi OKAY
E	1.8E+06	1.00	E` = 1.8E+06	neg fb =	0 psi OKAY
Fb_neg	1850	1.92	F`b neg = 3552 psi		



Provide: 5-1/8X12
24F-V4

NOTES: NO STRAP REQUIRED AT EITHER END OF MEMBER

L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO.	1939-001-192
DATE	11/6/2019

PROJECT:	Desert Rose Customs	SHEET		OF	
SUBJECT:	Beams & Headers	DESIGNED	NES	CHECKED	KAB

DESCRIPTION:	FB6	Roof Tributary =	0.00	ft	W_{DL}=	576.02	lb/ft	C_D =	1.00	$C_{V/L}$ =	1.00
SIZE:	6X6	Floor Tributary =	20.00	ft	W_{LL}=	800.00	lb/ft	C_t =	1.00	C_F =	1.00
GRADE:	DFL#1 (TIMBERS)	Wall Tributary =	4.00	ft	P_{DL} =	0.00	lb	I_u =	3.00		
LENGTH (ft)=	3.00	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%	
NOTES:					$d_{Point\ Load}$ =	0.00	ft	Reduction (L)		%	
	Actual		Req'd		Source:			L/xxx	Alternate Loads		
S =	27.73 in^3	>	13.76 in^3	O.K.	Max LL Deflection =	0.01 in =	L / 3013	360	#1	#2	LL
A =	30.25 in^2	>	12.65 in^2	O.K.	Max TL Deflection =	0.02 in =	L / 1751	240			
I =	76.26 in^4	>	10.45 in^4	O.K.		Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	2,064 lb	Post:	2X6	DFL#2	O.K.		lb		8.0	2.0	ft
Right Reaction =	2,064 lb	Post:	2X6	DFL#2	O.K.		lb		8.0	2.0	ft

DESCRIPTION:	FB7	Roof Tributary =	0.00	ft	W_{DL}=	576.02	lb/ft	C_D =	1.00	$C_{V/L}$ =	1.00
SIZE:	6X8	Floor Tributary =	20.00	ft	W_{LL}=	800.00	lb/ft	C_t =	1.00	C_F =	1.00
GRADE:	DFL#1 (TIMBERS)	Wall Tributary =	4.00	ft	P_{DL} =	0.00	lb	I_u =	5.50		
LENGTH (ft)=	5.50	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%	
NOTES:					$d_{Point\ Load}$ =	0.00	ft	Reduction (L)		%	
	Actual		Req'd		Source:			L/xxx	Alternate		
S =	51.56 in^3	>	46.25 in^3	O.K.	Max LL Deflection =	0.05 in =	L / 1240	360	#1	#2	LL
A =	41.25 in^2	>	25.80 in^2	O.K.	Max TL Deflection =	0.09 in =	L / 721	240			
I =	193.36 in^4	>	64.39 in^4	O.K.		Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	3,784 lb	Post:	2X6	DFL#2	O.K.		lb		9.0	2.0	ft
Right Reaction =	3,784 lb	Post:	2X6	DFL#2	O.K.		lb		9.0	2.0	ft

DESCRIPTION:	FB8	Roof Tributary =	0.00	ft	W_{DL}=	557.02	lb/ft	C_D =	1.25	$C_{V/L}$ =	1.00
SIZE:	4X8	Floor Tributary =	20.00	ft	W_{LL}=	800.00	lb/ft	C_t =	1.00	C_F =	1.30
GRADE:	DFL#2	Wall Tributary =	3.00	ft	P_{DL} =	0.00	lb	I_u =	4.00		
LENGTH (ft)=	4.00	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%	
NOTES:	Alt: 6x6				$d_{Point\ Load}$ =	0.00	ft	Reduction (L)		%	
	Actual		Req'd		Source:			L/xxx	Alternate		
S =	30.66 in^3	>	22.27 in^3	O.K.	Max LL Deflection =	0.03 in =	L / 1852	360	#1	#2	LL
A =	25.38 in^2	>	12.63 in^2	O.K.	Max TL Deflection =	0.04 in =	L / 1092	240			
I =	111.15 in^4	>	24.43 in^4	O.K.		Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	2,714 lb	Post:	2X4	DFL#2	O.K.		lb		8.0	2.0	ft
Right Reaction =	2,714 lb	Post:	2X4	DFL#2	O.K.		lb		8.0	2.0	ft

DESCRIPTION:		Roof Tributary =	0.00	ft	W_{DL}=	0.00	lb/ft	C_D =	1.00	$C_{V/L}$ =	1.00
SIZE:	4X6	Floor Tributary =	0.00	ft	W_{LL}=	0.00	lb/ft	C_t =	1.00	C_F =	1.30
GRADE:	DFL#2	Wall Tributary =	0.00	ft	P_{DL} =	0.00	lb	I_u =	1.00		
LENGTH (ft)=	1.00	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%	
NOTES:					$d_{Point\ Load}$ =	0.00	ft	Reduction (L)		%	
	Actual		Req'd		Source:			L/xxx	Alternate		
S =	17.65 in^3	>	0.00 in^3	O.K.	Max LL Deflection =	0.00 in =	N/A	360	#1	#2	LL
A =	19.25 in^2	>	0.00 in^2	O.K.	Max TL Deflection =	0.00 in =	N/A	240			
I =	48.53 in^4	>	0.00 in^4	O.K.		Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	0 lb	Post:	2X6	DFL#2	O.K.		lb		8.0	2.0	ft
Right Reaction =	0 lb	Post:	2X6	DFL#2	O.K.		lb		8.0	2.0	ft

DESCRIPTION:		Roof Tributary =	0.00	ft	W_{DL}=	0.00	lb/ft	C_D =	1.00	$C_{V/L}$ =	1.00
SIZE:	4X6	Floor Tributary =	0.00	ft	W_{LL}=	0.00	lb/ft	C_t =	1.00	C_F =	1.30
GRADE:	DFL#2	Wall Tributary =	0.00	ft	P_{DL} =	0.00	lb	I_u =	1.00		
LENGTH (ft)=	1.00	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%	
NOTES:					$d_{Point\ Load}$ =	0.00	ft	Reduction (L)		%	
	Actual		Req'd		Source:			L/xxx	Alternate		
S =	17.65 in^3	>	0.00 in^3	O.K.	Max LL Deflection =	0.00 in =	N/A	360	#1	#2	LL
A =	19.25 in^2	>	0.00 in^2	O.K.	Max TL Deflection =	0.00 in =	N/A	240			
I =	48.53 in^4	>	0.00 in^4	O.K.		Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	0 lb	Post:	2X6	DFL#2	O.K.		lb		8.0	2.0	ft
Right Reaction =	0 lb	Post:	2X6	DFL#2	O.K.		lb		8.0	2.0	ft

L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-001-192
DATE:

PROJECT: Desert Rose Customs | SHEET | OF
DESCRIPTION: Worst Case | DESIGNED NES | CHECKED KAB

STUD WALL DESIGN - ASCE 7 WIND PROVISIONS (BASIC LOAD COMBINATIONS)

Roof Dead Load =	22.00 psf
Floor Dead Load =	25.00 psf
Wall/Misc. Dead Load =	19.00 psf

Roof Live Load =	20.00 psf
Floor Live Load =	40.00 psf
Roof Snow Load =	0.00 psf

Wall Height =	10.00 ft
Roof Tributary Width =	16.00 ft
Floor Tributary Width =	0.00 ft
Wall/Misc. DL Width =	4.00 ft

Wind Zone (Interior or End) =	End

Stud Size =	2X6
Stud Spacing =	16 in o.c.
Lumber Grade =	DFL#2

Wind Speed =	100 mph
Exposure =	C
Mean Roof Height, h =	25.13 ft
Topo. Factor, K_{zt} =	1.00
Wind Dir. Factor, K_d =	0.85
GCpi +/- =	0.18

INTERACTION RESULTS FOR POSSIBLE CONTROLLING LOAD COMBINATIONS		
DL + FLL =	**0.085**	**OK**
DL + RLL =	**0.140**	**OK**
DL + SL =	**0.085**	**OK**
DL + 0.6*WIND =	**0.244**	**OK**
DL + 0.75*(FLL + RLL) =	**0.140**	**OK**
DL + 0.75*(FLL + SL) =	**0.080**	**OK**
DL + 0.75*(0.6*WIND + FLL + RLL) =	**0.201**	**OK**
DL + 0.75*(0.6*WIND + FLL + SL) =	**0.185**	**OK**
DEFLECTION L/? =	**1052.6**	**O.K.****

** Deflection Determined Using 0.42 x Wind Load

Exterior Finish =	Brittle

(L/240 Deflection Ratio)

F_b =	900 psi
F_c =	1,350 psi
E =	1,600,000 psi

Factors	
C_F (Bending) =	1.30
C_F (Compression) =	1.10
C_r =	1.15
C_D (DL) =	1.00
C_D (FLL) =	1.00
C_D (RLL) =	1.25
C_D (SL) =	1.00
C_D (Wind) =	1.60

Wall Uniform Loads	
Dead Load, DL =	428 lb/ft
Roof Live Load, RLL =	320 lb/ft
Floor Live Load, FLL =	0 lb/ft
Snow Load, SL =	0 lb/ft

Ultimate Design Pressure =	30.11 psf
Ultimate Max. Deflection =	0.27 in
Max. Deflection x 0.42 =	0.11 in
Max. Deflection x 0.60 =	N/A
L/d =	21.82

< 50 (OK)

Check Bearing Stress on Wall Plate	
Wall Plate Lumber Grade =	DFL#2
Max. Stud Reaction on Wall Plate =	997 lbs
Bearing Stress on Wall Plate =	120.89 psi
Allow. Bearing Stress on Wall Plate =	625 psi

(OK)

Ultimate Wall Pressures (psf)				
Zone	Interior Zone (Zone 4)		(Zone 5)	
Positive/Negative	Positive	Negative	Positive	Negative
Pressure (psf)	23.37 psf	-25.51 psf	23.37 psf	-30.11 psf
qh (psf)	21.33 psf	21.33 psf	21.33 psf	21.33 psf
GCp	0.916	-1.016	0.916	-1.231
Gcpi	0.180	-0.180	0.180	-0.180

Studs Part of Fire Assembly? No

Stud Section Properties		
Area =	8.25	in^2
Section Modulus =	7.56	in^3
Moment of Inertia =	20.80	in^4
Depth =	5.50 in	
Width =	1.50 in	

Axial Stresses				
	DL	FLL	RLL	SL
f_a =	69.17 psi	0.00 psi	51.72 psi	0.00 psi

Maximum Bending Moment & Stress	
M =	502 ft-lb
f_b =	796.22 psi

Allowable Bending Stress		
F'_b =	2,153 psi	

Allowable Axial Stresses					
c =	0.80	Overwrite c =			
$(L/d)^2$ =	476.03				
COV_E =	0.25	Overwrite COV_E =			
E_{min}' =	584,494 psi				
F_{cE} =	1009.29				
	DL	FLL	RLL	SL	WIND
F_c^* =	1,485.00	1,485.00	1,856.25	1,485.00	2,376.00
F_{cE}/F_c^* =	0.680	0.680	0.544	0.680	0.425
C_P =	0.547	0.547	0.464	0.547	0.379
F'c =	812.76	812.76	860.54	812.76	899.64

psi

Note: All wall studs in this calculator are braced in the weak direction.

