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

DATE: **May 3, 2021**

STRUCTURAL CALCULATIONS
for
732 Naples St
at

Henderson, Nevada
Design is per the 2018 IBC
for
Assured Development
May 3, 2021
Project #: 1939-001-192

PROFESSIONAL ENGINEER-STATE OF NEVADA
SARAH CAMILLE ILOTHO BARENG
Exp. 12-31-2022
CIVIL
No. 025375

Sarah Bareng

Digitally signed by Sarah Bareng
DN: CN=Sarah Bareng, E=sarah.bareng@lrneng.com
Reason: I am the author of this document
Date: 2021-05-03 11:15:53

5/3/2021

BY: _____
SARAH L. BARENG, P.E.
STRUCTURAL DEPARTMENT

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

6765 West Russell Road, Suite 200
Las Vegas, Nevada 89118
Ph. 702.798.7978
Fax. 702.451.2296

51 West 9000 South
Sandy Utah 84070
Ph. 801.565.8580
Fax. 801.565.9340

These engineering calculations are valid only for the address or lot listed above and shall be sealed by an engineer authorized to represent L.R. Nelson Consulting Engineers. They shall not be used for any other location or structure without the express written consent of this firm.

L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-001-192
DATE 11/21/2019

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= 1.00
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 2500 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 11/21/2019

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

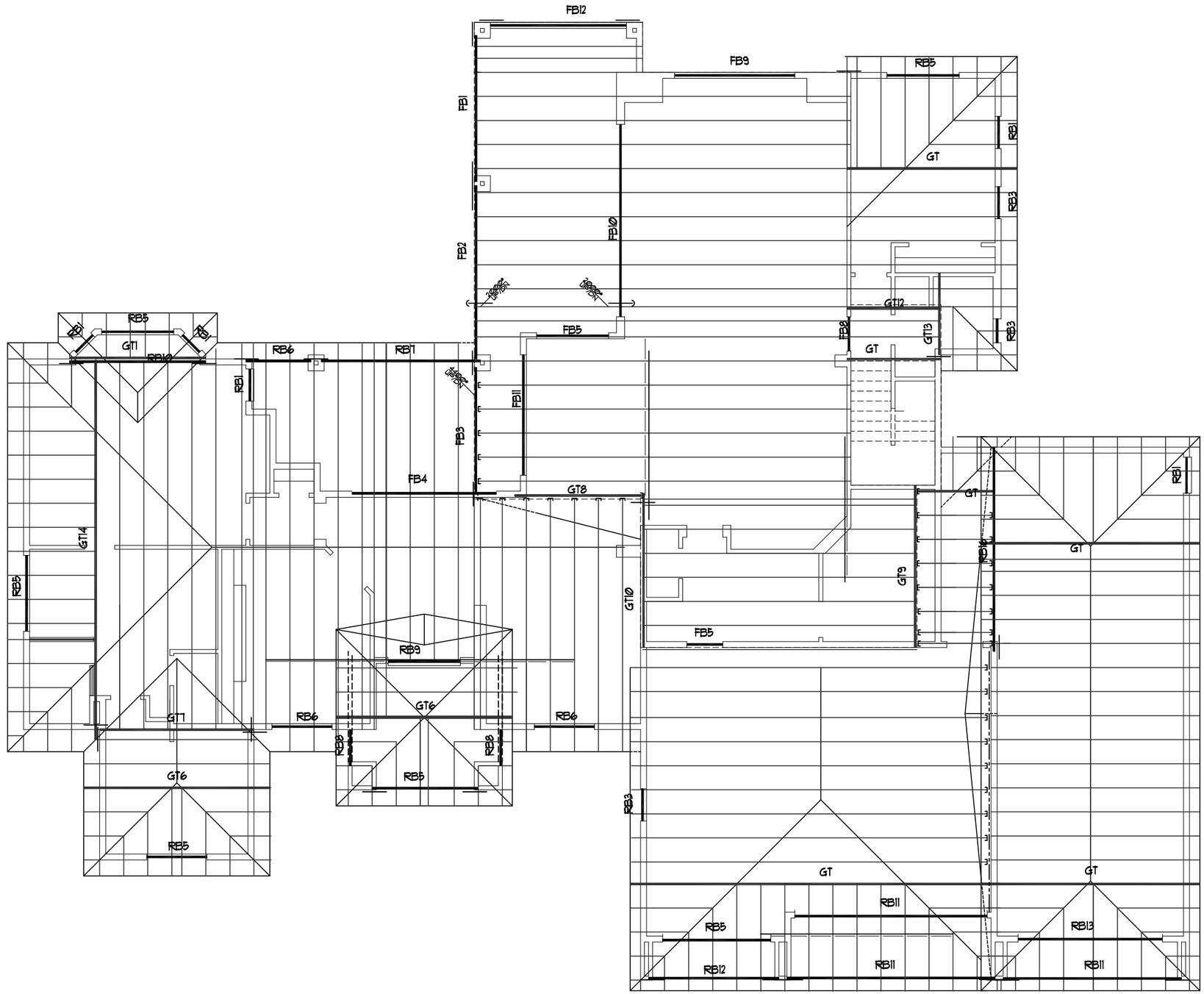
ALT. FLOOR LIVE LOAD

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 APN# 179-04-405-011

SUBJECT XXXX KEY PLAN

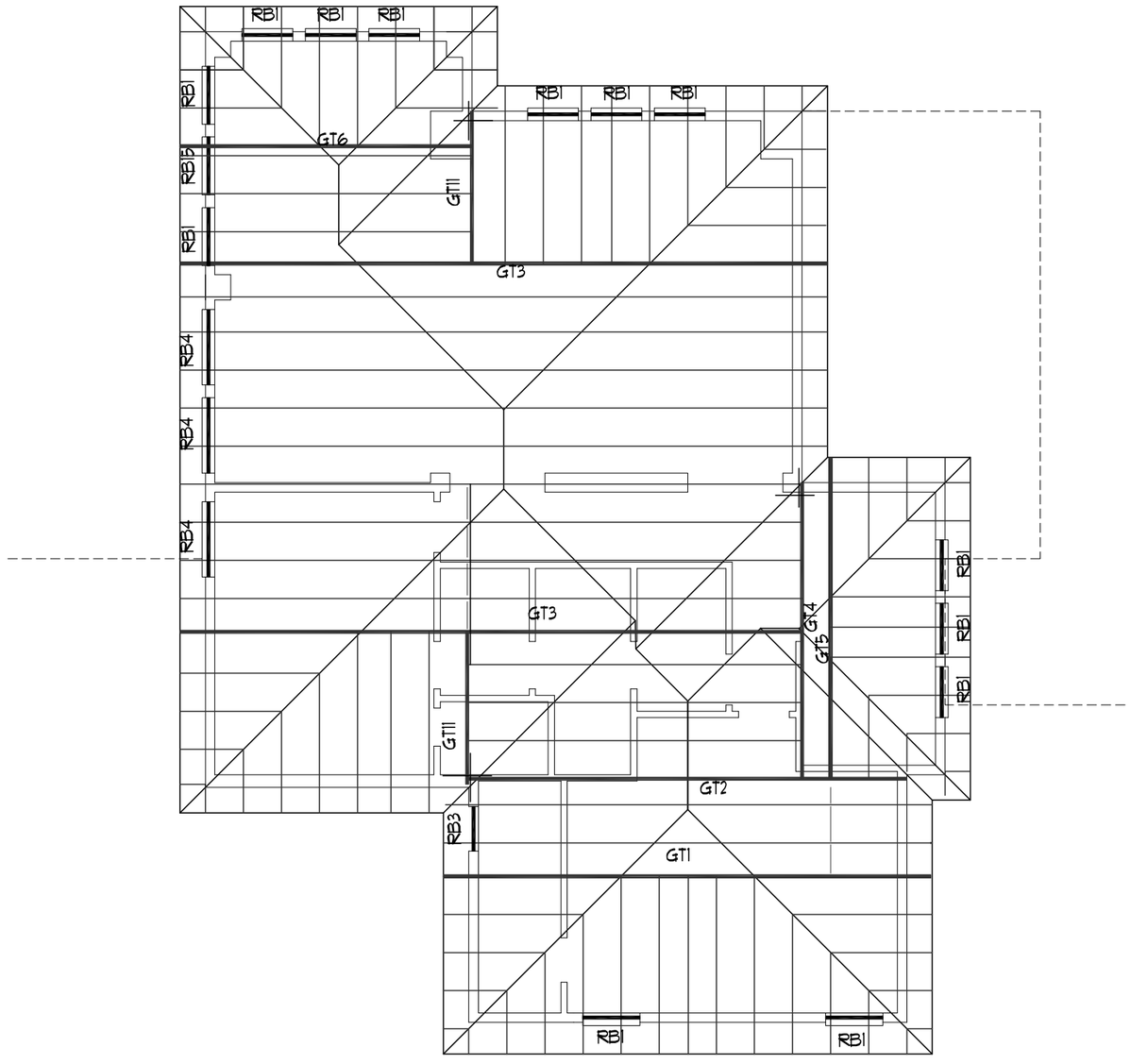
L. R. NELSON CONSULTING ENGINEERS INC

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

JOB NO. 1939-001-192
DATE 11/21/2019

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

L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-001-192
DATE 11/21/2019

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

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			

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			

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			

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			

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			

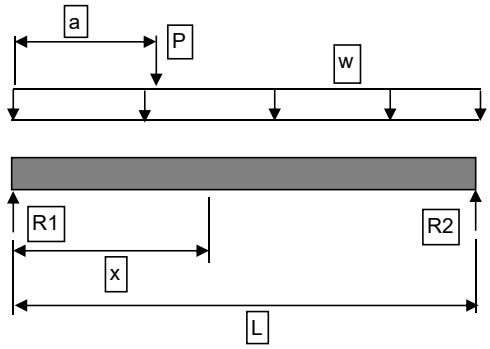
L. R. NELSON CONSULTING ENGINEERS

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.

