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20-191224 REV 01 RS

Structural Calculations

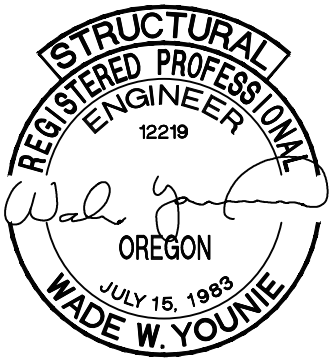
Permit Revisions
Lot 8

Wilson Subdivision

2135 S/NW 29th Ave. Portland, OR 97210

DCI Job Number 18031-0179.01

May 17, 2021



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CLT Panel

References: NDS 2018, CLT Handbook 2013, PRG 320 2019

Condition: **roof 3-ply CLT (max 16' span)**

CLT Panel manuf = PRG 320; grade = V2; 3 ply; layup ID = 0

$h = t_p$ = 4.13 in; A = 49.5 in^2/ft; G = 0.42; density = 28.5 pcf, based on MC = 15 % (NDS commentary eqn); w_{panel} = 9.8 psf

Panel ply thicknesses

# of outer || plies = 1

1st/outer ply t = 1.375 in

2nd outer or typ || ply t = 1.375 in (SPF No.1/No.2)

typ ⊥ ply t = 1.375 in (SPF No.3)

Design Values

	Major Direction (0)		Minor Direction (90)	
F_bS_{eff} =	2030	lb-ft/ft	= 160	lb-ft/ft
$(EI)_{eff}$ =	95	$x10^6$ lb-in^2/ft	= 3.1	$x10^6$ lb-in^2/ft
$(GA)_{eff}$ =	0.46	$x10^6$ lb/ft	= 0.52	$x10^6$ lb/ft
V_s =	1490	lb/ft	= 495	lb/ft
K_{cr} = 11.5 $(EI)_{app}$ =	89	$x10^6$ lb-in^2/ft		
F_b =	875	psi		

Span & Loading add self wt? Y

Span L = 16 ft; self wt override: psf

Trib width T.W. = 1 ft

Loads K_{cr} = 2

	D	SD	Lr	S	L
psf	9.8	12	20	25	0
w (plf)	9.8	12	20	25	0
% sustained	1	1	0	0	0.1

ASD Loads

	D	D+L	D+Lr	D+S	D+.75L +.75Lr	D+.75L +.75S	gov
combo	1	2	3a	3b	4a	4b	3b
C_d	0.9	1	1.25	1.15	1.25	1.15	1.15
w_a	22	22	42	47	37	41	47
w_a/C_d	24	22	33	41	29	35	41
$w_{addl\text{-}LT\text{-}defl}$	22	22	22	22			22

Strength

Flexure

M_a = 1498 lb-ft/ft

F_bS_{eff}' = 2335 lb-ft/ft

L_{max} = 19.98 ft

DCR = 0.64

Shear

V_a = 374 lb/ft

V_s' = 1714 lb/ft

L_{max} = 73.22 ft

DCR = 0.22

Deflection

Transient Load

S is gov load

w = 25 plf

$\Delta_{all\text{-}trans}$ = 360

Δ_{trans} = 0.41 in

L/ 465

L_{max} = 17.42 ft

DCR = 0.77

Total Load

3b is gov ASD combo

w = 47 plf

$\Delta_{all\text{-}TL}$ = 240

Δ_{TL} = 0.77 in

L/ 248

L_{max} = 16.18 ft

DCR = 0.97

Total Long Term

(minus initial dead load)

1 is gov ASD combo

$w_{addl\text{-}LT\text{-}defl}$ = 22 plf

$\Delta_{all\text{-}addl\text{-}LT}$ = 240

$\Delta_{addl\text{-}LT}$ = 0.36 in

L/ 533

L_{max} = 20.87

DCR = 0.45

Summary of Max Spans

	single span		double span
Flexure	L_{max} = 19.98 ft		19.98
Shear	L_{max} = 73.22 ft		58.58
Trans. load defl	L_{max} = 17.42 ft		19.34
Total load defl	L_{max} = 16.18 ft		19.15
Long term defl	L_{max} = 20.87 ft		
governing	L_{max} = 16.18 ft	16'-2''	19.15 19'-1''

General Beam Analysis

\31-Portland\2018 Portland Jobs\18031-0179.01 Wilson Townhouses\4- Calculations\Framing - 18031-0179.01.ec6
ENERCALC, INC. 1983-2018, Build:10.18.1.31, Ver:10.18.1.31