L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-001-192
DATE:

PROJECT: Desert Rose Customs | SHEET | OF
DESCRIPTION: FULL HEIGHT AT STAIRS | DESIGNED NES | CHECKED KAB

STUD WALL DESIGN - ASCE 7 WIND PROVISIONS (BASIC LOAD COMBINATIONS)

Roof Dead Load =	22.00 psf
Floor Dead Load =	25.00 psf
Wall/Misc. Dead Load =	19.00 psf

Roof Live Load =	20.00 psf
Floor Live Load =	40.00 psf
Roof Snow Load =	0.00 psf

Wall Height =	21.50 ft
Roof Tributary Width =	5.00 ft
Floor Tributary Width =	0.00 ft
Wall/Misc. DL Width =	11.00 ft

Wind Zone (Interior or End) =	End

Stud Size =	(2)2X6
Stud Spacing =	12 in o.c.
Lumber Grade =	DFL#2

Wind Speed =	100 mph
Exposure =	C
Mean Roof Height, h =	25.13 ft
Topo. Factor, K_{zt} =	1.00
Wind Dir. Factor, K_d =	0.85
GCpi +/- =	0.18

INTERACTION RESULTS FOR POSSIBLE CONTROLLING LOAD COMBINATIONS		
DL + FLL =	**0.091**	**OK**
DL + RLL =	**0.119**	**OK**
DL + SL =	**0.091**	**OK**
DL + 0.6*WIND =	**0.360**	**OK**
DL + 0.75*(FLL + RLL) =	**0.119**	**OK**
DL + 0.75*(FLL + SL) =	**0.091**	**OK**
DL + 0.75*(0.6*WIND + FLL + RLL) =	**0.282**	**OK**
DL + 0.75*(0.6*WIND + FLL + SL) =	**0.272**	**OK**
DEFLECTION L/? =	**339.2**	**O.K.****

** Deflection Determined Using 0.42 x Wind Load

Exterior Finish =	Brittle	(L/240 Deflection Ratio)

F_b =	900 psi
F_c =	1,350 psi
E =	1,600,000 psi

Factors	
C_F (Bending) =	1.30
C_F (Compression) =	1.10
C_r =	1.15
C_D (DL) =	1.00
C_D (FLL) =	1.00
C_D (RLL) =	1.25
C_D (SL) =	1.00
C_D (Wind) =	1.60

Wall Uniform Loads	
Dead Load, DL =	319 lb/ft
Roof Live Load, RLL =	100 lb/ft
Floor Live Load, FLL =	0 lb/ft
Snow Load, SL =	0 lb/ft

Ultimate Design Pressure =	25.07 psf	
Ultimate Max. Deflection =	1.81 in	
Max. Deflection x 0.42 =	0.76 in	
Max. Deflection x 0.60 =	N/A	
L/d =	46.91	< 50 (OK)

Check Bearing Stress on Wall Plate		
Wall Plate Lumber Grade =	DFL#2	
Max. Stud Reaction on Wall Plate =	419 lbs	
Bearing Stress on Wall Plate =	25.39 psi	(OK)
Allow. Bearing Stress on Wall Plate =	625 psi	

Ultimate Wall Pressures (psf)				
Zone	Interior Zone (Zone 4)		(Zone 5)	
Positive/Negative	Positive	Negative	Positive	Negative
Pressure (psf)	20.85 psf	-22.99 psf	20.85 psf	-25.07 psf
qh (psf)	21.33 psf	21.33 psf	21.33 psf	21.33 psf
GCp	0.798	-0.898	0.798	-0.995
Gcpi	0.180	-0.180	0.180	-0.180

Studs Part of Fire Assembly? No

Stud Section Properties		
Area =	16.50	in^2
Section Modulus =	15.13	in^3
Moment of Inertia =	41.59	in^4
Depth =	5.50 in	
Width =	3.00 in	

Axial Stresses				
	DL	FLL	RLL	SL
f_a =	19.33 psi	0.00 psi	6.06 psi	0.00 psi

Maximum Bending Moment & Stress	
M =	1448 ft-lb
f_b =	1149.13 psi

Allowable Bending Stress		
F'_b =	2,153 psi	

Allowable Axial Stresses					
c =	0.80	Overwrite c =			
$(L/d)^2$ =	2200.46				
COV_E =	0.25	Overwrite COV_E =			
E_{min}' =	584,494 psi				
F_{cE} =	218.34				
	DL	FLL	RLL	SL	WIND
F_c^* =	1,485.00	1,485.00	1,856.25	1,485.00	2,376.00
F_{cE}/F_c^* =	0.147	0.147	0.118	0.147	0.092
C_P =	0.142	0.142	0.115	0.142	0.090
F'c =	211.33	211.33	212.83	211.33	214.10 psi

Note: All wall studs in this calculator are braced in the weak direction.

L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-001-192
DATE:

PROJECT: Desert Rose Customs | SHEET | OF
DESCRIPTION: GARAGE | DESIGNED NES | CHECKED KAB

STUD WALL DESIGN - ASCE 7 WIND PROVISIONS (BASIC LOAD COMBINATIONS)

Roof Dead Load =	22.00 psf
Floor Dead Load =	25.00 psf
Wall/Misc. Dead Load =	19.00 psf

Roof Live Load =	20.00 psf
Floor Live Load =	40.00 psf
Roof Snow Load =	0.00 psf

Wall Height =	12.00 ft
Roof Tributary Width =	13.50 ft
Floor Tributary Width =	0.00 ft
Wall/Misc. DL Width =	4.00 ft

Wind Zone (Interior or End) =	End

Stud Size =	2X6
Stud Spacing =	16 in o.c.
Lumber Grade =	DFL#2

Wind Speed =	100 mph
Exposure =	C
Mean Roof Height, h =	16.00 ft
Topo. Factor, K_{zt} =	1.00
Wind Dir. Factor, K_d =	0.85
GCpi +/- =	0.18

INTERACTION RESULTS FOR POSSIBLE CONTROLLING LOAD COMBINATIONS		
DL + FLL =	**0.098**	**OK**
DL + RLL =	**0.164**	**OK**
DL + SL =	**0.098**	**OK**
DL + 0.6*WIND =	**0.316**	**OK**
DL + 0.75*(FLL + RLL) =	**0.164**	**OK**
DL + 0.75*(FLL + SL) =	**0.095**	**OK**
DL + 0.75*(0.6*WIND + FLL + RLL) =	**0.263**	**OK**
DL + 0.75*(0.6*WIND + FLL + SL) =	**0.239**	**OK**
DEFLECTION L/? =	**693.8**	**O.K.****

** Deflection Determined Using 0.42 x Wind Load

Exterior Finish =	Brittle	(L/240 Deflection Ratio)

F_b =	900 psi
F_c =	1,350 psi
E =	1,600,000 psi

Factors	
C_F (Bending) =	1.30
C_F (Compression) =	1.10
C_r =	1.15
C_D (DL) =	1.00
C_D (FLL) =	1.00
C_D (RLL) =	1.25
C_D (SL) =	1.00
C_D (Wind) =	1.60

Wall Uniform Loads	
Dead Load, DL =	373 lb/ft
Roof Live Load, RLL =	270 lb/ft
Floor Live Load, FLL =	0 lb/ft
Snow Load, SL =	0 lb/ft

Ultimate Design Pressure =	26.43 psf
Ultimate Max. Deflection =	0.49 in
Max. Deflection x 0.42 =	0.21 in
Max. Deflection x 0.60 =	N/A
L/d =	26.18 < 50 (OK)

Check Bearing Stress on Wall Plate		
Wall Plate Lumber Grade =	DFL#2	
Max. Stud Reaction on Wall Plate =	857 lbs	
Bearing Stress on Wall Plate =	103.92 psi	(OK)
Allow. Bearing Stress on Wall Plate =	625 psi	

Ultimate Wall Pressures (psf)				
Zone	Interior Zone (Zone 4)		(Zone 5)	
Positive/Negative	Positive	Negative	Positive	Negative
Pressure (psf)	20.86 psf	-22.82 psf	20.86 psf	-26.43 psf
qh (psf)	19.59 psf	19.59 psf	19.59 psf	19.59 psf
GCp	0.885	-0.985	0.885	-1.169
Gcpi	0.180	-0.180	0.180	-0.180

Studs Part of Fire Assembly? No

Stud Section Properties		
Area =	8.25	in^2
Section Modulus =	7.56	in^3
Moment of Inertia =	20.80	in^4
Depth =	5.50 in	
Width =	1.50 in	

Axial Stresses				
	DL	FLL	RLL	SL
f_a =	60.28 psi	0.00 psi	43.64 psi	0.00 psi

Maximum Bending Moment & Stress	
M =	634 ft-lb
f_b =	1006.62 psi

Allowable Bending Stress		
F'_b =	2,153 psi	

Allowable Axial Stresses					
c =	0.80	Overwrite c =			
$(L/d)^2$ =	685.49				
COV_E =	0.25	Overwrite COV_E =			
E_{min}' =	584,494 psi				
F_{cE} =	700.89				
	DL	FLL	RLL	SL	WIND
F_c^* =	1,485.00	1,485.00	1,856.25	1,485.00	2,376.00
F_{cE}/F_c^* =	0.472	0.472	0.378	0.472	0.295
C_P =	0.414	0.414	0.342	0.414	0.274
F'c =	614.24	614.24	634.89	614.24	651.64 psi

Note: All wall studs in this calculator are braced in the weak direction.

L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-001-192
DATE:

PROJECT: Desert Rose Customs **SHEET** **OF**
DESCRIPTION: 2x4 Interior Bearing **DESIGNED** NES **CHECKED** KAB

STUD WALL DESIGN - ASCE 7 WIND PROVISIONS (BASIC LOAD COMBINATIONS)

Roof Dead Load =	22.00 psf
Floor Dead Load =	25.00 psf
Wall/Misc. Dead Load =	19.00 psf

Roof Live Load =	20.00 psf
Floor Live Load =	40.00 psf
Roof Snow Load =	0.00 psf

Wall Height =	9.71 ft	
Roof Tributary Width =	16.00 ft	
Floor Tributary Width =	14.00 ft	
Wall/Misc. DL Width =	15.00 ft	
Wind Zone (Interior or End) =	End	N/A
Stud Size =	2X4	
Stud Spacing =	12 in o.c.	
Lumber Grade =	DFL#2	
Wind Speed =	N/A (Inside)	
Exposure =	C	N/A
Mean Roof Height, h =	16.00 ft	
Topo. Factor, K_{zt} =	1.00	
Wind Dir. Factor, K_d =	0.85	N/A
GCpi +/- =	0.18	N/A
		N/A

INTERACTION RESULTS FOR POSSIBLE CONTROLLING LOAD COMBINATIONS		
DL + FLL =	**0.728**	**OK**
DL + RLL =	**0.605**	**OK**
DL + SL =	**0.464**	**OK**
DL + 0.6*WIND =	**0.368**	**OK**
DL + 0.75*(FLL + RLL) =	**0.763**	**OK**
DL + 0.75*(FLL + SL) =	**0.651**	**OK**
DL + 0.75*(0.6*WIND + FLL + RLL) =	**0.819**	**OK**
DL + 0.75*(0.6*WIND + FLL + SL) =	**0.596**	**OK**
DEFLECTION L/? =	**999.5**	**OK****

** Deflection Checked for L/120 Ratio per Table 1604.3

Exterior Finish =	Brittle	(L/240 Deflection Ratio)

F_b =	900 psi
F_c =	1,350 psi
E =	1,600,000 psi

Factors	
C_F (Bending) =	1.50
C_F (Compression) =	1.15
C_r =	1.15
C_D (DL) =	1.00
C_D (FLL) =	1.00
C_D (RLL) =	1.25
C_D (SL) =	1.00
C_D (Wind) =	1.60

Wall Uniform Loads	
Dead Load, DL =	987 lb/ft
Roof Live Load, RLL =	320 lb/ft
Floor Live Load, FLL =	560 lb/ft
Snow Load, SL =	0 lb/ft

Ultimate Design Pressure =	8.33 psf	
Ultimate Max. Deflection =	0.19 in	
Max. Deflection x 0.42 =	N/A	
Max. Deflection x 0.60 =	0.12 in	
L/d =	33.29	< 50 (OK)

Check Bearing Stress on Wall Plate		
Wall Plate Lumber Grade =	DFL#2	
Max. Stud Reaction on Wall Plate =	1,867 lbs	
Bearing Stress on Wall Plate =	355.62 psi	(OK)
Allow. Bearing Stress on Wall Plate =	625 psi	

Ultimate Wall Pressures (psf)				
Zone	Interior Zone (Zone 4)		(Zone 5)	
Positive/Negative	Positive	Negative	Positive	Negative
Pressure (psf)	8.33 psf	8.33 psf	8.33 psf	8.33 psf
qh (psf)	N/A	N/A	N/A	N/A
GCp	N/A	N/A	N/A	N/A
Gcpi	N/A	N/A	N/A	N/A

Studs Part of Fire Assembly? No

Stud Section Properties		
Area =	5.25	in^2
Section Modulus =	3.06	in^3
Moment of Inertia =	5.36	in^4
Depth =	3.50 in	
Width =	1.50 in	

Axial Stresses				
	DL	FLL	RLL	SL
f_a =	188.00 psi	106.67 psi	60.95 psi	0.00 psi

Maximum Bending Moment & Stress	
M =	98 ft-lb
f_b =	384.68 psi

Allowable Bending Stress		
F'_b =	2,484 psi	

Allowable Axial Stresses					
c =	0.80	Overwrite c =			
$(L/d)^2$ =	1108.32				
COV_E =	0.25	Overwrite COV_E =			
E_{min}' =	584,494 psi				
F_{cE} =	433.50				
	DL	FLL	RLL	SL	WIND
F_c^* =	1,552.50	1,552.50	1,940.63	1,552.50	2,484.00
F_{cE}/F_c^* =	0.279	0.279	0.223	0.279	0.175
C_P =	0.261	0.261	0.212	0.261	0.168
F'c =	404.92	404.92	411.37	404.92	416.70 psi

Note: All wall studs in this calculator are braced in the weak direction.

