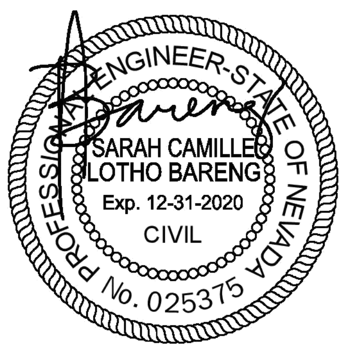
PROJECT NUMBER: **1939-005-201**

DATE: **June 26, 2020**

STRUCTURAL CALCULATIONS
for
Milan Lot 6
at

Henderson, NV
Design is per the 2018 IBC
for
Assured Real Estate
June 26, 2020
Project #: 1939-005-201



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: 2020-07-07 10:05:19

7/7/2020

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. 1939-005-201
DATE 6/26/2020

PROJECT: **Milan Lot 6** **SHEET** ______ **OF** ______
SUBJECT: **DESIGN CRITERIA** **DESIGNED** **BP** **CHECKED** **SLB**

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 =	52%	S1 =	17%	R =	6.5
Equivalent Lateral Force	Sds =	45%	Sd1 =	17%	Ie=	1.00
Seismic base shear, E = Cs x W			Site Class =	C	Cs =	0.07

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 ______ 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. 1939-005-201
DATE 6/26/2020

PROJECT: **Milan Lot 6** SHEET OF
SUBJECT: **BUILDING LOADS** DESIGNED **BP** CHECKED **SLB**

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 - CARPET, VINYL, ET	2.00
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	15.00
LL	40.00

ALTERNATE LOAD #2

Load Type

M,E & MISC.

DL 0.00

Description of Load

EXTERIOR WALLS

1 COAT STUCCO	3.50
2X4 @ 16" OC FRAMING	1.41
WALL INSULATION	1.00
15/32" SHEATHING	1.60
1/2" GYPSUM	2.20
MISC.	2.29
DL	12.00

OVERFILL (WHERE OCCURS)

15/32" SHEATHING	1.60
OVERFILL RAFTERS	3.40
DL	5.00

ALT. FLOOR LIVE LOAD

LL 60.00

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

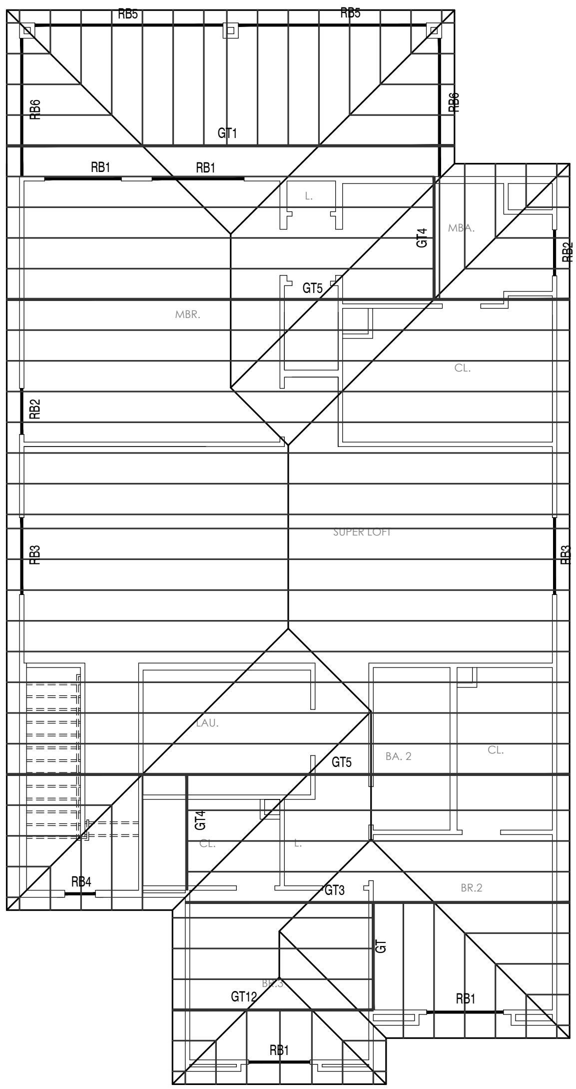
L. R. NELSON CONSULTING ENGINEERS INC.

JOB NO. 1939-005-201

DATE 06-09-20

PROJECT MILAN LOT 6 SHEET ____ OF ____

SUBJECT FRAMING KEY PLAN DESIGNED ____ CHECKED ____



ROOF FRAMING PLAN

1/4"=1'-0"

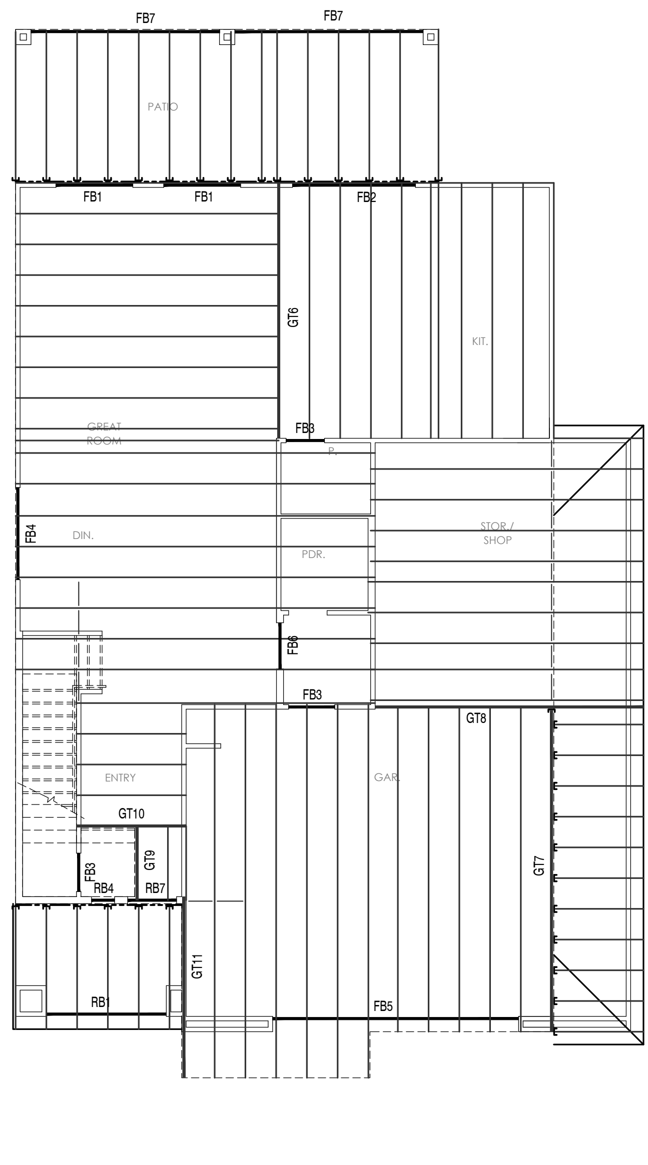
L. R. NELSON CONSULTING ENGINEERS INC.

JOB NO. 1939-005-201

DATE 06-09-20

PROJECT MILAN LOT 6 SHEET OF

SUBJECT FRAMING KEY PLAN DESIGNED CHECKED



FLOOR FRAMING PLAN

1/4"=1'-0"

L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-005-201
DATE 6/26/2020

PROJECT: **Milan Lot 6** **SHEET** ____ **OF** ____
SUBJECT: **Beams & Headers** **DESIGNED** **BP** **CHECKED** **SLB**

DESCRIPTION:	GT1	Roof Tributary =	5.00	ft	W_{DL}=	134.00	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 =	2.00	ft	P_{DL} =	0.00	lb	l_u =	27.25		
LENGTH (ft)=	27.25	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 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 =	3,188	lb	Post:	2X4			DFLSTUD	**O.K.**	lb		2.0	2.0	ft
Right Reaction =	3,188	lb	Post:	2X4			DFLSTUD	**O.K.**	lb		2.0	2.0	ft

DESCRIPTION:	GT2	Roof Tributary =	7.00	ft	W_{DL}=	154.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	#N/A
SIZE:		Floor Tributary =	0.00	ft	W_{LL}=	140.00	lb/ft	C_t =	1.00	C_F =	#N/A
GRADE:		Wall Tributary =	0.00	ft	P_{DL} =	0.00	lb	l_u =	8.00		
LENGTH (ft)=	8.00	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 =	#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 =	1,953	lb	Post:	2X4			DFLSTUD	**O.K.**	777 lb	GT12	9.0	2.0	ft
Right Reaction =	1,176	lb	Post:	HGR			DFLSTUD	-	lb		9.0	2.0	ft

DESCRIPTION:	GT3	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	l_u =	24.50		
LENGTH (ft)=	24.50	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%	
NOTES:	SEE ADD 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 =	2,104	lb	Post:	(2)2X4			DFLSTUD	**O.K.**	2,103.94 lb		9.0	2.0	ft
Right Reaction =	3,078	lb	Post:	(2)2X4			DFLSTUD	**O.K.**	3,077.81 lb		9.0	2.0	ft

DESCRIPTION:	GT4	Roof Tributary =	14.50	ft	W_{DL}=	319.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	#N/A
SIZE:		Floor Tributary =	0.00	ft	W_{LL}=	290.00	lb/ft	C_t =	1.00	C_F =	#N/A
GRADE:		Wall Tributary =	0.00	ft	P_{DL} =	0.00	lb	l_u =	8.00		
LENGTH (ft)=	8.00	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 =	#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 =	2,436	lb	Post:	(2)2X4			DFLSTUD	**O.K.**	lb		9.0	2.0	ft
Right Reaction =	2,436	lb	Post:	HGR			DFLSTUD	-	lb		8.0	2.0	ft

DESCRIPTION:	GT5	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	l_u =	35.00		
LENGTH (ft)=	35.00	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%	
NOTES:	SEE ADD CALC				$d_{Point\ Load}$ =	0.50	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 =	2,142	lb	Post:	(2)2X4			DFLSTUD	**O.K.**	2,142.00 lb		9.0	2.0	ft
Right Reaction =	4,242	lb	Post:	(3)2X6			DFLSTUD	**O.K.**	4,242.00 lb		20.0	2.0	ft

L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO.	1939-005-201
DATE	6/26/2020

PROJECT:	Milan Lot 6	SHEET		OF	
SUBJECT:	Beams & Headers	DESIGNED	BP	CHECKED	SLB

DESCRIPTION:	GT-6	Roof Tributary =	0.00	ft	W_{DL}=	142.51	lb/ft	C_D =	1.00	$C_{V/L}$ =	0.45
SIZE:	3-1/8X24	Floor Tributary =	9.50	ft	W_{LL}=	380.00	lb/ft	C_t =	1.00	C_F =	1.00
GRADE:	24F-V4	Wall Tributary =	0.00	ft	P_{DL} =	0.00	lb	l_u =	16.87		
LENGTH (ft)=	16.87	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%	
NOTES:					$d_{Point Load}$ =	0.00	ft	Reduction (L)		%	
					Source:						

	Actual			Req'd							L/xxx		Alternate Loads		
S =	300.00	in^3	>	205.60	in^3	O.K.	Max LL Deflection =	0.11 in =	L / 1894		360	#1	#2	LL	
A =	75.00	in^2	>	21.01	in^2	O.K.	Max TL Deflection =	0.15 in =	L / 1378		240				
I =	3600.00	in^4	>	684.16	in^4	O.K.			Addt'l Load to Post		Source		Lx	Ly	
Left Reaction =	4,407	lb	Post:	(3)2X4			DFLSTUD	O.K.		lb			9.0	2.0	ft
Right Reaction =	4,407	lb	Post:	(3)2X4			DFLSTUD	O.K.		lb			9.0	9.0	ft

DESCRIPTION:	GT7	Roof Tributary =	0.00	ft	W_{DL}=	0.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	0.98
SIZE:	3-1/8X24	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	l_u =	2.00		
LENGTH (ft)=	21.50	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%	
NOTES:	SEE ADD CALC				$d_{Point Load}$ =	0.00	ft	Reduction (L)		%	
					Source:						

	Actual			Req'd							L/xxx		Alternate		
S =	300.00	in^3	>	0.00	in^3	O.K.	Max LL Deflection =	0.00 in =	N/A		360	#1	#2	LL	
A =	75.00	in^2	>	0.00	in^2	O.K.	Max TL Deflection =	0.00 in =	N/A		240				
I =	3600.00	in^4	>	0.00	in^4	O.K.			Addt'l Load to Post		Source		Lx	Ly	
Left Reaction =	11,838	lb	Post:	4X10			DFL#2	O.K.	11,837.81	lb			9.0	9.0	ft
Right Reaction =	9,794	lb	Post:	HGR			DFLSTUD	-	9,793.63	lb			8.0	2.0	ft

DESCRIPTION:	GT8	Roof Tributary =	0.00	ft	W_{DL}=	0.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	0.96
SIZE:	5-1/8X24	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	l_u =	6.00		
LENGTH (ft)=	16.50	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%	
NOTES:	SEE ADD CALC				$d_{Point Load}$ =	0.00	ft	Reduction (L)		%	
					Source:						

	Actual			Req'd							L/xxx		Alternate		
S =	492.00	in^3	>	0.00	in^3	O.K.	Max LL Deflection =	0.00 in =	N/A		360	#1	#2	LL	
A =	123.00	in^2	>	0.00	in^2	O.K.	Max TL Deflection =	0.00 in =	N/A		240				
I =	5904.00	in^4	>	0.00	in^4	O.K.			Addt'l Load to Post		Source		Lx	Ly	
Left Reaction =	9,562	lb	Post:	4X8			DFL#2	O.K.	9,562.38	lb			9.0	9.0	ft
Right Reaction =	7,197	lb	Post:	4X6			DFL#2	O.K.	7,196.75	lb			9.0	9.0	ft

DESCRIPTION:	GT9	Roof Tributary =	0.00	ft	W_{DL}=	15.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	#N/A
SIZE:		Floor Tributary =	1.00	ft	W_{LL}=	40.00	lb/ft	C_t =	1.00	C_F =	#N/A
GRADE:		Wall Tributary =	0.00	ft	P_{DL} =	262.50	lb	l_u =	5.00		
LENGTH (ft)=	5.00	Additional Load =	0.00	plf	P_{LL} =	700.00	lb	Reduction (L_r)		%	
NOTES:					$d_{Point Load}$ =	2.50	ft	Reduction (L)		%	
					Source:	STRINGER					

	Actual			Req'd							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 =	619	lb	Post:	2X4			DFLSTUD	O.K.		lb			3.0	3.0	ft
Right Reaction =	619	lb	Post:	HGR			DFLSTUD	-		lb			8.0	2.0	ft

DESCRIPTION:	GT10	Roof Tributary =	0.00	ft	W_{DL}=	15.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	#N/A
SIZE:		Floor Tributary =	1.00	ft	W_{LL}=	40.00	lb/ft	C_t =	1.00	C_F =	#N/A
GRADE:		Wall Tributary =	0.00	ft	P_{DL} =	168.75	lb	l_u =	6.50		
LENGTH (ft)=	6.50	Additional Load =	0.00	plf	P_{LL} =	450.00	lb	Reduction (L_r)		%	
NOTES:					$d_{Point Load}$ =	3.00	ft	Reduction (L)		%	
					Source:	GT9					

	Actual			Req'd							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 =	512	lb	Post:	2X4			DFLSTUD	O.K.		lb			9.0	2.0	ft
Right Reaction =	464	lb	Post:	2X4			DFLSTUD	O.K.		lb			9.0	2.0	ft

L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-005-201
DATE

PROJECT Milan Lot 6 **SHEET** **OF**
SUBJECT GT8 **DESIGNED** BP **CHECKED** SLB

Beveled End Member Evaluation (for sloped cuts on the TOP side of a member)

This routine is used as a design aid to evaluate the affect of end cuts in solid-sawn lumber or glue-laminated beams within the rafter slope envelope at bearing supports.

