PDC-TR 18-02
September 2018

US Army Corps
of Engineers®

Protective Design Center Technical Report

Analysis Guidance for Cross-Laminated Timber Construction Exposed to Airblast Loading

Prepared for
USACE Protective Design Center
Omaha District

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1. **Background.** Protective Design Center Technical Report (PDC-TR) 06-08 defines quantitative response limits to correlate the output of single-degree-of-freedom (SDOF) dynamic analyses with one of four qualitative levels of protection (LOPs) [1]. These response limits are used to analyze Department of Defense (DoD) buildings of various types of construction for airblast loading deriving from terrorist threats. Presently, PDC-TR 06-08 does not define response limits for cross-laminated timber (CLT) construction.

2. **Purpose.** This PDC-TR provides analysis guidance for CLT construction exposed to airblast loads deriving from terrorist threats. As part of this analysis guidance, quantitative response limits for use with SDOF dynamic analysis models are defined.

3. **Applicability.** All DoD facilities.

   3.1. **Ultimate Recipients.** All major command (MAJCOM) civil engineer offices, base civil engineers (BCE), and responsible U.S. Army Corps of Engineers (USACE) and Naval Facilities Engineering Command (NAVFACENGCOM) offices acting as design/construction agents for DoD projects or facilities on DoD property.

   3.2. **Loading Range.** The analysis guidance included in this PDC-TR is applicable to CLT construction exposed to airblast loads deriving from far-field explosions associated with terrorism. Far-field explosions generate a planar shock front and are characterized by incident overpressures of less than 200 psi [2]. Close-in and direct hit loading ranges are outside the scope of this PDC-TR.

   3.3. **Analysis Method.** The analysis guidance included in this PDC-TR assumes the use of SDOF dynamic analysis methods.

   3.4. **Axial Load.** The analysis guidance included in this PDC-TR is intended for CLT construction where the actual compression stress parallel to grain, f_c, is less than 50 percent of the average expected dynamic compression design value parallel to grain, F_{dc} (see Section 8.2).

4. **Acronyms.**

APA	The Engineered Wood Association
ASD	Allowable Stress Design
BC	Boundary Condition
BCE	Base Civil Engineers
CLT	Cross-Laminated Timber
COV	Coefficient of Variation

DFL	Douglas Fir-Larch
DIF	Dynamic Increase Factor
DoD	Department of Defense
LOP	Level of Protection
MAJCOM	Major Command
MC	Moisture Content
MOR	Modulus of Rupture
MSR	Machine Stress Rated
NAVFACENGCOM	Naval Facilities Engineering Command
NDS	National Design Specification for Wood Construction
PDC-TR	Protective Design Center Technical Report
SDOF	Single-Degree-of-Freedom
SIF	Static Increase Factor
SPF	Spruce-Pine-Fir
UFGS	Unified Facilities Guide Specifications
USACE	U.S. Army Corps of Engineers

5. CLT Overview.

5.1. Description. CLT is an engineered wood panel that consists of multiple layers (i.e., plies) of dimensional lumber boards aligned edge to edge, stacked orthogonally, and bonded on their wide faces with structural adhesives. As depicted in Figure 1, the orientation of the outermost panel plies is termed the "major strength direction" and that of the crosswise panel plies is termed the "minor strength direction". CLT panels can be constructed using a variety of ply numbers (and ply thicknesses). Two grade classifications exist for CLT panels certified in accordance with ANSI/APA PRG 320 [4]: (1) "E" or engineered, which utilizes machine stress rated (MSR) lumber in the major strength direction and (2) "V" or visually-graded, which utilizes visually-graded lumber in the major strength direction. The specific combination of ply number, ply thickness(es), lumber species, and lumber grading is referred to as the CLT panel's "layup".

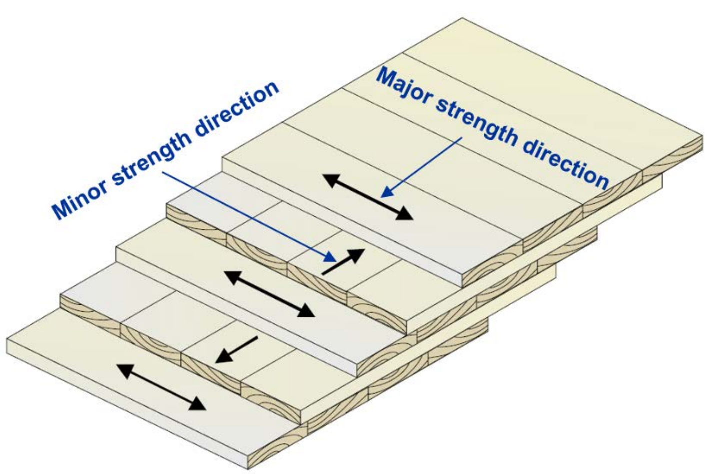


Figure 1. CLT Panel Illustration [5].

5.2. Standards & References.

5.2.1. ANSI/APA PRG 320-2018 [4]. ANSI/APA PRG 320-2018 *Standard for Performance Rated Cross-Laminated Timber* (hereinafter referred to as PRG 320) provides dimensions and tolerances, performance requirements, test methods, quality assurance, and trademarking for CLT panels. In addition, Annex A of PRG 320 defines the layups for four "E" grades and three "V" grades and includes allowable stress design (ASD) reference design values based on the Shear Analogy Method [5] for each grade. Custom grades not listed in Annex A are permitted provided they meet the requirements of Section 7.2.1 of PRG 320.

5.2.2. 2018 NDS [3]. The 2018 edition of the *National Design Specification for Wood Construction* (NDS) defines adjustment factors for CLT panel and connection reference design values.

5.2.3. CLT Handbook [5]. While the *CLT Handbook* is not a standard, it serves as a central repository of information related to CLT analysis, design, and construction.

5.3. Specification. Section 06 17 19 of the Unified Facilities Guide Specifications (UFGS) serves as the guide specification for the fabrication and erection of CLT panels for walls, floors, roofs, partitions, and all metal shapes and hardware required for their installation [6].

6. State of Research. Most research investigating the response of CLT construction to dynamic loads has focused on resisting seismic loads. As such, there are significant testing efforts devoted to investigating the in-plane shear response of CLT panels, their associated connections, and the global response of CLT structures to ground shaking [5].

In addition, several testing efforts have been performed to investigate the flatwise bending response of CLT construction and serve as the basis for the guidance presented in this PDC-TR:

- Quasi-static laboratory tests on Grades V1 (3-ply and 5-ply), E1 (3-ply), and SL-V4 (3-ply) CLT panels both with and without axial load were performed to investigate the post-peak flatwise bending response of CLT panels in their major strength direction under a uniformly-applied quasi-static load [7,8].

- Shock tube tests were performed to investigate the flatwise bending response of 3-ply, 5-ply, and 7-ply Grade E1 CLT panels to dynamic loads at BakerRisk [9,10] and the University of Ottawa [11].
- A series of blast tests were performed on three full-scale CLT structures

(i.e., constructed out of Grades E1, V1, and SL-V4 CLT) with and without superimposed load to investigate the ability of CLT construction to resist airblast loads [8,12].

7. Material.

7.1. Reference Design Values. The ASD reference design values for both the major and minor strength directions of a selection of layups certified by The Engineered Wood Association (APA) in accordance with PRG 320 [13,14,15,16] are included in Appendix A for illustrative purposes. The analyst shall obtain CLT panel information used for design directly from a CLT manufacturer's literature or code evaluation report per Section 10.2 of the NDS [3].

7.2. Static Increase Factor. The static increase factor (SIF) is used to transform ASD reference design values into average expected design values assuming normal load duration. The SIF for CLT panels varies depending on the reference design value, lumber grading (i.e., visual or MSR) of plies in the span direction, and lumber species.

Equation (1) indicates the subfactors needed to compute the SIF for CLT panels and Table 1 and Table 2 define the necessary subfactors. From Table 1, it can been seen that the SIF applied to the effective ASD reference flatwise bending moment value, $(F_bS)_{eff}$, varies based on the span direction of concern whereas the ASD reference flatwise shear value, V_s, and ASD reference compression value parallel to grain, F_c, do not. In addition to the SIF shown in Equation (1), reference design values shall be multiplied by the applicable adjustment factors required by the NDS as indicated in Section 8. Commentary concerning the derivation of CLT SIFs is included in Appendix B.

$$SIF = K_{char} * K_{avg} * K_{size} \quad (1)$$

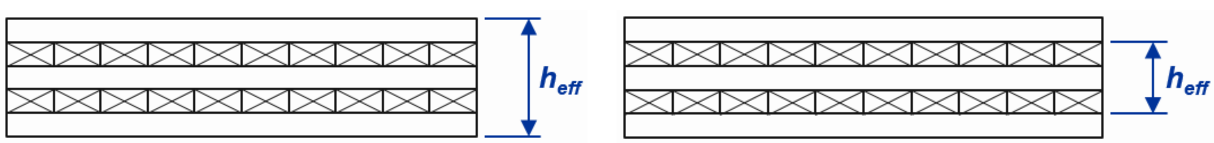
where:
- K_{char} = Subfactor to transform *ASD reference design* value into *characteristic* value
- K_{avg} = Subfactor to transform *characteristic* value into *50th percentile* value
- K_{size} = Subfactor to account for lumber size effect

Table 1. Subfactors Used to Determine CLT SIF.