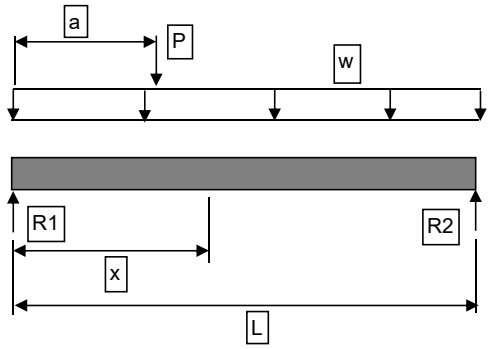
L. R. NELSON CONSULTING ENGINEERS

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 11/21/2019

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					

L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-001-192
DATE 11/21/2019

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

DESCRIPTION:	GT10	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 =	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:														
S =	#N/A	in^3	#N/A	#N/A	in^3	**#N/A**		Max LL Deflection =	#VALUE!	#VALUE!	L/xxx	360	Alternate Loads #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 =	3,760	lb	Post:	2X6		DFL#2	**O.K.**		3759.5	lb		GT10			10.0	2.0	ft					
Right Reaction =	19,593	lb	Post:	6X6		DFL#1 (TIMBERS)	**O.K.**		19592.5	lb		GT10			10.0	10.0	ft					

DESCRIPTION:	GT11	Roof Tributary =	10.00	ft	W_{DL}=	277.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	#N/A											
SIZE:		Floor Tributary =	0.00	ft	W_{LL}=	200.00	lb/ft	C_t =	1.00	C_F =	#N/A											
GRADE:		Wall Tributary =	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													
S =	#N/A	in^3	#N/A	#N/A	in^3	**#N/A**		Max LL Deflection =	#VALUE!	#VALUE!	L/xxx	360	Alternate #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 =	2,336	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					

DESCRIPTION:	GT14	Roof Tributary =	4.00	ft	W_{DL}=	126.00	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 =	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:														
S =	#N/A	in^3	#N/A	#N/A	in^3	**#N/A**		Max LL Deflection =	#VALUE!	#VALUE!	L/xxx	360	Alternate #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 =	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					

DESCRIPTION:	GT12	Roof Tributary =	5.50	ft	W_{DL}=	412.01	lb/ft	C_D =	1.25	$C_{V/L}$ =	#N/A											
SIZE:		Floor Tributary =	1.00	ft	W_{LL}=	150.00	lb/ft	C_t =	1.00	C_F =	#N/A											
GRADE:		Wall Tributary =	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													
S =	#N/A	in^3	#N/A	#N/A	in^3	**#N/A**		Max LL Deflection =	#VALUE!	#VALUE!	L/xxx	360	Alternate #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 =	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					

DESCRIPTION:		Roof Tributary =	0.00	ft	W_{DL}=	0.00	lb/ft	C_D =	1.00	$C_{V/L}$ =	#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 =	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:														
S =	#N/A	in^3	#N/A	#N/A	in^3	**#N/A**		Max LL Deflection =	#VALUE!	#VALUE!	L/xxx	360	Alternate #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 =	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

File = H:\Struct\Projects\P1900\1939001-192 Desert Rose Custom\CALCS\Beams.ec6 .
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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

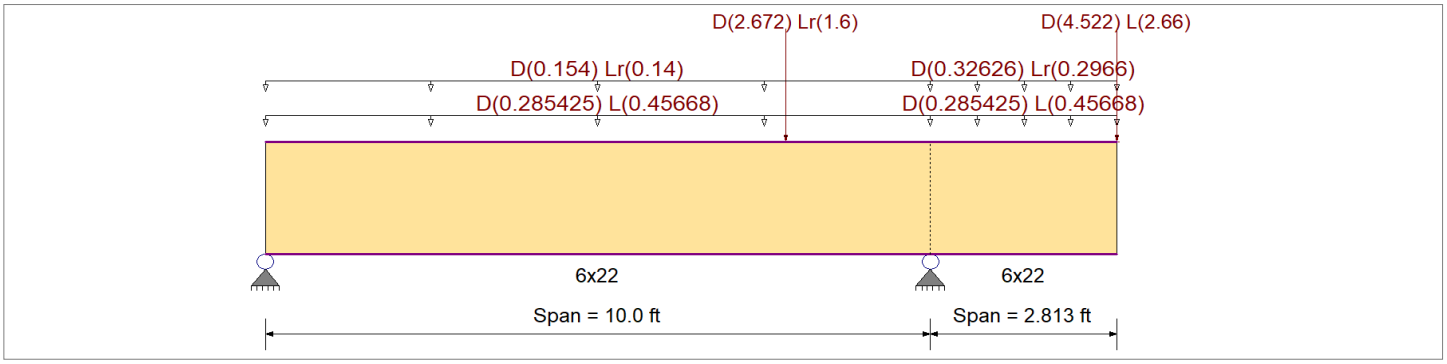
Wood Species : Douglas Fir-Larch
Wood Grade : No.2

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		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	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

File = H:\Struct\Projects\P1900\1939001-192 Desert Rose Custom\CALCS\Beams.ec6 .
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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	Max Stress Ratios V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	Moment Values fb	Moment Values F'b	Shear Values V	Shear Values fv	Shear Values 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

File = H:\Struct\Projects\P1900\1939001-192 Desert Rose Custom\CALCS\Beams.ec6 .
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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 11/21/2019

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)		%											
	Actual		Req'd				Source:			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:	NOT USED				$d_{Point\ Load}$ =	2.25	ft	Reduction (L)		%											
	Actual		Req'd				Source:	GT1		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)		%											
	Actual		Req'd				Source:			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)		%											
	Actual		Req'd				Source:			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)		%											
	Actual		Req'd				Source:			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 11/21/2019

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	l_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	l_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	l_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	l_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	l_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 >	72.47	in^3	O.K.	Max LL Deflection =	0.12 in =	L / 1066		360	#1	#2	LL									
A =	61.50	in^2 >	23.20	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						

L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-001-192
DATE 11/21/2019

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

DESCRIPTION:	RB11	Roof Tributary =	5.00	ft	W_{DL}=	262.01	lb/ft	C_D =	1.25	$C_{V/L}$ =	0.97										
SIZE:	6X12	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 =	8.00	ft	P_{DL} =	0.00	lb	I_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						

DESCRIPTION:	RB12	Roof Tributary =	5.00	ft	W_{DL}=	186.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	0.99										
SIZE:	6X8	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 =	4.00	ft	P_{DL} =	0.00	lb	I_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						

DESCRIPTION:	RB13	Roof Tributary =	5.00	ft	W_{DL}=	262.01	lb/ft	C_D =	1.25	$C_{V/L}$ =	0.99										
SIZE:	6X10	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 =	8.00	ft	P_{DL} =	0.00	lb	I_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						

DESCRIPTION:	RB14	Roof Tributary =	7.00	ft	W_{DL}=	230.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	1.00										
SIZE:	4X6	Floor Tributary =	0.00	ft	W_{LL}=	140.00	lb/ft	C_t =	1.00	C_F =	1.30										
GRADE:	DFL#2	Wall Tributary =	4.00	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)		%											
	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						

DESCRIPTION:	RB15	Roof Tributary =	5.00	ft	W_{DL}=	167.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	1.00										
SIZE:	6X8	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} =	1,169.02	lb	I_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 11/21/2019

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	$C_{V/L}$ =	0.95										
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)		%											
	Actual		Req'd				Source:				L/xxx	Alternate Loads									
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)		%											
	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				

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)		%											
	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:					$d_{Point\ Load}$ =	0.50	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				

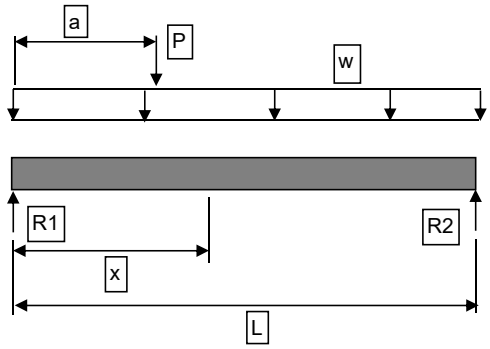
L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-001-192
DATE:

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

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

1.5xDL Deflection = **0.58** inches

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 11/21/2019

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

DESCRIPTION:	FB1	Roof Tributary =	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 Tributary =	6.00	ft	W_{LL}=	340.00	lb/ft	C_t =	1.00	C_F =	1.00										
GRADE:	24F-V4	Wall Tributary =	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)		%											
	Actual		Req'd				Source:	GT3		L/xxx	Alternate Loads										
S =	155.67	in^3 >	93.16	in^3	O.K.	Max LL Deflection =	0.16 in =	L / 1007		360	#1	#2	LL								
A =	69.19	in^2 >	50.05	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					

DESCRIPTION:	FB2	Roof Tributary =	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 Tributary =	6.00	ft	W_{LL}=	560.00	lb/ft	C_t =	1.00	C_F =	1.00										
GRADE:	24F-V4	Wall Tributary =	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)		%											
	Actual		Req'd				Source:			L/xxx	Alternate										
S =	155.67	in^3 >	135.32	in^3	O.K.	Max LL Deflection =	0.27 in =	L / 623		360	#1	#2	LL								
A =	69.19	in^2 >	36.08	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					

DESCRIPTION:	FB3	Roof Tributary =	5.00	ft	W_{DL}=	426.01	lb/ft	C_D =	1.25	$C_{V/L}$ =	1.00										
SIZE:	5-1/8X12	Floor Tributary =	2.00	ft	W_{LL}=	180.00	lb/ft	C_t =	1.00	C_F =	1.00										
GRADE:	24F-V4	Wall Tributary =	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)		%											
	Actual		Req'd				Source:	GT3		L/xxx	Alternate										
S =	123.00	in^3 >	93.24	in^3	O.K.	Max LL Deflection =	0.12 in =	L / 1109		360	#1	#2	LL								
A =	61.50	in^2 >	33.42	in^2	O.K.	Max TL Deflection =	0.34 in =	L / 384		240											
I =	738.00	in^4 >	461.19	in^4	O.K.				Addt'l Load to Post		Source			Lx	Ly						
Left Reaction =	5,356	lb	Post:	2X6		DFL#2	O.K.			lb				8.0	2.0	ft					
Right Reaction =	7,986	lb	Post:	2X6		DFL#2	O.K.			lb				8.0	2.0	ft					