Member Type & Size being evaluated: 5.125" x 24" (24F-V4) GLU-LAM Beam

Max. Moment	45549.8	FT-LBS	
Beam Span	16.50	FT	
Uniform Load	0.0	PLF	Equivalent Uniform Load ____ PLF
Calculated shear (if not uniformly loaded)	9562	LBS	
Beam Reaction	9562	LBS	Duration of Load Factor = 1.25
Roof slope ?/12	4	Inches	
Bearing length (bl)	3.5	Inches	
End cut height (do)	9.0	Inches	
Beam depth (d)	24	Inches	
Beam width (b)	5.125	Inches	
slope angle =	18.43	degree	
depth at support face (de) =	**10.17**	Inches	**Okay for notch depth**
min. depth at support =	8.00	Inches	
e = beveled length - bearing length =	41.50	Inches	
Maximum shear stress =	275	PSI	
Allowable shear stress =	300	PSI	**Okay for Shear**
Minimum bearing length =	4.15	Inches	
Overall Length of Taper Cut =	45		
Allowable Taper Cut Length =	72		**Okay for taper cut length**

```
---------------------\
                     \  -------------------------------------------
                      \           depth at sup. (de)
depth (d)              \   -------------------------
                        |       end cut (do)
------------------------------    ------------------------------------
                     |   |
                     Bearing
                     length  (bl)
```

Beveled Cut for 5.125 x 24 GLB is Acceptable (for parameters listed hereon)

Macro Library

\p /ppra1..h46~agpq

L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-005-201
DATE 6/26/2020

PROJECT: **Milan Lot 6** **SHEET** ______ **OF** ______
SUBJECT: **Beams & Headers** **DESIGNED** **BP** **CHECKED** **SLB**

DESCRIPTION:	GT11	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	l_u =	11.37		
LENGTH (ft)=	11.37	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%	
NOTES:	SEE ADD CALC ENERCALC				$d_{Point\ Load}$ =	0.00	ft	Reduction (L)		%	

	Actual			Req'd						L/xxx	Alternate Loads	
							Source:					
S =	#N/A	in³	#N/A	#N/A	in³	**#N/A**	Max LL Deflection =	#VALUE!	#VALUE!	360	#1	#2	LL
A =	#N/A	in²	#N/A	#N/A	in²	**#N/A**	Max TL Deflection =	#VALUE!	#VALUE!	240			
I =	#N/A	in⁴	#N/A	#VALUE!	in⁴	**#N/A**		Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	3,350	lb	Post:	(2)2X4	DFLSTUD	**O.K.**	3350	lb			9.0	2.0	ft
Right Reaction =	3,082	lb	Post:	(2)2X4	DFLSTUD	**O.K.**	3082	lb			9.0	2.0	ft

DESCRIPTION:	GT12	Roof Tributary =	3.00	ft	W_{DL}=	66.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	#N/A
SIZE:		Floor Tributary =	0.00	ft	W_{LL}=	60.00	lb/ft	C_t =	1.00	C_F =	#N/A
GRADE:		Wall Tributary =	0.00	ft	P_{DL} =	0.00	lb	l_u =	12.33		
LENGTH (ft)=	12.33	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						L/xxx	Alternate		
							Source:						
S =	#N/A	in³	#N/A	#N/A	in³	**#N/A**	Max LL Deflection =	#VALUE!	#VALUE!	360	#1	#2	LL
A =	#N/A	in²	#N/A	#N/A	in²	**#N/A**	Max TL Deflection =	#VALUE!	#VALUE!	240			
I =	#N/A	in⁴	#N/A	#VALUE!	in⁴	**#N/A**		Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	777	lb	Post:	2X4	DFLSTUD	**O.K.**		lb			9.0	2.0	ft
Right Reaction =	777	lb	Post:	2X4	DFLSTUD	**O.K.**		lb			9.0	2.0	ft

DESCRIPTION:		Roof Tributary =	0.00	ft	W_{DL}=	0.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	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	l_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						L/xxx	Alternate		
							Source:						
S =	17.65	in³	>	0.00	in³	**O.K.**	Max LL Deflection =	0.00 in =	N/A	360	#1	#2	LL
A =	19.25	in²	>	0.00	in²	**O.K.**	Max TL Deflection =	0.00 in =	N/A	240			
I =	48.53	in⁴	>	0.00	in⁴	**O.K.**		Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	0	lb	Post:	2X4	DFLSTUD	**O.K.**		lb			8.0	2.0	ft
Right Reaction =	0	lb	Post:	2X4	DFLSTUD	**O.K.**		lb			8.0	2.0	ft

DESCRIPTION:		Roof Tributary =	0.00	ft	W_{DL}=	0.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	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	l_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						L/xxx	Alternate		
							Source:						
S =	17.65	in³	>	0.00	in³	**O.K.**	Max LL Deflection =	0.00 in =	N/A	360	#1	#2	LL
A =	19.25	in²	>	0.00	in²	**O.K.**	Max TL Deflection =	0.00 in =	N/A	240			
I =	48.53	in⁴	>	0.00	in⁴	**O.K.**		Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	0	lb	Post:	2X4	DFLSTUD	**O.K.**		lb			8.0	2.0	ft
Right Reaction =	0	lb	Post:	2X4	DFLSTUD	**O.K.**		lb			8.0	2.0	ft

DESCRIPTION:		Roof Tributary =	0.00	ft	W_{DL}=	0.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	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	l_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						L/xxx	Alternate		
							Source:						
S =	17.65	in³	>	0.00	in³	**O.K.**	Max LL Deflection =	0.00 in =	N/A	360	#1	#2	LL
A =	19.25	in²	>	0.00	in²	**O.K.**	Max TL Deflection =	0.00 in =	N/A	240			
I =	48.53	in⁴	>	0.00	in⁴	**O.K.**		Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	0	lb	Post:	2X4	DFLSTUD	**O.K.**		lb			8.0	2.0	ft
Right Reaction =	0	lb	Post:	2X4	DFLSTUD	**O.K.**		lb			8.0	2.0	ft

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

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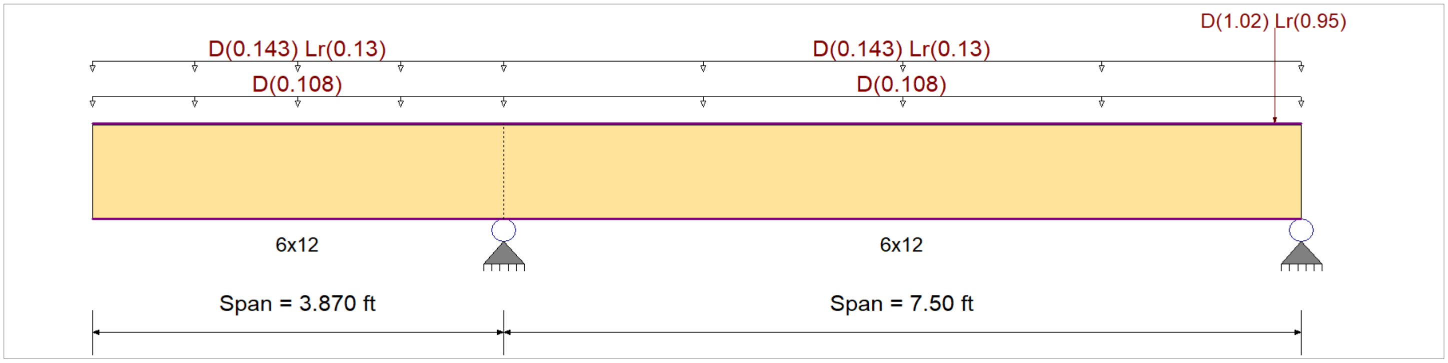
DESCRIPTION: GT11 FOR REACTION ONLY

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	Fb +	2,400.0 psi	*E : Modulus of Elasticity*		
Load Combination :	ASCE 7-16	Fb -	1,850.0 psi	Ebend- xx	1,800.0 ksi	
		Fc - Prll	1,650.0 psi	Eminbend - xx	950.0 ksi	
Wood Species	: DF/DF	Fc - Perp	650.0 psi	Ebend- yy	1,600.0 ksi	
Wood Grade	: 24F-V4	Fv	265.0 psi	Eminbend - yy	850.0 ksi	
		Ft	1,100.0 psi	Density	31.210 pcf	
Beam Bracing	: Beam is Fully Braced against lateral-torsional buckling					



Applied Loads

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

Load for Span Number 1
Uniform Load : D = 0.0120 ksf, Tributary Width = 9.0 ft, (WALL)
Uniform Load : D = 0.0220, Lr = 0.020 ksf, Tributary Width = 6.50 ft, (ROOF LOAD)
Load for Span Number 2
Uniform Load : D = 0.0120 ksf, Tributary Width = 9.0 ft, (WALL)
Uniform Load : D = 0.0220, Lr = 0.020 ksf, Tributary Width = 6.50 ft, (ROOF)
Point Load : D = 1.020, Lr = 0.950 k @ 7.250 ft, (GT3)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.122** : 1	Maximum Shear Stress Ratio	=	**0.109** : 1
Section used for this span		**6x12**	Section used for this span		**6x12**
	=	282.42 psi		=	36.13 psi
	=	2,312.50 psi		=	331.25 psi
Load Combination		+D+Lr+H, LL Comb Run (LL)	Load Combination		+D+Lr+H, LL Comb Run (LL)
Location of maximum on span	=	3.870 ft	Location of maximum on span	=	3.870 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.018 in	Ratio =	5156 >=360
Max Upward Transient Deflection	-0.014 in	Ratio =	6750 >=360
Max Downward Total Deflection	0.028 in	Ratio =	3368 >=240
Max Upward Total Deflection	-0.004 in	Ratio =	22116 >=240

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr+H, LL Comb Run (L*)	1	0.0276	0.000	Lr Only, LL Comb Run (*L)	-0.0003	3.784
+D+Lr+H, LL Comb Run (*L)	2	0.0152	4.148		0.0000	3.784

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum		3.349	3.082
Overall MINimum		1.152	1.276
+D+H		2.197	1.677
+D+L+H, LL Comb Run (*L)		2.197	1.677

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

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

DESCRIPTION: GT11 FOR REACTION ONLY

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
+D+L+H, LL Comb Run (L*)		2.197	1.677
+D+L+H, LL Comb Run (LL)		2.197	1.677
+D+Lr+H, LL Comb Run (*L)		2.716	3.082
+D+Lr+H, LL Comb Run (L*)		2.830	1.547
+D+Lr+H, LL Comb Run (LL)		3.349	2.953
+D+0.750Lr+0.750L+H, LL Comb Run (*		2.587	2.731
+D+0.750Lr+0.750L+H, LL Comb Run (L		2.672	1.579
+D+0.750Lr+0.750L+H, LL Comb Run (L		3.061	2.634
+D+0.750L+0.750S+H, LL Comb Run (*L		2.197	1.677
+D+0.750L+0.750S+H, LL Comb Run (L*		2.197	1.677
+D+0.750L+0.750S+H, LL Comb Run (LL		2.197	1.677
+D+0.750Lr+0.750L+0.450W+H, LL Comb		2.587	2.731
+D+0.750Lr+0.750L+0.450W+H, LL Comb		2.672	1.579
+D+0.750Lr+0.750L+0.450W+H, LL Comb		3.061	2.634
+D+0.750L+0.750S+0.450W+H, LL Comb		2.197	1.677
+D+0.750L+0.750S+0.450W+H, LL Comb		2.197	1.677
+D+0.750L+0.750S+0.450W+H, LL Comb		2.197	1.677
+D+0.750L+0.750S+0.5250E+H, LL Comb		2.197	1.677
+D+0.750L+0.750S+0.5250E+H, LL Comb		2.197	1.677
+D+0.750L+0.750S+0.5250E+H, LL Comb		2.197	1.677
D Only		2.197	1.677
Lr Only, LL Comb Run (*L)		0.519	1.406
Lr Only, LL Comb Run (L*)		0.633	-0.130
Lr Only, LL Comb Run (LL)		1.152	1.276
L Only, LL Comb Run (*L)			
L Only, LL Comb Run (L*)			
L Only, LL Comb Run (LL)			
W Only			

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JOB NO. 1939-005-201
DATE 6/26/2020

PROJECT: **Milan Lot 6** **SHEET** ______ **OF** ______
SUBJECT: **Beams & Headers** **DESIGNED** **BP** **CHECKED** **SLB**

DESCRIPTION:	RB1	Roof Tributary =	5.00	ft	W_{DL}=	158.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	1.00
SIZE:	4X6	Floor Tributary =	0.00	ft	W_{LL}=	100.00	lb/ft	C_t =	1.00	C_F =	1.30
GRADE:	DFL#2	Wall Tributary =	4.00	ft	P_{DL} =	0.00	lb	l_u =	8.00		
LENGTH (ft)=	8.00	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 =	17.65	in^3	>	16.94	in^3	O.K.	Max LL Deflection =	0.12 in =	L / 809	360	#1	#2	LL	
A =	19.25	in^2	>	6.09	in^2	O.K.	Max TL Deflection =	0.31 in =	L / 313	240				
I =	48.53	in^4	>	37.15	in^4	O.K.		Addt'l Load to Post		Source		Lx	Ly	
Left Reaction =	1,032	lb	Post:	2X4	DFLSTUD	O.K.			lb			8.0	2.0	ft
Right Reaction =	1,032	lb	Post:	2X4	DFLSTUD	O.K.			lb			8.0	2.0	ft

DESCRIPTION:	RB2	Roof Tributary =	18.33	ft	W_{DL}=	451.26	lb/ft	C_D =	1.25	$C_{V/L}$ =	1.00
SIZE:	4X6	Floor Tributary =	0.00	ft	W_{LL}=	366.60	lb/ft	C_t =	1.00	C_F =	1.30
GRADE:	DFL#2	Wall Tributary =	4.00	ft	P_{DL} =	0.00	lb	l_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 =	17.65	in^3	>	16.99	in^3	O.K.	Max LL Deflection =	0.04 in =	L / 1240	360	#1	#2	LL	
A =	19.25	in^2	>	9.77	in^2	O.K.	Max TL Deflection =	0.10 in =	L / 556	240				
I =	48.53	in^4	>	20.96	in^4	O.K.		Addt'l Load to Post		Source		Lx	Ly	
Left Reaction =	1,840	lb	Post:	2X4	DFLSTUD	O.K.			lb			8.0	2.0	ft
Right Reaction =	1,840	lb	Post:	2X4	DFLSTUD	O.K.			lb			8.0	2.0	ft