L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-001-192
DATE 11/6/19

PROJECT: Desert Rose Customs
SUBJECT: TOP PLATE DESIGN
SHEET **SECTION**
DESIGNED NES **CHECKED** KAB

Description and Location: 2x6 worst case

Top Plate Size:	2X6				Dead	Live		Tributary Area			Roof Truss Spacing:	24	in. o.c.
Species:	DFL#2			Roof	22	20	psf	Roof	20	ft	Floor Truss Spacing:	24	in. o.c.
	No. of Plates:	2		Floor	25.001	40	psf	Floor	0	ft			
STUDS	No.:	1		Wall/Misc.	19.001		psf	Wall	0	ft			
	Spacing:	16	in o.c.										
	Align Studs w/ trusses:	No											

F'_B	1681.875	C_D	1.25	C_L	1.00	P_{ALLOW} (due to Bending)	2601	lbs.
F'_V	225	C_M	1.00	C_F	1.30	P_{ALLOW} (due to Shear)	3232	lbs.
E'	1600000	C_t	1.00	C_{fu}	1.15	P_{ALLOW} (due to Crushing)	5156	lbs.
F_{CPERP}	625	Ci	1.00	C_r	1.00			

Allowable Point Load P_{ALLOW} = 2601 lbs.
P_{ACTUAL} = 1680 lbs

TOP PLATE IS SUFFICIENT

Description and Location: 2x6 Floor

Top Plate Size:	2X6				Dead	Live		Tributary Area			Roof Truss Spacing:	24	in. o.c.
Species:	DFL#2			Roof	22	20	psf	Roof	0	ft	Floor Truss Spacing:	24	in. o.c.
	No. of Plates:	2		Floor	25.001	40	psf	Floor	19	ft			
STUDS	No.:	1		Wall/Misc.	19.001		psf	Wall	0	ft			
	Spacing:	16	in o.c.										
	Align Studs w/ trusses:	No											

F'_B	1345.5	C_D	1.00	C_L	1.00	P_{ALLOW} (due to Bending)	2081	lbs.
F'_V	180	C_M	1.00	C_F	1.30	P_{ALLOW} (due to Shear)	2586	lbs.
E'	1600000	C_t	1.00	C_{fu}	1.15	P_{ALLOW} (due to Crushing)	5156	lbs.
F_{CPERP}	625	Ci	1.00	C_r	1.00			

Allowable Point Load P_{ALLOW} = 2081 lbs.
P_{ACTUAL} = 2470.029524 lbs

TOP PLATE IS INSUFFICIENT

Description and Location: 2x4 Interior Bearing

Top Plate Size:	2X4				Dead	Live		Tributary Area			Roof Truss Spacing:	24	in. o.c.
Species:	DFL#2			Roof	22	20	psf	Roof	23	ft	Floor Truss Spacing:	24	in. o.c.
	No. of Plates:	3		Floor	25.001	40	psf	Floor	14	ft			
STUDS	No.:	1		Wall/Misc.	19.001		psf	Wall	0	ft			
	Spacing:	12	in o.c.										
	Align Studs w/ trusses:	No											

F'_B	1856.25	C_D	1.25	C_L	1.00	P_{ALLOW} (due to Bending)	3654	lbs.
F'_V	225	C_M	1.00	C_F	1.50	P_{ALLOW} (due to Shear)	3436	lbs.
E'	1600000	C_t	1.00	C_{fu}	1.10	P_{ALLOW} (due to Crushing)	3281	lbs.
F_{CPERP}	625	Ci	1.00	C_r	1.00			

Allowable Point Load P_{ALLOW} = 3281 lbs.
P_{ACTUAL} = 3242.021754 lbs

TOP PLATE IS SUFFICIENT

Description and Location: Interior Bearing 2x6 alternate

Top Plate Size:	2X6				Dead	Live		Tributary Area			Roof Truss Spacing:	24	in. o.c.
Species:	DFL#2			Roof	22	20	psf	Roof	23	ft	Floor Truss Spacing:	24	in. o.c.
	No. of Plates:	2		Floor	25.001	40	psf	Floor	14	ft			
STUDS	No.:	1		Wall/Misc.	19.001		psf	Wall	0	ft			
	Spacing:	12	in o.c.										
	Align Studs w/ trusses:	No											

F'_B	1681.875	C_D	1.25	C_L	1.00	P_{ALLOW} (due to Bending)	3468	lbs.
F'_V	225	C_M	1.00	C_F	1.30	P_{ALLOW} (due to Shear)	3600	lbs.
E'	1600000	C_t	1.00	C_{fu}	1.15	P_{ALLOW} (due to Crushing)	5156	lbs.
F_{CPERP}	625	Ci	1.00	C_r	1.00			

Allowable Point Load P_{ALLOW} = 3468 lbs.
P_{ACTUAL} = 3242.021754 lbs

TOP PLATE IS SUFFICIENT

Description and Location:

Top Plate Size:	2X4				Dead	Live		Tributary Area			Roof Truss Spacing:	24	in. o.c.
Species:				Roof	22	20	psf	Roof	0	ft	Floor Truss Spacing:	24	in. o.c.
	No. of Plates:	2		Floor	25.001	40	psf	Floor	0	ft			
STUDS	No.:	1		Wall/Misc.	19.001		psf	Wall	0	ft			
	Spacing:	16	in o.c.										
	Align Studs w/ trusses:	No											

F'_B	#N/A	C_D	1.00	C_L	1.00	P_{ALLOW} (due to Bending)	#N/A	lbs.
F'_V	#N/A	C_M	1.00	C_F	1.50	P_{ALLOW} (due to Shear)	#N/A	lbs.
E'	#N/A	C_t	1.00	C_{fu}	1.10	P_{ALLOW} (due to Crushing)	#N/A	lbs.
F_{CPERP}	#N/A	Ci	1.00	C_r	1.00			

Allowable Point Load P_{ALLOW} = #N/A lbs.
P_{ACTUAL} = 0 lbs

#N/A

Description and Location:

Top Plate Size:	2X4				Dead	Live		Tributary Area			Roof Truss Spacing:	24	in. o.c.
Species:				Roof	22	20	psf	Roof	0	ft	Floor Truss Spacing:	24	in. o.c.
	No. of Plates:	2		Floor	25.001	40	psf	Floor	0	ft			
STUDS	No.:	1		Wall/Misc.	19.001		psf	Wall	0	ft			
	Spacing:	16	in o.c.										
	Align Studs w/ trusses:	No											

F'_B	#N/A	C_D	1.00	C_L	1.00	P_{ALLOW} (due to Bending)	#N/A	lbs.
F'_V	#N/A	C_M	1.00	C_F	1.50	P_{ALLOW} (due to Shear)	#N/A	lbs.
E'	#N/A	C_t	1.00	C_{fu}	1.10	P_{ALLOW} (due to Crushing)	#N/A	lbs.
F_{CPERP}	#N/A	Ci	1.00	C_r	1.00			

Allowable Point Load P_{ALLOW} = #N/A lbs.
P_{ACTUAL} = 0 lbs

#N/A

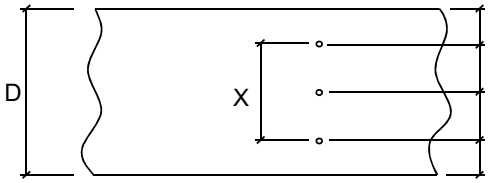
L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-001-192
DATE: November 6, 2019

PROJECT Desert Rose Customs **SHEET** **OF**

SUBJECT Wood Ledger **DESIGNED** NES **CHECKED** KAB

Minimum Nail Spacing - Per NDS



(d - Nail Diameter)

For 16d Sinkers: d = 0.148"

2x6 D = 5.5 inches
X = 5.5" - 2(2.5 x 0.148") = 5.5" - 0.74" = 4.76"
4.76 / 1.50 = 3.2 Spaces 2 Spaces @ 1.5" = 3.00" Use (3) Nails Maximum

2x8 D = 7.25 inches
X = 7.25" - 0.74" = 6.51" 4 Spaces @ 1.5" = 6.00" Use (5) Nails Maximum

2x10 D = 9.25 inches
X = 9.25" - 0.74" = 8.51" 5 Spaces @ 1.5" = 7.50" Use (6) Nails Maximum

2x12 D = 11.25 inches
X = 11.25" - 0.74" = 10.51" 7 Spaces @ 1.5" = 10.50" Use (8) Nails Maximum

Per Row
2 or 3 rows possible
where ledger nails to
end of 4x floor truss

DFL No. 2 Ledger to DFL Studs
Allowable Load for 1 1/2 Side Plate = 118 #/Nail

DFL No. 2 Ledger to SPF Studs or SPF Trusses
Allowable Load for 1 1/2 Side Plate = 100 #/Nail)

DFL VALUES
Capacity of 16d Sinkers @ 118 #/nail

	100% (Floor)	125% (Roof)
3 x 118 =	354 #	443 #
4 x 118 =	472 #	590 #
5 x 118 =	590 #	738 #
6 x 118 =	708 #	885 #
7 x 118 =	826 #	1033 #
8 x 118 =	944 #	1180 #

SPF VALUES
Capacity of 16d Sinkers @ 100 #/nail

	100% (Floor)	125% (Roof)
3 x 100 =	300 #	375 #
4 x 100 =	400 #	500 #
5 x 100 =	500 #	625 #
6 x 100 =	600 #	750 #
7 x 100 =	700 #	875 #
8 x 100 =	800 #	1000 #

L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-001-192
DATE: November 6, 2019

PROJECT Desert Rose Customs **SHEET** **OF**

SUBJECT Wood Ledger **DESIGNED** NES **CHECKED** KAB

Ledgers Supporting Roof Loads — Total Allowable Roof Load

Supported Span	Member	DFL Frmg = 70psf Nailing	SPF Frmg = 50 psf Min Hanger UNO
8ft or Less	2x6	(3) 16d at 16" o.c.	LUS24/JUS24 at16" oc
	2x8	(5) 16d at 24" o.c.	LUS24/JUS24 at 24" oc
16ft or Less	2x8	(5) 16d at 16" o.c.	LUS26/JUS26 at16" oc
	2x12	(8) 16d at 24" o.c.	LUS28/JUS28 at 24" oc
24ft or Less	2x12	(8) 16d at 16" o.c.	LUS28/JUS28 at16" oc
	2x12	(2) columns of (6) 16d at 24" o.c.	HUS28/HUS28 at 24" oc

Ledgers Supporting Floor Loads — Total Allowable Floor Load

Supported Span	Member	DFL Frmg = 89psf Nailing	SPF Frmg = 70 psf Min Hanger UNO
10ft or Less	2x8	(5) 16d at 16" o.c.	LUS26/JUS26 at16" oc
	2x10	(2) columns of (6) 16d at 24" o.c.	LUS26/JUS26 at 24" oc
20ft or Less	2x10	(2) columns of (6) 16d at 16" o.c.	LUS28/JUS28 at16" oc
	2x12	(2) columns of (8) 16d at 24" o.c.	HUS210/HUS210 at 24"oc

NOTES:
1. Minimum hangers are listed by manufacturer as Simpson Strong-tie/USP
2. Listed hangers are minimum hangers where not otherwise noted on the structural drawings
3. Two columns of nails require min. 3", (2)2x or 4x2 trusses for supporting members receiving nailing

L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-001-192
DATE 11/6/2019

PROJECT: Desert Rose Customs **SHEET** **OF**
SUBJECT: Allowable Connector Loads **DESIGNED** NES **CHECKED** KAB

16d SINKERS **Use 16d sinker nails (0.148" Diameter × 3 ¼")*
**Assume all nails penetrate through a 1½" plate, 23/32" floor sheathing and 1½" of solid blocking.*

$Z_{16d\ sinker}$ = 103 lbs/nail C_D = 1.6 C_t = 1.0
$Z'_{16d\ sinker}$ = $(Z_{16d\ sinker})(C_D)(C_M)(C_t)$ SG = 0.50 C_M = 1.0
= (103lbs/nail)(1.6)(1.0)(1.0)
= 165 lbs/nail

16d sinkers at	8 in o.c.	⇒	165 *lbs/nail* x	1.5	nails/ft =	**247**	***lbs/ft***
16d sinkers at	5 in o.c.	⇒	165 *lbs/nail* x	2.4	nails/ft =	**396**	***lbs/ft***
16d sinkers at	4 in o.c.	⇒	165 *lbs/nail* x	3	nails/ft =	**494**	***lbs/ft***
16d sinkers at	3 in o.c.	⇒	165 *lbs/nail* x	4	nails/ft =	**659**	***lbs/ft***
16d sinkers at	2 in o.c.	⇒	165 *lbs/nail* x	6	nails/ft =	**989**	***lbs/ft***

16d SINKERS **Use 16d sinker nails (0.148" Diameter × 3 ¼")*
**Assume all nails penetrate through a 1½" plate into 1½" (min) plate.*

$Z_{16d\ sinker}$ = 118 lbs/nail C_D = 1.6 C_t = 1.0
$Z'_{16d\ sinker}$ = $(Z_{16d\ sinker})(C_D)(C_M)(C_t)$ SG = 0.50 C_M = 1.0
= (118lbs/nail)(1.6)(1.0)(1.0)
= 189 lbs/nail

