ANNÉE STRUCTURAL ENGINEERING, LLC

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September 21, 2023

Jacob Halverson
City of Mercer Island
9611 SE 36th St
Mercer Island, WA 98040

Project: Madrona Crest Lot 9, 3605 86th Ave SE, Permit # 2306-185

Jacob:

I have reviewed the SDCI review correction notice and have the following responses to the Structural items. Please note that the numbers of my responses correspond to the numbers on the correction sheet.

Structural

General

1. Special inspection is required for post-installed anchors per Sheet S1.0. Please update the Mercer Island Cover Sheet to identify the special inspector and phone number with this requirement at epoxy-grouted and expansion anchors.

Response: by others

2. Please check off Connector Plate Wood Trusses under Deferred Submittals on the Mercer Island Cover Sheet.

Response: by others

3. The Structural Notes, Sheet S1.0, should be updated as noted below:
a. Wind Exposure D is noted on the drawings, but Exposure B was used for design.
b. Site Class E is noted on the drawings, but Site Class D was used for design.
c. The Seismic Design Base Shear noted on the drawings is 18k. This does not appear to coordinate with the actual design. Page 2 shows a total shear force of 13,988# for consideration at the roof and upper floor. Page 5 of the calculations shows an additional force at the lower floor but does not add the lower floor to evaluation (nor should it). It appears the design base shear (ASD value) is 13.99k. Please clarify.

Response: Wind Exposure is Category B, Site Class is D, the design base shear for the Roof and Upper Floor Diaphragms and tributary walls is 14.0k. This information has been corrected on Sheet S1.0. The Design Base Shear of 18.0k included the seismic weight of the Main Floor Diaphragm which is located at the top of the Concrete foundation walls or at h_i of 0.0 ft.

Gravity

4. The Prefabricated Connector Plate Wood Trusses notes on Sheet S2.2 identify the design criteria for the truss manufacturer. There is a top chord snow load of 35 psf identified on this sheet; is this really intended? The snow load is noted as 25 psf on Sheet S1.0 of the Structural Notes. Please resolve discrepancy.

Response: The design snow load is 25 psf, there are no requirements for drifting or unbalanced snow loads, this information has been corrected on Sheet S2.2.

5. Detail 2/S3.0 is cut at two locations between Grids 3 & 4 on the Roof Framing Plan, Sheet S2.2, at the scissor trusses. Is Detail 3/S3.0 actually intended?

Response: While there are vaults in the Scissor Truss profiles the bottom chord is level at these locations so detail 2/S3.0. is deemed more appropriate. Reference section 2/A4.0.

6. Detail 1/S3.1 is cut at porch roofs on Sheet S2.1. Verify uplift connection capacity.

Response: See the attached calculations. The designed connection is within 6% of allowable steel stresses so it is acceptable in my professional opinion.

7. Provide adequate reinforcing in the concrete plinth to confine reinforcing and anchor bolts at the top of the column in Details 11 & 12/S3.1 per ACI 318 10.7.6.1.6. Transverse reinforcement must be distributed within 5" of the top of the column or pedestal and must consist of at least (2) #4 or (3) #3 bars.

Response: The plinth in details 11 & 12/S3.1 can be defined as pedestals per ACI 318-14 Chapter 14 and can be designed with plain concrete. The pedestal should see no flexural load and the reinforcement is for serviceability issues such as reducing cracking and spalling over time due to exposure. I have added a max. height dimension to 11 & 12/S3.1 for clarity so that these items are defined as a pedestal.

8. Detail 1/S3.0 is cut on Sheet S2.0 in the middle of the garage. Please clarify intent.

Response: Cut and paste error, this reference has been removed from sheet S2.0.

9. Only sleeping areas can be designed with a 30 psf live load per IBC Table 1607.1, all other residential areas must use a live load of 40 psf. The typical upper floor TJI joist calculations only assume a 30 psf live load. It appears additional evaluation is needed for areas other than sleeping areas. See page 12 of the calculations. Verify other locations considered a live load of 40 psf at non-sleeping areas.