DESCRIPTION:	FB4	Roof Tributary =	16.25	ft	W_{DL}=	471.50	lb/ft	C_D =	1.25	$C_{V/L}$ =	0.98										
SIZE:	5-1/8X12	Floor Tributary =	0.00	ft	W_{LL}=	325.00	lb/ft	C_t =	1.00	C_F =	1.00										
GRADE:	24F-V4	Wall Tributary =	6.00	ft	P_{DL} =	3,589.06	lb	I_u =	12.50												
LENGTH (ft)=	12.50	Additional Load =	0.00	plf	P_{LL} =	1,766.90	lb	Reduction (L_r)		%											
NOTES:					$d_{Point\ Load}$ =	10.50	ft	Reduction (L)		%											
	Actual		Req'd				Source:	FB3		L/xxx	Alternate										
S =	123.00	in^3 >	87.59	in^3	O.K.	Max LL Deflection =	0.18 in =	L / 843		360	#1	#2	LL								
A =	61.50	in^2 >	39.31	in^2	O.K.	Max TL Deflection =	0.46 in =	L / 325		240											
I =	738.00	in^4 >	545.03	in^4	O.K.				Addt'l Load to Post		Source			Lx	Ly						
Left Reaction =	5,835	lb	Post:	2X6		DFL#2	O.K.			lb				10.0	2.0	ft					
Right Reaction =	9,477	lb	Post:	(2)2X6		DFL#2	O.K.			lb				10.0	2.0	ft					

DESCRIPTION:	FB5	Roof Tributary =	5.00	ft	W_{DL}=	426.01	lb/ft	C_D =	1.00	$C_{V/L}$ =	1.00										
SIZE:	6X6	Floor Tributary =	2.00	ft	W_{LL}=	180.00	lb/ft	C_t =	1.00	C_F =	1.00										
GRADE:	DFL#1 (TIMBERS)	Wall Tributary =	14.00	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:					$d_{Point\ Load}$ =	0.50	ft	Reduction (L)		%											
	Actual		Req'd				Source:			L/xxx	Alternate										
S =	27.73	in^3 >	24.24	in^3	O.K.	Max LL Deflection =	0.04 in =	L / 1674		360	#1	#2	LL								
A =	30.25	in^2 >	13.59	in^2	O.K.	Max TL Deflection =	0.14 in =	L / 497		240											
I =	76.26	in^4 >	36.82	in^4	O.K.				Addt'l Load to Post		Source			Lx	Ly						
Left Reaction =	1,818	lb	Post:	2X6		DFL#2	O.K.			lb				8.0	2.0	ft					
Right Reaction =	1,818	lb	Post:	2X6		DFL#2	O.K.			lb				8.0	2.0	ft					

L. R. NELSON CONSULTING ENGINEERS

		JOB NO.	1939-001-192
		DATE:	11/21/19
PROJECT	Desert Rose Customs	**SHEET**	**OF**
SUBJECT	BEAMS AND HEADERS	**DESIGNED** NES	**CHECKED** KAB

Span , ft 14.25 MEMBER: FB2 SEISMIC AND WIND CHECK

UNIFORM LOADS, plf (continuous across entire member)

w1DL	768.00	Continuous lateral support (1= yes, 0= no)?	1	Yes
w1LL	560.00	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	0.00	a1	0.00 ft	P3 D	0.00	a3	1.00 ft
P1 LL	0.00			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	1499.71	a E1	4.25 ft	PW1	808.84	a W1	4.25 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 =	10158 LB	1	151 LB	2
V_b =	9024 LB	1	1664 LB	2
RL =	10680 LB	1	329 LB	2
RR =	9546 LB	1	1842 LB	2

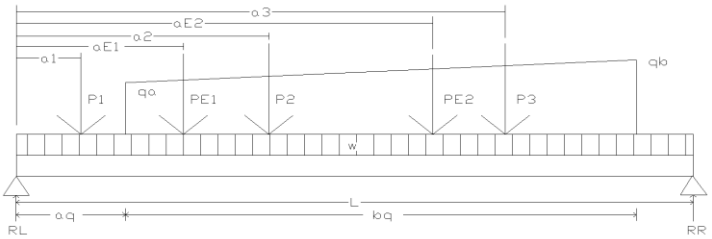
MEMBER DESIGN FORCES

NEGATIVE MOMENT	-2354 LB-FT
POSITIVE MOMENT	37264 LB-FT
MAXIMUM SHEAR	10158 LB

MEMBER DESIGN

		A in^2	S in^3	I in^4
Member Type	24F-V4			
Member Section	5-1/8X13-1/2	69.19	155.67	1050.79

	Allowable Stress psi	adjustment factor CD(CL)..(Cx)(Fasd)		Adjusted Stress psi		Member Stresses psi	
Fv	265.00	1.92	F`v =	509 psi	fv =	220 psi	OKAY
Fb	2400.00	1.92	F`b =	4608 psi	pos fb =	2873 psi	OKAY
E	1.8E+06	1.00	E` =	1.8E+06	neg fb =	27 psi	OKAY
Fb_neg	1850	1.92	F`b neg =	3552 psi			



Provide: 5-1/8X13-1/2
24F-V4

NOTES: NO STRAP REQUIRED AT EITHER END OF MEMBER

L. R. NELSON CONSULTING ENGINEERS

		JOB NO.	1939-001-192
		DATE:	11/21/19
PROJECT	Desert Rose Customs	**SHEET**	**OF**
SUBJECT	BEAMS AND HEADERS	**DESIGNED** NES	**CHECKED** KAB

Span , ft	11.00	MEMBER:	FB3			SEISMIC AND WIND CHECK	
UNIFORM LOADS, plf (continuous across entire member)							
w1DL	426.01	Continuous lateral support (1= yes, 0= no)?				1	Yes
w1LL	180.00	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	3589.06	a1	8.00 ft	GT3	P3 D	0.00	a3	1.00 ft
P1 LL	1766.90				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	868.26	a E1	10.00 ft		PW1	1099.18	a W1	10.00 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				CD =	1.60
qa DL	0.00	a q	1.00 ft	Fasd =	1.2
qa LL	0.00	b q	1.00 ft	Ω =	2.5
qb DL	0.00			SDS =	0.422
qb LL	0.00			*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

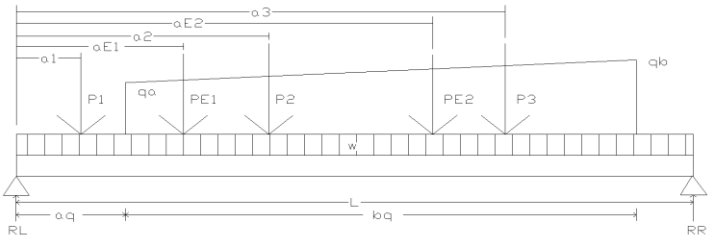
	MAXIMUM	CASE	MINIMUM	CASE
V_a =	4473 LB	1	1501 LB	2
V_b =	8111 LB	1	608 LB	2
RL =	4721 LB	1	1600 LB	2
RR =	8359 LB	1	706 LB	2

MEMBER DESIGN FORCES	
NEGATIVE MOMENT	0 LB-FT
POSITIVE MOMENT	19212 LB-FT
MAXIMUM SHEAR	8111 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	265.00	1.92	F`v =	509 psi	fv =	198 psi	OKAY
Fb	2400.00	1.92	F`b =	4608 psi	pos fb =	1874 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 11/21/2019

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	l_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	l_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	l_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:	FB9	Roof Tributary =	5.00	ft	W_{DL}=	350.01	lb/ft	C_D =	1.00	$C_{V/L}$ =	0.99											
SIZE:	6X10	Floor Tributary =	2.00	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	l_u =	10.50													
LENGTH (ft)=	10.50	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	>	65.46	in^3	O.K.	Max LL Deflection =	0.08 in =	L / 1609	360	#1	#2	LL									
A =	52.25	in^2	>	20.85	in^2	O.K.	Max TL Deflection =	0.23 in =	L / 547	240												
I =	392.96	in^4	>	172.56	in^4	O.K.				Addt'l Load to Post		Source		Lx	Ly							
Left Reaction =	2,783	lb	Post:	2X6		DFL#2	O.K.			lb				8.0	2.0	ft						
Right Reaction =	2,783	lb	Post:	2X6		DFL#2	O.K.			lb				8.0	2.0	ft						

DESCRIPTION:	FB10	Roof Tributary =	0.00	ft	W_{DL}=	506.77	lb/ft	C_D =	1.00	$C_{V/L}$ =	0.96											
SIZE:	5-1/8X16-1/2	Floor Tributary =	18.75	ft	W_{LL}=	675.00	lb/ft	C_t =	1.00	C_F =	1.00											
GRADE:	24F-V4	Wall Tributary =	2.00	ft	P_{DL} =	0.00	lb	l_u =	16.50													
LENGTH (ft)=	16.50	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%												
NOTES:					$d_{Point\ Load}$ =	0.00	ft	Reduction (L)	90	%												
Actual		Req'd			Source:			L/xxx	Alternate													
S =	232.55	in^3	>	210.05	in^3	O.K.	Max LL Deflection =	0.33 in =	L / 607	360	#1	#2	LL									
A =	84.56	in^2	>	45.99	in^2	O.K.	Max TL Deflection =	0.57 in =	L / 347	240												
I =	1918.51	in^4	>	1,327.16	in^4	O.K.				Addt'l Load to Post		Source		Lx	Ly							
Left Reaction =	9,750	lb	Post:	(2)2X6		DFL#2	O.K.			lb				8.0	2.0	ft						
Right Reaction =	9,750	lb	Post:	(2)2X6		DFL#2	O.K.			lb				8.0	2.0	ft						

L. R. NELSON CONSULTING ENGINEERS

		JOB NO.	1939-001-192
		DATE:	11/21/19
PROJECT	Desert Rose Customs	**SHEET**	**OF**
SUBJECT	BEAMS AND HEADERS	**DESIGNED** NES	**CHECKED** KAB