Factor	Design Value	Lumber Grading	
		MSR	*Visual*
K_{char}	$(F_bS)_{eff}$	1.30	
	F_c	1.20	
	V_s	2.00	
K_{avg}	$(F_bS)_{eff}$	1.35	See Table 2
	F_c	1.40	
	V_s	1.30	
K_{size} [1,2]	$(F_bS)_{eff}$	$(3.5 / h_{eff})^{0.29}$ $K_{size} \le 1.10$	$(11.25 / h_{eff})^{0.29}$ $K_{size} \le 1.65$
	F_c	1.00	
	V_s	1.00	

1 K_{size} is computed based on lumber grading type of the plies oriented in the span direction of concern.

2 h_{eff} is equal to the effective panel thickness *in inches* in the span direction of concern. For example, a 3-ply panel with 1.375-inch thick plies would have a h_{eff} of 4.125 if the panel was spanning in the major strength direction and a h_{eff} of 1.375 if the panel was spanning in the minor strength direction. Similarly, a 5-ply panel with 1.375-inch plies would have a h_{eff} of 6.875 if the panel was spanning in the major strength direction and a h_{eff} of 4.125 if the panel was spanning in the minor strength direction (see Figure 2).



(a) Panel spanning in Major Strength Direction.　*(b) Panel spanning in Minor Strength Direction.*

Figure 2. h_{eff} Example for 5-Ply CLT Panel.

Table 2. K_{avg} to be Applied to $(F_bS)_{eff}$ for Visually Graded Lumber Plies.

Species	Commercial Grade	K_{avg}
Douglas Fir-Larch	No. 2 ; No. 3	2.30
Eastern Softwoods, Northern Species, or Western Woods		2.05
Southern Pine		2.30
Spruce-Pine-Fir	No. 1/No. 2 ; No. 3	2.10
Spruce-Pine-Fir (South)	No. 2 ; No. 3	2.05
Other No. 2 or No. 3 visually graded lumber		2.05
Other visually graded lumber		1.35

7.3. Dynamic Increase Factor. The dynamic increase factor (DIF) is used to increase average expected design values based on normal load duration to account for the strength enhancement effects associated with increasing strain rate. A DIF of 2.0 shall be applied to $(F_bS)_{eff}$, V_s, and F_c. No DIF shall be applied to $(EI)_{eff}$ and $(GA)_{eff}$. Commentary concerning the derivation of this DIF is included in Appendix B.

8. Resistance Function. The resistance function used in SDOF dynamic analysis for a CLT panel subjected to out-of-plane airblast loading shall be computed assuming one-way action. An idealized bilinear or trilinear function with a perfectly plastic ultimate resistance shall be constructed using the guidance in this section. (Whether or not the resistance function form is bilinear or trilinear will be dictated by the assumed boundary conditions (BCs).) Commentary concerning the idealized CLT panel resistance function is included in Appendix C.

8.1. Stiffness. The apparent bending stiffness, $(EI)_{app}$, shall include both flexural and shear deformations as shown in Equation (2).

$$(EI)_{app} = \frac{(EI)_{eff}}{1 + \frac{K_s (EI)_{eff}}{(GA)_{eff} L^2}} \quad \text{[lb-in}^2\text{/ft]} \qquad (2)$$

where:
$(EI)_{eff}$ = Effective bending stiffness in flatwise bending (see Appendix A) [lb-in^2/ft]
$(GA)_{eff}$ = Effective shear stiffness in flatwise bending (see Appendix A) [lb-in^2/ft]
L = Span [in]
K_s = Shear deformation influence constant (see Table 3)

Table 3. K_s for Different Loading Distributions & Boundary Conditions.

Boundary Conditions	Loading	K_s
Pin-Roller	Uniformly Distributed	11.5
Pin-Roller	Concentrated at Midspan	14.4
Fixed-Fixed	Uniformly Distributed	57.6
Fixed-Fixed	Concentrated at Midspan	57.6
Fixed-Free	Uniformly Distributed	4.8
Fixed-Free	Concentrated at Free-End	3.6

Then, the generic bending deflection equation, Equation (3), may be used to compute a stiffness, k, suitable for use with SDOF analysis.

$$k = C_{adj\_EI} * \frac{(EI)_{app} b_w}{k_b b L^4} \quad \text{[psi/in]} \qquad (3)$$

where:
C_{adj_EI} = Apparent bending stiffness adjustment factors required by Table 10.3.1 of the NDS [3]
b_w = Section width [ft]
b = Loaded tributary width [in]
k_b = Bending influence constant (see Table 4)

Table 4. k_b for Different Loading Distributions & Boundary Conditions.

Boundary Conditions	Loading	k_b
Pin-Roller	Uniformly Distributed	5/384
Pin-Roller	Concentrated at Midspan	1/48
Fixed-Fixed	Uniformly Distributed	1/384
Fixed-Fixed	Concentrated at Midspan	1/192
Fixed-Free	Uniformly Distributed	1/8
Fixed-Free	Concentrated at Free-End	1/3

8.2. Resistance. The ultimate resistance, r_u, shall be based on two limit states: (1) flatwise bending and (2) flatwise shear.

8.2.1. Flatwise Bending without Axial Load. For members with compressive axial loads of less than 10 percent of the average expected dynamic compression design value parallel to grain, F_{dc}, the flatwise bending limit state shall be based on the average expected flatwise bending moment strength without axial load, M_n, computed in accordance with Equation (4).

$$M_n = 0.9 * SIF_b * DIF * C_{adj\_b} * (F_bS)_{eff} \quad \text{[lb-ft/ft]} \tag{4}$$

where:
- SIF_b = Static increase factor for flatwise bending (see Section 7.2)
- DIF = Dynamic increase factor (see Section 7.3)
- C_{adj_b} = Flatwise bending adjustment factors required by Table 10.3.1 of the NDS [3] absent those related to load duration
- $(F_bS)_{eff}$ = ASD reference flatwise bending design value (see Appendix A)

F_{dc} shall be computed in accordance with Equation (5):

$$F_{dc} = SIF_c * DIF * C_{adj\_c} * F_c \quad \text{[psi]} \tag{5}$$

where:
- SIF_c = Static increase factor for compression parallel to grain (see Section 7.2)
- C_{adj_c} = Compression parallel to grain adjustment factors required by Table 10.3.1 of the NDS [3] absent those related to load duration
- F_c = ASD reference compression design value parallel to grain (see Appendix A)

Based on the computed flatwise bending moment strength without axial load, the ultimate resistance for different idealized boundary conditions for one-way spanning members can be computed using Table 10-4 of UFC 3-340-01 [2].

8.2.2. Flatwise Bending with Axial Load. For members with compressive axial loads of less than 50 percent but greater than 10 percent of F_{dc}, the flatwise bending limit state shall be based on the average expected flatwise bending moment strength with axial load, M_{n_axial}, computed in accordance with Equations (6) and (7). These equations are based on Equation 15.4-1 of the NDS [3].

$$M_{n\_axial} = M_n * \left(1 - \frac{P}{P_{cE}}\right)\left[1 - \left(\frac{P}{F_{dc}A_{parallel}}\right)^2\right] - P\Delta\left(1 + 0.234\frac{P}{P_{cE}}\right) \quad \text{[lb-ft/ft]} \qquad (6)$$

$$P_{cE} = \frac{\pi^2 C_{adj\_EI}(EI)_{app}}{L_e^2} \quad \text{[lb/ft]} \qquad (7)$$

where:

	P	=	Axial load [lb/ft]
	$A_{parallel}$	=	Area of cross section of CLT layers with fibers parallel to the load direction (see Appendix A) [in^2/ft]
	Δ	=	Eccentricity of axial load measured perpendicular to the plane of the panel [ft]
	L_e	=	Effective column length (as defined in Section 3.7.1.2 of the NDS [3]) [in]

Based on the computed flatwise bending moment strength with axial load, the ultimate resistance for different idealized boundary conditions for one-way spanning members can be computed using Table 10-4 of UFC 3-340-01 [2].

8.2.3. Flatwise Shear. The shear limit state shall be based on the average expected flatwise shear strength, V_n, computed in accordance with Equation (8).

$$V_n = 0.9 * SIF_s * DIF * C_{adj\_s} * V_s \quad \text{[lb/ft]} \qquad (8)$$

where:

	SIF_s	=	Static increase factor for flatwise shear (see Section 7.2)
	C_{adj_s}	=	Flatwise shear adjustment factors required by Table 10.3.1 of the NDS [3] absent those related to load duration
	V_s	=	ASD reference flatwise shear strength (see Appendix A)

Based on the computed flatwise shear strength, the ultimate resistance for different idealized boundary conditions for one-way spanning members can be computed using Table 10-4 of UFC 3-340-01 [2].