16d sinkers at	8 in o.c.	⇒	165 *lbs/nail* x	1.5	nails/ft =	**283**	***lbs/ft***
16d sinkers at	5 in o.c.	⇒	165 *lbs/nail* x	2.4	nails/ft =	**453**	***lbs/ft***
16d sinkers at	4 in o.c.	⇒	165 *lbs/nail* x	3	nails/ft =	**566**	***lbs/ft***
16d sinkers at	3 in o.c.	⇒	165 *lbs/nail* x	4	nails/ft =	**755**	***lbs/ft***
16d sinkers at	2 in o.c.	⇒	165 *lbs/nail* x	6	nails/ft =	**1133**	***lbs/ft***

WOOD SCREWS
**Assume all screws penetrate through a 1½" plate, 23/32" floor sheathing and 1½" of solid blocking.*

$Z_{1/4"\ x\ 6\ WS}$ = 135 lbs/nail C_D = 1.6 C_t = 1.0
$Z'_{1/4"\ x\ 6\ WS}$ = $(Z_{1/4"x6\ WS})(C_D)(C_M)(C_t)$ SG = 0.50 C_M = 1.0
= (135lbs/screw)(1.6)(1.0)(1.0)
= 216 lbs/screw

1/4" x 6 WS	6 in o.c.	⇒	216 *lbs/screw* x	2	screws/ft =	**432**	***lbs/ft***
1/4" x 6 WS	4 in o.c.	⇒	216 *lbs/screw* x	3	screws/ft =	**648**	***lbs/ft***
1/4" x 6 WS	3 in o.c.	⇒	216 *lbs/screw* x	4	screws/ft =	**864**	***lbs/ft***
1/4" x 6 WS	2 in o.c.	⇒	216 *lbs/screw* x	6	screws/ft =	**1296**	***lbs/ft***

A35 **Assume full penetration of nails as required by manufacturer for full load*

SG = 0.50

A35 = 695 lbs

A35	32 in o.c.	⇒	695 *lbs/clip* x	0.4	clips/ft =	**261**	***lbs/ft***
A35	18 in o.c.	⇒	695 *lbs/clip* x	0.7	clips/ft =	**463**	***lbs/ft***
A35	12 in o.c.	⇒	695 *lbs/clip* x	1	clips/ft =	**695**	***lbs/ft***
A35	10 in o.c.	⇒	695 *lbs/clip* x	1.2	clips/ft =	**834**	***lbs/ft***
A35	8 in o.c.	⇒	695 *lbs/clip* x	1.5	clips/ft =	**1043**	***lbs/ft***

LTP4 **Assume full penetration of nails as required by manufacturer for full load*

SG = 0.50

LTP4 = 670 lbs

LTP4	32 in o.c.	⇒	670 *lbs/clip* x	0.4	clips/ft =	**251**	***lbs/ft***
LTP4	18 in o.c.	⇒	670 *lbs/clip* x	0.7	clips/ft =	**447**	***lbs/ft***
LTP4	12 in o.c.	⇒	670 *lbs/clip* x	1	clips/ft =	**670**	***lbs/ft***
LTP4	10 in o.c.	⇒	670 *lbs/clip* x	1.2	clips/ft =	**804**	***lbs/ft***
LTP4	8 in o.c.	⇒	670 *lbs/clip* x	1.5	clips/ft =	**1005**	***lbs/ft***

L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-001-192
DATE 11/06/19

PROJECT Desert Rose Customs **SHEET** **SECTION**
SUBJECT SEISMIC LOADS **DESIGNED** NES **CHECKED** KAB

SEISMIC LOADS (Main Lateral Force Resisting System)

Risk Category =	II
Seismic Design Category =	C

See below

Site Class:	C
R =	6.5
S_s =	0.487
S_1 =	0.164
F_a =	1.3
F_v =	1.5
S_{MS} =	0.63
S_{M1} =	0.25
S_{DS} =	0.422
S_{D1} =	0.164

SDC =	C
SDC =	C

N =	2	(number of stories)
C_T =	0.02	
h_n (ft) =	22.00	(to highest level)
T_a =	0.20	
x =	0.75	
T_L =	6	

Redundancy, ρ =	1.30

I_E =	1.00
C_{SMAX} =	0.124
C_S =	0.065
C_{SMIN} =	0.010
C_{SMIN} =	0

$C_{SCONTROL}$=	**0.065**
ρ*0.7*$C_{SCONTROL}$=	**0.059**

$E = \rho Q_E + 0.2 S_{DS} D$ =	0.065	x W +	0.084	x D
$E = \rho Q_E - 0.2 S_{DS} D$ =	0.065	x W -	0.084	x D
$0.7E = 0.7(\rho Q_E + 0.2 S_{DS} D)$ =	**0.059**	x W +	**0.059**	x D
$0.7E = 0.7(\rho Q_E - 0.2 S_{DS} D)$ =	**0.059**	x W -	**0.059**	x D

L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-001-192
DATE: 11/06/19

PROJECT Desert Rose Customs **SHEET** **OF**
SUBJECT WIND LOADS (Main Lateral Force Resisting System) **DESIGNED** NES **CHECKED** KAB

Directional Procedure: Rigid Buildings of All Heights **Parallel to Ridge**
$p = qGC_p - q_i(GC_{pi})$ psf ☐ FLEXIBLE STRUCTURE (natural frequency < 1 Hz)

V(mph) =	100 mph	Mean Roof Height, h (ft) =	20	ω =	1.0
Exposure =	C	Width of Building, B (ft) =	81		
Grnd Elev Factor, Ke =	0.95	Depth of Building, L (ft) =	93		
G =	0.85	Topo. Factor, K_{zt} =	1		
Roof Angle =	0.00	Wind Dir. Factor, K_d =	0.85		
Roof Slope =	/12	h/L =	0.22		

HEIGHT (ft)	K_Z	q_z (psf)	q_h (psf)	PRESSURE (psf)				
				Walls (Windward)	Walls (Leeward)	Roof (Windward, Positive, Normal)	Roof (Windward, Negative, Normal)	Roof (Leeward, Normal)
C_p =				0.80	-0.48	0.00	0.00	-0.30
0-15	0.85	17.58		**12.0**				
20	0.9	18.61	18.61	**12.7**	**-7.6**	**0**	**0**	**0**
25	0.94	19.44		**13.3**				
30	0.98	20.26		**13.8**				
40	1.04	21.50		**14.7**				
50	1.09	22.54		**15.4**				
60	1.13	23.36		**15.9**				
70	1.17	24.19		**16.5**				
80	1.21	25.02		**17.1**				
90	1.24	25.64		**17.5**				
100	1.26	26.05		**17.8**				
120	1.31	27.09		**18.5**				
140	1.36	28.12		**19.2**				
160	1.39	28.74		**19.6**				
180	1.43	29.57		**20.2**				
200	1.46	30.19		**20.6**				
250	1.53	31.63		**21.6**				

HEIGHT (ft)	Pressure (psf)					
	Side Walls	Roof (Normal w/ Slope Less Than 10 Degrees or Parallel to Ridge)				Internal +/-
		0 to h/2	h/2 to h	h to 2h	> 2h	Enclosed
C_p =	-0.70	-0.90	-0.90	-0.50	-0.30	0.18
0-15						
20	**-11.1**	**-14.3**	**-14.3**	**-8.0**	**-4.8**	**3.4**
25						
30						
40						
50						
60						
70						
80						
90						
100						
120						
140						
160						
180						
200						
250						

L. R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-001-192
DATE: 11/06/19

PROJECT Desert Rose Customs **SHEET** **OF**
SUBJECT WIND LOADS (Main Lateral Force Resisting System) **DESIGNED** NES **CHECKED** KAB

Directional Procedure: Rigid Buildings of All Heights **Perpendicular to Ridge**
$p = qGC_p - q_i(GC_{pi})$ psf ☐ FLEXIBLE STRUCTURE (natural frequency < 1 Hz)

V(mph) =	100 mph	Mean Roof Height, h (ft) =	20	ω =	1.0
Exposure =	B	Width of Building, B (ft) =	93		
Grnd Elev Factor, Ke =	0.95	Depth of Building, L (ft) =	81		
G =	0.85	Topo. Factor, K_{zt} =	1		
Roof Angle =	0.00	Wind Dir. Factor, K_d =	0.85		
Roof Slope =	/12	h/L =	0.25		

HEIGHT (ft)	K_Z	q_z (psf)	q_h (psf)	PRESSURE (psf)				
				Walls (Windward)	Walls (Leeward)	Roof (Windward, Positive, Normal)	Roof (Windward, Negative, Normal)	Roof (Leeward, Normal)
C_p =				0.80	-0.50	0.00	0.00	-0.30
0-15	0.57	11.79		**8.1**				
20	0.62	12.82	12.82	**8.8**	**-5.5**	**0**	**0**	**0**
25	0.66	13.65		**9.3**				
30	0.7	14.48		**9.9**				
40	0.76	15.72		**10.7**				
50	0.81	16.75		**11.4**				
60	0.85	17.58		**12.0**				
70	0.89	18.40		**12.6**				
80	0.93	19.23		**13.1**				
90	0.96	19.85		**13.5**				
100	0.99	20.47		**14.0**				
120	1.04	21.50		**14.7**				
140	1.09	22.54		**15.4**				
160	1.13	23.36		**15.9**				
180	1.17	24.19		**16.5**				
200	1.2	24.81		**16.9**				
250	1.28	26.47		**18.0**				

HEIGHT (ft)	Pressure (psf)					
	Side Walls	Roof (Normal w/ Slope Less Than 10 Degrees or Parallel to Ridge)				Internal +/-
		0 to h/2	h/2 to h	h to 2h	> 2h	Enclosed
C_p =	-0.70	-0.90	-0.90	-0.50	-0.30	0.18
0-15						
20	**-7.7**	**-9.9**	**-9.9**	**-5.5**	**-3.3**	**2.4**
25						
30						
40						
50						
60						
70						
80						
90						
100						
120						
140						
160						
180						
200						
250						

L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-001-192
DATE 11/06/19

PROJECT Desert Rose Customs
SUBJECT LATERAL LINE LOADS
SHEET
DESIGNED NES
SECTION
CHECKED KAB

Seismic Line Loads

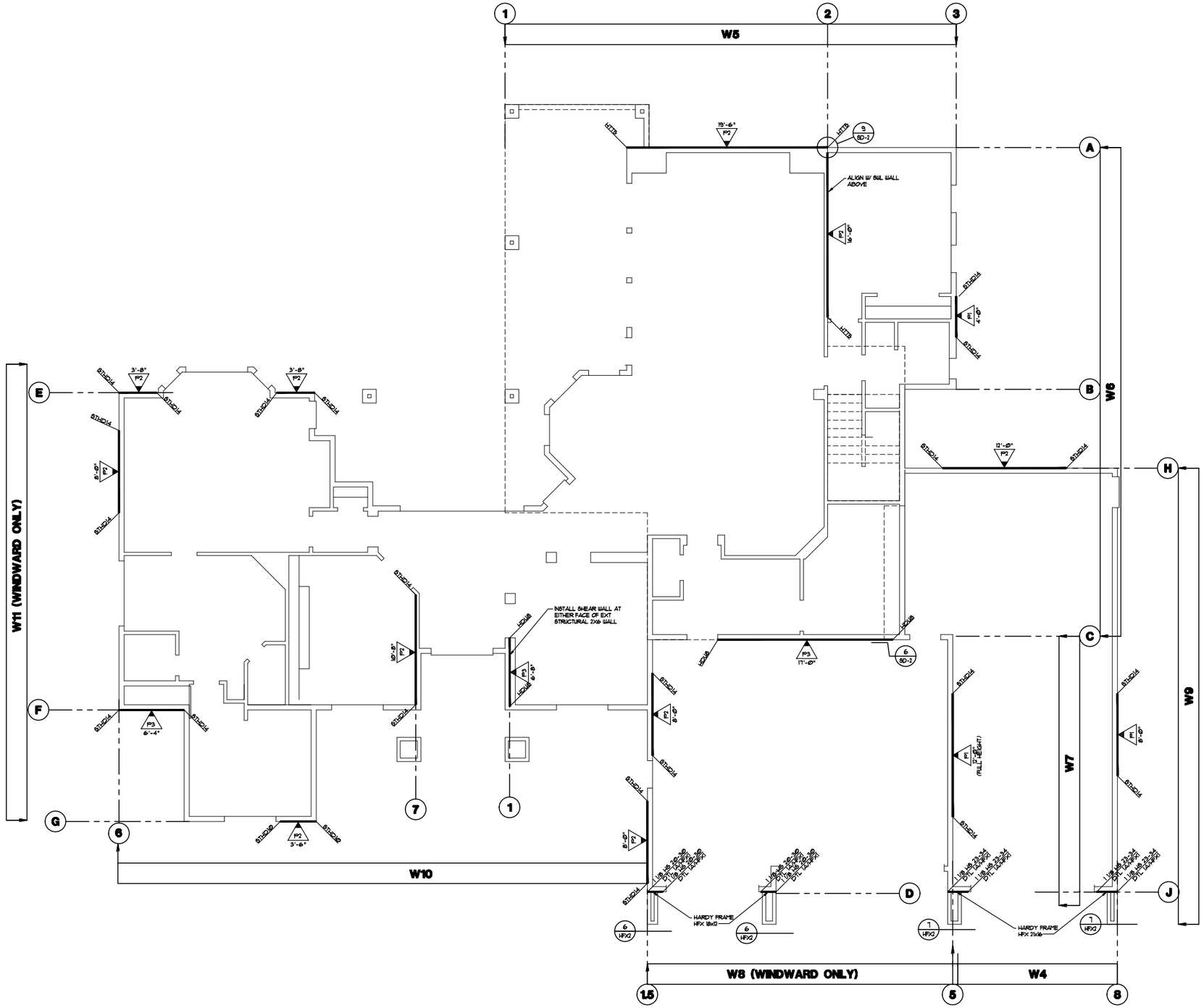
Label	Level	Roof Trib (ft)	Floor Trib (ft)	Wall Trib (ft)	Other (lb/ft)	Total Weight (lb/ft)	Total Force (lb/ft)	Redist Factor	Revised Force (lb/ft)
ω1	ROOF	40		10	200	1270	75	1.560	117
ω2	ROOF	42		10	210	1324	78	1.560	122
ω3						0	0	1.000	0
ω4	ROOF	47		16	235	1573	93	1.560	145
ω5	2nd FLOOR	50		15.5	600	1995	118	0.851	100
ω6	2nd FLOOR	40		15.5	480	1655	98	0.851	83
ω7	2nd FLOOR	34		12	408	1384	82	0.851	70
ω8	2nd FLOOR	35		12	420	1418	84	0.851	71
ω9	ROOF	20		16	100	844	50	1.560	78
ω10	2nd FLOOR	36		16	360	1456	86	0.851	73
ω11	2nd FLOOR	52		16	520	1968	116	0.851	99
ω12						0	0	1.000	0
ω13						0	0	1.000	0
ω14						0	0	1.000	0
ω15						0	0	1.000	0
ω16						0	0	1.000	0
ω17						0	0	1.000	0
ω18						0	0	1.000	0
ω19						0	0	1.000	0
ω20						0	0	1.000	0
ω21						0	0	1.000	0