Span , ft 16.50 MEMBER: FB10 SEISMIC AND WIND CHECK

UNIFORM LOADS, plf (continuous across entire member)

w1DL	506.00	Continuous lateral support (1= yes, 0= no)?	1	Yes
w1LL	675.00	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	0.00	a1	0.00 ft	P3 D	0.00	a3	1.00 ft
P1 LL	0.00			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	1499.71	a E1	1.00 ft	PW1	808.84	a W1	1.00 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 =	10736 LB	1	-1381 LB	2
V_b =	8265 LB	1	1914 LB	2
RL =	11178 LB	1	-1264 LB	2
RR =	8706 LB	1	2031 LB	2

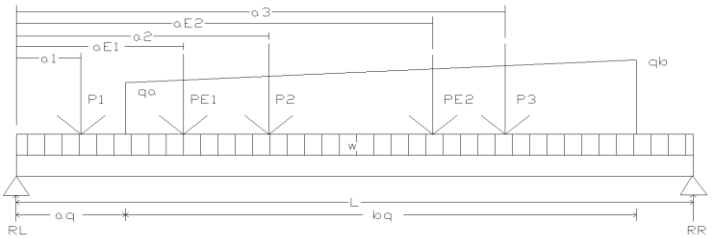
MEMBER DESIGN FORCES

NEGATIVE MOMENT	-1401 LB-FT
POSITIVE MOMENT	36617 LB-FT
MAXIMUM SHEAR	10736 LB

MEMBER DESIGN

		A in^2	S in^3	I in^4
Member Type	24F-V4			
Member Section	5-1/8X16-1/2	84.56	232.55	1918.51

	Allowable Stress psi	adjustment factor CD(CL)..(Cx)(Fasd)		Adjusted Stress psi		Member Stresses psi	
Fv	265.00	1.92	F`v =	509 psi	fv =	190 psi	OKAY
Fb	2400.00	1.92	F`b =	4608 psi	pos fb =	1890 psi	OKAY
E	1.8E+06	1.00	E` =	1.8E+06	neg fb =	9 psi	OKAY
Fb_neg	1850	1.92	F`b neg =	3552 psi			



Provide: 5-1/8X16-1/2
24F-V4

NOTES: CS 16
REQUIRED AT LEFT END
OF MEMBER T max= 1705 LBS

L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-001-192
DATE 11/21/2019

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

DESCRIPTION:	FB11	Roof Tributary =	0.00	ft	W_{DL}=	525.77	lb/ft	C_D =	1.00	$C_{V/L}$ =	0.99											
SIZE:	5-1/8X10-1/2	Floor Tributary =	18.75	ft	W_{LL}=	750.00	lb/ft	C_t =	1.00	C_F =	1.00											
GRADE:	24F-V4	Wall Tributary =	3.00	ft	P_{DL} =	0.00	lb	I_u =	10.50													
LENGTH (ft)=	10.50	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 Loads										
S =	94.17	in^3 >	88.99	in^3	O.K.	Max LL Deflection =	0.23 in =	L / 547	360	#1	#2	LL										
A =	53.81	in^2 >	31.59	in^2	O.K.	Max TL Deflection =	0.39 in =	L / 321	240													
I =	494.40	in^4 >	369.21	in^4	O.K.				Addt'l Load to Post		Source		Lx	Ly								
Left Reaction =	6,698	lb	Post:	2X6		DFL#2	O.K.			lb			8.0	2.0	ft							
Right Reaction =	6,698	lb	Post:	2X6		DFL#2	O.K.			lb			8.0	2.0	ft							

DESCRIPTION:	FB12	Roof Tributary =	4.00	ft	W_{DL}=	341.01	lb/ft	C_D =	1.00	$C_{V/L}$ =	0.99											
SIZE:	5-1/8X9	Floor Tributary =	1.00	ft	W_{LL}=	120.00	lb/ft	C_t =	1.00	C_F =	1.00											
GRADE:	24F-V4	Wall Tributary =	12.00	ft	P_{DL} =	0.00	lb	I_u =	11.50													
LENGTH (ft)=	11.50	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 =	69.19	in^3 >	38.52	in^3	O.K.	Max LL Deflection =	0.08 in =	L / 1638	360	#1	#2	LL										
A =	46.13	in^2 >	13.05	in^2	O.K.	Max TL Deflection =	0.32 in =	L / 426	240													
I =	311.34	in^4 >	175.28	in^4	O.K.				Addt'l Load to Post		Source		Lx	Ly								
Left Reaction =	2,651	lb	Post:	2X6		DFL#2	O.K.			lb			8.0	2.0	ft							
Right Reaction =	2,651	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							

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)		%												
	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:					$d_{Point\ Load}$ =	0.50	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:	Roof 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 =	9.71 ft
Roof Tributary Width =	23.00 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 =	DFLSTUD
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 =	0.153	OK
DL + RLL =	0.245	OK
DL + SL =	0.153	OK
DL + 0.6*WIND =	0.404	OK
DL + 0.75*(FLL + RLL) =	0.245	OK
DL + 0.75*(FLL + SL) =	0.137	OK
DL + 0.75*(0.6*WIND + FLL + RLL) =	0.352	OK
DL + 0.75*(0.6*WIND + FLL + SL) =	0.307	OK
DEFLECTION L/? =	1006.0	O.K.**

** Deflection Determined Using 0.42 x Wind Load

Exterior Finish =	Brittle	(L/240 Deflection Ratio)

F_b =	700 psi
F_c =	850 psi
E =	1,400,000 psi

Factors	
C_F (Bending) =	1.00
C_F (Compression) =	1.00
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 =	582 lb/ft
Roof Live Load, RLL =	460 lb/ft
Floor Live Load, FLL =	0 lb/ft
Snow Load, SL =	0 lb/ft

Ultimate Design Pressure =	30.11 psf	
Ultimate Max. Deflection =	0.28 in	
Max. Deflection x 0.42 =	0.12 in	
Max. Deflection x 0.60 =	N/A	
L/d =	21.19	< 50 (OK)

Check Bearing Stress on Wall Plate		
Wall Plate Lumber Grade =	DFL#2	
Max. Stud Reaction on Wall Plate =	1,389 lbs	
Bearing Stress on Wall Plate =	168.40 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)	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 =	94.06 psi	0.00 psi	74.34 psi	0.00 psi

Maximum Bending Moment & Stress	
M =	473 ft-lb
f_b =	750.70 psi

Allowable Bending Stress		
F'_b =	1,288 psi	

Allowable Axial Stresses					
c =	0.80	Overwrite c =			
$(L/d)^2$ =	448.82				
COV_E =	0.25	Overwrite COV_E =			
E_{min}' =	511,432 psi				
F_{cE} =	936.67				
	DL	FLL	RLL	SL	WIND
F_c^* =	850.00	850.00	1,062.50	850.00	1,360.00
F_{cE}/F_c^* =	1.102	1.102	0.882	1.102	0.689
C_P =	0.723	0.723	0.646	0.723	0.552
F'c =	614.93	614.93	686.28	614.93	751.25 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 =	DFLSTUD
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 =	**0.107**	**OK**
DL + RLL =	**0.138**	**OK**
DL + SL =	**0.107**	**OK**
DL + 0.6*WIND =	**0.606**	**OK**
DL + 0.75*(FLL + RLL) =	**0.138**	**OK**
DL + 0.75*(FLL + SL) =	**0.105**	**OK**
DL + 0.75*(0.6*WIND + FLL + RLL) =	**0.475**	**OK**
DL + 0.75*(0.6*WIND + FLL + SL) =	**0.458**	**OK**
DEFLECTION L/? =	**296.8**	**O.K.****

** Deflection Determined Using 0.42 x Wind Load

Exterior Finish =	Brittle	(L/240 Deflection Ratio)

F_b =	700 psi
F_c =	850 psi
E =	1,400,000 psi

Factors	
C_F (Bending) =	1.00
C_F (Compression) =	1.00
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 =	2.07 in	
Max. Deflection x 0.42 =	0.87 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 =	1,288 psi	

Allowable Axial Stresses					
c =	0.80	Overwrite c =			
$(L/d)^2$ =	2200.46				
COV_E =	0.25	Overwrite COV_E =			
E_{min}' =	511,432 psi				
F_{cE} =	191.05				
	DL	FLL	RLL	SL	WIND
F_c^* =	850.00	850.00	1,062.50	850.00	1,360.00
F_{cE}/F_c^* =	0.225	0.225	0.180	0.225	0.140
C_P =	0.213	0.213	0.173	0.213	0.136
F'c =	181.23	181.23	183.40	181.23	185.21 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: RV GARAGE - Interior **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 =	15.71 ft	
Roof Tributary Width =	22.50 ft	
Floor Tributary Width =	0.00 ft	
Wall/Misc. DL Width =	4.00 ft	
Wind Zone (Interior or End) =	End	N/A
Stud Size =	2X6	
Stud Spacing =	16 in o.c.	
Lumber Grade =	DFLSTUD	
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 =	**0.289**	**OK**
DL + RLL =	**0.502**	**OK**
DL + SL =	**0.289**	**OK**
DL + 0.6*WIND =	**0.417**	**OK**
DL + 0.75*(FLL + RLL) =	**0.502**	**OK**
DL + 0.75*(FLL + SL) =	**0.281**	**OK**
DL + 0.75*(0.6*WIND + FLL + RLL) =	**0.513**	**OK**
DL + 0.75*(0.6*WIND + FLL + SL) =	**0.331**	**OK**
DEFLECTION L/? =	**601.0**	**OK****

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

Exterior Finish =	Brittle	(L/240 Deflection Ratio)

F_b =	700 psi
F_c =	850 psi
E =	1,400,000 psi

Factors	
C_F (Bending) =	1.00
C_F (Compression) =	1.00
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 =	571 lb/ft
Roof Live Load, RLL =	450 lb/ft
Floor Live Load, FLL =	0 lb/ft
Snow Load, SL =	0 lb/ft

Ultimate Design Pressure =	8.33 psf	
Ultimate Max. Deflection =	0.52 in	
Max. Deflection x 0.42 =	N/A	
Max. Deflection x 0.60 =	0.31 in	
L/d =	34.28	< 50 (OK)

Check Bearing Stress on Wall Plate		
Wall Plate Lumber Grade =	DFL#2	
Max. Stud Reaction on Wall Plate =	1,361 lbs	
Bearing Stress on Wall Plate =	165.01 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 =	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 =	92.28 psi	0.00 psi	72.73 psi	0.00 psi