9. Rebound. Blast tests have indicated that CLT panels exposed to airblast loads commonly exhibit larger rebound responses than inbound responses [8,12]. As such, it is important that the negative phase of the airblast load be considered in analysis. Thus, two blast load cases shall be considered for CLT panels exposed to airblast loads: (1) positive-phase-only and (2) positive-plus-negative-phase.

10. Response Limits. Response limits for the flatwise response of CLT panels that are compatible with the LOPs defined in PDC-TR 06-08 [1] are listed in Table 5. Commentary concerning the response limits shown in Table 5 is included in Appendix C.

Table 5. Response Limits for CLT Construction.

Controlling Limit State	B1		B2		B3		B4	
	μ	θ	μ	θ	μ	θ	μ	θ
Flatwise Bending	1.0	-	1.5	-	1.75	-	2	-
Flatwise Shear	1.0	-	1.5	-	1.75	-	2	-

11. Connections. Connection capacity shall be designed to exceed the smaller of the demand imposed by the dynamic reaction force or the demand associated with the ultimate resistance, r_u, of the connected CLT panel.

11.1. Connection Capacities. Connection capacities shall be computed in accordance with relevant material specific building code with all relevant strength reduction factors and/or safety factors applied except as modified in Section 11.2.

11.2. Material Specific Guidance.

11.2.1. CLT. Except as indicated in Sections 11.2.1.1 through 11.2.1.3, connection capacities involving CLT panels shall be determined as specified by the NDS [3]. All fastener spacing requirements included in the NDS shall be adhered to. A load duration factor, C_D, of 2.0 may be assumed.

11.2.1.1. Lateral Design Values. The adjusted lateral design value, *Z'*, for *wood screws* computed in accordance with the NDS and assuming a load duration factor, C_D, of 2.0 may be multiplied by an additional increase factor of 2.5. This increase factor assumes the wood screw is oriented perpendicular with the broad face of the CLT panel and is spaced according to NDS requirements. Commentary concerning this increase factor can be found in Section C-2.3.1 of Appendix C.

11.2.1.2. Withdrawal Design Values. The adjusted withdrawal design value, *W'*, for *wood screws* computed in accordance with the NDS and assuming a load duration factor, C_D, of 2.0 may be multiplied by an increase factor of 1.5. Where the fastener engages all plies of the CLT panel, this increase factor may be increased to 2.0. Commentary concerning this increase factor can be found in Section C-2.3.2 of Appendix C.

11.2.1.3. Ultimate Tested Values. Ultimate capacities of prefabricated connection elements determined via testing and provided by manufacturers must be multiplied by a reduction factor of 0.625. It is assumed that these provided ultimate capacities will contain an effective load duration factor, C_D, of 1.6 and have no safety factor. Commentary concerning this increase factor can be found in Section C-2.3.3 of Appendix C.

11.2.2. Steel. The yield stress of steel, F_y, may be multiplied by the SIF and DIF defined in UFC 3-340-02 [17] when computing connection capacities. The strength reduction factor for limit states involving the yield stress of steel may be set equal to 1.0.

11.2.3. Other. Other building material properties may be multiplied by the applicable SIF and DIF (based on the connected CLT panel strain rate) defined in UFC 3-340-02 [17] when computing connection capacities. For materials outside of the bounds listed in UFC 3-340-02, SIF and DIF shall be set equal to 1.0.

12. Point of Contact (POC). Recommendations for improvements to this PDC-TR are encouraged and should be furnished to:

U.S. Army Corps of Engineers
Protective Design Center
1616 Capitol Avenue
Omaha, NE 68102
Phone: (402) 995-2366
e-mail: pdc.web@usace.army.mil

APPENDIX A
EXAMPLE CLT PANEL PROPERTIES

Table A-1. Lumber & Dimension Properties for Several CLT Layups [1,2].

Grade	No. of Plies	MAJOR STRENGTH DIRECTION				MINOR STRENGTH DIRECTION			
		Lumber Description	***Lumber Grading***	h_{eff} [3] [in]	$A_{parallel}$ [4] [in^2/ft]	***Lumber Description***	***Lumber Grading***	h_{eff} [3] [in]	$A_{parallel}$ [4] [in^2/ft]
E1	3	1950f-1.7E Spruce-Pine-Fir	MSR	4.125	33	No. 3 Spruce-Pine-Fir	Visual	1.375	16.5
	5			6.875	49.5			4.125	33
	7			9.625	66			6.875	49.5
E2	3	1650f-1.5E Douglas Fir-Larch	MSR	4.125	33	No. 3 Douglas Fir-Larch	Visual	1.375	16.5
	5			6.875	49.5			4.125	33
	7			9.625	66			6.875	49.5
V1	3	No. 2 Douglas Fir-Larch	Visual	4.125	33	No. 3 Douglas Fir-Larch	Visual	1.375	16.5
	5			6.875	49.5			4.125	33
	7			9.625	66			6.875	49.5
V2M1.1	3	No. 2 Spruce-Pine-Fir	Visual	4.125	33	No. 2 Spruce-Pine-Fir	Visual	1.375	16.5
	5			6.875	49.5			4.125	33
	7			9.625	66			6.875	49.5
SL-V4	3	No. 2 Spruce-Pine-Fir (South)	Visual	4.125	33	No. 2 Spruce-Pine-Fir (South)	Visual	1.375	16.5
	5			6.875	49.5			4.125	33
	7			9.625	66			6.875	49.5

1 The values shown in this table assume 1.375-inch thick plies stacked in alternating orthogonal directions.

2 The layups included in this table are a selection of the layups certified by APA in accordance with ANSI/APA PRG-320 as of the writing of this PDC-TR. They are included to illustrate the panel information necessary to analyze a CLT panel for airblast loading. Per Section 10.2 of the NDS [3], CLT panel information shall be obtained from the CLT manufacturer's literature or code evaluation report. Code evaluation reports for the layups shown in this table and others can be found at https://www.apawood.org/product-reports.

3 Determined as defined in Section 7.2.

4 Determined as defined in Section 1.6 of the NDS [3].

Table A-2. ASD Reference Design Values for Several CLT Layups [1,2,3].

Grade	No. of Plies	MAJOR STRENGTH DIRECTION						MINOR STRENGTH DIRECTION					
		F_b	F_c	$(F_bS)_{eff}$	$(EI)_{eff}$	$(GA)_{eff}$	V_s	F_b	F_c	$(F_bS)_{eff}$	$(EI)_{eff}$	$(GA)_{eff}$	V_s
		[psi]	[psi]	[lb-ft/ft]	[10^6 lb-in^2/ft]	[10^6 lb/ft]	[lb/ft]	[psi]	[psi]	[lb-ft/ft]	[10^6 lb-in^2/ft]	[10^6 lb/ft]	[lb/ft]
E1	3	1,950	1,800	4,525	115	0.46	1,430	500	650	160	3.1	0.61	495
	5			10,400	440	0.92	1,970			1,370	81	1.2	1,430
	7			18,375	1,089	1.4	2,490			3,125	309	1.8	1,960
E2	3	1,650	1,700	3,825	102	0.53	1,910	525	775	165	3.6	0.56	660
	5			8,825	389	1.1	2,625			1,430	95	1.1	1,910
	7			15,600	963	1.6	3,325			3,275	360	1.7	2,625
V1	3	900	1,350	2,090	108	0.53	1,910	525	775	165	3.6	0.59	660
	5			4,800	415	1.1	2,625			1,430	95	1.2	1,910
	7			8,500	1,027	1.6	3,325			3,275	360	1.8	2,625
V2M1.1	3	875	1,150	2,050	96	0.53	1,490	875	650	280	3.7	0.53	495
	5			4,725	367	1.1	2,480			2,410	96	1.1	1,490
	7			8,350	910	1.6	3,475			5,550	367	1.6	2,480
SL-V4	3	775	1,000	1,800	74	0.41	1,430	775	1,000	245	2.9	0.41	495
	5			4,150	286	0.83	1,980			2,120	74	0.83	1,430
	7			7,325	707	1.2	2,500			4,825	283	1.2	1,960

1 The values shown in this table assume 1.375-inch thick plies stacked in alternating orthogonal directions.

2 The layups included in this table are a selection of the layups certified by APA in accordance with ANSI/APA PRG-320 as of the writing of this PDC-TR. They are included to illustrate the panel information necessary to analyze a CLT panel for airblast loading. Per Section 10.2 of the NDS [3], CLT panel information shall be obtained from the CLT manufacturer's literature or code evaluation report. Code evaluation reports for the layups shown in this table and others can be found at https://www.apawood.org/product-reports.

3 The ASD reference design values shown in this table are as shown in [13,14,15,16].

APPENDIX B
STATIC & DYNAMIC INCREASE FACTORS FOR CLT CONSTRUCTION

B-1. Purpose. This appendix provides commentary concerning the static and dynamic increase factors specified in this PDC-TR.

B-2. Static Increase Factor. The static increase factor is used to transform minimum specified to average expected material properties. In the context of CLT, "minimum specified material properties" are ASD reference design values.