Response: See the attached calculations demonstrating that the joists can support 40 psf for the entire span if required.

10. Sheet S2.1 calls out typical LSLs which are 1-3/4 x 11-7/8 LSLs. Beam 8, identified on page 8 of the calculations, is designed as a 3-1/2 x 11-7/8 LSL per page 14 of the calculations. Please revise the callout on the Upper Floor Framing Plan.

Response: This beam call-out has been revised on Sheet S2.1 to match the required beam in the calculations.

11. Detail 1/S3.0 is cut at the isolated footing at the south patio along Grid 1 on Sheet S2.0. Is detail 11/S3.1 really intended?

Response: Sheet S2.0 has been revised to show detail 11/S3.1 at that location.

<u>Lateral</u>

12. Shear wall capacities in the Shear Wall Schedule 1/S1.1 seems off. SDPWS Table 4.3A shows the following capacities (plf) for HF framing: SW6 242 seismic/339 wind, SW4 353 seismic/485 wind, and SW3 456 seismic/637 wind. Considering the other connections in top plate nailing, A35 Clips, and anchor bolts, the seismic capacity appears to govern the design. Please update capacities in the schedule and verify lateral design is still in compliance.

Response: Per Note 1 in the SW schedule, 1/S1.1, the required shear wall sheathing is Struct-I plywood or OSB therefore the Struct-I row in SDPWS Table 4.3A was used to provide allowable shear values. The values have been double-checked and are in accordance with the code. The lateral design has been checked and is in compliance with the code.

13. Note 1 in the Shear Wall Schedule 1/S1.1 refers to the size and spacing of intermediate nails. All shear walls have 0.131 panel edge nailing, but this note specifies 0.113 nails for intermediate nailing. Is this really intended? If so, justify shear wall capacities.

Response: Note 2 in 1/S1.1 has been revised to provide clarity.

14. Where the required nominal unit shear capacity, vs, exceeds 700 plf or nail spacing of 2" or less is specified, 3x framing members and blocking must be provided at adjoining panel edges and nails must be staggered per SDPWS 4.3.7.1, Item 5. Alternatively, (2)2x's can be used where they are fastened together with fasteners designed to transfer the induced shear between members. Apply this requirement to the Shear Wall Schedule.

Response: Unit shears do not exceed 700 plf, nor is the panel edge nailing spaced at 2" or less so this requirement does not pertain to this project.

15. Holdown capacities appear somewhat off in the Holdown Schedule 4/S1.1. It appears the capacities should be reduced to 2,215# at HDU2 and 4,340# at HDU5. Please update capacity in schedule and verify lateral design is still in compliance. It appears HDU5 holdowns would typically be undersized based on the uplift loads specified on page 6 of the calculations.

Response: The Holdown Schedule, 4/S1.1 has been revised to show the lowered capacity at the HDU2 and to require Doug-Fir framing at the HDU5 holdowns, thereby increasing its listed capacity.

16. Horizontal diaphragm shear forces must be distributed to the vertical resisting elements based on tributary area for flexible diaphragms per SDPWS 4.2.5. Where offsets in reaction lines exceed 5', they should be evaluated as separate reaction lines. For example, the west wall of Bed 2 should be a separate reaction line at the upper floor which may require the shear walls along Grid C to be increased in capacity to an SW4. Please evaluate.

Response: Shear walls at grid line C at the Upper Floor Walls (ref. S2.2) would see approx. 49% of the seismic load, therefore v = .49*7,961#/(10.8' + 4.8') = 247 plf < 260 plf for SW6 (as discussed above – see #12). Therefore, the walls remain SW6 with either analysis. For future reference can you inform me where to find the 5' offset restriction in the code?

17. The typical gable-end condition in Detail 1/S3.0 only provides shear flow connection from the roof sheathing through the H1 clips at 48" on center before the wall sheathing is connected to the top chord of the gable end truss. Provide a connection sufficient for lateral load transfer at all shear wall conditions.