Roof DL	22.00	psf
Seismic Snow	0.00	psf
Floor DL	25.00	psf
Wall DL	19.00	psf
Seis Coeff	0.059	* V

Wind Line Loads **PROCEDURE:** Envelope: Not Used Directional: X

Label	Roof Pitch /12	Mean Height for Windward Pressure	Direction of Wind Parallel to Ridge?	Windward Wall (ft)	Leeward Wall (ft)	Windward Roof (ft)	Leeward Roof (ft)	Ultimate Wind Force (plf)		Ultimate 16 psf wall & 8 psf roof min. (plf)	Allowable Wind Force (plf)		Allowable 9.6 psf wall & 4.8 psf roof min. (plf)
ω1	0.00/12	28.25	Yes	8.125	8.125			173.88		130.00	104.33		78.00
ω2	0.00/12	28.25	No	5	5	3.125	3.125	77.00		105.00	46.20		63.00
ω3	0.00/12							0.00		0.00	0.00		0.00
ω4	0.00/12	18	Yes	11	11			223.30		176.00	133.98		105.60
ω5	0.00/12	12	Yes	12.25	12.25			240.10		196.00	144.06		117.60
ω6	0.00/12	12	No	12.25	12.25			166.60		196.00	99.96		117.60
ω7	0.00/12	16	No	6	6	7.3	7.3	85.80		154.40	51.48		92.64
ω8	0.00/12	16	Yes	10				127.00		160.00	76.20		96.00
ω9	0.00/12	17	No	8	4	2.6666667	2.67	92.40		149.33	55.44		89.60
ω10	0.00/12	15	No	5	5	6	6	68.00		128.00	40.80		76.80
ω11	0.00/12	15	Yes	11				132.00		176.00	79.20		105.60
ω12	0.00/12							0.00		0.00	0.00		0.00
ω13	0.00/12							0.00		0.00	0.00		0.00
ω14	0.00/12							0.00		0.00	0.00		0.00
ω15	0.00/12							0.00		0.00	0.00		0.00
ω16	0.00/12							0.00		0.00	0.00		0.00
ω17	0.00/12							0.00		0.00	0.00		0.00
ω18	0.00/12							0.00		0.00	0.00		0.00
ω19	0.00/12							0.00		0.00	0.00		0.00
ω20	0.00/12							0.00		0.00	0.00		0.00
ω21	0.00/12							0.00		0.00	0.00		0.00

Note: "16 psf wall & 8 psf roof min. (plf)" check is sometimes set to zero due to the line load being windward only or leeward only (16 psf wall & 8 psf roof min. (plf) check is for windward + leeward)



1ST STORY SHEAR WALL PLAN

1/4"=1'-0"

L. R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-001-192 DATE 11-11-2019

SHEET ____ OF ____

DESIGNED NES

PROJECT DESERT ROSE CUSTOM

SUBJECT KEY PLAN

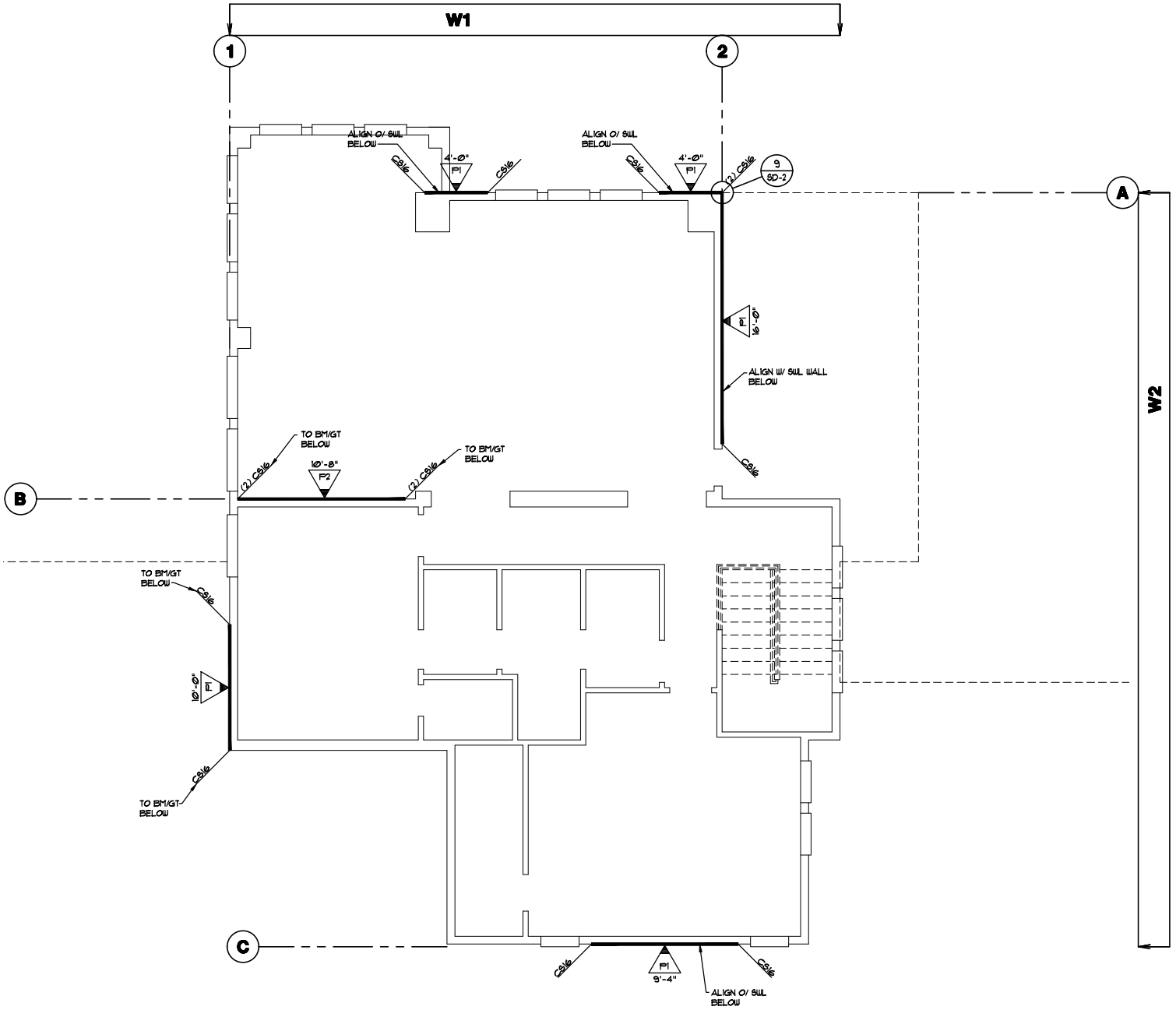
L. R. NELSON CONSULTING ENGINEERS INC.

JOB NO. 1939-001-192

DATE 11-11-2019

PROJECT APN# 179-04-405-011 SHEET ____ OF ____

SUBJECT SHEAR WALL KEY PLAN DESIGNED NES CHECKED ____



L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-001-192
DATE 11/06/19

PROJECT Desert Rose Customs
SUBJECT SHEARWALLS
SHEET **SECTION**
DESIGNED NES **CHECKED** KAB

Roof DL (psf)= 22
Floor DL (psf)= 25
Wall DL (psf)= 19

Allowable aspect ratio/1= 3.5 ω = 1.0

LINE: 1 2ND STORY Total Length (ft)= 10 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =							Wind (lb)	Seis (lb)
ω1	16.00	117.1	78.0	104.3						-8.58								
		0.0	0.0	0.0						-8.58							above=	
		0.0	0.0	0.0						-8.58							Total=	1669	1874

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
10.00				16,692	18,736	5,700			P1	CS16	Full	167	365	187	260	1,099	1,304	O.K.
											Full							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: 1.5 1ST STORY Total Length (ft)= 16 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 12 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 12

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =							Wind (lb)	Seis (lb)
ω5	16.00	100.3	117.6	144.1						-8.58								
ω7	15.00	69.6	92.6	51.5						-8.58							above=	
		0.0	0.0	0.0						-8.58							Total=	3271	2649

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
8.00				19,627	15,894	4,378			P2	STHD14 (C)	Full	204	533	166	380	1,906	1,440	O.K.
8.00				19,627	15,894	4,378			P2	STHD14 (C)	Corner	204	533	166	380	1,906	1,440	O.K.
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: 2 2ND STORY Total Length (ft)= 16 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =							Wind (lb)	Seis (lb)
ω1	24.00	117.1	78.0	104.3						-8.58								
		0.0	0.0	0.0						-8.58							above=	
		0.0	0.0	0.0						-8.58							Total=	2504	2810

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
16.00				25,038	28,104	14,593			P1	CS16	Full	156	365	176	260	653	844	O.K.
											Corner							
											Corner							
									SHARED HD W/ LINE A	(2) CS16	Full		#N/A		#N/A	821	1,551	O.K.
											Corner							
											Corner							

NOTES:

LINE: 2 1ST STORY Total Length (ft)= 16 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =							Wind (lb)	Seis (lb)
ω5	15.00	100.3	117.6	144.1						-8.58								
		0.0	0.0	0.0						-8.58							above=	2504	2810
		0.0	0.0	0.0						-8.58							Total=	4665	4315

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
16.00				46,647	43,151	14,593	260	653	P2	HTT5	Full	292	533	270	380	2,263	2,438	O.K.
											Corner							
											Corner							
									SHARED HD W/ LINE A	HDU8	Full		#N/A		#N/A	2,841	3,520	O.K.
											Corner							
											Corner							

NOTES:

L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-001-192
DATE 11/06/19

PROJECT Desert Rose Customs
SUBJECT SHEARWALLS
SHEET **SECTION**
DESIGNED NES **CHECKED** KAB

Roof DL (psf)= 22
Floor DL (psf)= 25
Wall DL (psf)= 19

Allowable aspect ratio/1= 3.5 ω = 1.0

LINE: 3 1ST STORY Total Length (ft)= 4 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω5	7.00	100.3	117.6	144.1						-8.58							Wind (lb)	Seis (lb)
		0.0	0.0	0.0						-8.58						above=		
		0.0	0.0	0.0						-8.58						Total=	1008	702

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
4.00				10,084	7,022	912			P1	STHD14 (C)	Corner	252	292	176	208	2,293	1,527	O.K.
											Corner							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: 5 1ST STORY Total Length (ft)= 12 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 12 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 12

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω8	15.00	71.3	96.0	76.2						-8.58							Wind (lb)	Seis (lb)
ω4	9.00	145.0	105.6	134.0						-8.58						above=		
		0.0	0.0	0.0						-8.58						Total=	2390	2375