DESCRIPTION:	RB3	Roof Tributary =	18.33	ft	W_{DL}=	451.26	lb/ft	C_D =	1.25	$C_{V/L}$ =	1.00
SIZE:	4X8	Floor Tributary =	0.00	ft	W_{LL}=	366.60	lb/ft	C_t =	1.00	C_F =	1.30
GRADE:	DFL#2	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 =	30.66	in^3	>	25.37	in^3	O.K.	Max LL Deflection =	0.04 in =	L / 1555	360	#1	#2	LL	
A =	25.38	in^2	>	11.70	in^2	O.K.	Max TL Deflection =	0.09 in =	L / 697	240				
I =	111.15	in^4	>	38.27	in^4	O.K.		Addt'l Load to Post		Source		Lx	Ly	
Left Reaction =	2,249	lb	Post:	2X4	DFLSTUD	O.K.			lb			8.0	2.0	ft
Right Reaction =	2,249	lb	Post:	2X4	DFLSTUD	O.K.			lb			8.0	2.0	ft

DESCRIPTION:	RB4	Roof Tributary =	5.00	ft	W_{DL}=	158.00	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 =	4.00	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:					$d_{Point\ Load}$ =	0.50	ft	Reduction (L)		%	

	Actual			Req'd				Source:		L/xxx		Alternate		
S =	27.73	in^3	>	9.69	in^3	O.K.	Max LL Deflection =	0.03 in =	L / 2369	360	#1	#2	LL	
A =	30.25	in^2	>	5.08	in^2	O.K.	Max TL Deflection =	0.08 in =	L / 918	240				
I =	76.26	in^4	>	19.93	in^4	O.K.		Addt'l Load to Post		Source		Lx	Ly	
Left Reaction =	839	lb	Post:	2X6	DFLSTUD	O.K.			lb			8.0	2.0	ft
Right Reaction =	839	lb	Post:	2X6	DFLSTUD	O.K.			lb			8.0	2.0	ft

DESCRIPTION:	RB5	Roof Tributary =	4.50	ft	W_{DL}=	111.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	0.97
SIZE:	4X10	Floor Tributary =	0.00	ft	W_{LL}=	90.00	lb/ft	C_t =	1.00	C_F =	1.20
GRADE:	DFL#2	Wall Tributary =	1.00	ft	P_{DL} =	0.00	lb	l_u =	12.50		
LENGTH (ft)=	12.50	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%	
NOTES:					$d_{Point\ Load}$ =	0.50	ft	Reduction (L)		%	

	Actual			Req'd				Source:		L/xxx		Alternate		
S =	49.91	in^3	>	36.00	in^3	O.K.	Max LL Deflection =	0.13 in =	L / 1121	360	#1	#2	LL	
A =	32.38	in^2	>	7.34	in^2	O.K.	Max TL Deflection =	0.30 in =	L / 502	240				
I =	230.84	in^4	>	110.41	in^4	O.K.		Addt'l Load to Post		Source		Lx	Ly	
Left Reaction =	1,256	lb	Post:	2X4	DFL#2	O.K.			lb			9.0	2.0	ft
Right Reaction =	1,256	lb	Post:	2X4	DFL#2	O.K.			lb			9.0	2.0	ft

L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-005-201
DATE 6/26/2020

PROJECT: **Milan Lot 6** **SHEET** ____ **OF** ____
SUBJECT: **Beams & Headers** **DESIGNED** **BP** **CHECKED** **SLB**

DESCRIPTION:	RB6	Roof Tributary =	0.00	ft	W_{DL}=	0.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	0.93
SIZE:	3-1/8X10-1/2	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	l_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:	0		L/xxx	Alternate Loads		
S =	57.42 in^3	>	0.00 in^3	O.K.	Max LL Deflection =	0.00 in =	N/A	360	#1	#2	LL
A =	32.81 in^2	>	0.00 in^2	O.K.	Max TL Deflection =	0.00 in =	N/A	240			
I =	301.46 in^4	>	0.00 in^4	O.K.		Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	3,805 lb	Post:	2X6	DFLSTUD	O.K.	3,804.96	lb		9.0	2.0	ft
Right Reaction =	1,263 lb	Post:	2X4	DFLSTUD	O.K.	1,262.81	lb		9.0	2.0	ft

DESCRIPTION:	RB7	Roof Tributary =	0.00	ft	W_{DL}=	0.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	1.00
SIZE:	6X6	Floor Tributary =	0.00	ft	W_{LL}=	0.00	lb/ft	C_t =	1.00	C_F =	1.00
GRADE:	DFL#1 (TIMBERS)	Wall Tributary =	0.00	ft	P_{DL} =	168.75	lb	l_u =	3.50		
LENGTH (ft)=	3.50	Additional Load =	0.00	plf	P_{LL} =	450.00	lb	Reduction (L_r)		%	
NOTES:					$d_{Point\ Load}$ =	1.00	ft	Reduction (L)		%	
	Actual		Req'd		Source:	GT9		L/xxx	Alternate		
S =	27.73 in^3	>	3.12 in^3	O.K.	Max LL Deflection =	0.00 in =	L / 9529	360	#1	#2	LL
A =	30.25 in^2	>	3.12 in^2	O.K.	Max TL Deflection =	0.01 in =	L / 6930	240			
I =	76.26 in^4	>	2.88 in^4	O.K.		Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	442 lb	Post:	2X4	DFLSTUD	O.K.		lb		8.0	2.0	ft
Right Reaction =	177 lb	Post:	2X4	DFLSTUD	O.K.		lb		8.0	2.0	ft

DESCRIPTION:		Roof Tributary =	0.00	ft	W_{DL}=	0.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	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	l_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:	2X4	DFLSTUD	O.K.		lb		8.0	2.0	ft
Right Reaction =	0 lb	Post:	2X4	DFLSTUD	O.K.		lb		8.0	2.0	ft

DESCRIPTION:		Roof Tributary =	0.00	ft	W_{DL}=	0.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	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	l_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:	2X4	DFLSTUD	O.K.		lb		8.0	2.0	ft
Right Reaction =	0 lb	Post:	2X4	DFLSTUD	O.K.		lb		8.0	2.0	ft

DESCRIPTION:		Roof Tributary =	0.00	ft	W_{DL}=	0.00	lb/ft	C_D =	1.25	$C_{V/L}$ =	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	l_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:	2X4	DFLSTUD	O.K.		lb		8.0	2.0	ft
Right Reaction =	0 lb	Post:	2X4	DFLSTUD	O.K.		lb		8.0	2.0	ft

L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-005-201
DATE 6/26/2020

PROJECT: **Milan Lot 6** **SHEET** ____ **OF** ____
SUBJECT: **Beams & Headers** **DESIGNED** **BP** **CHECKED** **SLB**

DESCRIPTION:	FB1	Roof Tributary =	2.00 ft	W_{DL}=	378.01 lb/ft	C_D = 1.25	$C_{V/L}$ = 1.00
SIZE:	4X8	Floor Tributary =	6.00 ft	W_{LL}=	280.00 lb/ft	C_t = 1.00	C_F = 1.30
GRADE:	DFL#2	Wall Tributary =	12.00 ft	P_{DL} =	0.00 lb	l_u = 5.25	
LENGTH (ft)=	5.25	Additional Load =	100.00 plf	P_{LL} =	0.00 lb	Reduction (L_r)	%
NOTES:	ADDITIONAL DECK LOAD ADDED			$d_{Point\ Load}$ =	0.00 ft	Reduction (L)	%

	Actual			Req'd						L/xxx	Alternate Loads		
							Source:				#1	#2	LL
S =	30.66	in^3	>	18.60	in^3	O.K.	Max LL Deflection =	0.03 in =	L / 2341	360			
A =	25.38	in^2	>	8.86	in^2	O.K.	Max TL Deflection =	0.06 in =	L / 996	240			
I =	111.15	in^4	>	26.78	in^4	O.K.		Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	1,727	lb	Post:	2X4	DFLSTUD	O.K.			lb		8.0	2.0	ft
Right Reaction =	1,727	lb	Post:	2X4	DFLSTUD	O.K.			lb		8.0	2.0	ft

DESCRIPTION:	FB2	Roof Tributary =	2.00 ft	W_{DL}=	490.52 lb/ft	C_D = 1.00	$C_{V/L}$ = 0.96
SIZE:	3-1/8X10-1/2	Floor Tributary =	13.50 ft	W_{LL}=	580.00 lb/ft	C_t = 1.00	C_F = 1.00
GRADE:	24F-V4	Wall Tributary =	12.00 ft	P_{DL} =	0.00 lb	l_u = 8.50	
LENGTH (ft)=	8.50	Additional Load =	100.00 plf	P_{LL} =	0.00 lb	Reduction (L_r)	%
NOTES:	ADDITIONAL DECK LOAD ADDED			$d_{Point\ Load}$ =	0.00 ft	Reduction (L)	%

	Actual			Req'd						L/xxx	Alternate		
							Source:				#1	#2	LL
S =	57.42	in^3	>	50.24	in^3	O.K.	Max LL Deflection =	0.13 in =	L / 812	360			
A =	32.81	in^2	>	22.58	in^2	O.K.	Max TL Deflection =	0.23 in =	L / 440	240			
I =	301.46	in^4	>	164.36	in^4	O.K.		Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	4,550	lb	Post:	(2)2X4	DFLSTUD	O.K.			lb		8.0	2.0	ft
Right Reaction =	4,550	lb	Post:	(2)2X4	DFLSTUD	O.K.			lb		8.0	2.0	ft

DESCRIPTION:	FB3	Roof Tributary =	0.00 ft	W_{DL}=	204.01 lb/ft	C_D = 1.00	$C_{V/L}$ = 1.00
SIZE:	4X6	Floor Tributary =	12.00 ft	W_{LL}=	480.00 lb/ft	C_t = 1.00	C_F = 1.30
GRADE:	DFL#2	Wall Tributary =	2.00 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						L/xxx	Alternate		
							Source:				#1	#2	LL
S =	17.65	in^3	>	7.89	in^3	O.K.	Max LL Deflection =	0.01 in =	L / 3195	360			
A =	19.25	in^2	>	5.94	in^2	O.K.	Max TL Deflection =	0.02 in =	L / 2242	240			
I =	48.53	in^4	>	5.47	in^4	O.K.		Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	1,026	lb	Post:	2X4	DFLSTUD	O.K.			lb		8.0	2.0	ft
Right Reaction =	1,026	lb	Post:	2X4	DFLSTUD	O.K.			lb		8.0	2.0	ft

DESCRIPTION:	FB4	Roof Tributary =	ft	W_{DL}=	0.00 lb/ft	C_D = 1.00	$C_{V/L}$ = 0.97
SIZE:	3-1/8X10-1/2	Floor Tributary =	ft	W_{LL}=	0.00 lb/ft	C_t = 1.00	C_F = 1.00
GRADE:	24F-V4	Wall Tributary =	ft	P_{DL} =	lb	l_u = 6.50	
LENGTH (ft)=	6.50	Additional Load =	plf	P_{LL} =	lb	Reduction (L_r)	%
NOTES:	SEE ADD CALC			$d_{Point\ Load}$ =	ft	Reduction (L)	%

	Actual			Req'd						L/xxx	Alternate		
							Source:				#1	#2	LL
S =	57.42	in^3	>	0.00	in^3	O.K.	Max LL Deflection =	0.00 in =	N/A	360			
A =	32.81	in^2	>	0.00	in^2	O.K.	Max TL Deflection =	0.00 in =	N/A	240			
I =	301.46	in^4	>	0.00	in^4	O.K.		Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	3,853	lb	Post:	(2)2X4	DFLSTUD	O.K.		3,852.88	lb		8.0	2.0	ft
Right Reaction =	5,535	lb	Post:	(3)2X4	DFLSTUD	O.K.		5,534.99	lb		8.0	2.0	ft

DESCRIPTION:	FB5	Roof Tributary =	0.00 ft	W_{DL}=	0.00 lb/ft	C_D = 1.00	$C_{V/L}$ = 1.00
SIZE:	5-1/8X13-1/2	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	l_u = 2.00	
LENGTH (ft)=	16.50	Additional Load =	0.00 plf	P_{LL} =	0.00 lb	Reduction (L_r)	%
NOTES:	SEE ADD CALC ENERCALC			$d_{Point\ Load}$ =	0.50 ft	Reduction (L)	%

	Actual			Req'd						L/xxx	Alternate		
							Source:				#1	#2	LL
S =	155.67	in^3	>	0.00	in^3	O.K.	Max LL Deflection =	0.00 in =	N/A	360			
A =	69.19	in^2	>	0.00	in^2	O.K.	Max TL Deflection =	0.00 in =	N/A	240			
I =	1050.79	in^4	>	0.00	in^4	O.K.		Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	6,554	lb	Post:	(2)2X6	DFLSTUD	O.K.		6,554.00	lb		8.0	2.0	ft
Right Reaction =	5,578	lb	Post:	(2)2X6	DFLSTUD	O.K.		5,578.00	lb		8.0	2.0	ft

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Title Block Line 6

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Engineer:
Project ID:
Project Descr:

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Wood Beam

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

DESCRIPTION: FB5

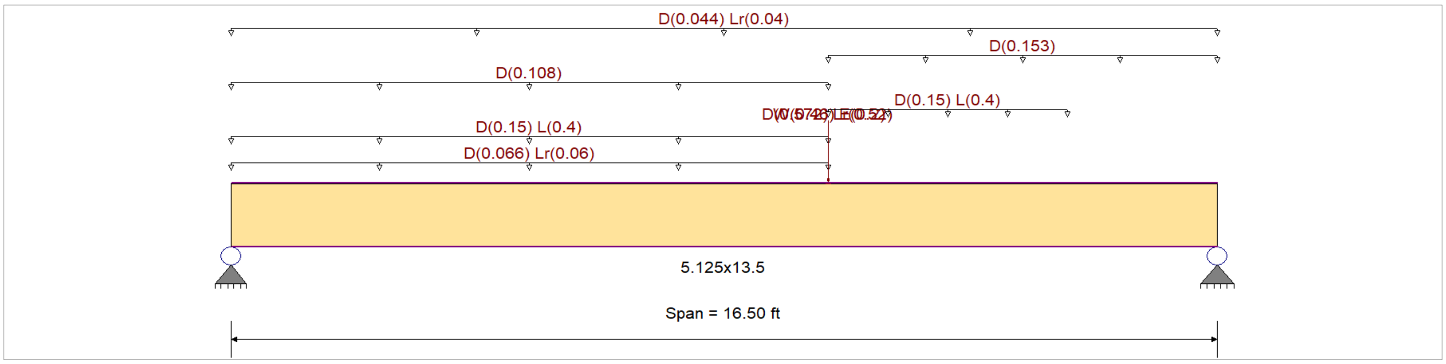
CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,400.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	1,850.0 psi	Ebend- xx	1,800.0 ksi
		Fc - Prll	1,650.0 psi	Eminbend - xx	950.0 ksi
Wood Species	: DF/DF	Fc - Perp	650.0 psi	Ebend- yy	1,600.0 ksi
Wood Grade	: 24F-V4	Fv	265.0 psi	Eminbend - yy	850.0 ksi
		Ft	1,100.0 psi	Density	31.210 pcf