Response: Worst case shear transfer thru the H1 clips is at lateral line 5/6: 4,795#/43.54' = 110#/ft. H1 clips provide 440#/4.0' = 110#/ft. This detail is adequate to transfer the required design load.

18. It appears that holdowns are missing from both ends of the 4' long shear wall along Grid 2 at the south wall of the Family Room, Sheet S2.0. See page 6 of the calculations.

Response: These holdowns have been added to the plan, see Sheet S2.0.

19. Please verify that the CS16 holdown straps have sufficient capacity for the shear walls along Grids B/C on the Upper Floor Framing Plan, Sheet S2.1. Where strapping to an LSL, it appears these straps would not have a sufficient end length to get the full capacity. At this location, Detail 1/S3.2 is cut which shows an MSTC48B3 strap instead of a CS16 strap. Please resolve discrepancy.

Response: The straps along this lateral line have been revised to MSTC40 for walls and deep beams, or to 1/S3.2 at the 11-7/8" LSL beam, see revised Sheet S2.1.

20. Does Detail 2/S3.2 apply to this project? If so, please cut in plan.

Response: The strap call-outs on Sheet S2.1 along grids 2 & 5 have been revised to reference this detail where applicable. 2/S3.2 has been revised to reference the MSTC48BC strap to match the required over-turning forces in the calculations.

21. Per ASCE 12.10.2, collector elements must be provided that are capable of transferring the seismic forces originating in other portions of the structure to the element providing the resistance to those forces. It appears that a drag member and connector may need to be provided along Grid C on the Upper Floor Framing Plan, Sheet S2.1. Please verify.

Response: Beam #8 along grid C on the Upper Floor Framing (ref. S2.1) has been denoted as a Drag Strut ('DS') per Floor Framing Note #4 on Sheet S2.1 and a DSC2 connector has been indicated on the plans to transfer the load to the shear walls farther along that lateral line.

22. How are lateral loads resolved at porch roofs?

Response: See the attached calculations for the required force to resist rotation the low porch diaphragms. Connector notes have been added at each low roof porch framing, see S2.1.

23. Provide an evaluation of horizontal diaphragms to show adequacy per IBC 2305.1, SDPWS 4.2.1, and ASCE 12.10.1. Diaphragms must be designed for both the shear and bending stresses resulting from design forces. At diaphragm discontinuities, such as openings and re-entrant corners, the design must assure that the dissipation or transfer of edge (chord) forces combined with other forces in the diaphragm is within shear and tension capacity of the diaphragm.

Response: See the attached calculations for calculation of diaphragm shears and force transfer around the opening. Connectors have been added to the corners of the opening of the diaphragm opening, see Sheet S2.1.

24. Floor framing notes on Sheets S2.0 & S2.1 call for 3/4″ sheathing with 0.113 nails at 6″ on center at panel edges and 12″ on center at intermediate framing. Will these nails be sufficient for diaphragm shears?

Response: The maximum diaphragm shear is 73.2 plf < 140 plf (SDPWS, Table 4.2C. 6d nails, Case 4) therefore the 0.113″ nails at 6″ and 12″ are adequate.

25. Floor framing note 5 refers to Detail 2/S1.1 for information at the CS16 strap. Should this be Detail 1/S1.1?

Response: The Strap Schedule and Top Plate Splice detail have been re-named to 2/S1.1 and 3/S1.1 respectively to align with the General Framing Notes and to avoid conflict with the Shear Wall Schedule (1/S1.1).

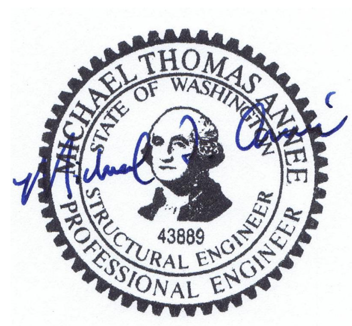
26. Redundancy was assumed to equal 1.0. Please justify by providing an evaluation per the parameters in ASCE 12.3.4.