Maximum Bending Moment & Stress	
M =	343 ft-lb
f_b =	543.70 psi

Allowable Bending Stress		
F'_b =	1,288 psi	

Allowable Axial Stresses					
c =	0.80	Overwrite c =			
$(L/d)^2$ =	1174.87				
COV_E =	0.25	Overwrite COV_E =			
E_{min}' =	511,432 psi				
F_{cE} =	357.82				
	DL	FLL	RLL	SL	WIND
F_c^* =	850.00	850.00	1,062.50	850.00	1,360.00
F_{cE}/F_c^* =	0.421	0.421	0.337	0.421	0.263
C_P =	0.376	0.376	0.309	0.376	0.247
F'c =	319.38	319.38	328.44	319.38	335.80 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:	RV GARAGE - Exterior	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 =	15.71 ft
Roof Tributary Width =	10.00 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 =	DFLSTUD
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 =	0.150	OK
DL + RLL =	0.244	OK
DL + SL =	0.150	OK
DL + 0.6*WIND =	0.887	OK
DL + 0.75*(FLL + RLL) =	0.244	OK
DL + 0.75*(FLL + SL) =	0.146	OK
DL + 0.75*(0.6*WIND + FLL + RLL) =	0.752	OK
DL + 0.75*(0.6*WIND + FLL + SL) =	0.671	OK
DEFLECTION L/? =	289.5	O.K.**

** Deflection Determined Using 0.42 x Wind Load

Exterior Finish =	Brittle	(L/240 Deflection Ratio)

F_b =	700 psi
F_c =	850 psi
E =	1,400,000 psi

Factors	
C_F (Bending) =	1.00
C_F (Compression) =	1.00
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 =	296 lb/ft
Roof Live Load, RLL =	200 lb/ft
Floor Live Load, FLL =	0 lb/ft
Snow Load, SL =	0 lb/ft

Ultimate Design Pressure =	24.70 psf	
Ultimate Max. Deflection =	1.55 in	
Max. Deflection x 0.42 =	0.65 in	
Max. Deflection x 0.60 =	N/A	
L/d =	34.28	< 50 (OK)

Check Bearing Stress on Wall Plate		
Wall Plate Lumber Grade =	DFL#2	
Max. Stud Reaction on Wall Plate =	661 lbs	
Bearing Stress on Wall Plate =	80.16 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)	19.99 psf	-21.95 psf	19.99 psf	-24.70 psf
qh (psf)	19.59 psf	19.59 psf	19.59 psf	19.59 psf
GCp	0.841	-0.941	0.841	-1.081
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 =	47.84 psi	0.00 psi	32.32 psi	0.00 psi

Maximum Bending Moment & Stress	
M =	1016 ft-lb
f_b =	1612.41 psi

Allowable Bending Stress		
F'_b =	1,288 psi	

Allowable Axial Stresses						
c =	0.80	Overwrite c =				
$(L/d)^2$ =	1174.87					
COV_E =	0.25	Overwrite COV_E =				
E_{min}' =	511,432 psi					
F_{cE} =	357.82					
	DL	FLL	RLL	SL	WIND	
F_c^* =	850.00	850.00	1,062.50	850.00	1,360.00	
F_{cE}/F_c^* =	0.421	0.421	0.337	0.421	0.263	
C_P =	0.376	0.376	0.309	0.376	0.247	
F'c =	319.38	319.38	328.44	319.38	335.80	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 - Family Room	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

Wall Height =	9.71 ft	
Roof Tributary Width =	23.00 ft	
Floor Tributary Width =	10.00 ft	
Wall/Misc. DL Width =	12.00 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

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

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

INTERACTION RESULTS FOR POSSIBLE CONTROLLING LOAD COMBINATIONS		
DL + FLL =	0.651	OK
DL + RLL =	0.669	OK
DL + SL =	0.463	OK
DL + 0.6*WIND =	0.366	OK
DL + 0.75*(FLL + RLL) =	0.754	OK
DL + 0.75*(FLL + SL) =	0.595	OK
DL + 0.75*(0.6*WIND + FLL + RLL) =	0.800	OK
DL + 0.75*(0.6*WIND + FLL + SL) =	0.504	OK
DEFLECTION L/? =	999.5	OK**

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

Exterior Finish =	Brittle	(L/240 Deflection Ratio)

Wall Uniform Loads	
Dead Load, DL =	984 lb/ft
Roof Live Load, RLL =	460 lb/ft
Floor Live Load, FLL =	400 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 =	1,844 lbs	
Bearing Stress on Wall Plate =	351.24 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 =	187.43 psi	76.19 psi	87.62 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:	
PROJECT:	Desert Rose Customs	SHEET	OF
DESCRIPTION:	2x4 Interior Bearing - 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

Wall Height =	9.71 ft	
Roof Tributary Width =	0.00 ft	
Floor Tributary Width =	14.00 ft	
Wall/Misc. DL Width =	5.00 ft	
Wind Zone (Interior or End) =	End	N/A
Stud Size =	2X4	
Stud Spacing =	16 in o.c.	
Lumber Grade =	DFLSTUD	
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

F_b =	700 psi
F_c =	850 psi
E =	1,400,000 psi

Factors	
C_F (Bending) =	1.10
C_F (Compression) =	1.05
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

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

INTERACTION RESULTS FOR POSSIBLE CONTROLLING LOAD COMBINATIONS		
DL + FLL =	0.755	OK
DL + RLL =	0.325	OK
DL + SL =	0.334	OK
DL + 0.6*WIND =	0.410	OK
DL + 0.75*(FLL + RLL) =	0.755	OK
DL + 0.75*(FLL + SL) =	0.755	OK
DL + 0.75*(0.6*WIND + FLL + RLL) =	0.769	OK
DL + 0.75*(0.6*WIND + FLL + SL) =	0.769	OK
DEFLECTION L/? =	655.9	OK**

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

Exterior Finish =	Brittle	(L/240 Deflection Ratio)

Wall Uniform Loads	
Dead Load, DL =	445 lb/ft
Roof Live Load, RLL =	0 lb/ft
Floor Live Load, FLL =	560 lb/ft
Snow Load, SL =	0 lb/ft

Ultimate Design Pressure =	8.33 psf	
Ultimate Max. Deflection =	0.30 in	
Max. Deflection x 0.42 =	N/A	
Max. Deflection x 0.60 =	0.18 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 =	1,340 lbs	
Bearing Stress on Wall Plate =	255.24 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 =	113.02 psi	142.22 psi	0.00 psi	0.00 psi

Maximum Bending Moment & Stress	
M =	131 ft-lb
f_b =	512.91 psi

Allowable Bending Stress		
F'_b =	1,417 psi	

Allowable Axial Stresses					
c =	0.80	Overwrite c =			
$(L/d)^2$ =	1108.32				
COV_E =	0.25	Overwrite COV_E =			
E_{min}' =	511,432 psi				
F_{cE} =	379.31				
	DL	FLL	RLL	SL	WIND
F_c^* =	892.50	892.50	1,115.63	892.50	1,428.00
F_{cE}/F_c^* =	0.425	0.425	0.340	0.425	0.266
C_P =	0.379	0.379	0.312	0.379	0.249
F'c =	338.08	338.08	347.80	338.08	355.71 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 11/21/19

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

Description and Location: 2x6 worst case ROOF

Top Plate Size:	2X6			Dead	Live		Tributary Area			Roof Truss Spacing:	24 in. o.c.
Species:	DFL#2		Roof	22	20	psf	Roof	25	ft	Floor Truss Spacing:	24 in. o.c.
STUDS	No. of Plates:	2	Floor	25.001	40	psf	Floor	0	ft		
	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	Allowable Point Load P_{ALLOW} =	2601	lbs.
F'_V	225	C_M	1.00	C_F	1.30	P_{ALLOW} (due to Bending)	2601	lbs.
E'	1600000	C_t	1.00	C_{fu}	1.15	P_{ALLOW} (due to Shear)	3232	lbs.
F_{CPERP}	625	Ci	1.00	C_r	1.00	P_{ALLOW} (due to Crushing)	5156	lbs.
						P_{ACTUAL} =	2100	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.
STUDS	No. of Plates:	2	Floor	25.001	40	psf	Floor	16	ft		
	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	Allowable Point Load P_{ALLOW} =	2081	lbs.
F'_V	180	C_M	1.00	C_F	1.30	P_{ALLOW} (due to Bending)	2081	lbs.
E'	1600000	C_t	1.00	C_{fu}	1.15	P_{ALLOW} (due to Shear)	2586	lbs.
F_{CPERP}	625	Ci	1.00	C_r	1.00	P_{ALLOW} (due to Crushing)	5156	lbs.
						P_{ACTUAL} =	2080.024862	lbs

TOP PLATE IS SUFFICIENT

Description and Location: 2x4 Interior Bearing Family Room

Top Plate Size:	2X4			Dead	Live		Tributary Area			Roof Truss Spacing:	24 in. o.c.
Species:	DFL#2		Roof	22	20	psf	Roof	6	ft	Floor Truss Spacing:	24 in. o.c.
STUDS	No. of Plates:	2	Floor	25.001	40	psf	Floor	10	ft		
	No.:	1	Wall/Misc.	19.001		psf	Wall	12	ft		
	Spacing:	16	in o.c.								
Align Studs w/ trusses:	No										

F'_B	1856.25	C_D	1.25	C_L	1.00	Allowable Point Load P_{ALLOW} =	1827	lbs.
F'_V	225	C_M	1.00	C_F	1.50	P_{ALLOW} (due to Bending)	1827	lbs.
E'	1600000	C_t	1.00	C_{fu}	1.10	P_{ALLOW} (due to Shear)	2057	lbs.
F_{CPERP}	625	Ci	1.00	C_r	1.00	P_{ALLOW} (due to Crushing)	3281	lbs.
						P_{ACTUAL} =	1772.024862	lbs

TOP PLATE IS SUFFICIENT

Description and Location: Interior Bearing 2x6 garage

Top Plate Size:	2X6			Dead	Live		Tributary Area			Roof Truss Spacing:	24 in. o.c.
Species:	DFL#2		Roof	22	20	psf	Roof	6	ft	Floor Truss Spacing:	24 in. o.c.
STUDS	No. of Plates:	2	Floor	25.001	40	psf	Floor	2	ft		
	No.:	1	Wall/Misc.	19.001		psf	Wall	12	ft		
	Spacing:	12	in o.c.								
Align Studs w/ trusses:	No										