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



Applied Loads

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

Beam self weight calculated and added to loads
Load for Span Number 1
- Uniform Load : D = 0.0220, Lr = 0.020 ksf, Extent = 0.0 -->> 10.0 ft, Tributary Width = 3.0 ft, (ROOF)
- Point Load : D = 0.5720, Lr = 0.520 k @ 10.0 ft, (GT2)
- Uniform Load : D = 0.150, L = 0.40 k/ft, Extent = 0.0 -->> 10.0 ft, Tributary Width = 1.0 ft, (FLOOR)
- Uniform Load : D = 0.150, L = 0.40 k/ft, Extent = 10.0 -->> 14.0 ft, Tributary Width = 1.0 ft, (FLOOR)
- Uniform Load : D = 0.1080 k/ft, Extent = 0.0 -->> 10.0 ft, Tributary Width = 1.0 ft, (WALL)
- Uniform Load : D = 0.0120 ksf, Extent = 10.0 -->> 16.50 ft, Tributary Width = 12.750 ft, (WALL)
- Point Load : W = 0.460, E = 0.20 k @ 10.0 ft, (LATERAL)
- Uniform Load : D = 0.0440, Lr = 0.040 , Tributary Width = 1.0 ft, (ROOF)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.881**: 1	Maximum Shear Stress Ratio	=	**0.467** : 1
Section used for this span		**5.125x13.5**	Section used for this span		**5.125x13.5**
	=	2,114.43 psi		=	123.69 psi
	=	2,400.00 psi		=	265.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	8.370 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.336 in	Ratio =	590 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.725 in	Ratio =	273 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

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 = 16.50 ft	1	0.517	0.265	0.90	1.000	1.00	1.00	1.00	1.00	1.00	14.47	1,115.75	2160.00	2.91	63.19	238.50
+D+L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.50 ft	1	0.881	0.467	1.00	1.000	1.00	1.00	1.00	1.00	1.00	27.43	2,114.43	2400.00	5.71	123.69	265.00

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Title Block Line 6

Project Title:
Engineer:
Project ID:
Project Descr:

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Wood Beam

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DESCRIPTION: FB5

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
+D+Lr+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.50 ft	1	0.488	0.246	1.25	1.000	1.00	1.00	1.00	1.00	1.00	18.99	1,463.69	3000.00	3.76	81.50	331.25
+D+0.750Lr+0.750L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.50 ft	1	0.708	0.369	1.25	1.000	1.00	1.00	1.00	1.00	1.00	27.55	2,123.73	3000.00	5.64	122.30	331.25
+D+0.750Lr+0.750L+0.450W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.50 ft	1	0.567	0.293	1.60	1.000	1.00	1.00	1.00	1.00	1.00	28.25	2,177.67	3840.00	5.72	124.07	424.00
+D+0.750L+0.750S+0.450W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.50 ft	1	0.499	0.260	1.60	1.000	1.00	1.00	1.00	1.00	1.00	24.88	1,917.70	3840.00	5.09	110.33	424.00

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	1	0.7253	8.310		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 — Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	6.554	5.578
Overall MINimum	0.181	0.279
+D+H	3.330	3.050
+D+L+H	6.554	5.426
+D+Lr+H	4.283	3.877
+D+0.750Lr+0.750L+H	6.463	5.452
+D+0.750L+0.750S+H	5.748	4.832
+D+0.750Lr+0.750L+0.450W+H	6.544	5.578
+D+0.750L+0.750S+0.450W+H	5.829	4.957
+D+0.750L+0.750S+0.5250E+H	5.789	4.896
D Only	3.330	3.050
Lr Only	0.953	0.827
L Only	3.224	2.376
W Only	0.181	0.279

L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-005-201
DATE 6/26/2020

PROJECT: **Milan Lot 6** **SHEET** ______ **OF** ______
SUBJECT: **Beams & Headers** **DESIGNED** **BP** **CHECKED** **SLB**

DESCRIPTION:	FB6	Roof Tributary =	0.00	ft	W_{DL}=	196.51	lb/ft	C_D =	1.00	$C_{V/L}$ =	1.00
SIZE:	6X6	Floor Tributary =	11.50	ft	W_{LL}=	460.00	lb/ft	C_t =	1.00	C_F =	1.00
GRADE:	DFL#1 (TIMBERS)	Wall Tributary =	2.00	ft	P_{DL} =	0.00	lb	l_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 Loads		
S =	27.73	in^3	>	14.77	in^3	O.K.	Max LL Deflection =	0.03 in =	L / 1552	360	#1	#2	LL
A =	30.25	in^2	>	10.38	in^2	O.K.	Max TL Deflection =	0.05 in =	L / 1088	240			
I =	76.26	in^4	>	17.68	in^4	O.K.		Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	1,477	lb	Post:	2X4	DFLSTUD	O.K.			lb		8.0	2.0	ft
Right Reaction =	1,477	lb	Post:	2X4	DFLSTUD	O.K.			lb		8.0	2.0	ft

DESCRIPTION:	FB7	Roof Tributary =	0.00	ft	W_{DL}=	75.00	lb/ft	C_D =	1.00	$C_{V/L}$ =	0.93
SIZE:	3-1/8X10-1/2	Floor Tributary =	5.00	ft	W_{LL}=	300.00	lb/ft	C_t =	1.00	C_F =	1.00
GRADE:	24F-V4	Wall Tributary =	0.00	ft	P_{DL} =	0.00	lb	l_u =	12.50		
LENGTH (ft)=	12.50	Additional Load =	0.00	plf	P_{LL} =	0.00	lb	Reduction (L_r)		%	
NOTES:					$d_{Point\ Load}$ =	0.00	ft	Reduction (L)		%	

	Actual			Req'd				Source:		L/xxx	Alternate		
S =	57.42	in^3	>	39.31	in^3	O.K.	Max LL Deflection =	0.30 in =	L / 494	360	#1	#2	LL
A =	32.81	in^2	>	12.60	in^2	O.K.	Max TL Deflection =	0.38 in =	L / 395	240			x
I =	301.46	in^4	>	219.73	in^4	O.K.		Addt'l Load to Post		Source	Lx	Ly	
Left Reaction =	3,600	lb	Post:	(2)2X4	DFLSTUD	O.K.		1,256	lb	RB5	9.0	2.0	ft
Right Reaction =	3,600	lb	Post:	(2)2X4	DFLSTUD	O.K.		1,256	lb	RB5	9.0	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	l_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:	2X4	DFLSTUD	O.K.			lb		8.0	2.0	ft
Right Reaction =	0	lb	Post:	2X4	DFLSTUD	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	l_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:	2X4	DFLSTUD	O.K.			lb		8.0	2.0	ft
Right Reaction =	0	lb	Post:	2X4	DFLSTUD	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	l_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:	2X4	DFLSTUD	O.K.			lb		8.0	2.0	ft
Right Reaction =	0	lb	Post:	2X4	DFLSTUD	O.K.			lb		8.0	2.0	ft

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

L. R. NELSON CONSULTING ENGINEERS

		JOB NO.	1939-005-201
		DATE:	
PROJECT:	Milan Lot 6	SHEET	OF
DESCRIPTION:	ROOF AND FLOOR EXT Great Room	DESIGNED BP	CHECKED SLB

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

Roof Dead Load =	22.00 psf
Floor Dead Load =	15.00 psf
Wall/Misc. Dead Load =	12.00 psf

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

Wall Height =	8.71 ft	
Roof Tributary Width =	18.33 ft	
Floor Tributary Width =	8.50 ft	
Wall/Misc. DL Width =	14.00 ft	
Wind Zone (Interior or End) =	Interior	
		ALT:
Stud Size =	2X4	2X4
Stud Spacing =	12 in o.c.	16 in o.c.
Lumber Grade =	DFL STUD	DFL#2

INTERACTION RESULTS FOR POSSIBLE CONTROLLING LOAD COMBINATIONS		
DL + FLL =	0.489	OK
DL + RLL =	0.483	OK
DL + SL =	0.329	OK
DL + 0.6*WIND =	0.637	OK
DL + 0.75*(FLL + RLL) =	0.557	OK
DL + 0.75*(FLL + SL) =	0.432	OK
DL + 0.75*(0.6*WIND + FLL + RLL) =	0.871	OK
DL + 0.75*(0.6*WIND + FLL + SL) =	0.651	OK
DEFLECTION L/? =	582.6	O.K.**

** Deflection Determined Using 0.42 x Wind Load

Wind Speed =	100 mph
Exposure =	C
Mean Roof Height, h =	22.00 ft
Topo. Factor, K_{zt} =	1.00
Wind Dir. Factor, K_d =	0.85
GCpi +/- =	0.18

Exterior Finish =	Brittle	(L/240 Deflection Ratio)

Wall Uniform Loads	
Dead Load, DL =	699 lb/ft
Roof Live Load, RLL =	367 lb/ft
Floor Live Load, FLL =	340 lb/ft
Snow Load, SL =	0 lb/ft

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

Ultimate Design Pressure =	24.75 psf	
Ultimate Max. Deflection =	0.43 in	
Max. Deflection x 0.42 =	0.18 in	
Max. Deflection x 0.60 =	N/A	
L/d =	29.86	< 50 (OK)

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

Check Bearing Stress on Wall Plate		
Wall Plate Lumber Grade =	DFL#2	
Max. Stud Reaction on Wall Plate =	1,405 lbs	
Bearing Stress on Wall Plate =	267.69 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)	22.70 psf	-24.75 psf	22.70 psf	-29.45 psf
qh (psf)	20.46 psf	20.46 psf	20.46 psf	20.46 psf
GCp	0.930	-1.030	0.930	-1.259
Gcpi	0.180	-0.180	0.180	-0.180

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 =	133.10 psi	64.76 psi	69.83 psi	0.00 psi

Maximum Bending Moment & Stress	
M =	235 ft-lb
f_b =	919.69 psi

Allowable Bending Stress	
F'_b =	1,417 psi

Allowable Axial Stresses					
c =	0.80	Overwrite c =			
$(L/d)^2$ =	891.79				
COV_E =	0.25	Overwrite COV_E =			
E_{min}' =	511,432 psi				
F_{cE} =	471.41				
	DL	FLL	RLL	SL	WIND
F_c^* =	892.50	892.50	1,115.63	892.50	1,428.00
F_{cE}/F_c^* =	0.528	0.528	0.423	0.528	0.330
C_P =	0.453	0.453	0.377	0.453	0.304
F'c =	404.40	404.40	420.53	404.40	433.60 psi

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

L. R. NELSON CONSULTING ENGINEERS

JOB NO.	1939-005-201
DATE:	
PROJECT:	Milan Lot 6
DESCRIPTION:	ROOF AND FLOOR Rear Kitchen
SHEET	OF
DESIGNED	BP
CHECKED	SLB

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

Roof Dead Load =	22.00 psf
Floor Dead Load =	15.00 psf
Wall/Misc. Dead Load =	12.00 psf

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

		ALT:
Wall Height =	8.71 ft	
Roof Tributary Width =	2.00 ft	
Floor Tributary Width =	16.00 ft	
Wall/Misc. DL Width =	14.00 ft	
Wind Zone (Interior or End) =	Interior	
Stud Size =	2X4	2X4
Stud Spacing =	12 in o.c.	16 in oc
Lumber Grade =	DFL STUD	DFL #2

INTERACTION RESULTS FOR POSSIBLE CONTROLLING LOAD COMBINATIONS		
DL + FLL =	**0.514**	**OK**
DL + RLL =	**0.223**	**OK**
DL + SL =	**0.213**	**OK**
DL + 0.6*WIND =	**0.516**	**OK**
DL + 0.75*(FLL + RLL) =	**0.436**	**OK**
DL + 0.75*(FLL + SL) =	**0.422**	**OK**
DL + 0.75*(0.6*WIND + FLL + RLL) =	**0.656**	**OK**
DL + 0.75*(0.6*WIND + FLL + SL) =	**0.636**	**OK**
DEFLECTION L/? =	**582.6**	**O.K.****

** Deflection Determined Using 0.42 x Wind Load

Wind Speed =	100 mph
Exposure =	C
Mean Roof Height, h =	22.00 ft
Topo. Factor, K_{zt} =	1.00
Wind Dir. Factor, K_d =	0.85
GCpi +/- =	0.18

Exterior Finish =	Brittle	(L/240 Deflection Ratio)

Wall Uniform Loads	
Dead Load, DL =	452 lb/ft
Roof Live Load, RLL =	40 lb/ft
Floor Live Load, FLL =	640 lb/ft
Snow Load, SL =	0 lb/ft

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

Ultimate Design Pressure =	24.75 psf	
Ultimate Max. Deflection =	0.43 in	
Max. Deflection x 0.42 =	0.18 in	
Max. Deflection x 0.60 =	N/A	
L/d =	29.86	< 50 (OK)

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

Check Bearing Stress on Wall Plate		
Wall Plate Lumber Grade =	DFL#2	
Max. Stud Reaction on Wall Plate =	1,132 lbs	
Bearing Stress on Wall Plate =	215.62 psi	(OK)
Allow. Bearing Stress on Wall Plate =	625 psi	

Ultimate Wall Pressures (psf)				
Zone	Interior Zone (Zone 4)		(Zone 5)	
Positive/Negative	Positive	Negative	Positive	Negative
Pressure (psf)	22.70 psf	-24.75 psf	22.70 psf	-29.45 psf
qh (psf)	20.46 psf	20.46 psf	20.46 psf	20.46 psf
GCp	0.930	-1.030	0.930	-1.259
Gcpi	0.180	-0.180	0.180	-0.180

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 =	86.10 psi	121.90 psi	7.62 psi	0.00 psi

Maximum Bending Moment & Stress	
M =	235 ft-lb
f_b =	919.69 psi

Allowable Bending Stress	
F'_b =	1,417 psi

Allowable Axial Stresses					
c =	0.80	Overwrite c =			
$(L/d)^2$ =	891.79				
COV_E =	0.25	Overwrite COV_E =			
E_{min}' =	511,432 psi				
F_{cE} =	471.41				
	DL	FLL	RLL	SL	WIND
F_c^* =	892.50	892.50	1,115.63	892.50	1,428.00
F_{cE}/F_c^* =	0.528	0.528	0.423	0.528	0.330
C_P =	0.453	0.453	0.377	0.453	0.304
F'c =	404.40	404.40	420.53	404.40	433.60 psi

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

L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-005-201
DATE:

PROJECT: Milan Lot 6
DESCRIPTION: ROOF AND FLOOR TYP

SHEET OF
DESIGNED BP CHECKED SLB

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

Roof Dead Load =	22.00 psf
Floor Dead Load =	15.00 psf
Wall/Misc. Dead Load =	12.00 psf