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
12.00				28,685	28,501	9,850			P1	STHD14 (C)	Corner	199	365	198	260	1,570	1,554	O.K.
											Corner							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: 6 1ST STORY Total Length (ft)= 8 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω10	15.00	73.2	76.8	40.8						-8.58							Wind (lb)	Seis (lb)
		0.0	0.0	0.0						-8.58						above=		
		0.0	0.0	0.0						-8.58						Total=	1152	1098

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
8.00				11,520	10,984	3,648			P2	STHD14 (C)	Corner	144	533	137	380	984	917	O.K.
											Corner							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: 7 1ST STORY Total Length (ft)= 10.67 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω10	20.00	73.2	76.8	40.8						-8.58							Wind (lb)	Seis (lb)
		0.0	0.0	0.0						-8.58						above=		
		0.0	0.0	0.0						-8.58						Total=	1536	1465

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
10.67				15,360	14,646	6,490			P2	STHD14 (C)	Corner	144	533	137	380	831	764	O.K.
											Corner							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-001-192
DATE 11/06/19

PROJECT Desert Rose Customs
SUBJECT SHEARWALLS
SHEET **SECTION**
DESIGNED NES **CHECKED** KAB

Roof DL (psf)= 22
Floor DL (psf)= 25
Wall DL (psf)= 19

Allowable aspect ratio/1= 3.5 ω = 1.0

LINE: 1 1ST STORY Total Length (ft)= 6.67 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω10	12.00	73.2	76.8	40.8						-8.58							Wind (lb)	Seis (lb)
		0.0	0.0	0.0						-8.58						above=	1669	1874
		0.0	0.0	0.0						-8.58						Total=	2591	2752

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
6.67				25,908	27,523	2,536			P3	HDU8	Corner	388	685	413	490	3,504	3,746	O.K.
											Corner							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: A 2ND STORY Total Length (ft)= 8 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω2	10.00	122.1	63.0	46.2						-8.58							Wind (lb)	Seis (lb)
		0.0	0.0	0.0						-8.58						above=		
		0.0	0.0	0.0						-8.58						Total=	630	1221

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
4.00				3,150	6,104	912			P1	CS16	Full	79	292	153	208	559	1,298	O.K.
4.00				3,150	6,104	912			P1	CS16	Full	79	292	153	208	559	1,298	O.K.
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: A 1ST STORY Total Length (ft)= 19.5 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω5	24.00	100.3	117.6	144.1						-8.58							Wind (lb)	Seis (lb)
		0.0	0.0	0.0						-8.58						above=	1376	2667
		0.0	0.0	0.0						-8.58						Total=	4834	5075

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
19.50				48,339	50,748	21,675	559	1,298	P2	HTT5	Full	248	533	260	380	1,927	2,789	O.K.
											Corner							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: B 2ND STORY Total Length (ft)= 10.67 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω2	24.00	122.1	63.0	46.2						-8.58							Wind (lb)	Seis (lb)
		0.0	0.0	0.0						-8.58						above=		
		0.0	0.0	0.0						-8.58						Total=	1512	2930

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
10.67				15,120	29,299	6,490			P2	(2) CS16	Full	142	533	275	380	809	2,138	O.K.
											Corner							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

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JOB NO. 1939-001-192
DATE 11/06/19

PROJECT Desert Rose Customs
SUBJECT SHEARWALLS
SHEET **SECTION**
DESIGNED NES **CHECKED** KAB

Roof DL (psf)= 22
Floor DL (psf)= 25
Wall DL (psf)= 19

Allowable aspect ratio/1= 3.5 ω = 1.0

LINE: C 2ND STORY Total Length (ft)= 9.33 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω2	15.00	122.1	63.0	46.2						-8.58							Wind (lb)	Seis (lb)
		0.0	0.0	0.0						-8.58						above=		
		0.0	0.0	0.0						-8.58						Total=	945	1831

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
9.33				9,450	18,312	4,962			P1	CS16	Full	101	365	196	260	481	1,431	O.K.
											Corner							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: C 1ST STORY Total Length (ft)= 17 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 12 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 12

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω6	24.00	83.2	117.6	100.0						-8.58							Wind (lb)	Seis (lb)
ω7	14.00	69.6	92.6	51.5						-8.58						above=	1709	3312
		0.0	0.0	0.0						-8.58						Total=	5828	6283

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
17.00				69,940	75,398	19,768	481	1,431	P3	HDU8	Corner	343	685	370	490	3,432	4,703	O.K.
											Corner							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: D 1ST STORY Total Length (ft)= 2 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 12 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 12

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω7	20.00	69.6	92.6	51.5						-8.58							Wind (lb)	Seis (lb)
		0.0	0.0	0.0						-8.58						above=		
		0.0	0.0	0.0						-8.58						Total=	1853	1392

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
1.00				11,117	8,353	68			HFX18X12-1 1/8 HS		Full	926	2,160	696	2,160	11,048	8,284	
1.00				11,117	8,353	68			HFX18X12-1 1/8 HS		Full	926	2,160	696	2,160	11,048	8,284	
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: E 1ST STORY Total Length (ft)= 7.34 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω11	12.00	99.0	105.6	79.2						-8.58							Wind (lb)	Seis (lb)
		0.0	0.0	0.0						-8.58						above=		
		0.0	0.0	0.0						-8.58						Total=	1267	1188

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
3.67				6,336	5,939	768			P2	STHD14 (C)	Corner	173	391	162	279	1,517	1,409	O.K.
3.67				6,336	5,939	768			P2	STHD14 (C)	Corner	173	391	162	279	1,517	1,409	O.K.
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

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DATE 11/06/19

PROJECT Desert Rose Customs
SUBJECT SHEARWALLS
SHEET **SECTION**
DESIGNED NES **CHECKED** KAB

Roof DL (psf)= 22
Floor DL (psf)= 25
Wall DL (psf)= 19

Allowable aspect ratio/1= 3.5 ω = 1.0

LINE: F 1ST STORY Total Length (ft)= 6.33 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω11	21.00	99.0	105.6	79.2						-8.58							Wind (lb)	Seis (lb)
		0.0	0.0	0.0						-8.58						above=		
		0.0	0.0	0.0						-8.58						Total=	2218	2079

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
6.33				22,176	20,785	2,284			P3	STHD14 (C)	Corner	350	685	328	490	3,142	2,923	O.K.
											Corner							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: G 1ST STORY Total Length (ft)= 3.54 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 10 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 10

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω11	6.00	99.0	105.6	79.2						-8.58							Wind (lb)	Seis (lb)
		0.0	0.0	0.0						-8.58						above=		
		0.0	0.0	0.0						-8.58						Total=	634	594

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
3.54				6,336	5,939	714			P2	STHD10 (C)	Corner	179	377	168	269	1,588	1,476	O.K.
											Corner							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: H 1ST STORY Total Length (ft)= 12 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 16 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 16

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω9	23.00	77.8	89.6	55.4						-8.58							Wind (lb)	Seis (lb)
		0.0	0.0	0.0						-8.58						above=		
		0.0	0.0	0.0						-8.58						Total=	2061	1790

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
12.00				32,973	28,638	13,133			P2	STHD14 (C)	Corner	172	533	149	380	1,653	1,292	O.K.
											Corner							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: J 1ST STORY Total Length (ft)= 2 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 16 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 16

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω9	23.00	77.8	89.6	55.4						-8.58							Wind (lb)	Seis (lb)
		0.0	0.0	0.0						-8.58						above=		
		0.0	0.0	0.0						-8.58						Total=	2061	1790

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	Wall Type	Holdown Strap	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
1.00				16,486	14,319	91			HFX-21x16		Corner	1,030	2,295	895	1,850	16,395	14,228	
1.00				16,486	14,319	91			HFX-21x16		Full	1,030	2,295	895	1,850	16,395	14,228	
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

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PROJECT Desert Rose Customs
SUBJECT SHEARWALLS
SHEET **SECTION**
DESIGNED NES **CHECKED** KAB

Roof DL (psf)= 22
Floor DL (psf)= 25
Wall DL (psf)= 19

Allowable aspect ratio/1= 3.5 ω = 1.0

LINE: 8 1ST STORY Total Length (ft)= 8 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 16 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 16

Load	Trib w (ft)	I_E*Seis	ω (min)	ω						Allow. Uplift Pressure (psf) =								
ω4	9.00	145.0	105.6	134.0						-8.58							Wind (lb)	Seis (lb)
		0.0	0.0	0.0						-8.58						above=		
		0.0	0.0	0.0						-8.58						Total=	1206	1305

Shear-Wall Length (ft)	$Roof_{DL}$ 'w' (ft)	$Floor_{DL}$ 'w' (ft)	$Other_{DL}$ 'w' (plf)	OTM - Wind (ft-lb)	OTM - Seismic (ft-lb)	0.6*RM (ft-lb)	Tension From Above: Wind (lb)	Tension From Above: Seismic (lb)	**Wall Type**	**Holdown Strap**	HD Capacity Rating	Wind Shear (plf)	Wind Wall Capacity (plf)	Seismic Shear (plf)	Seismic Wall Capacity (plf)	Tension: Wind (lb)	Tension: Seismic (lb)	HD Capacity (lb)
8.00				19,293	20,886	5,837			**P1**	**STHD14 (C)**	Corner	151	365	163	260	1,682	1,881	O.K.
											Corner							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

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PROJECT Desert Rose Customs **SHEET** **OF**
SUBJECT DIAPRHAGM FORCES **DESIGNED** NES **CHECKED** KAB

ROOF DIAPRAGM SHEAR: Min Diaphragm Width: 16 Eave Ht.

Location	Line Load	End Zone	Governing Force	Dist. Btwn. Shear lines (ft)	Diaph'm Depth (ft)	Diaph'm Shear (lb/ft)	Allowable Shear (lb/ft)	Diaphragm Construction	Case
1	ω1	None	SEIS	38.833333	39.66667	57.3	165.6	15/32 shthg, 8d nails @ 6" o.c.	CASES 2-6
2	ω2	None	SEIS	39.666667	38.83333	62.3	165.6	15/32 shthg, 8d nails @ 6" o.c.	CASES 2-7
3	ω4	None	SEIS	16	44.16667	26.3	165.6	15/32 shthg, 8d nails @ 6" o.c.	CASES 2-8
4	ω9	None	WIND	44.166667	16	123.7	232.3	15/32 shthg, 8d nails @ 6" o.c.	CASES 2-9
5		None							
6		None							

Specific Gravity of Framing 0.42

FLOOR DIAPRAGM SHEAR: Min Diaphragm Width: Eave Ht.

Location	Line Load	End Zone	Governing Force	Dist. Btwn. Shear lines (ft)	Diaph'm Depth (ft)	Diaph'm Shear (lb/ft)	Allowable Shear (lb/ft)	Diaphragm Construction	Case
7	ω5	None	WIND	43.833333	47.5	66.5	276	23/32 shthg, 10d nails @ 6" o.c.	CASES 2-6
8	ω6	None	WIND	47.5	43.83333	63.7	276	23/32 shthg, 10d nails @ 6" o.c.	CASES 2-7
9	ω7	None	WIND	26.166667	29.66667	40.9	276	23/32 shthg, 10d nails @ 6" o.c.	CASES 2-8
10	ω8	None	WIND	29.666667	26.16667	54.4	276	23/32 shthg, 10d nails @ 6" o.c.	CASES 2-9
11	ω10	None	WIND	51.5	44.33333	44.6	276	23/32 shthg, 10d nails @ 6" o.c.	CASES 2-10
12	ω11	None	WIND	44.333333	51.5	45.5	276	23/32 shthg, 10d nails @ 6" o.c.	CASES 2-11

Specific Gravity of Framing 0.42

CHORD FORCES:

Location	Tension (lbs)	Number of Plies	Grade	Size	Unity	16d Nails Req'd	Number Provided	Cd
1	556.5	2	DFL#2	2x4	0.04	2.947483	12	1.6
2	618.3	2	DFL#2	2x4	0.04	3.274904	12	1.6
4	1365.5	2	DFL#2	2x4	0.09	7.232448	12	1.6
7	728.4	2	DFL#2	2x4	0.05	3.858041	12	1.6
8	756.7	2	DFL#2	2x4	0.05	4.007726	12	1.6
10	403.6	2	DFL#2	2x4	0.03	2.137806	12	1.6

Splice Connection:	(12) 16d Sinkers

Minimum Splice per IBC 2304 is (8) 16d common Nails

L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-001-192
DATE 11/6/2019

PROJECT: Desert Rose Customs **SHEET** **OF**
SUBJECT: ALLOWABLE ANCHOR BOLT LOADS **DESIGNED** NES **CHECKED** KAB

A307 Anchor Bolts - 1/2" Diameter

**Assumes 1/2" diameter A307 anchor bolts in a 1 1/2" Dougleas-Fir larch bottom plate and 6" (min) embed, parallel to grain*
**Assumes wood controls*