Response: The only lateral line that does not meet the definition of two bays of seismic-force resisting perimeter framing and that also has wall piers with a H:W of greater than 1:1 is Line 2 at the Main Floor walls where the loss of the 4.4′ wall would result in a loss of 22% capacity along that line. By inspection this would not trigger an extreme torsional irregularity.

If you have any questions concerning the responses or require additional information, please contact me via e-mail (mike@anneestructural.com) or phone (206-658-5169).

Sincerely,

Michael T. Année, SE
Année Structural Engineering, LLC



(6) $W = 414^{\#}$/SCREW

$Z = 186^{\#}$/SCREW → SEE ATTACHED CALC.

→ SEE ATTACHED CALC.

UPLIFT @ RAFTERS AT SOUTH COVERED ROOF (WORST CASE) $= 273^{\#} \leq (2)\,186^{\#}$

$M = 273^{\#}(1" - 0.125") = 239$ IN-#

$S = \frac{239}{0.6(36{,}000)} = 0.011\ in^3$; $S_{1/8" \times 2" \times 2" \times 4"\ R} = \frac{4(0.125)^2}{6} = 0.0104\ in^3$ ∴ WITHIN 6% ∴ OK

(22) WIND @ S. PORCH ROOF; $W = 65(7.1-(-4.5)) + 14(18.6) = 1{,}014^{\#}$

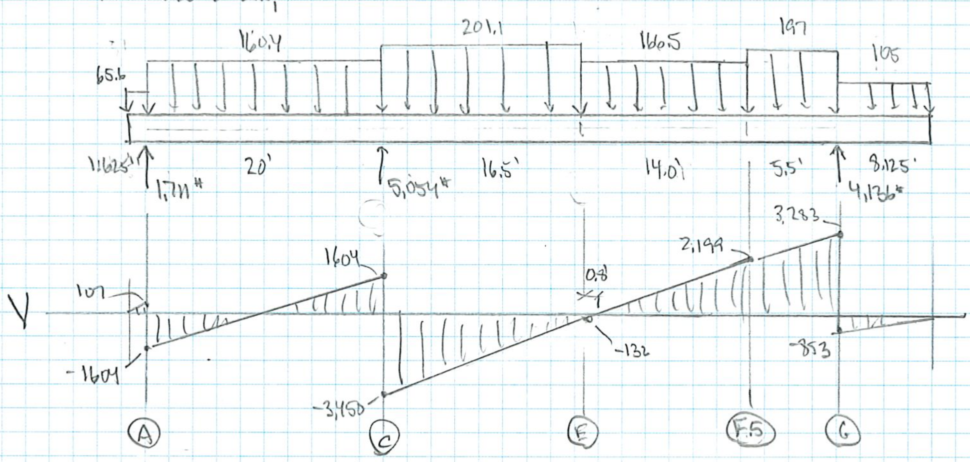
$E = 10\%(6{,}127^{\#}) = 613^{\#}$ ∴ WIND CONTROLS

$T = C = 1{,}014^{\#}\left(\frac{12.125'}{2}\right)/27.46' = 224^{\#}$ ∴ H2.5 @ EACH END OF EACH ROOF

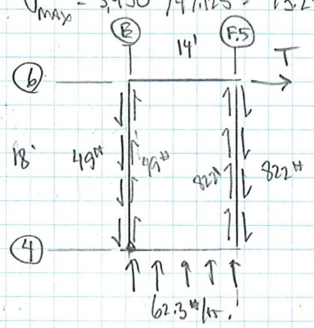
(23) UPPER FLOOR DIAPHRAGM SHEAR DIAGRAMS; $F_p = 15{,}573^{\#} \times 0.7 = 10{,}901^{\#}$ (ASD)   3,273 ft²

62.4 N
104.1 S

TRANSVERSE DIR, N-S



$V_{MAX} = 3{,}450^{\#}/47.125' = 73.2^{\#}/ft. \leq 140^{\#}/ft.$ ∴ UNBLOCKED DIAPHRAGM OK



$\Sigma M_{B4} = 0; -822^{\#}(14') + 62.3\left(\frac{14^2}{2}\right) + T(18') = 0; T = 979^{\#}$