F'_B	1681.875	C_D	1.25	C_L	1.00	Allowable Point Load P_{ALLOW} =	3468	lbs.
F'_V	225	C_M	1.00	C_F	1.30	P_{ALLOW} (due to Bending)	3468	lbs.
E'	1600000	C_t	1.00	C_{fu}	1.15	P_{ALLOW} (due to Shear)	3600	lbs.
F_{CPERP}	625	Ci	1.00	C_r	1.00	P_{ALLOW} (due to Crushing)	5156	lbs.
						P_{ACTUAL} =	892.0124311	lbs

TOP PLATE IS SUFFICIENT

Description and Location: 2x4 Interior Bearing Stairs

Top Plate Size:	2X4			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.
STUDS	No. of Plates:	2	Floor	25.001	40	psf	Floor	14	ft		
	No.:	1	Wall/Misc.	19.001		psf	Wall	5	ft		
	Spacing:	12	in o.c.								
Align Studs w/ trusses:	No										

F'_B	1485	C_D	1.00	C_L	1.00	Allowable Point Load P_{ALLOW} =	1832	lbs.
F'_V	180	C_M	1.00	C_F	1.50	P_{ALLOW} (due to Bending)	1949	lbs.
E'	1600000	C_t	1.00	C_{fu}	1.10	P_{ALLOW} (due to Shear)	1832	lbs.
F_{CPERP}	625	Ci	1.00	C_r	1.00	P_{ALLOW} (due to Crushing)	3281	lbs.
						P_{ACTUAL} =	1915.025639	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.
STUDS	No. of Plates:	2	Floor	25.001	40	psf	Floor	0	ft		
	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	Allowable Point Load P_{ALLOW} =	#N/A	lbs.
F'_V	#N/A	C_M	1.00	C_F	1.50	P_{ALLOW} (due to Bending)	#N/A	lbs.
E'	#N/A	C_t	1.00	C_{fu}	1.10	P_{ALLOW} (due to Shear)	#N/A	lbs.
F_{CPERP}	#N/A	Ci	1.00	C_r	1.00	P_{ALLOW} (due to Crushing)	#N/A	lbs.
						P_{ACTUAL} =	0	lbs

#N/A

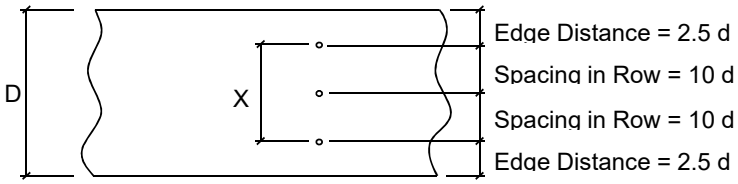
L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-001-192
DATE: November 21, 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 21, 2019

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

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

Ledgers Supporting Roof Loads

Total Allowable Roof Load
DFL Frmg = 70psf SPF Frmg = 50 psf

Supported Span	Member	Nailing	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
DFL Frmg = 89psf SPF Frmg = 70 psf

Supported Span	Member	Nailing	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 11/21/2019

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 11/21/19

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 =	0.486
S_1 =	0.163
F_a =	1.3
F_v =	1.5
S_{MS} =	0.63
S_{M1} =	0.24
S_{DS} =	0.421
S_{D1} =	0.163

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 =	1.00

SDC =	C
SDC =	C

I_E =	1.00
C_{SMAX} =	0.123
C_S =	0.065
C_{SMIN} =	0.010
C_{SMIN} =	0

$C_{SCONTROL}$=	**0.065**
ρ*0.7*$C_{SCONTROL}$=	**0.045**

$E = \rho Q_E + 0.2S_{DS}D$ =	0.065	x W	+	0.084	x D
$E = \rho Q_E - 0.2S_{DS}D$ =	0.065	x W	-	0.084	x D
$0.7E = 0.7(\rho Q_E + 0.2S_{DS}D)$ =	**0.045**	x W	+	**0.059**	x D
$0.7E = 0.7(\rho Q_E - 0.2S_{DS}D)$ =	**0.045**	x W	-	**0.059**	x D

L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-001-192
DATE: 11/21/19

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) 0 to h/2	h/2 to h	h to 2h	> 2h	Internal +/- 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: 11/21/19

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 11/21/19

PROJECT Desert Rose Customs
SUBJECT LATERAL LINE LOADS

SHEET
DESIGNED NES
SECTION
CHECKED KAB

Seismic Line Loads

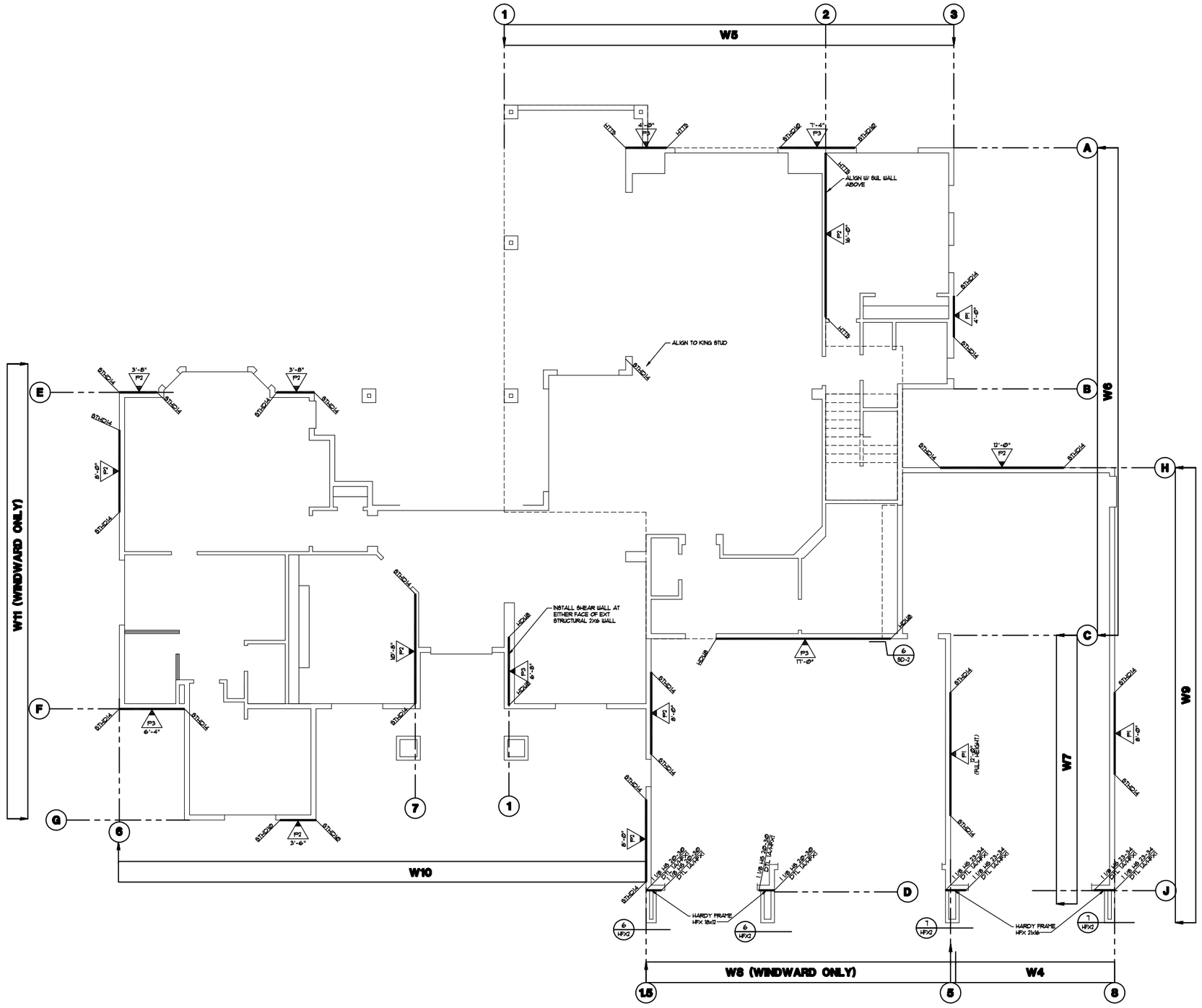
Label	Level	Roof Trib (ft)	Floor Trib (ft)	Wall Trib (ft)	Other (lb/ft)	Total Weight (lb/ft)	Total Force (lb/ft)	Redist Factor	Revised Force (lb/ft)
ω1	ROOF	40		10	200	1270	58	1.560	90
ω2	ROOF	42		10	210	1324	60	1.560	94
ω3						0	0	1.000	0
ω4	ROOF	47		16	235	1573	71	1.560	111
ω5	2nd FLOOR	50		15.5	600	1995	90	0.851	77
ω6	2nd FLOOR	40		15.5	480	1655	75	0.851	64
ω7	2nd FLOOR	34		12	408	1384	63	0.851	53
ω8	2nd FLOOR	35		12	420	1418	64	0.851	55
ω9	ROOF	20		16	100	844	38	1.560	60
ω10	2nd FLOOR	36		16	360	1456	66	0.851	56
ω11	2nd FLOOR	52		16	520	1968	89	0.851	76
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	0.045	* 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 APN# 179-04-405-011