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

Wall Height =	8.71 ft
Roof Tributary Width =	18.33 ft
Floor Tributary Width =	1.00 ft
Wall/Misc. DL Width =	12.00 ft
Wind Zone (Interior or End) =	Interior
Stud Size =	2X4
Stud Spacing =	16 in o.c.
Lumber Grade =	DFL STUD
Wind Speed =	100 mph
Exposure =	C
Mean Roof Height, h =	22.00 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 =	**0.378**	**OK**
DL + RLL =	**0.561**	**OK**
DL + SL =	**0.353**	**OK**
DL + 0.6*WIND =	**0.854**	**OK**
DL + 0.75*(FLL + RLL) =	**0.524**	**OK**
DL + 0.75*(FLL + SL) =	**0.358**	**OK**
DL + 0.75*(0.6*WIND + FLL + RLL) =	**0.989**	**OK**
DL + 0.75*(0.6*WIND + FLL + SL) =	**0.692**	**OK**
DEFLECTION L/? =	**436.9**	**O.K.*** **

** Deflection Determined Using 0.42 x Wind Load

Exterior Finish =	Brittle	(L/240 Deflection Ratio)

Wall Uniform Loads	
Dead Load, DL =	562 lb/ft
Roof Live Load, RLL =	367 lb/ft
Floor Live Load, FLL =	40 lb/ft
Snow Load, SL =	0 lb/ft

Ultimate Design Pressure =	24.75 psf	
Ultimate Max. Deflection =	0.57 in	
Max. Deflection x 0.42 =	0.24 in	
Max. Deflection x 0.60 =	N/A	
L/d =	29.86	< 50 (OK)

Check Bearing Stress on Wall Plate		
Wall Plate Lumber Grade =	DFL#2	
Max. Stud Reaction on Wall Plate =	1,292 lbs	
Bearing Stress on Wall Plate =	246.10 psi	(OK)
Allow. Bearing Stress on Wall Plate =	625 psi	

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

Studs Part of Fire Assembly? No

Ultimate Wall Pressures (psf)				
Zone	Interior Zone (Zone 4)		(Zone 5)	
Positive/Negative	Positive	Negative	Positive	Negative
Pressure (psf)	22.70 psf	-24.75 psf	22.70 psf	-29.45 psf
qh (psf)	20.46 psf	20.46 psf	20.46 psf	20.46 psf
GCp	0.930	-1.030	0.930	-1.259
Gcpi	0.180	-0.180	0.180	-0.180

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 =	142.82 psi	10.16 psi	93.12 psi	0.00 psi

Maximum Bending Moment & Stress	
M =	313 ft-lb
f_b =	1226.26 psi

Allowable Bending Stress	
F'_b =	1,417 psi

Allowable Axial Stresses						
c =	0.80	Overwrite c =				
$(L/d)^2$ =	891.79					
COV_E =	0.25	Overwrite COV_E =				
E_{min}' =	511,432 psi					
F_{cE} =	471.41					
	DL	FLL	RLL	SL	WIND	
F_c^* =	892.50	892.50	1,115.63	892.50	1,428.00	
F_{cE}/F_c^* =	0.528	0.528	0.423	0.528	0.330	
C_P =	0.453	0.453	0.377	0.453	0.304	
F'c =	404.40	404.40	420.53	404.40	433.60	psi

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

L. R. NELSON CONSULTING ENGINEERS

		JOB NO.	1939-005-201
		DATE:	
PROJECT:	Milan Lot 6	SHEET	OF
DESCRIPTION:	FLOOR TYP Interior Bearing Wall	DESIGNED BP	CHECKED SLB

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

Roof Dead Load =	22.00 psf	Roof Live Load =	20.00 psf
Floor Dead Load =	15.00 psf	Floor Live Load =	40.00 psf
Wall/Misc. Dead Load =	12.00 psf	Roof Snow Load =	0.00 psf

Wall Height =	8.71 ft
Roof Tributary Width =	0.00 ft
Floor Tributary Width =	16.00 ft
Wall/Misc. DL Width =	4.00 ft
Wind Zone (Interior or End) =	End N/A
Stud Size =	2X4
Stud Spacing =	16 in o.c.
Lumber Grade =	DFL STUD
Wind Speed =	N/A (Inside)
Exposure =	C N/A
Mean Roof Height, h =	22.00 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 =	0.583	OK
DL + RLL =	0.174	OK
DL + SL =	0.181	OK
DL + 0.6*WIND =	0.235	OK
DL + 0.75*(FLL + RLL) =	0.583	OK
DL + 0.75*(FLL + SL) =	0.583	OK
DL + 0.75*(0.6*WIND + FLL + RLL) =	0.426	OK
DL + 0.75*(0.6*WIND + FLL + SL) =	0.426	OK
DEFLECTION L/? =	908.8	OK**

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

Exterior Finish =	Brittle	(L/240 Deflection Ratio)

Wall Uniform Loads	
Dead Load, DL =	288 lb/ft
Roof Live Load, RLL =	0 lb/ft
Floor Live Load, FLL =	640 lb/ft
Snow Load, SL =	0 lb/ft

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

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 =	29.86	< 50 (OK)

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

Check Bearing Stress on Wall Plate		
Wall Plate Lumber Grade =	DFL#2	
Max. Stud Reaction on Wall Plate =	1,237 lbs	
Bearing Stress on Wall Plate =	235.69 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 =	73.15 psi	162.54 psi	0.00 psi	0.00 psi

Maximum Bending Moment & Stress	
M =	105 ft-lb
f_b =	412.70 psi

Allowable Bending Stress	
F'_b =	1,417 psi

Allowable Axial Stresses					
c =	0.80	Overwrite c =			
$(L/d)^2$ =	891.79				
COV_E =	0.25	Overwrite COV_E =			
E_{min}' =	511,432 psi				
F_{cE} =	471.41				
	DL	FLL	RLL	SL	WIND
F_c^* =	892.50	892.50	1,115.63	892.50	1,428.00
F_{cE}/F_c^* =	0.528	0.528	0.423	0.528	0.330
C_P =	0.453	0.453	0.377	0.453	0.304
F'c =	404.40	404.40	420.53	404.40	433.60 psi

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

L. R. NELSON CONSULTING ENGINEERS

		JOB NO.	1939-005-201
		DATE:	
PROJECT:	**Milan Lot 6**	**SHEET**	**OF**
DESCRIPTION:	STAIR CASE	**DESIGNED** BP	**CHECKED** SLB

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

Roof Dead Load =	22.00 psf	
Floor Dead Load =	15.00 psf	
Wall/Misc. Dead Load =	12.00 psf	
Wall Height =	19.71 ft	
Roof Tributary Width =	18.00 ft	
Floor Tributary Width =	0.00 ft	
Wall/Misc. DL Width =	9.86 ft	
Wind Zone (Interior or End) =	Interior	
		ALT:
Stud Size =	2X6	2X6
Stud Spacing =	8 in o.c.	12 in o.c.
Lumber Grade =	DFL STUD	DFL#2
Wind Speed =	100 mph	
Exposure =	C	
Mean Roof Height, h =	22.00 ft	
Topo. Factor, K_{zt} =	1.00	
Wind Dir. Factor, K_d =	0.85	
GCpi +/- =	0.18	

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 =	**0.195**	**OK**
DL + RLL =	**0.327**	**OK**
DL + SL =	**0.195**	**OK**
DL + 0.6*WIND =	**0.689**	**OK**
DL + 0.75*(FLL + RLL) =	**0.327**	**OK**
DL + 0.75*(FLL + SL) =	**0.192**	**OK**
DL + 0.75*(0.6*WIND + FLL + RLL) =	**0.638**	**OK**
DL + 0.75*(0.6*WIND + FLL + SL) =	**0.526**	**OK**
DEFLECTION L/? =	**324.9**	**O.K.****

** Deflection Determined Using 0.42 x Wind Load

Exterior Finish =	Brittle	(L/240 Deflection Ratio)

Wall Uniform Loads	
Dead Load, DL =	514 lb/ft
Roof Live Load, RLL =	360 lb/ft
Floor Live Load, FLL =	0 lb/ft
Snow Load, SL =	0 lb/ft

Ultimate Design Pressure =	22.29 psf	
Ultimate Max. Deflection =	1.73 in	
Max. Deflection x 0.42 =	0.73 in	
Max. Deflection x 0.60 =	N/A	
L/d =	43.00	< 50 (OK)

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

Check Bearing Stress on Wall Plate		
Wall Plate Lumber Grade =	DFL#2	
Max. Stud Reaction on Wall Plate =	583 lbs	
Bearing Stress on Wall Plate =	70.65 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)	20.24 psf	-22.29 psf	20.24 psf	-24.53 psf
qh (psf)	20.46 psf	20.46 psf	20.46 psf	20.46 psf
GCp	0.809	-0.909	0.809	-1.019
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 =	41.56 psi	0.00 psi	29.09 psi	0.00 psi

Maximum Bending Moment & Stress	
M =	722 ft-lb
f_b =	1145.01 psi

Allowable Bending Stress	
F'_b =	1,288 psi

Allowable Axial Stresses						
c =	0.80	Overwrite c =				
$(L/d)^2$ =	1849.31					
COV_E =	0.25	Overwrite COV_E =				
E_{min}' =	511,432 psi					
F_{cE} =	227.33					
	DL	FLL	RLL	SL	WIND	
F_c^* =	850.00	850.00	1,062.50	850.00	1,360.00	
F_{cE}/F_c^* =	0.267	0.267	0.214	0.267	0.167	
C_P =	0.251	0.251	0.204	0.251	0.161	
F'c =	213.07	213.07	216.27	213.07	218.93	psi

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

L. R. NELSON CONSULTING ENGINEERS

JOB NO.	1939-005-201
DATE:	
PROJECT:	Milan Lot 6
DESCRIPTION:	ROOF TYPICAL
SHEET	OF
DESIGNED	BP
CHECKED	SLB

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

Roof Dead Load =	22.00 psf
Floor Dead Load =	15.00 psf
Wall/Misc. Dead Load =	12.00 psf

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

Wall Height =	8.71 ft
Roof Tributary Width =	19.00 ft
Floor Tributary Width =	0.00 ft
Wall/Misc. DL Width =	4.00 ft
Wind Zone (Interior or End) =	End
Stud Size =	2X4
Stud Spacing =	16 in o.c.
Lumber Grade =	DFL STUD

INTERACTION RESULTS FOR POSSIBLE CONTROLLING LOAD COMBINATIONS		
DL + FLL =	0.293	OK
DL + RLL =	0.511	OK
DL + SL =	0.293	OK
DL + 0.6*WIND =	0.900	OK
DL + 0.75*(FLL + RLL) =	0.511	OK
DL + 0.75*(FLL + SL) =	0.281	OK
DL + 0.75*(0.6*WIND + FLL + RLL) =	0.972	OK
DL + 0.75*(0.6*WIND + FLL + SL) =	0.693	OK
DEFLECTION L/? =	367.2	O.K.**

** Deflection Determined Using 0.42 x Wind Load

Wind Speed =	100 mph
Exposure =	C
Mean Roof Height, h =	22.00 ft
Topo. Factor, K_{zt} =	1.00
Wind Dir. Factor, K_d =	0.85
GCpi +/- =	0.18

Exterior Finish =	Brittle	(L/240 Deflection Ratio)

Wall Uniform Loads	
Dead Load, DL =	466 lb/ft
Roof Live Load, RLL =	380 lb/ft
Floor Live Load, FLL =	0 lb/ft
Snow Load, SL =	0 lb/ft

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

Ultimate Design Pressure =	29.45 psf	
Ultimate Max. Deflection =	0.68 in	
Max. Deflection x 0.42 =	0.28 in	
Max. Deflection x 0.60 =	N/A	
L/d =	29.86	< 50 (OK)

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

Check Bearing Stress on Wall Plate		
Wall Plate Lumber Grade =	DFL#2	
Max. Stud Reaction on Wall Plate =	1,128 lbs	
Bearing Stress on Wall Plate =	214.86 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)	22.70 psf	-24.75 psf	22.70 psf	-29.45 psf
qh (psf)	20.46 psf	20.46 psf	20.46 psf	20.46 psf
GCp	0.930	-1.030	0.930	-1.259
Gcpi	0.180	-0.180	0.180	-0.180

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 =	118.35 psi	0.00 psi	96.51 psi	0.00 psi

Maximum Bending Moment & Stress	
M =	372 ft-lb
f_b =	1459.12 psi

Allowable Bending Stress	
F'_b =	1,417 psi

Allowable Axial Stresses					
c =	0.80	Overwrite c =			
$(L/d)^2$ =	891.79				
COV_E =	0.25	Overwrite COV_E =			
E_{min}' =	511,432 psi				
F_{cE} =	471.41				
	DL	FLL	RLL	SL	WIND
F_c^* =	892.50	892.50	1,115.63	892.50	1,428.00
F_{cE}/F_c^* =	0.528	0.528	0.423	0.528	0.330
C_P =	0.453	0.453	0.377	0.453	0.304
F'c =	404.40	404.40	420.53	404.40	433.60 psi

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

L. R. NELSON CONSULTING ENGINEERS

JOB NO.	1939-005-201
DATE	6/26/20
PROJECT:	Milan Lot 6
SUBJECT:	TOP PLATE DESIGN
SHEET	
SECTION	
DESIGNED	BP
CHECKED	SLB

Description and Location: ROOF WORSE CASE

Top Plate Size:	2X4			Dead	Live		Tributary Area			Roof Truss Spacing:	24	in. o.c.
Species:	DFL#2		Roof	22	20	psf	Roof	20	ft	Floor Truss Spacing:	24	in. o.c.
	No. of Plates:	2	Floor	15.001	40	psf	Floor	0	ft			
STUDS	No.:	1	Wall/Misc.	12.001		psf	Wall	0	ft			
	Spacing:	16	in o.c.									
	Align Studs w/ trusses:	No					Allowable Point Load P_{ALLOW} =	1827	lbs.	TOP PLATE IS SUFFICIENT		

F'_B	1856.25	C_D	1.25	C_L	1.00	P_{ALLOW} (due to Bending)	1827	lbs.
F'_V	225	C_M	1.00	C_F	1.50	P_{ALLOW} (due to Shear)	2057	lbs.
E'	1600000	C_t	1.00	C_{fu}	1.10	P_{ALLOW} (due to Crushing)	3281	lbs.
F_{CPERP}	625	Ci	1.00	C_r	1.00	P_{ACTUAL} =	1680	lbs

Description and Location: FLOOR WORSE CASE

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.
	No. of Plates:	2	Floor	15.001	40	psf	Floor	13	ft			
STUDS	No.:	1	Wall/Misc.	12.001		psf	Wall	0	ft			
	Spacing:	16	in o.c.									
	Align Studs w/ trusses:	No					Allowable Point Load P_{ALLOW} =	1461	lbs.	TOP PLATE IS SUFFICIENT		