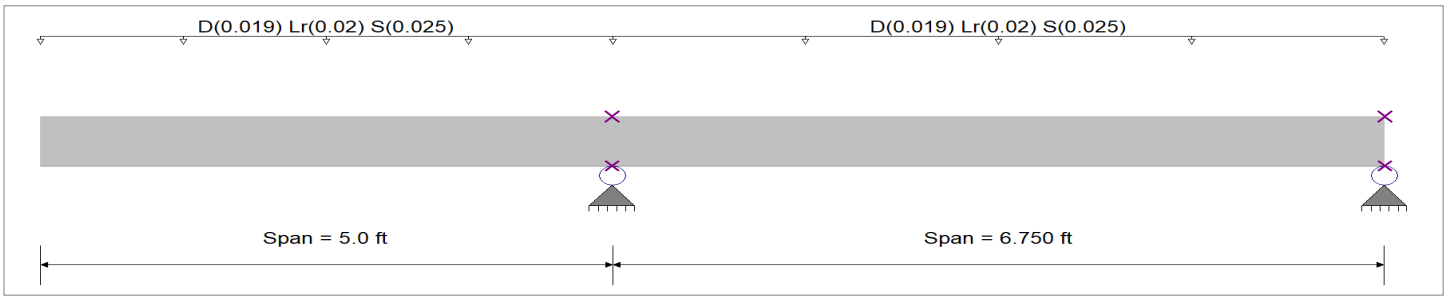
Lic. # : KW-06001666 **Licensee : DCI ENGINEERS**

Description : 3-ply CLT cantilever at roof

General Beam Properties

Elastic Modulus 2000 ksi

Span #1	Span Length =	5.0 ft	Area =	10.0 in^2	Moment of Inertia =	37.0 in^4
Span #2	Span Length =	6.750 ft	Area =	10.0 in^2	Moment of Inertia =	37.0 in^4



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 = 0.0190, Lr = 0.020, S = 0.0250 k/ft, Tributary Width = 1.0 ft, (roof)

Load for Span Number 2

Uniform Load : D = 0.0190, Lr = 0.020, S = 0.0250 k/ft, Tributary Width = 1.0 ft, (roof)

Less than CLT capacity (see previous page)

DESIGN SUMMARY

Maximum Bending =	0.550 k-ft	Maximum Shear =	0.230 k
Load Combination	+D+S+H	Load Combination	+D+S+H
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1
Location of maximum on span	5.000 ft	Location of maximum on span	5.000 ft

Maximum Deflection

Max Downward Transient Deflection	0.091 in	1324
Max Upward Transient Deflection	-0.007 in	11224
Max Downward Total Deflection	0.159 in	752
Max Upward Total Deflection	-0.013 in	6377

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values (k-ft) Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Shear Values (k) Va Max	Vnx	Vnx/Omega
Overall MAXimum Envelope													
Dsgn. L = 5.00 ft	1				-0.55	0.55					0.23		
Dsgn. L = 6.75 ft	2			0.05	-0.55	0.55					0.23		
+D+H													
Dsgn. L = 5.00 ft	1				-0.24	0.24					0.10		
Dsgn. L = 6.75 ft	2			0.02	-0.24	0.24					0.10		
+D+L+H													
Dsgn. L = 5.00 ft	1				-0.24	0.24					0.10		
Dsgn. L = 6.75 ft	2			0.02	-0.24	0.24					0.10		
+D+Lr+H													
Dsgn. L = 5.00 ft	1				-0.49	0.49					0.20		
Dsgn. L = 6.75 ft	2			0.05	-0.49	0.49					0.20		
+D+S+H													
Dsgn. L = 5.00 ft	1				-0.55	0.55					0.23		
Dsgn. L = 6.75 ft	2			0.05	-0.55	0.55					0.23		
+D+0.750Lr+0.750L+H													
Dsgn. L = 5.00 ft	1				-0.43	0.43					0.18		
Dsgn. L = 6.75 ft	2			0.04	-0.43	0.43					0.18		
+D+0.750L+0.750S+H													
Dsgn. L = 5.00 ft	1				-0.47	0.47					0.20		
Dsgn. L = 6.75 ft	2			0.04	-0.47	0.47					0.20		
+D+0.60W+H													
Dsgn. L = 5.00 ft	1				-0.24	0.24					0.10		
Dsgn. L = 6.75 ft	2			0.02	-0.24	0.24					0.10		
+D+0.750Lr+0.750L+0.450W+H													
Dsgn. L = 5.00 ft	1				-0.43	0.43					0.18		