SUBJECT XXXX KEY PLAN

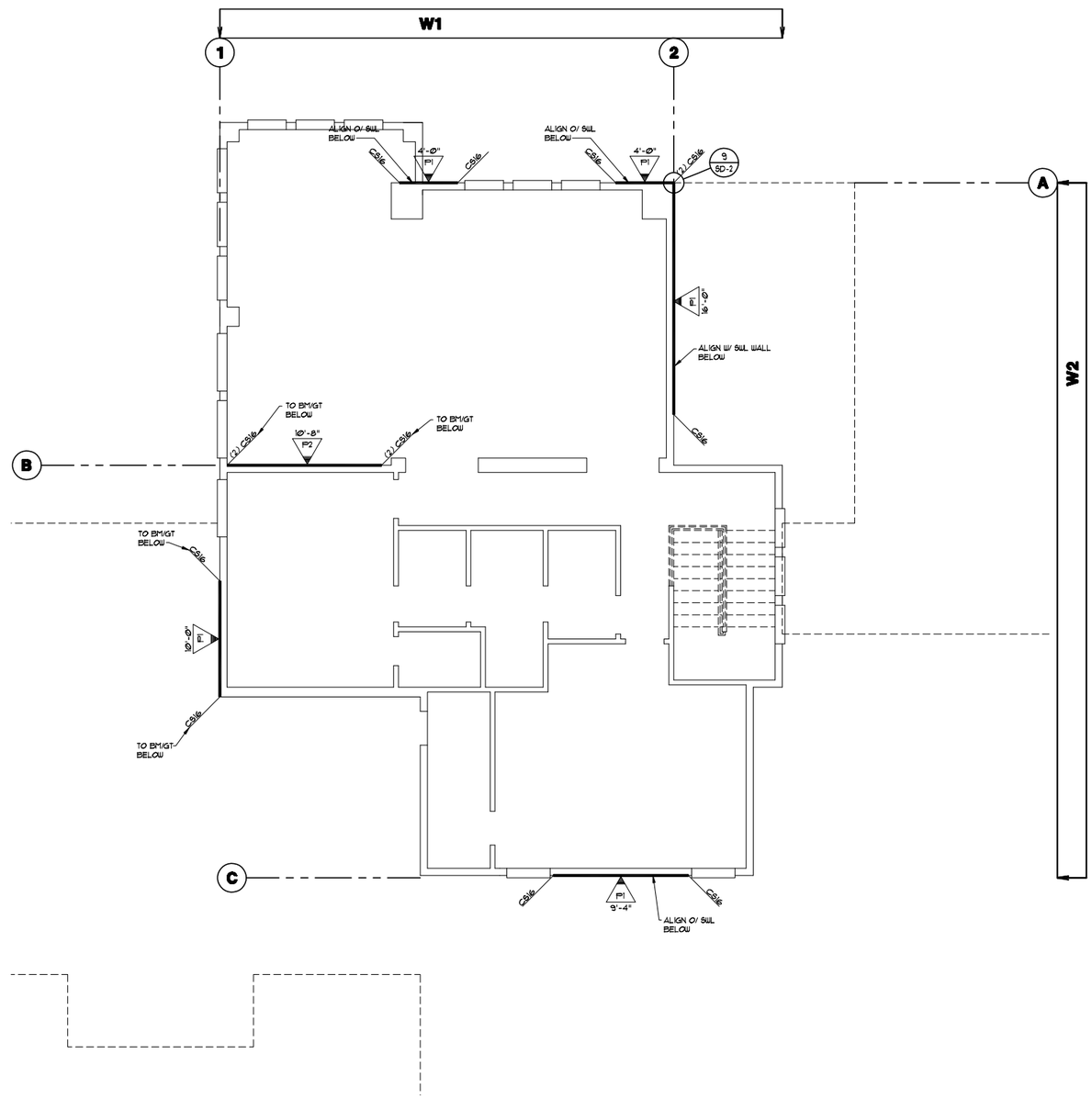
L. R. NELSON CONSULTING ENGINEERS INC

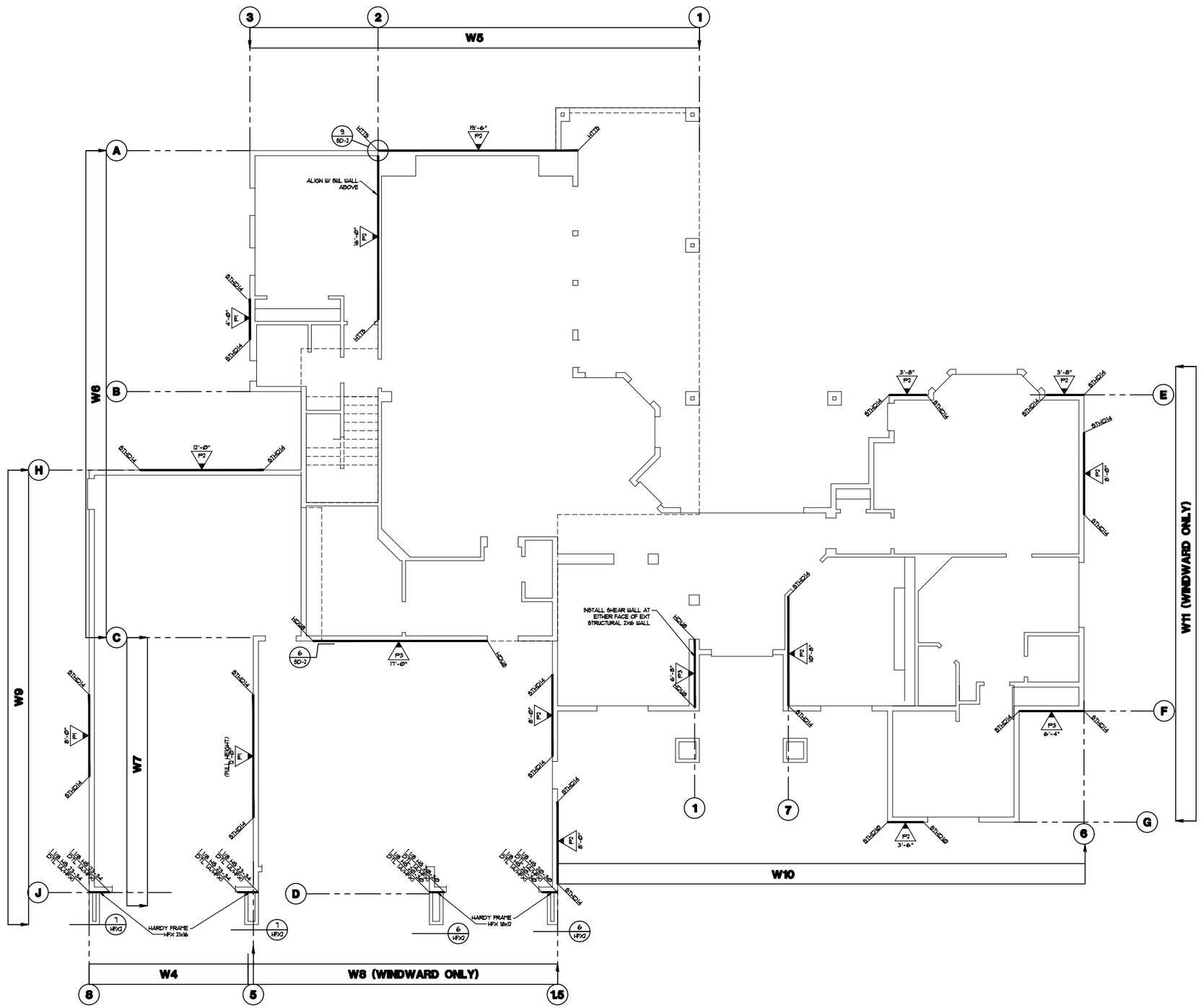
JOB NO. 1939-001-192

DATE 11-11-2019

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

SUBJECT SHEAR WALL KEY PLAN DESIGNED NES CHECKED ____





1ST STORY SHEAR WALL PLAN

1/4"=1'-0"

L. R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-001-192 DATE 11-21-2019

SHEET ____ OF ____

DESIGNED NES

PROJECT DESERT ROSE CUSTOMS

SUBJECT KEY PLAN

L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-001-192
DATE 11/21/19

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) =								
ω1	16.00	89.9	78.0	104.3						-8.58							Wind (lb)	Seis (lb)
		0.0	0.0	0.0						-8.58						above=		
		0.0	0.0	0.0						-8.58						Total=	1669	1438
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	14,383	5,700			**P1**	**CS16**	Full	167	365	144	260	1,099	868	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) =								
ω5	16.00	77.0	117.6	144.1						-8.58							Wind (lb)	Seis (lb)
ω8	15.00	54.7	96.0	76.2						-8.58						above=		
ω10	7.00	56.2	76.8	40.8						-8.58						Total=	3859	2447
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				23,155	14,681	4,378			**P2**	**STHD14 (C)**	Full	241	533	153	380	2,347	1,288	O.K.
8.00				23,155	14,681	4,378			**P2**	**STHD14 (C)**	Corner	241	533	153	380	2,347	1,288	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) =								
ω1	24.00	89.9	78.0	104.3						-8.58							Wind (lb)	Seis (lb)
		0.0	0.0	0.0						-8.58						above=		
		0.0	0.0	0.0						-8.58						Total=	2504	2157
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	21,574	14,593			**P1**	**CS16**	Full	156	365	135	260	653	436	O.K.
											Corner							
											Corner							
									SHARED HD W/ LINE A	**(2) CS16**	Full		#N/A		#N/A	821	1,074	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) =								
ω5	15.00	77.0	117.6	144.1						-8.58							Wind (lb)	Seis (lb)
		0.0	0.0	0.0						-8.58						above=	2504	2157
		0.0	0.0	0.0						-8.58						Total=	4665	3312
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	33,125	14,593	260	653	**P2**	**HTT5**	Full	292	533	207	380	2,263	1,811	O.K.
											Corner							
											Corner							
											Full							
											Corner							
											Corner							

NOTES:

L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-001-192
DATE 11/21/19

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	77.0	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	539

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	5,390	912			P1	STHD14 (C)	Corner	252	292	135	208	2,293	1,120	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	54.7	96.0	76.2						-8.58							Wind (lb)	Seis (lb)
ω4	9.00	111.3	105.6	134.0						-8.58						above=		
		0.0	0.0	0.0						-8.58						Total=	2390	1823

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	21,879	9,850			P1	STHD14 (C)	Corner	199	365	152	260	1,570	1,002	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	56.2	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	843

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	8,432	3,648			P2	STHD14 (C)	Corner	144	533	105	380	984	598	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	56.2	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	1124

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	11,243	6,490			P2	STHD14 (C)	Corner	144	533	105	380	831	445	O.K.
											Corner							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-001-192
DATE 11/21/19