Z_{II} = 650 lbs/bolt C_D = 1.6 C_t = 1.0
$Z'_{II} = Z*C_D*C_M*C_t$ SG = 0.5 C_M = 1.0
= (650lbs/nail)(1.6)(1.0)(1.0)
= 1040 lbs/nail

1/2" Anchor Bolts at 72 in o.c.	⇒	1040 *lbs/bolt* ×	0.2	nails/ft =	**173**	***lbs/ft***
1/2" Anchor Bolts at 36 in o.c.	⇒	1040 *lbs/bolt* ×	0.3	nails/ft =	**347**	***lbs/ft***
1/2" Anchor Bolts at 32 in o.c.	⇒	1040 *lbs/bolt* ×	0.4	nails/ft =	**390**	***lbs/ft***
1/2" Anchor Bolts at 28 in o.c.	⇒	1040 *lbs/bolt* ×	0.4	nails/ft =	**446**	***lbs/ft***
1/2" Anchor Bolts at 24 in o.c.	⇒	1040 *lbs/bolt* ×	0.5	nails/ft =	**520**	***lbs/ft***
1/2" Anchor Bolts at 20 in o.c.	⇒	1040 *lbs/bolt* ×	0.6	nails/ft =	**624**	***lbs/ft***
1/2" Anchor Bolts at 16 in o.c.	⇒	1040 *lbs/bolt* ×	0.8	nails/ft =	**780**	***lbs/ft***
1/2" Anchor Bolts at 12 in o.c.	⇒	1040 *lbs/bolt* ×	1	nails/ft =	**1040**	***lbs/ft***
1/2" Anchor Bolts at 9 in o.c.	⇒	1040 *lbs/bolt* ×	1.3	nails/ft =	**1387**	***lbs/ft***

A307 Anchor Bolts - 5/8" Diameter

**Assumes 5/8" diameter A307 anchor bolts in a 1 1/2" Douglas-Fir larch bottom plate and 6" (min) embed, parallel to grain*
**Assumes wood controls*

Z_{II} = 930 lbs/bolt C_D = 1.6 C_t = 1.0
$Z'_{II} = Z*C_D*C_M*C_t$ SG = 0.5 C_M = 1.0
= (930lbs/nail)(1.6)(1.0)(1.0)
= 1488 lbs/nail

5/8" Anchor Bolts at 72 in o.c.	⇒	1488 *lbs/bolt* ×	0.2	nails/ft =	**248**	***lbs/ft***
5/8" Anchor Bolts at 36 in o.c.	⇒	1488 *lbs/bolt* ×	0.3	nails/ft =	**496**	***lbs/ft***
5/8" Anchor Bolts at 32 in o.c.	⇒	1488 *lbs/bolt* ×	0.4	nails/ft =	**558**	***lbs/ft***
5/8" Anchor Bolts at 28 in o.c.	⇒	1488 *lbs/bolt* ×	0.4	nails/ft =	**638**	***lbs/ft***
5/8" Anchor Bolts at 24 in o.c.	⇒	1488 *lbs/bolt* ×	0.5	nails/ft =	**744**	***lbs/ft***
5/8" Anchor Bolts at 20 in o.c.	⇒	1488 *lbs/bolt* ×	0.6	nails/ft =	**893**	***lbs/ft***
5/8" Anchor Bolts at 16 in o.c.	⇒	1488 *lbs/bolt* ×	0.8	nails/ft =	**1116**	***lbs/ft***
5/8" Anchor Bolts at 12 in o.c.	⇒	1488 *lbs/bolt* ×	1	nails/ft =	**1488**	***lbs/ft***
5/8" Anchor Bolts at 9 in o.c.	⇒	1488 *lbs/bolt* ×	1.3	nails/ft =	**1984**	***lbs/ft***

A307 Anchor Bolts - 5/8" Diameter

**Assumes 5/8" diameter A307 anchor bolts in a 2 1/2" Douglas-Fir larch bottom plate and 6" (min) embed, parallel to grain*
**Assumes wood controls*

Z_{II} = 1180 lbs/bolt C_D = 1.6 C_t = 1.0
$Z'_{II} = Z*C_D*C_M*C_t$ SG = 0.5 C_M = 1.0
= (1180lbs/nail)(1.6)(1.0)(1.0)
= 1888 lbs/nail

5/8" Anchor Bolts at 72 in o.c.	⇒	1888 *lbs/bolt* ×	0.2	nails/ft =	**315**	***lbs/ft***
5/8" Anchor Bolts at 36 in o.c.	⇒	1888 *lbs/bolt* ×	0.3	nails/ft =	**629**	***lbs/ft***
5/8" Anchor Bolts at 32 in o.c.	⇒	1888 *lbs/bolt* ×	0.4	nails/ft =	**708**	***lbs/ft***
5/8" Anchor Bolts at 28 in o.c.	⇒	1888 *lbs/bolt* ×	0.4	nails/ft =	**809**	***lbs/ft***
5/8" Anchor Bolts at 24 in o.c.	⇒	1888 *lbs/bolt* ×	0.5	nails/ft =	**944**	***lbs/ft***
5/8" Anchor Bolts at 20 in o.c.	⇒	1888 *lbs/bolt* ×	0.6	nails/ft =	**1133**	***lbs/ft***
5/8" Anchor Bolts at 16 in o.c.	⇒	1888 *lbs/bolt* ×	0.8	nails/ft =	**1416**	***lbs/ft***
5/8" Anchor Bolts at 12 in o.c.	⇒	1888 *lbs/bolt* ×	1	nails/ft =	**1888**	***lbs/ft***
5/8" Anchor Bolts at 9 in o.c.	⇒	1888 *lbs/bolt* ×	1.3	nails/ft =	**2517**	***lbs/ft***

L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-001-192
DATE 11/6/2019

PROJECT: Desert Rose Customs **SHEET** **OF**
SUBJECT: ALLOWABLE ANCHOR BOLT LOADS **DESIGNED** NES **CHECKED** KAB

A307 Anchor Bolts - 1/2" Diameter

**Assumes 1/2" diameter A307 anchor bolts in a 1 1/2" Douglas-Fir larch bottom plate and 6" (min) embed, perpendicular to grain*
**Assumes wood controls*

$Z_\perp$ = 380 lbs/bolt C_D = 1.6 C_t = 1.0
$Z'_\perp = Z*C_D*C_M*C_t$ SG = 0.5 C_M = 1.0
= (380lbs/nail)(1.6)(1.0)(1.0)
= 608 lbs/nail

1/2" Anchor Bolts at 72 in o.c.	⇒	608 *lbs/bolt* ×	0.2	nails/ft =	**101**	***lbs/ft***
1/2" Anchor Bolts at 36 in o.c.	⇒	608 *lbs/bolt* ×	0.3	nails/ft =	**203**	***lbs/ft***
1/2" Anchor Bolts at 32 in o.c.	⇒	608 *lbs/bolt* ×	0.4	nails/ft =	**228**	***lbs/ft***
1/2" Anchor Bolts at 28 in o.c.	⇒	608 *lbs/bolt* ×	0.4	nails/ft =	**261**	***lbs/ft***
1/2" Anchor Bolts at 24 in o.c.	⇒	608 *lbs/bolt* ×	0.5	nails/ft =	**304**	***lbs/ft***
1/2" Anchor Bolts at 20 in o.c.	⇒	608 *lbs/bolt* ×	0.6	nails/ft =	**365**	***lbs/ft***
1/2" Anchor Bolts at 16 in o.c.	⇒	608 *lbs/bolt* ×	0.8	nails/ft =	**456**	***lbs/ft***
1/2" Anchor Bolts at 12 in o.c.	⇒	608 *lbs/bolt* ×	1	nails/ft =	**608**	***lbs/ft***
1/2" Anchor Bolts at 9 in o.c.	⇒	608 *lbs/bolt* ×	1.3	nails/ft =	**811**	***lbs/ft***

A307 Anchor Bolts - 5/8" Diameter

**Assumes 5/8" diameter A307 anchor bolts in a 1 1/2" Douglas-Fir larch bottom plate and 6" (min) embed, perpendicular to grain*
**Assumes wood controls*

$Z_\perp$ = 530 lbs/bolt C_D = 1.6 C_t = 1.0
$Z'_\perp = Z*C_D*C_M*C_t$ SG = 0.5 C_M = 1.0
= (530lbs/nail)(1.6)(1.0)(1.0)
= 848 lbs/nail

5/8" Anchor Bolts at 72 in o.c.	⇒	848 *lbs/bolt* ×	0.2	nails/ft =	**141**	***lbs/ft***
5/8" Anchor Bolts at 36 in o.c.	⇒	848 *lbs/bolt* ×	0.3	nails/ft =	**283**	***lbs/ft***
5/8" Anchor Bolts at 32 in o.c.	⇒	848 *lbs/bolt* ×	0.4	nails/ft =	**318**	***lbs/ft***
5/8" Anchor Bolts at 28 in o.c.	⇒	848 *lbs/bolt* ×	0.4	nails/ft =	**363**	***lbs/ft***
5/8" Anchor Bolts at 24 in o.c.	⇒	848 *lbs/bolt* ×	0.5	nails/ft =	**424**	***lbs/ft***
5/8" Anchor Bolts at 20 in o.c.	⇒	848 *lbs/bolt* ×	0.6	nails/ft =	**509**	***lbs/ft***
5/8" Anchor Bolts at 16 in o.c.	⇒	848 *lbs/bolt* ×	0.8	nails/ft =	**636**	***lbs/ft***
5/8" Anchor Bolts at 12 in o.c.	⇒	848 *lbs/bolt* ×	1	nails/ft =	**848**	***lbs/ft***
5/8" Anchor Bolts at 9 in o.c.	⇒	848 *lbs/bolt* ×	1.3	nails/ft =	**1131**	***lbs/ft***

A307 Anchor Bolts - 5/8" Diameter

**Assumes 5/8" diameter A307 anchor bolts in a 2 1/2" Douglas-Fir larch bottom plate and 6" (min) embed, perpendicular to grain*
**Assumes wood controls*

$Z_\perp$ = 610 lbs/bolt C_D = 1.6 C_t = 1.0
$Z'_\perp = Z*C_D*C_M*C_t$ SG = 0.5 C_M = 1.0
= (610lbs/nail)(1.6)(1.0)(1.0)
= 976 lbs/nail

5/8" Anchor Bolts at 72 in o.c.	⇒	976 *lbs/bolt* ×	0.2	nails/ft =	**163**	***lbs/ft***
5/8" Anchor Bolts at 36 in o.c.	⇒	976 *lbs/bolt* ×	0.3	nails/ft =	**325**	***lbs/ft***
5/8" Anchor Bolts at 32 in o.c.	⇒	976 *lbs/bolt* ×	0.4	nails/ft =	**366**	***lbs/ft***
5/8" Anchor Bolts at 28 in o.c.	⇒	976 *lbs/bolt* ×	0.4	nails/ft =	**418**	***lbs/ft***
5/8" Anchor Bolts at 24 in o.c.	⇒	976 *lbs/bolt* ×	0.5	nails/ft =	**488**	***lbs/ft***
5/8" Anchor Bolts at 20 in o.c.	⇒	976 *lbs/bolt* ×	0.6	nails/ft =	**586**	***lbs/ft***
5/8" Anchor Bolts at 16 in o.c.	⇒	976 *lbs/bolt* ×	0.8	nails/ft =	**732**	***lbs/ft***
5/8" Anchor Bolts at 12 in o.c.	⇒	976 *lbs/bolt* ×	1	nails/ft =	**976**	***lbs/ft***
5/8" Anchor Bolts at 9 in o.c.	⇒	976 *lbs/bolt* ×	1.3	nails/ft =	**1301**	***lbs/ft***

L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-001-192
DATE

PROJECT Desert Rose Customs
SUBJECT Foundation
SHEET OF
DESIGNED NES CHECKED KAB

Conventional Foundation

ALLOWABLE LOADS TO SLAB

LINE LOADS TO SLAB

ROOF:	DEAD LOAD =	22 psf	LIVE / SNOW LOAD =	20 psf
FLOOR:	DEAD LOAD =	25 psf	LIVE LOAD =	40 psf
MISC.:	DEAD LOAD =	19 psf		

PERIMETER LINE LOADS

Maximum Line Load		Minimum Line Load	
Roof Tributary =	16.92 ft	Roof Tributary =	7.17 ft
Floor Tributary =	6.75 ft	Floor Tributary =	0.00 ft
Wall/Misc. Tributary =	22.00 ft	Wall/Misc. Tributary =	10.00 ft
Total Dead + Live Load =	1567 plf	Total Dead + Live Load =	491 plf
Dead Load Only =	959 plf	Dead Load Only =	348 plf

INTERIOR LINE LOAD

Maximum Line Load	
Roof Tributary =	23.92 ft
Floor Tributary =	15.75 ft
Wall/Misc. Tributary =	22.00 ft
Total Dead + Live Load =	2446 plf
Dead Load Only =	1338 plf