B-2.1. Notes Concerning ASD Reference Design Values. In the United States, ASD reference design values for CLT panels are based on the qualification and mechanical test requirements specified in ANSI/APA PRG 320 [4]. ASD reference design values of interest for the flatwise response of CLT panels include:

- Bending stress, F_b
- Compression stress parallel to grain, F_c
- Effective flatwise bending moment capacity, $(F_bS)_{eff}$
- Effective flatwise bending stiffness, $(EI)_{eff}$
- Effective shear stiffness in flatwise bending, $(GA)_{eff}$
- Flatwise shear capacity, V_s

The F_b and F_c values are based on the species and grade of lumber in the span direction (i.e., the major strength direction or the minor strength direction). The remaining values are computed values based on the species and grade of lumber in the span direction. ASD reference design values for 3-ply, 5-ply, and 7-ply panels of several CLT grades certified by APA in accordance with PRG 320 are included in Appendix A. ASD reference design values for other CLT layups can be found in APA Product Reports located at https://www.apawood.org/product-reports.

The ASD reference design values *for laminations* specified in PRG 320 and APA Product Reports are identical to the corresponding reference design values recorded in Table 4A (visually graded dimension lumber, 2" – 4" thick), Table 4B (visually graded Southern Pine dimension lumber, 2" – 4" thick), and Table 4C (mechanically graded dimension lumber) of the NDS Supplement [18]. There is no reference planar (rolling) shear stress, F_s, in the NDS Supplement; the F_s values shown in PRG 320 are obtained by dividing the reference edgewise shear stress, F_v, by three.

The $(F_bS)_{eff}$, $(EI)_{eff}$, and $(GA)_{eff}$ values are computed using the ASD reference design values for laminations and the Shear Analogy Model as shown in [5]. Two items are important to note concerning these computed values:

- PRG 320 notes that the computed $(F_bS)_{eff}$ values in the major strength direction are "multiplied by a factor of 0.85 for conservatism" [4]. Although the basis for this 0.85 reduction factor is unclear, it appears to have been introduced to account for the difference between the bending strength computed using the Shear Analogy Method and that observed in testing [19]. This phenomenon was observed in both the University of Maine [7] and University of Ottawa [11] quasi-static testing on Grade E1 panels.

- The computed minor strength direction ASD reference design values ignore the contribution of the outermost plies running parallel with the major strength direction when computing $(F_bS)_{eff}$ and $(EI)_{eff}$ but not when computing $(GA)_{eff}$. This simplification appears to result from the gap between the narrow edges of boards in a given ply not being defined in PRG 320. Without such a tolerance limit, it is not possible to derive a relationship for how these outermost plies impact the flexural strength and stiffness of the panel. As such, the ASD reference design values in the minor strength direction should be considered as lower bound values.

B-2.2. Process to Transform ASD Reference Design into Average Expected Values. The following subsections provide commentary on the derivation of the three subfactors (i.e., K_{char}, K_{avg}, and K_{size}) used to generate the CLT panel SIFs.

B-2.2.1. K_{char} Derivation. This factor is defined in Table 1 of PRG 320 and is a function of the stress type. As the factors shown in Table 1 of PRG 320 are used to transform tested values with a ten-minute load duration into reference design values with a normal load duration, they include a load duration factor of 1.6. Thus, to derive K_{char} assuming normal load duration, the factors shown in Table 1 of PRG 320 must be divided by 1.6. Table B-1 records the K_{char} associated with each design value. (The values in Table B-1 are rounded to the nearest 0.05.)

Table B-1. K_{char} by Design Value Type.

Design Value	Factor in Table 1 of PRG 320	K_{char}
$(F_bS)_{eff}$	2.10	**1.30**
F_c	1.90	**1.20**
V_s	3.15	**2.00**

B-2.2.2. K_{avg} Derivation. Table 5-6 of the *Wood Handbook* [20] lists average coefficients of variation (COVs) for some mechanical properties of clear wood. The COVs shown in this table are based on the results of tests of green wood from approximately 50 species. Specific properties of interest include modulus of rupture (MOR) (16%), compression parallel to grain (18%), and shear parallel to grain (14%). As these values are for clear wood (i.e., wood without imperfections such as knots), they represent an average lower bound value for mechanical property variation. Assuming a normal statistical distribution and an infinite number of samples, 50th percentile values, $C_{50\%}$, can be computed using 5th percentile values, $C_{5\%}$, based on Equation (B-1).

$$C_{50\%} = \frac{C_{5\%}}{1 - (1.645 * COV)} \qquad \text{(B-1)}$$

Thus, using the COVs listed in Table 5-6 of the *Wood Handbook* [20], the factors shown in Table B-2 can be multiplied by 5th percentile values to obtain approximate lower bound 50th percentile values. (The values in Table B-2 are rounded to the nearest 0.05.)

Table B-2. Lower Bound 5th to 50th Percentile Transformation Factors.

Design Value	Factor
$(F_bS)_{eff}$	1.35
F_c	1.40
V_s	1.30

During testing performed to investigate the response of CLT to blast loads, it was observed that the bending transformation factor in Table B-2 significantly underestimated (i.e., by over 200 percent in some cases) the bending strength of CLT layups with only visually graded lumber [7,8]. To obtain a better factor to make this 5th to 50th percentile translation, the testing used to generate the reference design values shown in the NDS Supplement was investigated further.

The In-Grade Testing Program was started in 1977 by the USDA Forest Service and was used to generate the reference design values shown in Tables 4A and 4B of the NDS Supplement. At its conclusion, the program had tested more than 70,000 specimens totaling approximately 1,000,000 board feet of lumber in bending, tension parallel to grain, and compression parallel to grain. The results of this testing are documented in the *Mechanical Properties of Visually Graded Lumber* [21]. Similar testing was performed in Canada around the same time; the results of this testing are documented in *Canadian Lumber Properties* [22].

Among the information gained through these parallel efforts were 5th and 50th percentile values for the MOR for a collection of wood species. The MOR values along with the corresponding number of tests (n), lumber sizes, and moisture contents (MCs) are displayed in Table B-3 through Table B-7 for the species groups included in the PRG 320 visually graded layups. (Note that Spruce-Pine-Fir (South), while not currently included in any of the PRG 320 layups, is used by one of the current CLT manufacturers in North America and thus is tabulated in Table B-7.) As the Eastern Softwoods, Northern Species, Western Woods, Spruce-Pine-Fir, and Spruce-Pine-Fir (South) species groups are composed of several species, Table 6-7 of the *Wood Handbook* [20] was used to determine the relevant species. Using this MOR data, an average value to translate 5th percentile values to 50th percentile values was determined. The values for the five species groups tabulated are summarized in Table 2.

Unfortunately, according to the Forest Products Laboratory, it appears as if the test data performed in the United States undergirding Table 4C in the NDS Supplement has been lost. Although a small amount of testing on MSR lumber is documented in [22], the connection of this data with Table 4C is not known. Thus, in the absence of better information, the transformation factor included in Table B-2 is the basis for K_{avg} for MSR lumber.

Table B-3. No. 2 Douglas Fir-Larch MOR Data.

Source	Species	n	Size	% MC	[A] $MOR_{5\%}$ [psi]	[B] $MOR_{50\%}$ [psi]	[C] [B]/[A]	[D] $(n/\Sigma n)$ *[C]
[21]	Douglas Fir-Larch	386	2x4	12	3768	8253	2.19	0.13
		386		15	3855	7859	2.04	0.12
		1964	2x8	12	2472	5838	2.36	0.70
		1964		15	2551	5773	2.26	0.67
		388	2x10	12	2156	5064	2.35	0.14
		388		15	2215	5075	2.29	0.13
[22]		370	2x4	15	2975	7175	2.41	0.14
		370	2x8	15	2284	5262	2.30	0.13
		374	2x10	15	2077	4619	2.22	0.13
Σ		**6590**						**2.29**

Table B-4. No. 2 Southern Pine MOR Data.

Source	Species	n	Size	% MC	[A] $MOR_{5\%}$ [psi]	[B] $MOR_{50\%}$ [psi]	[C] [B]/[A]	[D] $(n/\Sigma n)$ *[C]
[21]	Southern Pine	413	2x4	12	3758	7819	2.08	0.16
		413		15	3856	7540	1.96	0.16
		413	2x6	12	2865	7000	2.44	0.19
		413		15	2960	6833	2.31	0.18
		1367	2x8	12	2544	6303	2.48	0.65
		1367		15	2626	6220	2.37	0.62
		412	2x10	12	2735	6174	2.26	0.18
		412		15	2826	6106	2.16	0.17
Σ		**5210**						**2.32**

Table B-5. No. 2 Eastern Softwoods, Northern Species, or Western Woods MOR Data.