AT E, F.5, 1, 3: $T = \frac{1{,}377^{\#}(14') + 104.2\left(\frac{14^2}{2}\right)}{30.125'} = 979^{\#}$ ∴ H6 or DSC2 @ EA. CORNER

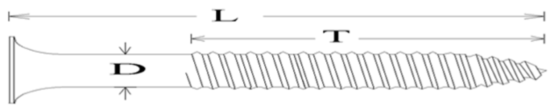
WOOD SCREW SINGLE SHEAR WOOD-WOOD or WOOD-METAL and COMBINED WITHDRAWAL

DFL = Douglas Fir-Larch
DFS = Douglas Fir-South
HF = Hem-Fir

Main Member:	
WOOD species:	HF
Fem	3500 psi
tm	5.5 inch

Side Member:			
WOOD species:	DFL	***METAL***	
Fes	4650 psi	Fes or Fu	58000 psi
ts	0 inch	ts	0.125 inch

Wood Screw:	
Gage	10 g
Dr	0.1520 inch
Fyb	80000 psi



L = Screw length
T = Thread length
D = Unthreaded shank diameter
S = Unthreaded shank length
N = Number of threads per inch

Re	0.060
K(D)	2.200
k3	8.031

Combined Lateral and Withdrawal:

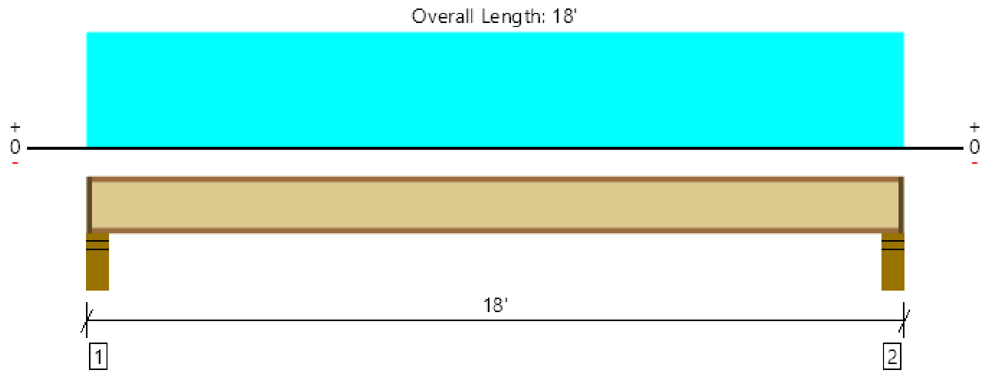
			alpha	0 degrees	
Z =	**118 lbs**	**shear**	penetration into main member	1.5 inches	0.912 in min
W =	**100 lbs/in**	**withdrawal**	thread length in main member	1 inches	

Single Shear (two member)	***Z' =***	***186***	**lbs**
Lag Screw Withdrawal	***W' =***	***160***	**lbs**
Combined Lateral and Withdrawal Design Value	***Z a' =***	***186***	**lbs**

Z	failure mode								
0	Mode Is	Adjustment Factor	C D	C M	C t	C g	C delta	C d	C eg
118	Mode IIIs	Z	1.60	1.00	1.00	1.00	1.00	0.99	1.00
139	Mode IV	W	1.60	1.00	1.00				

Notes:
1. Metal side members: Fu = 58000psi for ASTM A36 steel, and Fu = 45,000psi for ASTM A446 Grade A steel.
2. Depth factor, Cd, (NDS 9.3.3) based on penetration into main member of p = 10 * shank diameter of screw, with minimur
3. Thread length is approximately 2/3 the total wood screw length.