General Beam Analysis

Description : 3-ply CLT cantilever at roof

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values (k-ft) Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Shear Values (k) Va Max	Vnx	Vnx/Omega
Dsgn. L = 6.75 ft	2			0.04	-0.43	0.43					0.18		
+D+0.750L+0.750S+0.450W+H													
Dsgn. L = 5.00 ft	1				-0.47	0.47					0.20		
Dsgn. L = 6.75 ft	2			0.04	-0.47	0.47					0.20		
+0.60D+0.60W+0.60H													
Dsgn. L = 5.00 ft	1				-0.14	0.14					0.06		
Dsgn. L = 6.75 ft	2			0.01	-0.14	0.14					0.06		
+1.126D+1.750E+0.60H													
Dsgn. L = 5.00 ft	1				-0.27	0.27					0.11		
Dsgn. L = 6.75 ft	2			0.02	-0.27	0.27					0.11		
+1.095D+0.750L+0.750S+1.313E+H													
Dsgn. L = 5.00 ft	1				-0.49	0.49					0.21		
Dsgn. L = 6.75 ft	2			0.05	-0.49	0.49					0.21		
+0.4740D+1.750E+H													
Dsgn. L = 5.00 ft	1				-0.11	0.11					0.05		
Dsgn. L = 6.75 ft	2			0.01	-0.11	0.11					0.05		

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S+H	1	0.1593	0.000		0.0000	0.000
	2	0.0000	0.000	+D+S+H	-0.0127	1.869

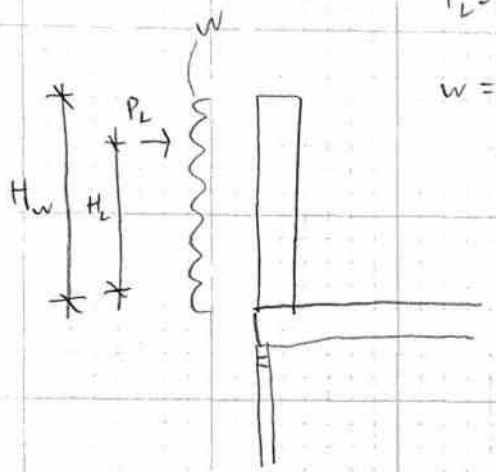
Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum		0.450	0.067
Overall MINimum		0.117	0.017
+D+H		0.194	0.029
+D+L+H		0.194	0.029
+D+Lr+H		0.399	0.059
+D+S+H		0.450	0.067
+D+0.750Lr+0.750L+H		0.348	0.052
+D+0.750L+0.750S+H		0.386	0.057
+D+0.60W+H		0.194	0.029
+D+0.750Lr+0.750L+0.450W+H		0.348	0.052
+D+0.750L+0.750S+0.450W+H		0.386	0.057
+0.60D+0.60W+0.60H		0.117	0.017
+D+0.70E+0.60H		0.194	0.029
+D+0.750L+0.750S+0.5250E+H		0.386	0.057
+0.60D+0.70E+H		0.117	0.017
D Only		0.194	0.029
Lr Only		0.205	0.030
L Only			
S Only		0.256	0.038
W Only			
E Only			
H Only			

DCI ENGINEERS	Project No.	Sheet No.
Project: Wilson		Date
Subject		By

3-ply CLT parapet



$P_L = [50\ plf]\ L$

$w = [65\ psf]\ w$ (parapet wind loading, per ASCE 7 ch 30.8)

$H_w = H_L = 1'-3" + 3'-6" = 4'-9"$ (condition at 3rd flr deck)

$M_w = 65\ psf \frac{(4.75')^2}{2} = 733\ lb\cdot ft/ft = 8796\ lb\cdot in/ft$ $\quad .6M_w = 440\ lb\cdot ft/ft = 5278\ lb\cdot in/ft$

$M_L = 50\ plf\ (4.75') = 238\ lb\cdot ft/ft = 2856\ lb\cdot in/ft$

*Note: $\frac{.6M_w}{1.6} = 3299 > M_L$, so wind load governs

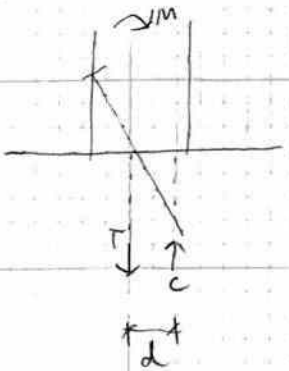
3-Ply CLT capacities

$M_r = F_b S_{eff} = 2030\ lb\cdot ft/ft$

$V_r = V_{s-0} = 1490\ lb/ft$

∴ CLT panel ok

Toe screw attachment



$T/c = \frac{M}{d}$ $\quad d = \frac{t_p}{2}(.9) = \frac{4.125"}{2}(.9) = 1.86"$ ← assumption to be verified