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	56.2	76.8	40.8						-8.58							Wind (lb)	Seis (lb)
		0.0	0.0	0.0						-8.58						above=	1669	1438
		0.0	0.0	0.0						-8.58						Total=	2591	2113
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	21,128	2,536			**P3**	**HDU8**	Corner	388	685	317	490	3,504	2,787	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	93.7	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	937
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	4,686	912			**P1**	**CS16**	Full	79	292	117	208	559	943	O.K.
4.00				3,150	4,686	912			**P1**	**CS16**	Full	79	292	117	208	559	943	O.K.
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: A 1ST STORY Total Length (ft)= 11.33 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) =								
ω6	24.00	63.9	117.6	100.0						-8.58							Wind (lb)	Seis (lb)
		0.0	0.0	0.0						-8.58						above=	1376	2048
		0.0	0.0	0.0						-8.58						Total=	4199	3581
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				14,824	12,641	912	559	943	**P3**	**HTT5**	Full	371	548	316	392	4,037	3,876	O.K.
7.33				27,165	23,165	3,063			**P2**	**STHD10 (C)**	Corner	371	533	316	380	3,288	2,742	O.K.
											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	93.7	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	2249
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	22,492	6,490			**P2**	**(2) CS16**	Full	142	533	211	380	809	1,500	O.K.
											Corner							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-001-192
DATE 11/21/19

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	93.7	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	1406

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	14,057	4,962			P1	CS16	Full	101	365	151	260	481	975	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	63.9	117.6	100.0						-8.58							Wind (lb)	Seis (lb)
ω7	14.00	53.4	92.6	51.5						-8.58						above=	1709	2542
		0.0	0.0	0.0						-8.58						Total=	5828	4823

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	57,879	19,768	481	975	P3	HDU8	Corner	343	685	284	490	3,432	3,217	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	53.4	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	1069

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	6,412	68			HFX18X12-1 1/8 HS		Full	926	2,160	534	2,160	11,048	6,344	
1.00				11,117	6,412	68			HFX18X12-1 1/8 HS		Full	926	2,160	534	2,160	11,048	6,344	
											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	76.0	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	912

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	4,559	768			P2	STHD14 (C)	Corner	173	391	124	279	1,517	1,033	O.K.
3.67				6,336	4,559	768			P2	STHD14 (C)	Corner	173	391	124	279	1,517	1,033	O.K.
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-001-192
DATE 11/21/19

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	76.0	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	1596

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	15,956	2,284			P3	STHD14 (C)	Corner	350	685	252	490	3,142	2,160	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	76.0	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	456

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	4,559	714			P2	STHD10 (C)	Corner	179	377	129	269	1,588	1,086	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	59.7	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	1374

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	21,984	13,133			P2	STHD14 (C)	Corner	172	533	115	380	1,653	738	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	59.7	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	1374

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	10,992	91			HFX-21x16		Corner	1,030	2,295	687	1,850	16,395	10,901	
1.00				16,486	10,992	91			HFX-21x16		Full	1,030	2,295	687	1,850	16,395	10,901	
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-001-192
DATE 11/21/19

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: 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	111.3	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	1002

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	16,033	5,837			**P1**	STHD14 (C)	Corner	151	365	125	260	1,682	1,274	O.K.
											Corner							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-001-192
DATE 11/21/19

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	WIND	38.833333	39.66667	51.1	232.3	15/32 shthg, 8d nails @ 6" o.c.	CASES 2-6
2	ω2	None	SEIS	39.666667	38.83333	47.9	165.6	15/32 shthg, 8d nails @ 6" o.c.	CASES 2-7
3	ω4	None	WIND	16	44.16667	24.3	232.3	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	495.8	2	DFL#2	2x4	0.03	2.625911	12	1.6
2	474.6	2	DFL#2	2x4	0.03	2.513984	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 11/21/2019

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 11/21/2019

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

$P_{max} = F_b 2H^2/(\beta/4)$ = 1.64 kips / ft
Where $\beta = (E_{cr} H^3 / 3 k_s)^{0.25}$ = 23 < St,b = 32
k_s = 100 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

$P_{max} = F_b 2H^2/(\beta/4)$ = 1.38 kips / ft
Where $\beta = (E_{cr} H^3 / 3 k_s)^{0.25}$ = 21 < St,b = 24.5
k_s = 100 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

$P_{max} = F_b 2H^2/(\beta/4)$ = 1.89 kips / ft
Where $\beta = (E_{cr} H^3 / 3 k_s)^{0.25}$ = 26 < St,b = 40.5
k_s = 100 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

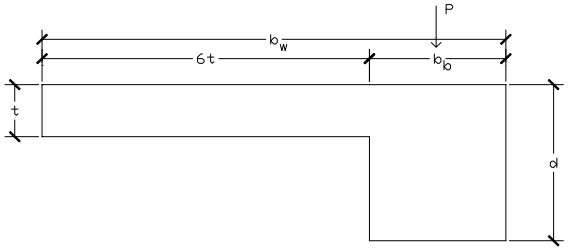
d inches	b_b in	t slab in	b_w in	I in^4	S_b in^3	A in^2
12	12	4	36	2778	365	240

$P_{max} = F_b 4S_b / \beta =$ 9576.94 lbs

Where $\beta = [4E_{cr} I / (bw \times k_s)]^{0.25} =$ 46

$k_s =$ 100 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)

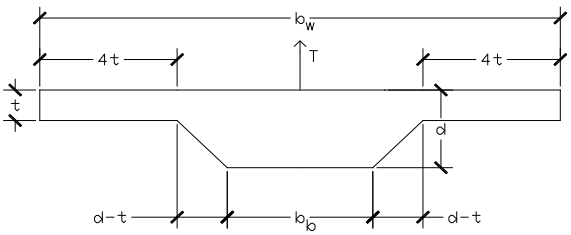
	Beam	Slab
∴ back substitution and solve for M1 =	$wL^2/2$	M2 = $wL^2/2$
	∴ L_1 = 6.33 ft	L_2 ft = 5.75 ft

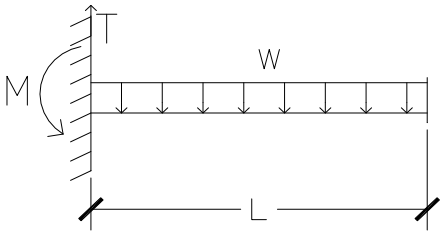
∴ Assume minimum 2 foot shearwall, L` = 8.33 ft

∴ 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

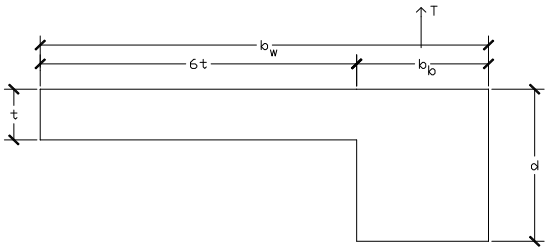
w = 423 lb/ft

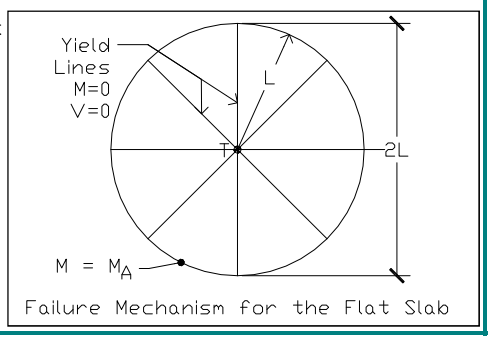
∴ back substitution and solve for M = $wL^2/2$

∴ L_1 = 8.64 ft

∴ Assume minimum 2 foot shearwall, L` = 10.64 ft

∴ T = 4501 lb





L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-001-192
DATE 11/21/19

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

FOOTINGS

Assumed Soil Bearing Pressure

q=	2,500	psf not to exceed ------	2,500	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	2,350	2
18	12	5,288	2
24	12	9,400	3
30	12	14,688	4
36	12	21,150	4
42	12	28,788	5
48	12	37,600	5
54	16	46,575	8
60	16	57,500	9

Continuous Footings:

Width (in)	Loads (plf)	#4 Long. Bars	#4 Trans. Bars
12	2350	1	N.A.
18	3525	2	12 in. o.c.
24	4700	2	9 in. o.c.
30	5875	3	9 in. o.c.
36	7050	3	9 in. o.c.
42	8225	4	9 in. o.c.
48	9400	4	9 in. o.c.
54	10350	6	7 in. o.c.
60	11500	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	3760	0.4	150	OK
18	3	3.5	8.50	399.5	8460	12.2	150	OK
24	3	3.5	8.50	399.5	15040	28.6	150	OK
30	3	3.5	8.50	399.5	23500	49.8	150	OK
36	3	3.5	8.50	399.5	33840	75.7	150	OK
42	3	3.5	8.50	399.5	46060	106.3	150	OK
48	3	3.5	8.50	399.5	60160	141.6	150	OK
54	3	3.5	12.50	787.5	74520	86.6	150	OK
60	3	3.5	12.50	787.5	92000	108.8	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	3760	26.1	0.0	75	OK
18	8.5	3	0	8460	26.1	0.0	75	OK
24	8.5	3	2	15040	26.1	6.1	75	OK
30	8.5	3	5	23500	26.1	15.4	75	OK
36	8.5	3	8	33840	26.1	24.6	75	OK
42	8.5	3	11	46060	26.1	33.8	75	OK
48	8.5	3	14	60160	26.1	43.0	75	OK
54	12.5	3	13	74520	25.6	26.6	75	OK
60	12.5	3	16	92000	25.6	32.7	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	26.1	3173	0.01	0.20	2	2
18	8.5	3	26.1	13219	0.03	0.30	2	2
24	8.5	3	26.1	34545	0.08	0.52	3	2
30	8.5	3	26.1	71381	0.17	0.65	4	2
36	8.5	3	26.1	127958	0.30	0.78	4	3
42	8.5	3	26.1	208504	0.49	0.91	5	3
48	8.5	3	26.1	317250	0.75	1.04	5	4
54	12.5	3	25.6	448673	0.71	1.56	8	5
60	12.5	3	25.6	622725	1.00	1.73	9	6

(Minimum reinforcement per ACI 318, Section 10.5)