Upper Floor Framing, 4 - Joist
1 piece(s) 11 7/8" TJI® 210 @ 16" OC



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	617 @ 4 1/2"	1460 (3.50")	Passed (42%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	592 @ 5 1/2"	1655	Passed (36%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2579 @ 9'	3795	Passed (68%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.310 @ 9'	0.431	Passed (L/668)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.403 @ 9'	0.863	Passed (L/514)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	46	40	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Weyerhaeuser Edge™ Panel (24" Span Rating) that is glued and nailed down.
- Additional considerations for the TJ-Pro™ Rating include: 5/8" Gypsum ceiling.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Stud wall - HF	5.50"	4.25"	1.75"	144	480	624	1 1/4" Rim Board
2 - Stud wall - HF	5.50"	4.25"	1.75"	144	480	624	1 1/4" Rim Board

- Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	4' 6" o/c	
Bottom Edge (Lu)	17' 10" o/c	

- TJI joists are only analyzed using Maximum Allowable bracing solutions.
- Maximum allowable bracing intervals based on applied load.

Vertical Load	Location	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 18'	16"	12.0	40.0	Default Load

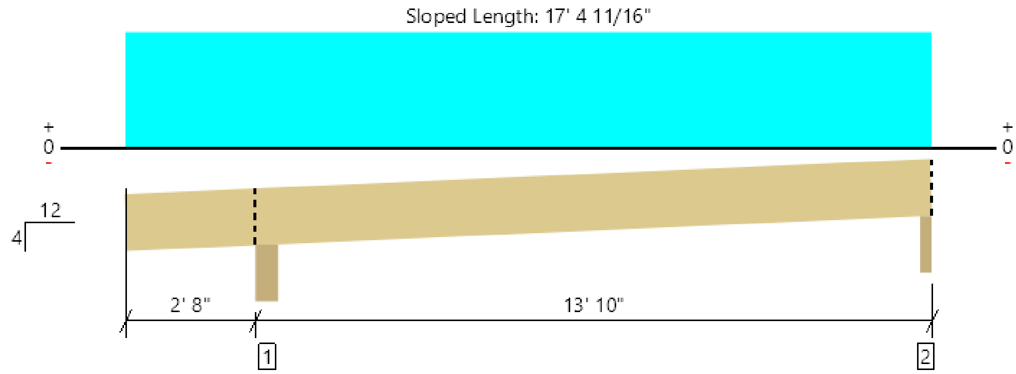
Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Mike Annee Annee Structural Engineering LLC (206) 658-5169 mike@anneestructural.com	

MEMBER REPORT

PASSED

Upper Floor Framing, 12 - Rafters
1 piece(s) 2 x 10 HF No.2 @ 24" OC



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Member Length : 17' 7 13/16"

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	544 @ 16' 4 1/4"	1671 (2.75")	Passed (33%)	--	1.0 D + 1.0 S (Alt Spans)
Shear (lbs)	496 @ 3' 10 1/4"	1596	Passed (31%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	1731 @ 9' 10 1/16"	2204	Passed (79%)	1.15	1.0 D + 1.0 S (Alt Spans)
Live Load Defl. (in)	0.204 @ 0	0.305	Passed (2L/360)	--	1.0 D + 0.6 W (Alt Spans)
Total Load Defl. (in)	0.480 @ 9' 8 7/16"	0.946	Passed (L/354)	--	1.0 D + 1.0 S (Alt Spans)

System : Roof
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 4/12

- Deflection criteria: LL (L/240) and TL (L/180).
- Overhang deflection criteria: LL (2L/240) and TL (2L/180).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.
- -273 lbs uplift at support located at 2' 10 3/4". Strapping or other restraint may be required.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Snow	Wind	Factored	
1 - Beveled Plate - DF	5.50"	5.50"	1.50"	314	497	-768	811/-273	Blocking
2 - Beveled Plate - DF	2.75"	2.75"	1.50"	208	336	24/-532	544/-194	Blocking

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	4' 10" o/c	
Bottom Edge (Lu)	16' 6" o/c	

- Maximum allowable bracing intervals based on applied load.

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Snow (1.15)	Wind (1.60)	Comments
1 - Uniform (PSF)	0 to 16' 6"	24"	15.0	25.0	-38.7	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Mike Annee Annee Structural Engineering LLC (206) 658-5169 mike@anneestructural.com	