Source	Species	n	Size	% MC	[A] $MOR_{5\%}$ [psi]	[B] $MOR_{50\%}$ [psi]	[C] [B]/[A]	[D] $(n/\Sigma n)$ *[C]
[21]	Balsam Fir	60	2x4	12	2842	5921	2.08	0.03
[21]	Balsam Fir	60	2x4	15	2901	5652	1.95	0.03
[21]	Balsam Fir	60	2x6	12	2384	4979	2.09	0.03
[21]	Balsam Fir	60	2x6	15	2448	4849	1.98	0.03
[21]	Balsam Fir	60	2x8	12	2400	4528	1.89	0.03
[21]	Balsam Fir	60	2x8	15	2464	4456	1.81	0.02
[21]	Eastern Hemlock	61	2x4	12	3143	6366	2.03	0.03
[21]	Eastern Hemlock	61	2x4	15	3209	6099	1.90	0.03
[21]	Eastern Hemlock	60	2x6	12	2643	6621	2.51	0.03
[21]	Eastern Hemlock	60	2x6	15	2717	6314	2.32	0.03
[21]	Eastern Hemlock	60	2x8	12	2094	5167	2.47	0.03
[21]	Eastern Hemlock	60	2x8	15	2153	5068	2.35	0.03
[21]	Eastern White Pine	60	2x4	12	2615	5121	1.96	0.03
[21]	Eastern White Pine	60	2x4	15	2683	4995	1.86	0.03
[21]	Eastern White Pine	62	2x6	12	1722	3369	1.96	0.03
[21]	Eastern White Pine	62	2x6	15	1756	3414	1.94	0.03
[21]	Eastern White Pine	60	2x8	12	1789	3864	2.16	0.03
[21]	Eastern White Pine	60	2x8	15	1828	3876	2.12	0.03
[21]	Jack Pine	40	2x4	12	2563	6260	2.44	0.02
[21]	Jack Pine	40	2x4	15	2639	6038	2.29	0.02
[21]	Jack Pine	41	2x6	12	2225	5500	2.47	0.02
[21]	Jack Pine	41	2x6	15	2291	5382	2.35	0.02
[21]	Jack Pine	39	2x8	12	2124	4101	1.93	0.02
[21]	Jack Pine	39	2x8	15	2184	4127	1.89	0.02
[21]	Engelmann Spruce	100	2x4	12	2714	5709	2.10	0.05
[21]	Engelmann Spruce	100	2x4	15	2780	5490	1.97	0.04
[21]	Engelmann Spruce	94	2x6	12	2323	5184	2.23	0.05
[21]	Engelmann Spruce	94	2x6	15	2388	5042	2.11	0.04
[21]	Engelmann Spruce	40	2x8	12	2062	5205	2.52	0.02
[21]	Engelmann Spruce	40	2x8	15	2118	5059	2.39	0.02
[21]	Jack Pine	40	2x4	12	2563	6260	2.44	0.02
[21]	Jack Pine	40	2x4	15	2639	6038	2.29	0.02
[21]	Jack Pine	41	2x6	12	2225	5500	2.47	0.02
[21]	Jack Pine	41	2x6	15	2291	5382	2.35	0.02
[21]	Jack Pine	39	2x8	12	2124	4101	1.93	0.02
[21]	Jack Pine	39	2x8	15	2184	4127	1.89	0.02

Table B-5. No. 2 Eastern Softwoods, Northern Species, or Western Woods MOR Data. (Cont'd)

Source	Species	n	Size	% MC	[A] MOR$_{5\%}$ [psi]	[B] MOR$_{50\%}$ [psi]	[C] [B]/[A]	[D] (n/Σn) *[C]
[21]	Lodgepole Pine	80	2x4	12	2838	5312	1.87	0.03
		80		15	2916	5220	1.79	0.03
		80	2x6	12	2252	5102	2.27	0.04
		80		15	2318	5035	2.17	0.04
		60	2x8	12	1556	4125	2.65	0.04
		60		15	1568	4150	2.65	0.04
	Ponderosa Pine	100	2x4	12	2647	4848	1.83	0.04
		100		15	2714	4749	1.75	0.04
		70	2x6	12	2191	3653	1.67	0.03
		70		15	2253	3676	1.63	0.03
		100	2x8	12	2051	4431	2.16	0.05
		100		15	2106	4381	2.08	0.05
	Red Pine	58	2x4	12	2712	5218	1.92	0.03
		58		15	2794	5157	1.85	0.02
		60	2x6	12	2475	4153	1.68	0.02
		60		15	2550	4190	1.64	0.02
		60	2x8	12	1775	3473	1.96	0.03
		60		15	1808	3545	1.96	0.03
	Sitka Spruce	40	2x4	12	3467	6569	1.89	0.02
		40		15	3511	6241	1.78	0.02
		60	2x6	12	2880	5239	1.82	0.02
		60		15	2948	5109	1.73	0.02
	Subalpine Fir	101	2x4	12	2373	5564	2.34	0.05
		101		15	2432	5314	2.19	0.05
		100	2x6	12	2514	5052	2.01	0.05
		100		15	2573	4882	1.90	0.04
		60	2x8	12	2034	4255	2.09	0.03
		60		15	2089	4190	2.01	0.03
	Tamarack	60	2x4	12	3353	7799	2.33	0.03
		60		15	3446	7467	2.17	0.03
		60	2x6	12	2829	5510	1.95	0.03
		60		15	2920	5475	1.88	0.03
		63	2x8	12	2659	4915	1.85	0.03
		63		15	2744	4934	1.80	0.03
Σ		**4458**						**2.05**

Table B-6. No. 2 Spruce-Pine-Fir MOR Data.

Source	Species	n	Size	% MC	[A] $MOR_{5\%}$ [psi]	[B] $MOR_{50\%}$ [psi]	[C] [B]/[A]	[D] (n/Σn) *[C]
[21]	Balsam Fir	60	2x4	12	2842	5921	2.08	0.03
		60		15	2901	5652	1.95	0.03
		60	2x6	12	2384	4979	2.09	0.03
		60		15	2448	4849	1.98	0.03
		60	2x8	12	2400	4528	1.89	0.03
		60		15	2464	4456	1.81	0.03
	Engelmann Spruce	100	2x4	12	2714	5709	2.10	0.05
		100		15	2780	5490	1.97	0.05
		94	2x6	12	2323	5184	2.23	0.05
		94		15	2388	5042	2.11	0.05
		40	2x8	12	2062	5205	2.52	0.03
		40		15	2118	5059	2.39	0.02
	Jack Pine	40	2x4	12	2563	6260	2.44	0.03
		40		15	2639	6038	2.29	0.02
		41	2x6	12	2225	5500	2.47	0.03
		41		15	2291	5382	2.35	0.02
		39	2x8	12	2124	4101	1.93	0.02
		39		15	2184	4127	1.89	0.02
	Lodgepole Pine	80	2x4	12	2838	5312	1.87	0.04
		80		15	2916	5220	1.79	0.04
		80	2x6	12	2252	5102	2.27	0.05
		80		15	2318	5035	2.17	0.04
		60	2x8	12	1556	4125	2.65	0.04
		60		15	1568	4150	2.65	0.04
	Subalpine Fir	101	2x4	12	2373	5564	2.34	0.06
		101		15	2432	5314	2.19	0.06
		100	2x6	12	2514	5052	2.01	0.05
		100		15	2573	4882	1.90	0.05
		60	2x8	12	2034	4255	2.09	0.03
		60		15	2089	4190	2.01	0.03
[22]	Spruce-Pine-Fir	440	2x4	15	3126	6463	2.07	0.23
		986	2x8	15	2500	5126	2.05	0.52
		441	2x10	15	2097	4434	2.11	0.24
Σ		**3897**						**2.10**

Table B-7. No. 2 Spruce-Pine-Fir (South) MOR Data.

Source	Species	n	Size	% MC	[A] $MOR_{5\%}$ [psi]	[B] $MOR_{50\%}$ [psi]	[C] [B]/[A]	[D] (n/Σn) *[C]
[21]	Balsam Fir	60	2x4	12	2842	5921	2.08	0.06
		60		15	2901	5652	1.95	0.06
		60	2x6	12	2384	4979	2.09	0.06
		60		15	2448	4849	1.98	0.06
		60	2x8	12	2400	4528	1.89	0.05
		60		15	2464	4456	1.81	0.05
	Engelmann Spruce	100	2x4	12	2714	5709	2.10	0.10
		100		15	2780	5490	1.97	0.10
		94	2x6	12	2323	5184	2.23	0.10
		94		15	2388	5042	2.11	0.10
		40	2x8	12	2062	5205	2.52	0.05
		40		15	2118	5059	2.39	0.05
	Jack Pine	40	2x4	12	2563	6260	2.44	0.05
		40		15	2639	6038	2.29	0.04
		41	2x6	12	2225	5500	2.47	0.05
		41		15	2291	5382	2.35	0.05
		39	2x8	12	2124	4101	1.93	0.04
		39		15	2184	4127	1.89	0.04
	Lodgepole Pine	80	2x4	12	2838	5312	1.87	0.07
		80		15	2916	5220	1.79	0.07
		80	2x6	12	2252	5102	2.27	0.09
		80		15	2318	5035	2.17	0.08
		60	2x8	12	1556	4125	2.65	0.08
		60		15	1568	4150	2.65	0.08
	Red Pine	58	2x4	12	2712	5218	1.92	0.05
		58		15	2794	5157	1.85	0.05
		60	2x6	12	2475	4153	1.68	0.05
		60		15	2550	4190	1.64	0.05
		60	2x8	12	1775	3473	1.96	0.06
		60		15	1808	3545	1.96	0.06
	Sitka Spruce	40	2x4	12	3467	6569	1.89	0.04
		40		15	3511	6241	1.78	0.03
		60	2x6	12	2880	5239	1.82	0.05
		60		15	2948	5109	1.73	0.05
Σ		**2064**						**2.05**

B-2.2.3. K_{size} Derivation. Bending, tension, and compression mechanical properties of wood have been shown to vary according to member volume (specifically depth and length) and loading condition. This phenomenon is related to the brittle nature of wood. Essentially, wood acts analogous to a series of links on a chain; if one of the links fails, the entire chain fails. Thus, the more volume of wood that is present, the more opportunity (statistically speaking) for there to be a region of extremely low strength, and thus an area that could ultimately dictate the capacity of the member. This phenomenon is statistically described using the weakest-link theory. More information on the weakest-link theory as it relates to bending members can be found in Chapter 6 of [22]. The NDS size factors applied to visually graded lumber reference design values listed in Tables 4A and 4B of the NDS Supplement are a result of this observed size effect.