F'_B	1485	C_D	1.00	C_L	1.00	P_{ALLOW} (due to Bending)	1461	lbs.
F'_V	180	C_M	1.00	C_F	1.50	P_{ALLOW} (due to Shear)	1645	lbs.
E'	1600000	C_t	1.00	C_{fu}	1.10	P_{ALLOW} (due to Crushing)	3281	lbs.
F_{CPERP}	625	Ci	1.00	C_r	1.00	P_{ACTUAL} =	1430.020201	lbs

Description and Location:

Top Plate Size:	2X4			Dead	Live		Tributary Area			Roof Truss Spacing:	24	in. o.c.
Species:			Roof	22	20	psf	Roof	0	ft	Floor Truss Spacing:	24	in. o.c.
	No. of Plates:	2	Floor	15.001	40	psf	Floor	0	ft			
STUDS	No.:	1	Wall/Misc.	12.001		psf	Wall	0	ft			
	Spacing:	16	in o.c.									
	Align Studs w/ trusses:	No					Allowable Point Load P_{ALLOW} =	#N/A	lbs.	#N/A		

F'_B	#N/A	C_D	1.00	C_L	1.00	P_{ALLOW} (due to Bending)	#N/A	lbs.
F'_V	#N/A	C_M	1.00	C_F	1.50	P_{ALLOW} (due to Shear)	#N/A	lbs.
E'	#N/A	C_t	1.00	C_{fu}	1.10	P_{ALLOW} (due to Crushing)	#N/A	lbs.
F_{CPERP}	#N/A	Ci	1.00	C_r	1.00	P_{ACTUAL} =	0	lbs

Description and Location:

Top Plate Size:	2X4			Dead	Live		Tributary Area			Roof Truss Spacing:	24	in. o.c.
Species:			Roof	22	20	psf	Roof	0	ft	Floor Truss Spacing:	24	in. o.c.
	No. of Plates:	2	Floor	15.001	40	psf	Floor	0	ft			
STUDS	No.:	1	Wall/Misc.	12.001		psf	Wall	0	ft			
	Spacing:	16	in o.c.									
	Align Studs w/ trusses:	No					Allowable Point Load P_{ALLOW} =	#N/A	lbs.	#N/A		

F'_B	#N/A	C_D	1.00	C_L	1.00	P_{ALLOW} (due to Bending)	#N/A	lbs.
F'_V	#N/A	C_M	1.00	C_F	1.50	P_{ALLOW} (due to Shear)	#N/A	lbs.
E'	#N/A	C_t	1.00	C_{fu}	1.10	P_{ALLOW} (due to Crushing)	#N/A	lbs.
F_{CPERP}	#N/A	Ci	1.00	C_r	1.00	P_{ACTUAL} =	0	lbs

Description and Location:

Top Plate Size:	2X4			Dead	Live		Tributary Area			Roof Truss Spacing:	24	in. o.c.
Species:			Roof	22	20	psf	Roof	0	ft	Floor Truss Spacing:	24	in. o.c.
	No. of Plates:	2	Floor	15.001	40	psf	Floor	0	ft			
STUDS	No.:	1	Wall/Misc.	12.001		psf	Wall	0	ft			
	Spacing:	16	in o.c.									
	Align Studs w/ trusses:	No					Allowable Point Load P_{ALLOW} =	#N/A	lbs.	#N/A		

F'_B	#N/A	C_D	1.00	C_L	1.00	P_{ALLOW} (due to Bending)	#N/A	lbs.
F'_V	#N/A	C_M	1.00	C_F	1.50	P_{ALLOW} (due to Shear)	#N/A	lbs.
E'	#N/A	C_t	1.00	C_{fu}	1.10	P_{ALLOW} (due to Crushing)	#N/A	lbs.
F_{CPERP}	#N/A	Ci	1.00	C_r	1.00	P_{ACTUAL} =	0	lbs

Description and Location:

Top Plate Size:	2X4			Dead	Live		Tributary Area			Roof Truss Spacing:	24	in. o.c.
Species:			Roof	22	20	psf	Roof	0	ft	Floor Truss Spacing:	24	in. o.c.
	No. of Plates:	2	Floor	15.001	40	psf	Floor	0	ft			
STUDS	No.:	1	Wall/Misc.	12.001		psf	Wall	0	ft			
	Spacing:	16	in o.c.									
	Align Studs w/ trusses:	No					Allowable Point Load P_{ALLOW} =	#N/A	lbs.	#N/A		

F'_B	#N/A	C_D	1.00	C_L	1.00	P_{ALLOW} (due to Bending)	#N/A	lbs.
F'_V	#N/A	C_M	1.00	C_F	1.50	P_{ALLOW} (due to Shear)	#N/A	lbs.
E'	#N/A	C_t	1.00	C_{fu}	1.10	P_{ALLOW} (due to Crushing)	#N/A	lbs.
F_{CPERP}	#N/A	Ci	1.00	C_r	1.00	P_{ACTUAL} =	0	lbs

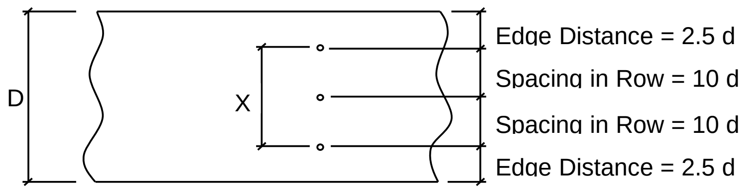
L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-005-201
DATE: June 26, 2020

PROJECT Milan Lot 6 **SHEET** **OF**

SUBJECT Wood Ledger **DESIGNED** BP **CHECKED** SLB

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 #

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L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-005-201
DATE: June 26, 2020

PROJECT Milan Lot 6 **SHEET** ____ **OF** ____

SUBJECT Wood Ledger **DESIGNED** BP **CHECKED** SLB

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

JOB NO. 1939-005-201
DATE 06/26/20

PROJECT	Milan Lot 6	**SHEET**		**SECTION**	
SUBJECT	SEISMIC LOADS	**DESIGNED**	BP	**CHECKED**	SLB

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) =	20.00	(to highest level)
T_a =	0.19	
x =	0.75	
T_L =	6	

Redundancy, ρ =	1.00

I_E =	1.00
C_{SMAX} =	0.133
C_S =	0.065
C_{SMIN} =	0.010
C_{SMIN} =	0

SDC =	C
SDC =	C

$C_{SCONTROL}$=	**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.	1939-005-201
		DATE	06/26/20
PROJECT	Milan Lot 6	**SHEET**	
		SECTION	
SUBJECT	VERTICAL REDISTRIBUTION	**DESIGNED**	BP
		CHECKED	SLB

VERTICAL REDISTRIBUTION OF SEISMIC FORCES

Number of Stories =	2

Seismic Coefficient, C_S =	0.065
0.7 x C_S =	**0.045**

Building Period, T =	0.19	<	0.5	=> k =	1

LEVEL	H (ft)	W (lb or psf)	ADJUSTED SEISMIC FACTOR AT LEVEL	SEISMIC ADJUSTMENT FACTOR AT LEVEL
2nd FLOOR	10.00	54,651 lbs	**0.030**	**0.653**
ROOF	20.00	61,863 lbs	**0.059**	**1.306**
N/A	0.00	0 lbs	N/A	N/A
N/A	0.00	0 lbs	N/A	N/A
N/A	0.00	0 lbs	N/A	N/A

CALCULATED VALUES:				Vertical Distribution of Shear	% of Base Shear
Level (i)	Height (h_i)	Weight (w_i)	$h_i^k * w_i$		
2nd FLOOR	10.00	54,651	546,506	30.64%	100.00%
ROOF	20.00	61,863	1,237,255	69.36%	69.36%
N/A	0.00	0	0	0.00%	0.00%
N/A	0.00	0	0	0.00%	0.00%
N/A	0.00	0	0	0.00%	0.00%
	Σ =	116,513	1,783,762	100.00%	

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L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-005-201
DATE: 06/26/20

PROJECT Milan Lot 6 **SHEET** **OF**
SUBJECT **WIND LOADS (Main Lateral Force Resisting System)** **DESIGNED** BP **CHECKED** SLB

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) =	24	ω =	1.0
Exposure =	C	Width of Building, B (ft) =	35		
Grnd Elev Factor, Ke =	0.94	Depth of Building, L (ft) =	54		
G =	0.85	Topo. Factor, K_{zt} =	1		
Roof Angle =	18.43	Wind Dir. Factor, K_d =	0.85		
Roof Slope =	4 /12	h/L =	0.44		

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	-0.40	0.00	-0.46	-0.56
0-15	0.85	17.39		**11.9**				
20	0.9	18.41		**12.6**				
25	0.94	19.23	19.23	**13.1**	**-6.6**	**0.0**	**-7.6**	**-9.2**
30	0.98	20.05		**13.7**				
40	1.04	21.28		**14.5**				
50	1.09	22.30		**15.2**				
60	1.13	23.12		**15.8**				
70	1.17	23.94		**16.3**				
80	1.21	24.75		**16.9**				
90	1.24	25.37		**17.3**				
100	1.26	25.78		**17.6**				
120	1.31	26.80		**18.3**				
140	1.36	27.82		**19.0**				
160	1.39	28.44		**19.4**				
180	1.43	29.25		**19.9**				
200	1.46	29.87		**20.4**				
250	1.53	31.30		**21.3**				

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						
25	**-11.5**	**-14.8**	**-14.8**	**-8.2**	**-5.0**	**3.5**
30						
40						
50						
60						
70						
80						
90						
100						
120						
140						
160						
180						
200						
250						

L. R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-005-201
DATE: 06/26/20

PROJECT Milan Lot 6 — SHEET ____ OF ____
SUBJECT WIND LOADS (Main Lateral Force Resisting System) — DESIGNED BP — CHECKED SLB

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) =	24	ω =	1.0
Exposure =	C	Width of Building, B (ft) =	54		
Grnd Elev Factor, Ke =	0.94	Depth of Building, L (ft) =	35		
G =	0.85	Topo. Factor, K_{zt} =	1		
Roof Angle =	18.43	Wind Dir. Factor, K_d =	0.85		
Roof Slope =	4 /12	h/L =	0.69		

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	-0.61	-0.57
0-15	0.85	17.39		**11.9**				
20	0.9	18.41		**12.6**				
25	0.94	19.23	19.23	**13.1**	**-8.2**	**0.0**	**-10.0**	**-9.4**
30	0.98	20.05		**13.7**				
40	1.04	21.28		**14.5**				
50	1.09	22.30		**15.2**				
60	1.13	23.12		**15.8**				
70	1.17	23.94		**16.3**				
80	1.21	24.75		**16.9**				
90	1.24	25.37		**17.3**				
100	1.26	25.78		**17.6**				
120	1.31	26.80		**18.3**				
140	1.36	27.82		**19.0**				
160	1.39	28.44		**19.4**				
180	1.43	29.25		**19.9**				
200	1.46	29.87		**20.4**				
250	1.53	31.30		**21.3**				

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	-1.02	-0.84	-0.56	-0.42	0.18
0-15						
20						
25	**-11.5**	**-16.7**	**-13.8**	**-9.2**	**-6.9**	**3.5**
30						
40						
50						
60						
70						
80						
90						
100						
120						
140						
160						
180						
200						
250						

L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-005-201
DATE 06/26/20

PROJECT Milan Lot 6
SUBJECT LATERAL LINE LOADS

SHEET | **SECTION**
DESIGNED BP | **CHECKED** SLB

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	65		9	365	1903	86	1.306	113
ω2	ROOF	36.67		9	220	1135	51	1.306	67
ω3	2nd FLOOR	10	55	22	455	1764	80	0.653	52
ω4	2nd FLOOR	5	35	22	265	1164	53	0.653	34
ω5	2nd FLOOR		35	22	265	1054	48	0.653	31
ω6	2nd FLOOR	36.67				807	37	0.653	24
ω7	ROOF	27.5				605	27	1.306	36
ω8						0	0	1.000	0
ω9						0	0	1.000	0
ω10						0	0	1.000	0
ω11						0	0	1.000	0
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	15.00	psf
Wall DL	12.00	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	4.00/12	23	Yes	7.42	7.42			146.11		118.67	87.67		71.20
ω2	4.00/12	23	No	4.50	4.50	5.83	5.83	150.68		118.67	90.41		71.20
ω3	4.00/12	9	Yes	11.00	11.00			203.50		176.00	122.10		105.60
ω4	4.00/12	9	No	11.00	9.33		1.67	223.10		176.00	133.86		105.60
ω5	4.00/12	9	No	11.00	11.00			221.10		176.00	132.66		105.60
ω6	4.00/12	9	Yes	1.67	1.67			30.83		26.67	18.50		16.00
ω7	4.00/12	23		1.00	1.00	2.17	2.17	39.63		33.33	23.78		20.00
ω8	4.00/12							0.00		0.00	0.00		0.00
ω9	4.00/12							0.00		0.00	0.00		0.00
ω10	4.00/12							0.00		0.00	0.00		0.00
ω11	4.00/12							0.00		0.00	0.00		0.00
ω12	4.00/12							0.00		0.00	0.00		0.00
ω13	4.00/12							0.00		0.00	0.00		0.00
ω14	4.00/12							0.00		0.00	0.00		0.00
ω15	4.00/12							0.00		0.00	0.00		0.00
ω16	4.00/12							0.00		0.00	0.00		0.00
ω17	4.00/12							0.00		0.00	0.00		0.00
ω18	4.00/12							0.00		0.00	0.00		0.00
ω19	4.00/12							0.00		0.00	0.00		0.00
ω20	4.00/12							0.00		0.00	0.00		0.00
ω21	4.00/12							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)

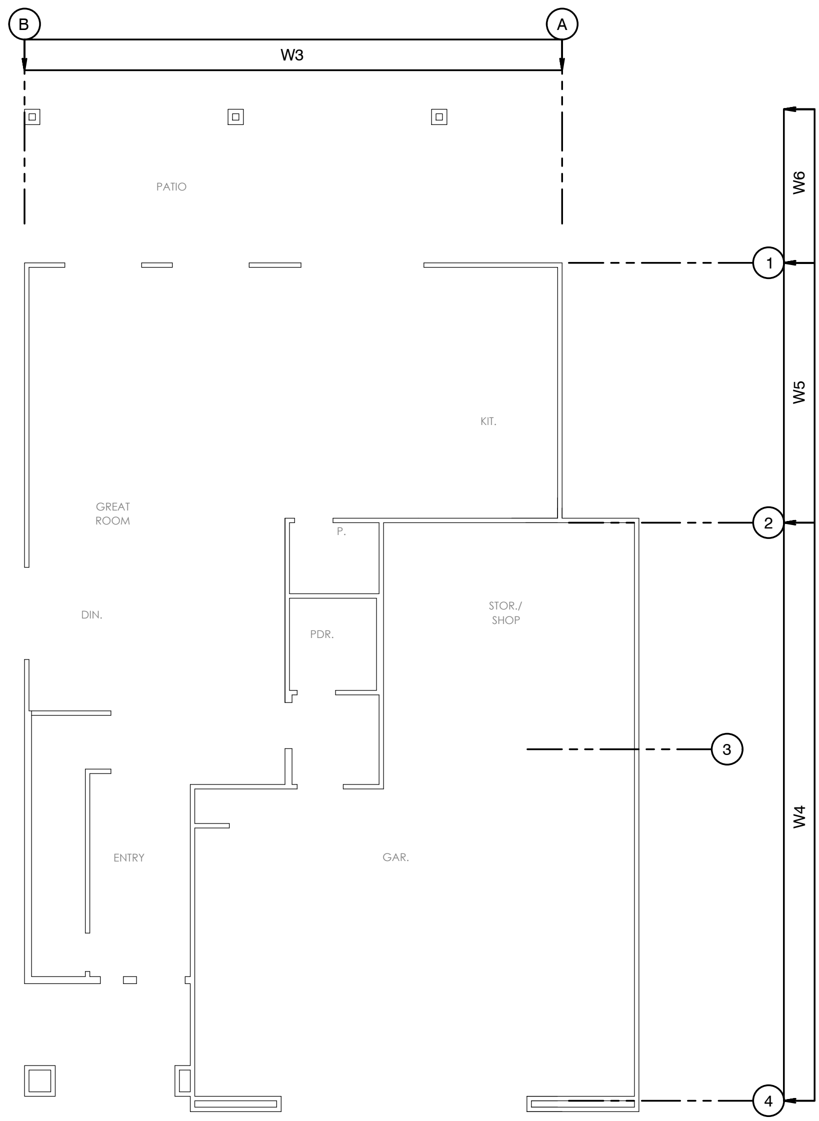
L. R. NELSON CONSULTING ENGINEERS. INC.