$T/c = \frac{.6M_w}{d} = \frac{5278\ lb\ in/ft}{1.86"} = 2838\ lb/ft$

see attached screw design

Check $F_{c\perp}$ at compression block: $f_{c\perp} = \frac{2838\ lb}{\frac{.2(4.125")}{2}(12")} = 573\ psi < 625\ psi$ ✓

Note: DTT22 is provided for redundancy at roof decks

Fully Threaded Screw

References: NDS 2018

Condition: **3-ply CLT parapet attachment**

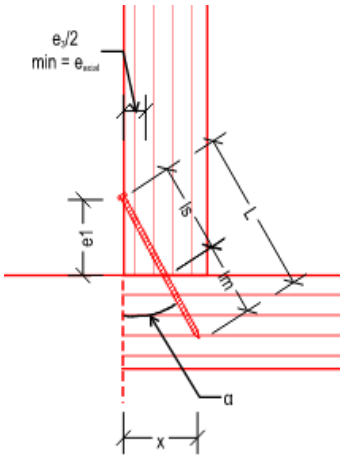
Screw manuf = MTC

screw type = ASSY VG Cyl

D =	0.3125	in	F_{yb} =	132500	psi
L =	10.25	in	V_{all} =	1105	lb
D_r =	0.197	in	T_{all} =	1775	lb
a =	0.551	in	G =	0.42	
L_t =	0.315	in	W_{90} =	212	lb/in

Toe Screw at "T" Joint

	α =	22	deg		
	e1 =	5	in	min 15D =	4.69 in
(ls+lm/2)sin(α)	e2 =	2.93	in	min 7D =	2.19 in
ls*sin(α)	e3 =	2.02	in	min 2*3D =	1.88 in
L*sin(α)	x =	3.84	in		



	side	main	
l_s or l_m =	5.39	4.86	in, ls=e1/cos(α) lm=L-ls
G =	0.42	0.42	
W_{90} =	212	212	lb/in
angle to grain =	22	90	
Rα =	0.75	1.00	(per MTC design guide)
C_d =	1.6	1.6	
p_s or p_m =	4.84	4.54	in, ps=ls-a pm=lm-lt
.9W'p =	1109	1387	lb T_{all} = 1775 lb

governing value, W'p = 1109 lb (side member)

W'p*sin(α) = 415 lb, allowable screw "shear" at joint (horizontal component)

W'p*cos(α) = 1028 lb, allowable screw "tension" at joint (vertical component)

SQ axial = 2.5D = 0.78 in (min screw spacing along CLT)

Demand, T = 2838 lb/ft

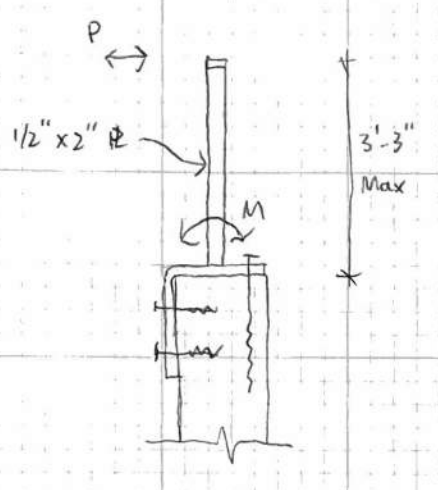
s_{req} = 4.35 in

.: use 5/16"ø x 10.25" ASSY VG Cyl screws @ 4"OC

DCI ENGINEERS	Project No.	Sheet No.
Project Wilson		Date
Subject		By

Steel guardrail attachment to 3-ply CLT

per detail 2/S50.04



P = 200 lb (50 plf load not applicable, per ASCE 7 sec 4.5.1.1 for one & two family dwellings)

since only the 200 lb is applicable, assume the continuous top rail provides load share to adjacent vertical stanchions, such that the max load to the base of a stanchion is 60% of the 200 lb