Presently, there is no established size effect factor prescribed to be applied to CLT. In fact, Table A1 of PRG 320 explicitly prohibits the use of a size adjustment for CLT panels. However, testing in the panel's major strength direction by several sources indicates a drop in relative capacity as the number of plies increases (see Table B-8).

Table B-8. $(F_bS)_{eff}$ Data Illustrating CLT Size Effect.

Source	Panel Description	[A] Average Tested $(F_bS)_{eff}$ [1] [lb-ft/ft]	[B] Qualification $(F_bS)_{eff}$ [lb-ft/ft]	$\frac{[\mathbf{A}]}{[\mathbf{B}]} * \frac{[\mathbf{B}]_{3-\mathrm{ply}}}{[\mathbf{A}]_{3-\mathrm{ply}}}$
[7] [2]	3-ply Grade V1	6,086	2,090	1.00
	5-ply Grade V1	13,029	4,800	0.93
[11] [3]	3-ply Grade E1	6,134	4,525	1.00
	5-ply Grade E1	11,523	10,400	0.82

1 Average tested value shown includes 2.1 factor safety factor specified in Table 1 of PRG 320.
2 Uniformly applied load on 4-foot thick panel.
3 Concentrated loads at third points on roughly 1.5-foot thick panel.

Thus, in the interest of arriving at expected 50th percentile values, it is important to consider the effect of size. In the United States, the size effect for visually graded lumber is defined in ASTM D1990 [23] by the formula shown in Equation (B-2):

$$F_2 = F_1 \left(\frac{W_1}{W_2}\right)^w \left(\frac{L_1}{L_2}\right)^l \left(\frac{T_1}{T_2}\right)^t \tag{B-2}$$

where W_1, L_1, and T_1 are the width (depth), length, and thickness, respectively, of the reference piece of wood and W_2, L_2, and T_2 are the same properties for the piece of wood of concern. The coefficients of w, l, and t are defined as 0.29, 0.14, and 0 in ASTM D1990. (The lack of a thickness coefficient is based on research performed by Bohannan [24].) It should be noted that this formula has not been verified for widths (depths) less than 3.5 inches or greater than 9.25 inches and for nominal thicknesses of 2 and 4 inches.

The visually graded reference design values (i.e., for SS, No.1, No. 2, and No. 3) shown in Tables 4A and 4B of the NDS Supplement are based on a 2x12 that is 240 inches long [25]. While the length could conceivably be adjusted as well, this adjustment is not performed by the NDS and is not applied here for consistency and simplicity. Thus, the K_{size} factor to be applied to CLT panels with visually graded lumber in the span direction of concern is equal to Equation (B-3):

$$K_{size} = \left(\frac{11.25}{h_{eff}}\right)^{0.29} \tag{B-3}$$

where h_{eff} is the effective width (depth) of the panel in the span direction of concern. The upper limit of 1.65 is based on the product of the largest NDS size factor, C_F, (i.e., 1.50) and flat use factor, C_{fu}, (i.e., 1.10). (Section C4.3.7 of the NDS commentary indicates that C_F and C_{fu} should be "used cumulatively" [3].)

Although size effect has been observed for mechanically graded lumber as well [26], the machines responsible for sorting mechanically graded lumber explicitly account for this size effect [27] and thus there is no explicit size effect factor in the NDS. However, as the smallest size of lumber that is likely to be used in a CLT panel is a 2x4, Equation (B-3) is modified accordingly in Equation (B-4) for CLT panels with MSR lumber in the span direction of concern.

$$K_{size} = \left(\frac{3.5}{h_{eff}}\right)^{0.29} \tag{B-4}$$

The above discussion and resulting K_{size} factors are clearly based on limited information. In addition, the thickness of panel may in fact need to be considered as well. As the state of CLT science in North America advances, this size adjustment will need to be reviewed and adjusted as well.

B-3. Dynamic Increase Factor. The dynamic increase factor (DIF) is used to increase average expected design values at normal load duration to account for the effect of strain rate. A DIF of 2.0 is recommended for all design values. This factor is equivalent to the load duration factor, C_D, for impact loading included in the NDS.

Absent more testing at a material level at various levels of strain rate on different types of lumber, this factor served as a good approximate of the strain rate increase observed in wood under dynamic loads [8,12].

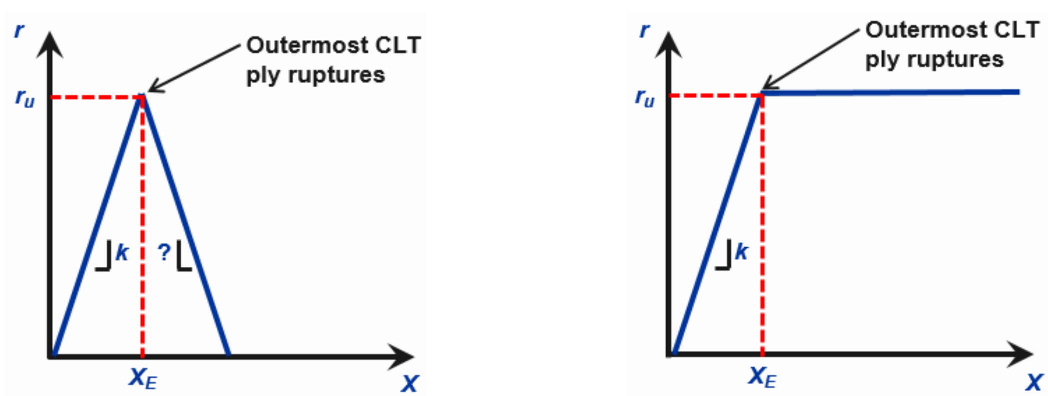
It should be noted that Poulin et al. [11] indicated that a DIF of 1.28 well approximated their shock tube tests on CLT panels. Although not stated, it is assumed that this factor is in addition to a 1.6 load duration factor hidden in the 2.1 bending factor prescribed in Table 1 of PRG 320. Multiplying 1.28 by 1.6 yields 2.05, which is in good agreement with the DIF proposed herein.

APPENDIX C
IDEALIZED RESISTANCE FUNCTION & RESPONSE LIMITS

C-1. Purpose. The purpose of this appendix is to provide commentary concerning the idealized resistance function and response limits recommended in this PDC-TR for CLT construction.

C-2. Resistance Function.

C-2.1. Resistance Function Form. Two forms are offered as potential candidates for a CLT panel resistance function. One explicitly models the softening response observed in one-way quasi-static testing of CLT panels [7,8]; this resistance function form is shown in Figure C-1a. The other resistance function form is simply an elastoplastic idealization. With this form, response limits are used to limit displacement ductility, μ, based on qualitative observations of damage observed in blast tests (see Section C-3). This resistance function form is shown in Figure C-1b. Note that for both resistance function forms, μ is equal to X_{max} divided by X_E, where X_{max} is the peak computed displacement in either inbound or rebound and X_E is the displacement corresponding to yield in the idealized resistance function.



(a) Post-Peak Softening. *(b) Elastoplastic.*

Figure C-1. Potential Idealized Resistance Functions Forms.

While the resistance function form with softening better matches the post-peak load versus displacement response of brittle materials such as wood, several observations complicate its usage in practice:

- The response of an SDOF dynamic analysis is very sensitive to the slope of the softening stiffness. However, it is very difficult to identify a representative softening stiffness value based on analysis or testing. Indeed, as observed in [7,8], the softening stiffness for CLT panels can vary significantly, even for duplicate tests, due to the brittle nature of wood.

- SDOF analyses conducted following blast tests on CLT structures indicated that the elastoplastic resistance function yielded comparable or better matching of the test data that that with softening [12]. Several SDOF analyses that utilized a resistance function with softening indicated a panel blowout when in fact the panel only exhibited localized panel rupture near midspan.

Furthermore, the SDOF response limits currently included in DoD antiterrorism criteria [1] generally assume an elastoplastic resistance function, even for brittle materials (e.g., cold-formed steel wall studs).

As such, the elastoplastic resistance function form is recommended with the response limits indicated in Section 10. A reduction factor of 0.9 is applied to the ultimate resistance (see Equations (4) and (8)) because the elastoplastic resistance function form overestimates the residual capacity of CLT panels within the range of allowable displacement ductility defined in Section 10.