CONCRETE SLAB PROPERTIES

f`c = 2500 psi E_c = 2850000 psi E_{cr} = 1425000 psi

ALLOWABLE CONCRETE STRESSES

FLEXURAL TENSILE STRESS	$f_{t,allow} = -6 (f_c')^{0.5}$ =	-300.000	psi
FLEXURAL COMPRESSIVE STRESS	$f_{c,allow} = -0.45 f_c'$ =	1125.000	psi
RESIDUAL COMPRESSIVE STRESS	$P_r = (Pe - SG) / A$ =	0.000	psi
ALLOWABLE BENDING TENSILE STRESS	$F_b = P_r - f_{t,allow}$ =	300.000	psi
ALLOWABLE SHEAR STRESS	$f_v = 1.7*f`c^{0.5} + 0.2*f_p$ =	85.000	psi

CHECK ALLOWABLE LINE LOADS TO SLAB

ALLOWABLE LINE LOAD FOR 4 INCH SLAB H = 4.00 inches

$P_{max} = F_b 2H^2/(\beta/4)$ = 1.71 kips / ft
Where $\beta = (E_{cr} H^3 / 3 k_s)^{0.25}$ = 22 < St,b = 32
k_s = 120 lb / in^3 Modulus of Subgrade Reaction, See Soils Report
(In absence of Soils Report, use Tables for Typical Values)

ALLOWABLE LINE LOAD FOR 3.5 INCH SLAB H = 3.50 inches

$P_{max} = F_b 2H^2/(\beta/4)$ = 1.45 kips / ft
Where $\beta = (E_{cr} H^3 / 3 k_s)^{0.25}$ = 20 < St,b = 24.5
k_s = 120 lb / in^3 Modulus of Subgrade Reaction, See Soils Report
(In absence of Soils Report, use Tables for Typical Values)

ALLOWABLE LINE LOAD FOR 4.5 INCH SLAB H = 4.50 inches

$P_{max} = F_b 2H^2/(\beta/4)$ = 1.98 kips / ft
Where $\beta = (E_{cr} H^3 / 3 k_s)^{0.25}$ = 25 < St,b = 40.5
k_s = 120 lb / in^3 Modulus of Subgrade Reaction, See Soils Report
(In absence of Soils Report, use Tables for Typical Values)

L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-001-192
DATE

PROJECT Desert Rose Customs
SUBJECT Foundation
SHEET OF
DESIGNED NES CHECKED KAB

TYPICAL VALUES OF MODULUS OF SUBGRADE REACTIONS, K_s (lb / in^3)

TYPE OF MATERIAL	MOISTURE CONTENT							
	1 to 4%	5 to 8%	9 to 12%	13 to 16%	17 to 20%	21 to 24%	25 to 28%	> 28%
Silts and clays (liquid limit >50) (OH, CH, MH)	-	175	150	125	100	75	50	25
Silts and clays (liquid limit <50) (OL, CL, ML)	-	200	175	150	125	100	75	50
Silty and clay sands (SM & SH)	300	250	225	200	150	-	-	-
Gravelly sands (SW & SP)	> 300	300	250	-	-	-	-	-
Silty and clayey gravels (GM & GC)	> 300	> 300	300	-	-	-	-	-
Gravel and sand	> 300	> 300	-	-	-	-	-	-

(Values for undisturbed soils)

PTI TABLE 6.1 TYPICAL VALUES K_S	
TYPE OF MATERIAL	**k_s lb/in^3**
Lightly compacted, high plastic Compressible soils	4
Compacted, low plastic soils	40
Stiff, compacted,select granular or stablized fill	400

Allowable Bearing Pressure	k_s lb/in^3
3000	**200**
2500	**185**
2000	**160**
1500	**125**
1000	**60**

(Based on chart for correlation of CBR values to strength or property values, PCA and E.I. DuPont)

RECOMMENDATIONS:
IN ABSENCE OF SOILS REPORT USE 300 lb/in^3 FOR COMPACTED BUILDING PADS USING GRANULAR FILL
IN ABSENCE OF SOILS REPORT USE 125 lb/in^3 FOR 1500 PSF ALLOWABLE BEARING (OR PER TABLE ABOVE)

L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-001-192
DATE

PROJECT Desert Rose Customs
SUBJECT Foundation
SHEET **OF**
DESIGNED NES **CHECKED** KAB

CHECK ALLOWABLE POINT LOAD TO PERIMETER BEAM

ALLOWABLE POINT LOAD FOR 4 INCH SLAB W/ 12 INCH PERIMETER BEAM

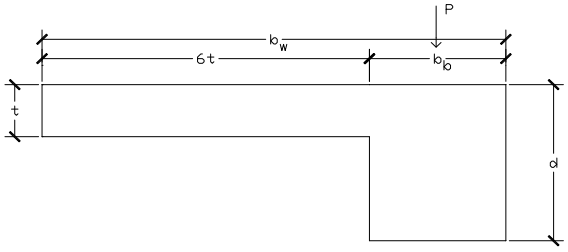
d inches	b_b in	t slab in	b_w in	I in^4	S_b in^3	A in^2
12	12	4	36	2778	365	240

$P_{max} = F_b 4 S_b / \beta$ = 10023.56 lbs

Where $\beta = [4E_{cr} I / (bw \times k_s)]^{0.25}$ = 44

k_s = 120 lb / in^3

Modulus of Subgrade Reaction, See Soils Report
(In absence of Soils Report, use Tables for Typical Values)



CHECK ALLOWABLE UPLIFT LOAD TO SLAB

1. Assume tee beam cantilevered beyond point load, w/12 in footing

2. Assume slab cantilevered each way, no footing, circular failure plane

d inches	b_b in	t slab in	b_w in	I in^4	S_t in^3	S_b in^3	A in^2	S_{slab} in^3	M_{slab} ft-lb
12	12	4	60	2702	620	354	384	32	800

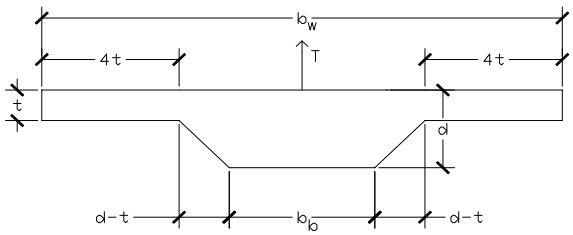
T-beam Allowables
M_A = 15491 ft-lb
V_A = 32640 lb
w = 774 lb/ft

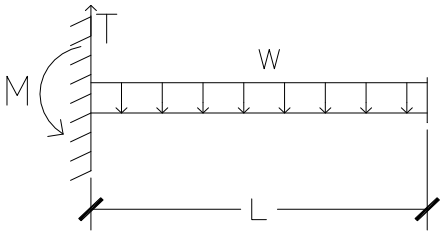
Slab Allowables
M_A = 800 ft-lb per foot
V_A = 16320 lb
w = 48 lb/ft

Sum forces about T with M_A = M, and T = w x L;
(Note: d = t for flat slab design and T = w x pi x L^2)

	Beam	Slab
∴ back substitution and solve for M1 =	$wL^2/2$	M2 = $wL^2/2$
∴ L_1 =	6.33 ft	L_2 ft = 5.75 ft

∴ Assume minimum 2 foot shearwall, L` = 8.33 ft
∴ T = 6443 lb at shear wall with footing
∴ T = 9792 lb at isolated uplift on T - Beam
∴ T = 5027 lb at isolated uplift on 4 inch slab





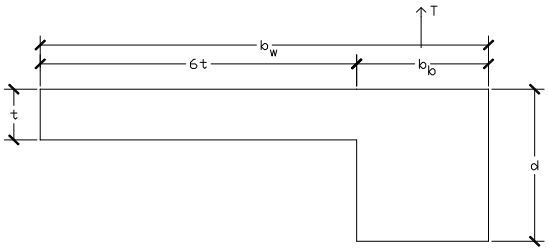
CHECK ALLOWABLE UPLIFT LOAD TO PERIMETER BEAM

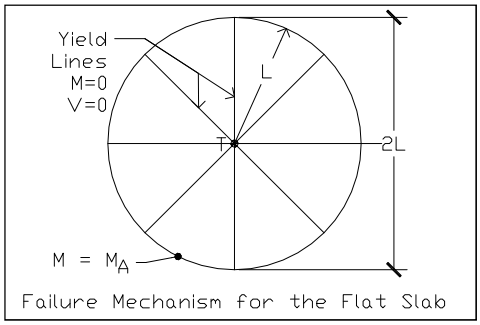
1. Assume L- beam cantilevered beyond point load, w/12 in footing

d inches	b_b in	t slab in	b_w in	I in^4	S_t in^3	S_b in^3	A in^2	S_{slab} in^3	M_{slab} ft-lb
12	12	4	36.0	2778	631	365	240	32	800

M_A = 15782 ft-lb
V_A = 20400 lb
w = 423 lb/ft

∴ back substitution and solve for M = $wL^2/2$
∴ L_1 = 8.64 ft
∴ Assume minimum 2 foot shearwall, L` = 10.64 ft
∴ T = 4501 lb





Failure Mechanism for the Flat Slab

L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-001-192
DATE 11/06/19

PROJECT Desert Rose Customs
SUBJECT FOOTINGS
SHEET
DESIGNED NES
SECTION
CHECKED KAB

FOOTINGS

Assumed Soil Bearing Pressure

q=	2,000	psf not to exceed ------	2,000	psf
	0	psf/ft width greater than	0	in

Concrete Weight:	150 pcf
Concrete Strength:	2500 psi
Steel Grade	60000 psi

Bearing Pressure Spread Footings

Width (in)	Depth (in)	Loads (lb)	#4 Bars
12	12	1,850	2
18	12	4,163	2
24	12	7,400	3
30	12	11,563	4
36	12	16,650	4
42	12	22,663	5
48	12	29,600	5
54	16	36,450	8
60	16	45,000	9

Continuous Footings:

Width (in)	Loads (plf)	#4 Long. Bars	#4 Trans. Bars
12	1850	1	N.A.
18	2775	2	12 in. o.c.
24	3700	2	9 in. o.c.
30	4625	3	9 in. o.c.
36	5550	3	9 in. o.c.
42	6475	4	9 in. o.c.
48	7400	4	9 in. o.c.
54	8100	6	7 in. o.c.
60	9000	7	7 in. o.c.

(The number of longitudinal bars required as per ACI 318, Section 7.12)

Punching Shear

Footing Width	X_{COLUMN} (in)	Y_{COLUMN} (in)	Effective Depth (in)	A_p (in^2)	P_u (lbs)	v_u (psi)	v_c (psi)	Check
12	3	3.5	8.50	399.5	2960	0.3	150	OK
18	3	3.5	8.50	399.5	6660	9.6	150	OK
24	3	3.5	8.50	399.5	11840	22.5	150	OK
30	3	3.5	8.50	399.5	18500	39.2	150	OK
36	3	3.5	8.50	399.5	26640	59.6	150	OK
42	3	3.5	8.50	399.5	36260	83.7	150	OK
48	3	3.5	8.50	399.5	47360	111.4	150	OK
54	3	3.5	12.50	787.5	58320	67.8	150	OK
60	3	3.5	12.50	787.5	72000	85.1	150	OK

One-Way Shear

Footing Width	Effective Depth (in)	X_{min} (in)	e (in)	P_u (lbs)	q_u (psi)	vu (psi)	v_c (psi)	Check
12	8.5	3	0	2960	20.6	0.0	75	OK
18	8.5	3	0	6660	20.6	0.0	75	OK
24	8.5	3	2	11840	20.6	4.8	75	OK
30	8.5	3	5	18500	20.6	12.1	75	OK
36	8.5	3	8	26640	20.6	19.3	75	OK
42	8.5	3	11	36260	20.6	26.6	75	OK
48	8.5	3	14	47360	20.6	33.9	75	OK
54	12.5	3	13	58320	20.0	20.8	75	OK
60	12.5	3	16	72000	20.0	25.6	75	OK

Bending

Footing Width	Effective Depth (in)	X_{min} (in)	q_u (psi)	M_u (lb-in)	Required A_s (in^2)	Min A_s (in^2)	#4 Bars	#5 Bars
12	8.5	3	20.6	2498	0.01	0.20	2	2
18	8.5	3	20.6	10406	0.02	0.30	2	2
24	8.5	3	20.6	27195	0.06	0.52	3	2
30	8.5	3	20.6	56194	0.13	0.65	4	2
36	8.5	3	20.6	100733	0.23	0.78	4	3
42	8.5	3	20.6	164141	0.38	0.91	5	3
48	8.5	3	20.6	249750	0.59	1.04	5	4
54	12.5	3	20.0	351135	0.56	1.56	8	5
60	12.5	3	20.0	487350	0.78	1.73	9	6

(Minimum reinforcement per ACI 318, Section 10.5)