$M = .60(200\text{ lb})(39") = 4680\text{ lb-in}$

Steel rail

$M_u = 1.6(4680\text{ lb-in}) = 7488\text{ lb-in}$

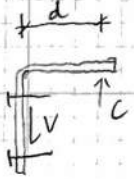
$\phi M_n = .9(36\text{ ksi})\frac{(.5")(2")^2}{4} = 16{,}200\text{ lb-in} > M_u$ ✓

for a 1/4" fillet weld all around, $S = bd + \frac{d^2}{3} = .5"(2") + \frac{(2")^2}{3} = 2.33\text{ in}^2$ (weld treated as line)

$\phi M_n = 1.392D(S) = 1.392(4)(2.33\text{ in}^2) = 13.0\text{ k-in}$

Connection to CLT

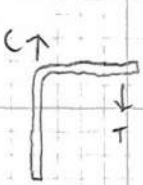
load away (screws in shear)



$V_a = T/C = \frac{M}{d} = \frac{4680\text{ lb-in}}{.9(4.125")} = 1261\text{ lb}$

for (4) SDS screws,
$Z' = 4(420\text{ lb}) = 1680\text{ lb} > V_a$ ✓

load inwards (lag screw in tension)



$T_a = \frac{4680\text{ lb-in}}{(4.125" - 1")} = 1498\text{ lb}$

for (2) 1/4"ϕ x 6" fully threaded lag screws:

$W' = 2(.75)(179\text{ lb/in})(6" - 1/4" - 5/32")$
(C_{eg} for $G = .42$)
$= 1502\text{ lb} > T_a$ ✓

Wood Beam

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ENERCALC, INC. 1983-2018, Build:10.18.1.31, Ver:10.18.1.31

Lic. # : KW-06001666 — Licensee : DCI ENGINEERS

Description : 3x decking (DF commercial grade)

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : ASCE 7-16

Material Properties

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

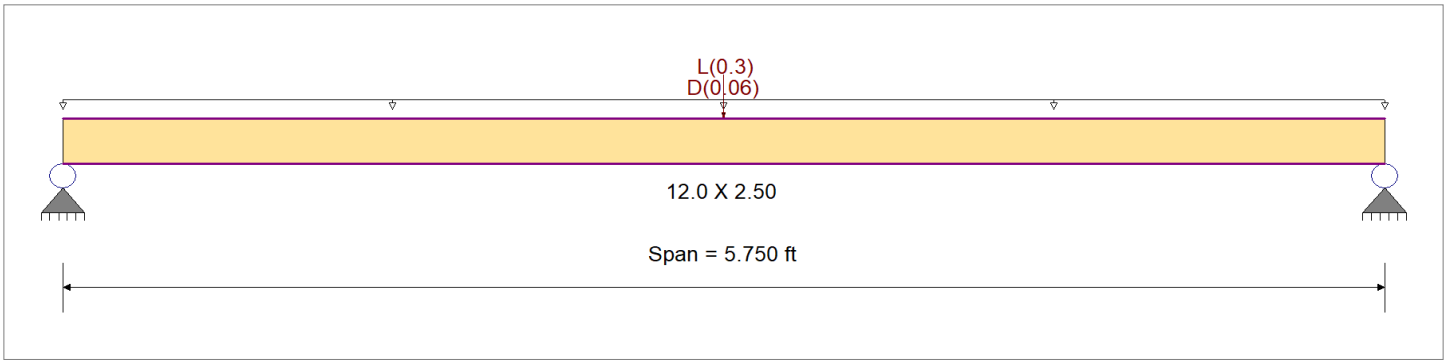
Wood Species : DF larch commercial decking
Wood Grade : Manufactured

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

Fb +	1650 psi
Fb -	1650 psi
Fc - Prll	1350 psi
Fc - Perp	625 psi
Fv	180 psi
Ft	575 psi

E : Modulus of Elasticity

Ebend- xx	1700 ksi
Eminbend - xx	620 ksi
Density	35 pcf



Applied Loads

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

Beam self weight calculated and added to loads

Uniform Load : D = 0.060 , Tributary Width = 1.0 ft, (stone entry)

Point Load : L = 0.30 k @ 2.875 ft, (point live load)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.413 : 1	Maximum Shear Stress Ratio	=	0.092 : 1
Section used for this span		12.0 X 2.50	Section used for this span		12.0 X 2.50
fb : Actual	=	680.98 psi	fv : Actual	=	16.54 psi
FB : Allowable	=	1,650.00 psi	Fv : Allowable	=	180.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	2.875 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	0.078 in	Ratio =	887 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.140 in	Ratio =	491 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240