C-2.2. Two-Way Action. Two-way action is inherent in CLT construction. Flexural and shear tests of one-way panel action in both the major and minor strength directions are performed as part of the APA certification process. Interaction of the two span directions is not well documented, particularly as it relates to the ultimate two-way resistance of the panel and the ensuing post-peak residual capacity. Thus, common practice when sizing CLT panels is to idealize the panel as a one-way spanning member in either the major or minor strength directions. This simplification is adopted in this PDC-TR.

C-2.3. Connection Design. As described above, several idealizations are made to the analytical resistance function to simplify the analysis of CLT panels exposed to blast loads (i.e., elastoplastic resistance function form with a 0.9 reduction factor applied to flatwise strength, ignoring two-way action). These idealizations, while conservative from a panel design perspective, may serve to underestimate the actual ultimate resistance of the CLT panel. In addition, other physical phenomena (e.g., compression membrane action, axial load arching, partial rotational restraint at panel boundary conditions, lumber variability) may serve to further augment the actual ultimate resistance of a CLT panel from that computed using the guidance in this PDC-TR. Thus, it is important that connection designs address this divergence between actual and computed ultimate resistance.

The preferred method of addressing this divergence is to perform another analysis that explicitly considers relevant two-way action, compression membrane response, boundary condition rotational restraint, etc. In addition, the connection detailing guidance included in Section 11 is provided to promote connections with controlling limit states that are ductile rather than brittle.

C-2.3.1. Lateral Design Values. Section C11.3 of the NDS commentary indicates that the lateral design values for wood screw connections at reference conditions (i.e., seasoned dry, normal load duration) are approximately 20 percent of maximum tested capacities [3]. Included in this safety factor of 5 is a 1.6 factor because testing conditions are associated with a ten-minute load duration. Thus, assuming impact load duration (i.e., C_D equal to 2.0), the total safety factor for blast-level strain rates is extrapolated to be 6.25.

Provided the wood screw is oriented perpendicular with the plane of the CLT panel and the minimum spacing requirements in the NDS are adhered to, the ultimate limit state associated with the fastener's lateral design value is expected to either be wood crushing or steel yielding, both of which exhibit significant post-peak residual strength and deformation capability [28] and are not expected to portend a complete loss of capacity in the event of being overstressed by an extremely short duration load. For this ultimate limit state, a net safety factor of 1.25 is recommended. Thus, dividing the total safety factor (i.e., 6.25) by net safety factor of 1.25 and a C_D of 2 indicates that the lateral design value obtained from NDS can be further increased by a factor of 2.5.

C-2.3.2. Withdrawal Design Values. Section C11.2.2 of the NDS commentary indicates that the withdrawal design values for wood screw connections at reference conditions (i.e., seasoned dry, normal load duration) are approximately 20 percent of maximum tested capacities [3]. Thus, per the logic noted in Section C-2.3.1, the total safety factor for blast-level strain rates is extrapolated to be 6.25.

Although wood screw withdrawal failures may exhibit ductile post-peak response, testing has indicated, particularly when the screw is oriented at 45 degrees with the broad face of the CLT panel, that the post-peak response is relatively non-ductile as compared to wood screw lateral failures [28]. As such, a net safety factor of 2.0 is recommended. Thus, dividing the total safety factor (i.e., 6.25) by net safety factor of 2.0 and a C_D of 2.0 indicates that the lateral design value obtained from NDS can be further increased by a factor of roughly 1.5.

Where the fastener in withdrawal engages all CLT panel plies, the net safety factor may be reduced to 1.5. Quasi-static testing performed at the University of Maine indicated that such a detailing practice served to reinforce the panel and preclude rolling shear failures that led to ply detachment [7].

C-2.3.3. Ultimate Tested Values. Ultimate values determined via testing have varying levels of ductility associated with their post-peak response. In the absence of further data concerning the post-peak response of the connection assembly, ultimate tested values shall be assumed to exhibit limited post-peak ductility and be designed with a net safety factor of 2.0. Assuming the ultimate tested values are based on a ten-minute test duration, it should be reduced by a factor of 0.625 (i.e., ultimate tested value multiplied by a net load duration factor of 1.25 (i.e., 2.0 divided by 1.6) and divided by a net safety factor of 2.0).

The net safety factor applied to ultimate tested values may be reduced to 1.25 if the post-peak response of the tested connection element exhibits a deformation ductility of at least five while still maintaining 75 percent of its peak load.

C-3. Response Limits. The response limits reported in this PDC-TR rely heavily on data and observations obtained from seven blast tests performed on three two-story, single-bay CLT structures at Tyndall Air Force Base. These tests are documented in two test reports [8,12]. Based on the post-test photographs included in [8,12], the observed damage in the first-floor front and side wall panels was correlated with the component damage level definitions included in Table 2-4 of PDC-TR 06-08 [1]. This information is included in Table C-1.

Table C-1. First-Floor Wall Panel Damage from Tests 1-7 of [8,12] Correlated to PDC-TR 06-08 Component Damage Levels.

Component Damage Level[1]	Description of Component Damage[1]	Examples from Tests 1 Through 7
Blowout	Component is overwhelmed by the blast load causing debris with significant velocities	• 3-ply E1 front wall following Test 7
Hazardous Failure	Component has failed, and debris velocities range from insignificant to very significant	No test data
Heavy Damage	Component has not failed, but it has significant permanent deflections causing is to be unrepairable	No test data
Moderate Damage	Component has some permanent deflection. It is generally repairable, if necessary, although replacement may be more economical and aesthetic	• All front walls following Tests 3 & 5 • 5-ply V1 front wall following Test 7 • All 3-ply side walls following Test 7
Superficial Damage	Component has no visible permanent damage	• All walls following Tests 1, 2, 4, & 6

1 As defined in [1].

SDOF dynamic analyses of the first-floor wall panels were performed using the resistance function, SIFs, and DIF documented in this PDC-TR. The pressure histories recorded during Tests 1 through 7 of [8,12] were used as the input blast loads. The peak computed displacement ductility for each of these analyses is plotted in the bar chart included as Figure C-2. In addition, Figure C-2 indicates where the PDC-TR 06-08 component damage levels fall based on the qualitative damage listed in Table C-1.

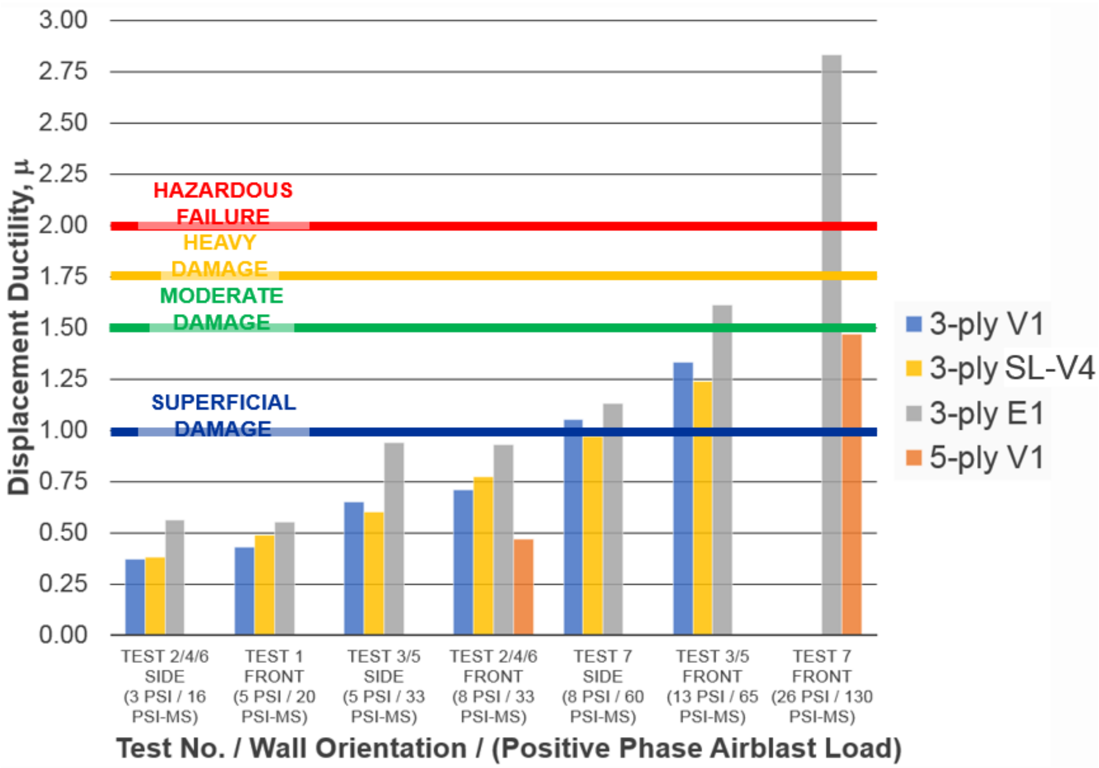


Figure C-2. Applied Blast Load vs. Computed Displacement Ductility with PDC-TR 06-08 Component Damage Levels Indicated [8].