JOB NO. 1939-005-201

DATE 06-09-20

PROJECT MILAN LOT 6 SHEET ____ OF ____

SUBJECT SHEAR WALL KEY PLAN DESIGNED ____ CHECKED ____



1ST LEVEL SHEAR WALL PLAN

1/4"=1'-0"

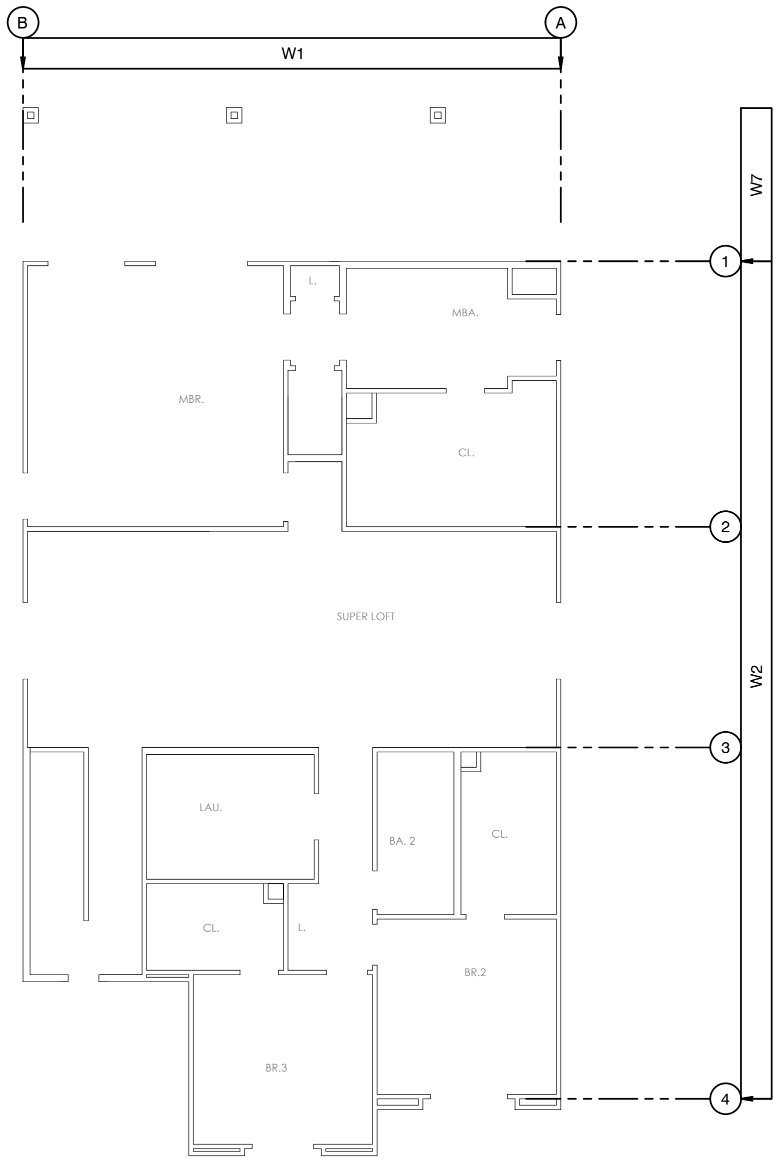
L. R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-005-201

DATE 06-09-20

PROJECT MILAN LOT 6 SHEET OF

SUBJECT SHEAR WALL KEY PLAN DESIGNED CHECKED



2ND LEVEL SHEAR WALL PLAN

1/4"=1'-0"

L. R. NELSON CONSULTING ENGINEERS

JOB NO.	1939-005-201
DATE	06/26/20
PROJECT	Milan Lot 6
SUBJECT	SHEARWALLS
SHEET	
SECTION	
DESIGNED	BP
CHECKED	SLB

Roof DL (psf)= 22
Floor DL (psf)= 15
Wall DL (psf)= 12

Allowable aspect ratio/1= 3.5 ω = 1.0

LINE: 1 2ND STORY

Total Length (ft)= 14 | Effective Shearwall Length Factor: 1.00 | Wall Height, h (ft)= 9 | Reduce for aspect ratio Y | Shear pier height(ft)= 9 | Calc A.B. N

Load	Trib w (ft)	I_E*Seis	ω (min)	ω	Allow. Uplift Pressure (psf) =
ω2	9.00	67.2	71.2	90.4	-10.02
ω7	10.00	35.9	20.0	23.8	-10.02
		0.0	0.0	0.0	-10.02

	Wind (lb)	Seis (lb)
above=		
Total=	1051	964

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)
14.00	2			9,463	8,673	8,938			P1		Corner	75	365	69	260	178	-19	
											Corner							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

HOLDOWN NOT REQ'D

LINE: 1 1ST STORY

Total Length (ft)= 8 | Effective Shearwall Length Factor: 1.00 | Wall Height, h (ft)= 9 | Reduce for aspect ratio Y | Shear pier height(ft)= 9 | Calc A.B. N

Load	Trib w (ft)	I_E*Seis	ω (min)	ω	Allow. Uplift Pressure (psf) =
ω5	8.50	31.2	105.6	132.7	-10.02
ω6	10.00	23.9	16.0	18.5	-10.02
		0.0	0.0	0.0	-10.02

	Wind (lb)	Seis (lb)
above=	1051	964
Total=	2364	1468

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	4			21,277	13,213	3,763			P1	STHD10 (C)	Corner	296	365	184	260	2,350	1,181	O.K.
											Corner							
											Corner							
									SHARED HOLDOWN	STHD10 (C)	Corner		#N/A		#N/A	2,662	1,589	O.K.
											Corner							
											Corner							

NOTES:

LINE: 2 2ND STORY

Total Length (ft)= 16.92 | Effective Shearwall Length Factor: 1.00 | Wall Height, h (ft)= 9 | Reduce for aspect ratio Y | Shear pier height(ft)= 9 | Calc A.B. N

Load	Trib w (ft)	I_E*Seis	ω (min)	ω	Allow. Uplift Pressure (psf) =
ω2	16.50	67.2	71.2	90.4	-10.02
		0.0	0.0	0.0	-10.02
		0.0	0.0	0.0	-10.02

	Wind (lb)	Seis (lb)
above=		
Total=	1492	1109

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)
16.92				13,426	9,985	9,276			P1		Corner	88	365	66	260	245	42	
											Corner							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

HOLDOWN NOT REQ'D

LINE: 2 1ST STORY

Total Length (ft)= 16 | Effective Shearwall Length Factor: 1.00 | Wall Height, h (ft)= 9 | Reduce for aspect ratio Y | Shear pier height(ft)= 9 | Calc A.B. N

Load	Trib w (ft)	I_E*Seis	ω (min)	ω	Allow. Uplift Pressure (psf) =
ω5	8.50	31.2	105.6	132.7	-10.02
ω4	19.00	34.5	105.6	133.9	-10.02
		0.0	0.0	0.0	-10.02

	Wind (lb)	Seis (lb)
above=	2504	1862
Total=	6175	2783

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				55,576	25,049	8,295			P2	STHD10 (C)	Corner	386	533	174	380	2,955	1,047	O.K.
											Corner							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

NEVADA - 6765 West Russell Road, Suite #200 - Las Vegas, NV 89118 - (702) 798-7978 - Fax (702) 451-2296
UTAH - 51 West 9000 South - Sandy, UT 84070 - (801) 565-8580 - Fax (801) 565-9340

L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-005-201
DATE 06/26/20

PROJECT Milan Lot 6 **SHEET** **SECTION**
SUBJECT SHEARWALLS **DESIGNED** BP **CHECKED** SLB

Roof DL (psf)= 22
Floor DL (psf)= 15
Wall DL (psf)= 12

Allowable aspect ratio/1= 3.5 ω = 1.0

LINE: 3 2ND STORY Total Length (ft)= 23.417 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 9 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 9

Load	Trib w (ft)	I_E*Seis	ω (min)	ω	Allow. Uplift Pressure (psf) =
ω2	18.50	67.2	71.2	90.4	-10.02
		0.0	0.0	0.0	-10.02
		0.0	0.0	0.0	-10.02

	Wind (lb)	Seis (lb)
above=		
Total=	1673	1244

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)
11.92	1			7,661	5,698	5,539			P1		Corner	71	365	53	260	238	13	
11.50	1			7,393	5,498	5,158			P1		Corner	71	365	53	260	252	30	
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

HOLDOWN NOT REQ'D

LINE: 4 2ND STORY Total Length (ft)= 11.5 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 9 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 9

Load	Trib w (ft)	I_E*Seis	ω (min)	ω	Allow. Uplift Pressure (psf) =
ω2	11.50	67.2	71.2	90.4	-10.02
		0.0	0.0	0.0	-10.02
		0.0	0.0	0.0	-10.02

	Wind (lb)	Seis (lb)
above=		
Total=	1040	773

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)
11.50				9,357	6,960	4,285			P1	CS16	Corner	90	365	67	260	441	233	O.K.
											Corner							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

SEE ADD'L CALC FOR PERFORATED SWL

LINE: 4 1ST STORY Total Length (ft)= 11.92 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 9 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 9

Load	Trib w (ft)	I_E*Seis	ω (min)	ω	Allow. Uplift Pressure (psf) =
ω4	17.50	34.5	105.6	133.9	-10.02
		0.0	0.0	0.0	-10.02
		0.0	0.0	0.0	-10.02

	Wind (lb)	Seis (lb)
above=	1700	1264
Total=	4042	1868

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				20,358	9,407	1,442			P1	STHD10 (C)	Corner	339	365	157	260	2,836	1,194	O.K.
5.25				16,024	7,404	893			P1	STHD10 (C)	Corner	339	365	157	260	2,882	1,240	O.K.
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: A 2ND STORY Total Length (ft)= 8 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 9 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 9

Load	Trib w (ft)	I_E*Seis	ω (min)	ω	Allow. Uplift Pressure (psf) =
ω1	17.50	112.8	71.2	87.7	-10.02
		0.0	0.0	0.0	-10.02
		0.0	0.0	0.0	-10.02

	Wind (lb)	Seis (lb)
above=		
Total=	1534	1973

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	17			13,807	17,761	9,255			P1	CS16	Corner	192	365	247	260	1,250	1,063	O.K.
											Corner							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

L.R. NELSON CONSULTING ENGINEERS, INC.

JOB NO. 1939-005-201
DATE

PROJECT: Milan Lot 6
SHEET
OF
SUBJECT: LINE 4 SECOND STORY
DESIGNED BP
CHECKED SLB

SHEAR WALL GEOMETRY AND FORCES

WIND LATERAL LOAD = 1040 LB
SEISMIC LATERAL LOAD = 773 LB
UPPER WALL = YES
CONNECTION HARDWARE = SIMPSON
ADD'L WIND UP/DOWN FROM ABOVE =
ADD'L SEISMIC UP/DOWN FROM ABOVE =

PANEL DIMS: L1 = 3.00 ft, L2 = 5.25 ft, L3 = 3.25 ft
H1 = 3.00 ft, H2 = 5.00 ft, H3 = 1.00 ft
cD = 4.0

KING STUDS = 2X6 DFLSTUD
END STUDS = (2)2X4 DFLSTUD
APA PANEL GRADE = PS2 Sht'g/Siding OSB
PANEL THICKNESS = 3/8 ON 1 SIDE(s)
NAIL SIZE, COMMON = 8d @ 6 in o.c.
SPECIFIC GRAVITY OF FRAMING = 0.5
STUD SPACING = 16
RESISTING WALL DEAD LOAD = 8.00 ft @ 0 psf = 0 plf
RESISTING ROOF DEAD LOAD = 5.00 ft @ 0 psf = 0 plf
RESISTING FLOOR DEAD LOAD = 0.00 ft @ 0 psf = 0 plf
SILL ANCHOR SIZE = 16d
DEFLECTION ASSUMES da = 0.150 in

DESIGN SUMMARY THE WALL DESIGN IS ADEQUATE

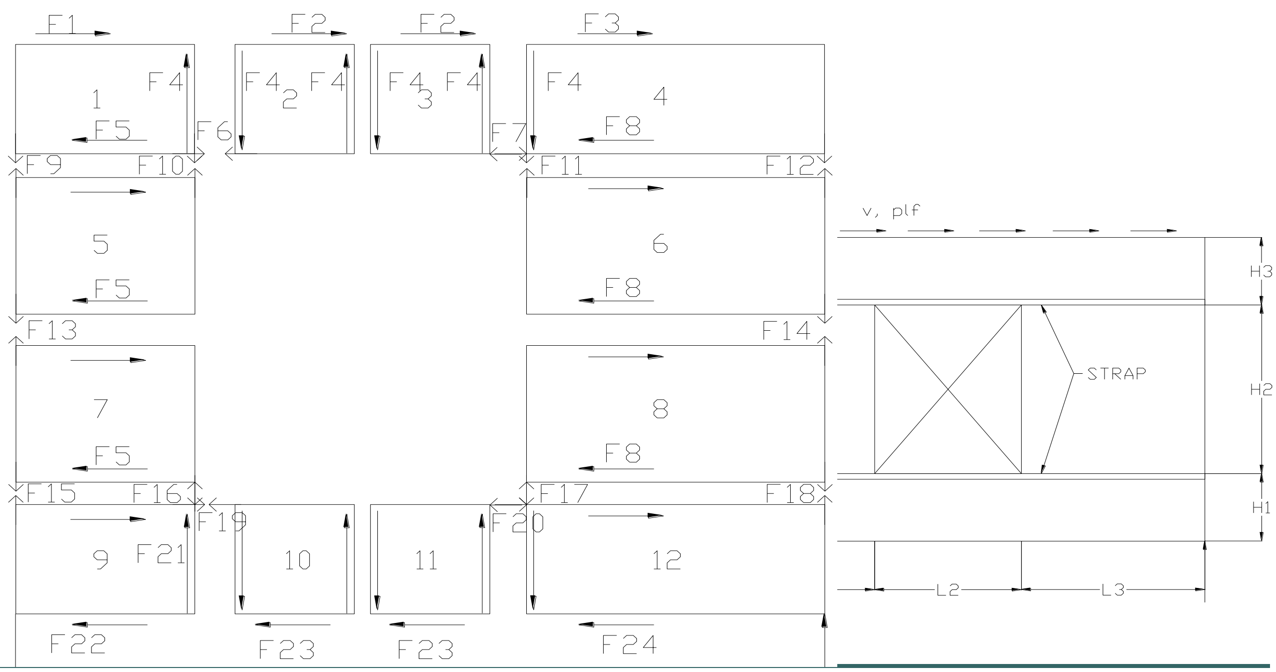
SHEATHING = 3/8 PS2 Sht'g/Siding OSB
SIDES SHT'G APPLIED TO = 1 SIDE(s)
BOUNDRY AND EDGE NAILING = 8d COMMON NAILS AT 6 in O.C.
FIELD NAILING = 8d COMMON NAILS AT 12 in O.C.
WALL END REACTIONS RL =: 814 lbs
RR = 814 lbs
STRAP FORCE: T = 831 lbs
MINIMUM STRAP REQUIRED = CS20 SIMPSON STRAP
KING STUDS = 2X6 DFLSTUD
END STUDS = (2)2X4 DFLSTUD
WALL DEFLECTION = 0.54in Wind, 1.696in Seismic, OKAY
SILL PLATE ANCHORS = 16d SINKERS AT 21 in O.C.
MINIMUM HOLDOWNS REQUIRED = CS22 SIMPSON HOLDOWN
MAXIMUM WIND UNIT SHEAR = 316 plf < Va = 364.0
MAXIMUM SEISMIC UNIT SHEAR = 235 plf < Va = 260.0
MAXIMUM HEIGHT/WIDTH RATIO = 1.7 <= 3.5

PANEL FORCES

	WIND	SEISMIC
F1 =	-322 lbs	-240 lbs
F2 =	831 lbs	618 lbs
F3 =	-299 lbs	-223 lbs
F4 =	316 lbs	235 lbs
F5 =	509 lbs	378 lbs
F6 =	831 lbs	618 lbs
F7 =	831 lbs	618 lbs
F8 =	531 lbs	395 lbs
F9 =	-107 lbs	-80 lbs
F10 =	424 lbs	315 lbs
F11 =	409 lbs	304 lbs
F12 =	-92 lbs	-69 lbs
F13 =	316 lbs	235 lbs
F14 =	316 lbs	235 lbs
F15 =	740 lbs	551 lbs
F16 =	424 lbs	315 lbs
F17 =	409 lbs	304 lbs
F18 =	725 lbs	539 lbs
F19 =	435 lbs	324 lbs
F20 =	435 lbs	324 lbs
F21 =	497 lbs	370 lbs
F22 =	73 lbs	55 lbs
F23 =	435 lbs	324 lbs
F24 =	96 lbs	71 lbs

PANEL SHEARS

	WIND	SEISMIC
V1 =	-107 plf	-80 plf
V2 =	316 plf	235 plf
V3 =	316 plf	235 plf
V4 =	-92 plf	-69 plf
V5 =	170 plf	126 plf
V6 =	163 plf	122 plf
V7 =	170 plf	126 plf
V8 =	163 plf	122 plf
V9 =	24 plf	18 plf
V10 =	166 plf	123 plf
V11 =	166 plf	123 plf
V12 =	30 plf	22 plf



L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-005-201
DATE 06/26/20

PROJECT Milan Lot 6 SHEET SECTION
SUBJECT SHEARWALLS DESIGNED BP CHECKED SLB

Allowable aspect ratio/1= 3.5 ω = 1.0

Roof DL (psf)= 22
Floor DL (psf)= 15
Wall DL (psf)= 12

LINE: A 1ST STORY Total Length (ft)= 28 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 9 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 9

Load	Trib w (ft)	I_E*Seis	ω (min)	ω	Allow. Uplift Pressure (psf) =
ω3	18.00	52.3	105.6	122.1	-10.02
		0.0	0.0	0.0	-10.02
		0.0	0.0	0.0	-10.02

	Wind (lb)	Seis (lb)
above=	1534	1973
Total=	3732	2914

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				19,193	14,987	8,295	1,250	1,063	P1	LSTHD8 (C)	Full	133	365	104	260	1,932	1,482	O.K.
12.00	17	4		14,395	11,241	23,415			P1		Corner	133	365	104	260	270	-1,015	
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: B 2ND STORY Total Length (ft)= 8 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 9 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 9

Load	Trib w (ft)	I_E*Seis	ω (min)	ω	Allow. Uplift Pressure (psf) =
ω1	17.50	112.8	71.2	87.7	-10.02
		0.0	0.0	0.0	-10.02
		0.0	0.0	0.0	-10.02

	Wind (lb)	Seis (lb)
above=		
Total=	1534	1973

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	17			13,807	17,761	9,255			P1	CS16	Corner	192	365	247	260	1,250	1,063	O.K.
											Corner							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

LINE: B 1ST STORY Total Length (ft)= 16 Effective Shearwall Length Factor: 1.00 Wall Height, h (ft)= 9 Calc A.B. N
Reduce for aspect ratio Y
Shear pier height(ft)= 9

Load	Trib w (ft)	I_E*Seis	ω (min)	ω	Allow. Uplift Pressure (psf) =
ω3	18.00	52.3	105.6	122.1	-10.02
		0.0	0.0	0.0	-10.02
		0.0	0.0	0.0	-10.02

	Wind (lb)	Seis (lb)
above=	1534	1973
Total=	3732	2914

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				33,587	26,228	8,295	1,250	1,063	P1	STHD10 (C)	Corner	233	365	182	260	2,831	2,184	O.K.
											Corner							
											Corner							
											Corner							
											Corner							
											Corner							

NOTES:

L. R. NELSON CONSULTING ENGINEERS

JOB NO.	1939-005-201
DATE	06/26/20

PROJECT	Milan Lot 6	**SHEET**		**OF**	
SUBJECT	DIAPRHAGM FORCES	**DESIGNED**	BP	**CHECKED**	SLB

ROOF 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
1	ω1	None	SEIS	35	46	42.9	165.6	15/32 shthg, 8d nails @ 6" o.c.	CASES 2-6
2	ω2	None	WIND	22	24	41.4	232.3	15/32 shthg, 8d nails @ 6" o.c.	CASES 2-6
3		None							
4		None							
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	ω3	None	WIND	35	54	39.6	276	23/32 shthg, 10d nails @ 6" o.c.	CASES 2-6
8	ω5	None	WIND	39	35	73.9	276	23/32 shthg, 10d nails @ 6" o.c.	CASES 2-6
9		None							
10		None							
11		None							
12		None							

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	375.4	2	DFL#2	2x4	0.03	1.98824	12	1.6
2	227.9	2	DFL#2	2x4	0.02	1.207143	12	1.6
7	346.2	2	DFL#2	2x4	0.02	1.833859	12	1.6
8	720.6	2	DFL#2	2x4	0.05	3.816886	12	1.6
								1.6
								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. 1939-005-201
DATE 6/26/2020

PROJECT: **Milan Lot 6** SHEET ______ OF ______
SUBJECT: **Allowable Connector Loads** DESIGNED **BP** CHECKED **SLB**

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.*

$Z_{1/4" \times 6\ WS}$ = 135 lbs/nail C_D = 1.6 C_t = 1.0
$Z'_{1/4" \times 6\ WS}$ = $(Z_{1/4"x6\ WS})(C_D)(C_M)(C_t)$ 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, INC.

JOB NO. 1939-005-201
DATE 6/26/2020

PROJECT: Milan Lot 6 SHEET OF
SUBJECT: ALLOWABLE ANCHOR BOLT LOADS DESIGNED BP CHECKED SLB

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. 1939-005-201
DATE 6/26/2020

PROJECT: Milan Lot 6 SHEET ____ OF ____
SUBJECT: ALLOWABLE ANCHOR BOLT LOADS DESIGNED BP CHECKED SLB

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, LLC.

		JOB NO.	1939-005-201
		DATE	
PROJECT	Milan Lot 6	SHEET	OF
SUBJECT:		DESIGNED BP	CHECKED SLB

Conventional Foundation

ALLOWABLE LOADS TO SLAB

LINE LOADS TO SLAB

ROOF:	DEAD LOAD =	22 psf	LIVE / SNOW LOAD =	20 psf
FLOOR:	DEAD LOAD =	15 psf	LIVE LOAD =	40 psf
MISC.:	DEAD LOAD =	12 psf		

PERIMETER LINE LOADS

Maximum Line Load		Minimum Line Load	
Roof Tributary =	18.33 ft	Roof Tributary =	3.00 ft
Floor Tributary =	8.50 ft	Floor Tributary =	0.00 ft
Wall/Misc. Tributary =	22.00 ft	Wall/Misc. Tributary =	9.00 ft
Total Dead + Live Load =	1501 plf	Total Dead + Live Load =	234 plf
Dead Load Only =	795 plf	Dead Load Only =	174 plf

INTERIOR LINE LOAD

Maximum Line Load	
Roof Tributary =	0.00 ft
Floor Tributary =	12.00 ft
Wall/Misc. Tributary =	9.00 ft
Total Dead + Live Load =	768 plf
Dead Load Only =	288 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

$P_{max} = F_b 2H^2/(\beta/4)$ = 1.73 kips / ft

Where $\beta = (E_{cr} H^3 / 3 k_s)^{0.25}$ = 22 < St,b = 32

k_s = 125 lb / in³   Modulus of Subgrade Reaction, See Soils Report
(In absence of Soils Report, use Tables for Typical Values)

ALLOWABLE LINE LOAD FOR 4 INCH SLAB   H = 4.00 inches

$P_{max} = F_b 2H^2/(\beta/4)$ = 1.73 kips / ft

Where $\beta = (E_{cr} H^3 / 3 k_s)^{0.25}$ = 22 < St,b = 32

k_s = 125 lb / in³   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

$P_{max} = F_b 2H^2/(\beta/4)$ = 2.00 kips / ft

Where $\beta = (E_{cr} H^3 / 3 k_s)^{0.25}$ = 24 < St,b = 40.5

k_s = 125 lb / in³   Modulus of Subgrade Reaction, See Soils Report
(In absence of Soils Report, use Tables for Typical Values)

L.R. NELSON CONSULTING ENGINEERS, LLC.

		JOB NO.	1939-005-201		
		DATE			
PROJECT	Milan Lot 6	SHEET		OF	
SUBJECT:		DESIGNED	BP	CHECKED	SLB

CHECK ALLOWABLE POINT LOAD TO PERIMETER BEAM

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

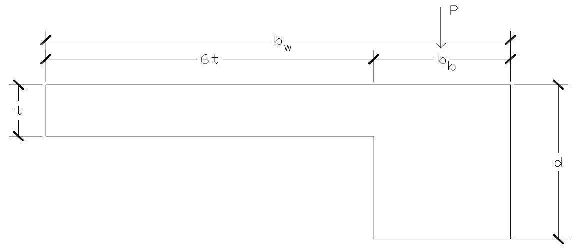
d inches	b_b in	t slab in	b_w in	I in^4	S_b in^3	A in^2
12	12	4	36	2778	365	240

$P_{max} = F_b 4 S_b / \beta =$ 10126.38 lbs

Where $\beta = [4E_{cr} I / (bw \times k_s)]^{0.25} =$ 43

$k_s =$ 125 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

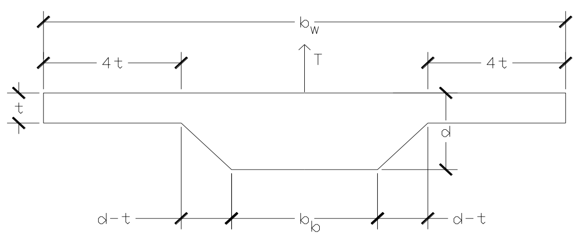
Sum forces about T with M_A = M, and T = w x L;
(Note: d = t for flat slab design and T = w x pi x L^2)

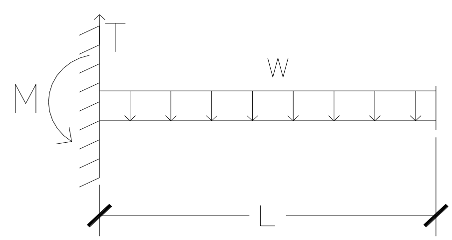
	Beam		Slab
∴ back substitution and solve for M1 =	$wL^2/2$	M2 =	$wL^2/2$
∴ L_1 =	6.33 ft	L_2 ft =	5.75 ft
∴ Assume minimum 2 foot shearwall, L` =	8.33 ft		

Slab Allowables

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

∴ T = 6443 lb at shear wall with footing
∴ T = 9792 lb at isolated uplift on T - Beam
∴ T = 5027 lb at isolated uplift on 4 inch slab





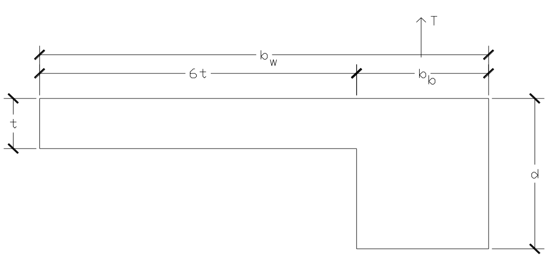
CHECK ALLOWABLE UPLIFT LOAD TO PERIMETER BEAM

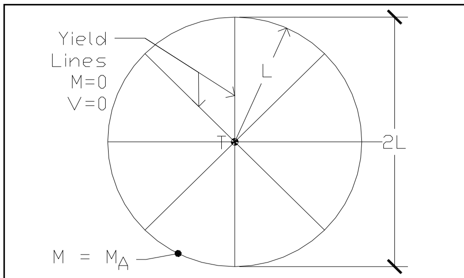
1. Assume L- beam cantilevered beyond point load, w/12 in footing

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

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

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





Failure Mechanism for the Flat Slab

L. R. NELSON CONSULTING ENGINEERS

JOB NO. 1939-005-201
DATE 06/26/20

PROJECT Milan Lot 6 SHEET SECTION
SUBJECT FOOTINGS DESIGNED BP CHECKED SLB

FOOTINGS
Assumed Soil Bearing Pressure

q=	2,500	psf not to exceed ------	2,000	psf
	0	psf/ft width greater than	0	in

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

Bearing Pressure Spread Footings

Width (in)	Depth (in)	Loads (lb)	#4 Bars
12	15	2,313	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	2313	2	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	11.50	678.5	3700	-2.8	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	11.5	3	0	3700	25.7	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	11.5	3	25.7	3122	0.01	0.25	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)