The response limit associated with each damage level shown in Table 5 are based on the results displayed in Figure C-2. Further commentary concerning the correlation of the displacement ductility value assigned to each component damage level is included below:

- **Superficial Damage (μ < 1.0)**: When the computed displacement ductility is less than one, no visual signs of damage were observed in the CLT wall panels.

- **Moderate Damage (1 ≤ μ < 1.5)**: The extent of the damage observed in most of the front wall panels following Tests 3, 5, and 7 was limited and localized near midspan. It is thought that such a wall panel could be repaired relatively easily with additional lumber boards and/or thin gauge steel plates.

- **Heavy Damage (1 ≤ μ < 1.75)**: No examples matching the “heavy damage” description appear in the testing performed. This response limit is simply placed halfway between the “moderate damage” and “hazardous failure” component damage levels.

- **Hazardous Failure (1.75 ≤ μ < 2)**: The displacement ductility value of two is based on testing investigating the flatwise bending response of axially-loaded CLT panels in their major strength direction under a uniformly-applied quasi-static load [8]. In this testing effort, it was observed that several panels completely lost flatwise bending strength at a displacement ductility of two.

- **Blowout (μ ≥ 2)**: The front wall panel in the Grade E1 structure following Test 7 was completely overwhelmed by the blast load and exhibited a displacement ductility well over two.

APPENDIX D
EXAMPLE PROBLEM

PROBLEM

Generate a resistance function capable for use in SDOF dynamic analysis for a CLT panel having the following input parameters:

CLT Panel Description:	3-ply Grade V1
(Major Strength Direction) Span, L:	120 in
Axial Load, P:	3,000 lb/ft
Eccentricity of Axial Load, Δ:	3 in
Idealized Boundary Conditions:	Pin-Roller
Load Distribution:	Uniform
$C_M = C_t = 1.0$	

SOLUTION

Step 1: Obtain corresponding input parameters from Appendix A.

Lumber Description: No. 2 Douglas Fir-Larch
Lumber Grading: Visual

h_{eff}	=	4.125 in
$A_{parallel}$	=	33 in^2/ft
F_b	=	900 psi
F_c	=	1,350 psi
$(F_bS)_{eff}$	=	2,090 lb-ft/ft
$(EI)_{eff}$	=	108,000,000 lb-in^2/ft
$(GA)_{eff}$	=	530,000 lb/ft
V_s	=	1,910 lb/ft

Step 2: Determine flatwise bending stiffness.

$K_s = 11.5$ (Table 3 - Pin-Roller BCs / Uniformly Distributed Loading)

Use Equation (2) to compute apparent bending stiffness:

$$(EI)_{app} = \frac{(EI)_{eff}}{1 + \frac{K_s (EI)_{eff}}{(GA)_{eff} L^2}} = \frac{108{,}000{,}000}{1 + \frac{11.5 * 108{,}000{,}000}{530{,}000 * 120^2}} = 92{,}884{,}381 \text{ lb-in}^2\text{/ft}$$

$C_{adj\_EI} = C_M C_t = 1.0 * 1.0 = 1.0$ (Table 10.3.1 of [3] defines adjustment factors needed)

k_b = 5/384 (Table 4 - Pin-Roller BCs / Uniformly Distributed Loading)

Section width is equivalent to the loaded tributary width:
b_w = 1.0 ft
b = 12 in

Use Equation (3) to compute stiffness:

$$k = C_{adj\_EI} * \frac{(EI)_{app} b_w}{k_b b L^4} = 1.0 * \frac{92{,}884{,}381 * 1.0}{\frac{5}{384} * 12 * 120^4} = 2.87 \text{ psi/in}$$

Step 3: Determine SIF to be applied to each design value.

Grade V1 CLT uses No. 2 Douglas Fir-Larch lumber as the major strength direction plies. Thus, per Table 2, the K_{avg} factor to be applied to $(F_bS)_{eff}$ is equal to 2.30.

Per Table 1, the K_{size} factor to be applied to $(F_bS)_{eff}$ is equal to:

$$K_{size\_b} = \left(\frac{11.25}{h_{eff}}\right)^{0.29} = \left(\frac{11.25}{4.125}\right)^{0.29} = 1.34$$

The remainder of the SIF subfactors are as defined in Table 1. Table D-1 summarizes the subfactors and resulting SIF for each design value using Equation (1).

Table D-1. SIF Summary.

Design Value	K_{char}	K_{avg}	K_{size}	**SIF**
$(F_bS)_{eff}$	1.30	2.30	1.34	4.01
F_c	1.20	1.40	1.00	1.68
V_s	2.00	1.30	1.00	2.60

Step 4: Determine if axial load must be considered when computing flatwise bending moment capacity.

Determine effective section modulus:

$$S_{eff} = \frac{(F_bS)_{eff}}{F_b} = \frac{2{,}090 * 12 \text{ in/ft}}{900} = 27.9 \text{ in}^3\text{/ft}$$

Compute maximum compressive stress parallel to grain:

$$f_c = \frac{P}{A_{parallel}} + \frac{P\Delta}{S_{eff}} = \frac{3{,}000}{33} + \frac{3{,}000 * 3}{27.9} = 413 \text{ psi}$$

From Appendix G of [3], K_e is equal to 1.0 for Pin-Roller boundary conditions.

$L_e = K_e L = 1.0 * 120 = 120$ in

Use Equation (7) to compute the critical buckling design value:

$$P_{cE} = \frac{\pi^2 C_{adj\_EI}(EI)_{app}}{L_e^2} = \frac{\pi^2 * 1.0 * 92{,}884{,}381}{120^2} = 63{,}662 \text{ lb/ft}$$

(<u>Note</u>: Table 10.4.1.1 of [3] defines the K_s for column buckling as 11.8 and the K_s for uniformly distributed load as 11.5. As the use of 11.8 rather than 11.5 will lead to a difference in $(EI)_{app}$ of less than one percent, the $(EI)_{app}$ computed in Step 2 is used here as well.)

Compute the reference compression design value parallel to grain multiplied by all applicable adjustment and increase factors except for the column stability factor, C_P:

$$P_c^* = C_M * C_t * SIF_c * DIF * F_c * A_{parallel} = 1.0 * 1.0 * 1.68 * 2 * 1{,}350 * 33 = 149{,}688 \text{ lb/ft}$$

$$P_{cE}/P_c^* = 63{,}662/149{,}688 = 0.43$$

Compute C_P in accordance with Equation 3.7-1 of [3]:

$$C_p = \frac{1 + \left(P_{cE}/P_c^*\right)}{2c} - \sqrt{\left[\frac{1 + \left(P_{cE}/P_c^*\right)}{2c}\right]^2 - \frac{\left(P_{cE}/P_c^*\right)}{c}} = \frac{1 + 0.43}{2 * 0.9} - \sqrt{\left[\frac{1 + 0.43}{2 * 0.9}\right]^2 - \frac{0.43}{0.9}} = 0.40$$

Determine dynamic compressive stress capacity parallel to grain using Equation (5):

$C_{adj\_c} = C_M C_t C_P = 1.0 * 1.0 * 0.40 = 0.40$ (Table 10.3.1 of [3] defines adjustment factors needed)

$$F_{dc} = C_{adj\_c} * SIF_c * DIF * F_c = 0.40 * 1.68 * 2 * 1{,}350 = 1{,}814 \text{ psi}$$

Comparing the actual compressive stress to the dynamic compressive stress capacity indicates that the actual compressive stress exceeds $0.1F_{dc}$:

$0.1F_{dc} = 181$ psi $< f_c = 413$ psi

Thus, axial compression loads must be considered when computing flatwise bending moment.

Step 5: Compute flatwise bending moment capacity.

The panel's depth does not exceed its breadth. Thus, per Section 3.3.3.1 of [3], the beam stability factor, C_L, is equal to 1.0. Thus, the net adjustment factor for bending is equal to:

$C_{adj\_b} = C_M C_t C_L = 1.0 * 1.0 * 1.0 = 1.0$ (Table 10.3.1 of [3] defines adjustment factors needed)

Use Equation (4) to compute flatwise moment bending capacity without axial load:

$$M_n = 0.9 * C_{adj\_b} * SIF_b * DIF * (F_b S)_{eff} = 0.9 * 1.0 * 4.01 * 2 * 2{,}090 = 15{,}086 \text{ lb-ft/ft}$$

Use Equation (6) to modify the moment capacity for the applied axial load:

$$M_{n\_axial} = M_n * \left(1 - \frac{P}{P_{cE}}\right)\left[1 - \left(\frac{P}{F_{dc}A_{parallel}}\right)^2\right] - P\Delta\left(1 + 0.234\frac{P}{P_{cE}}\right)$$
$$= 15{,}086 * \left(1 - \frac{3{,}000}{63{,}662}\right)\left[1 - \left(\frac{3{,}000}{1{,}814 * 33}\right)^2\right] - \left(3{,}000 * \frac{3}{12}\right)\left(1 + 0.234 * \frac{3{,}000}{63{,}662}\right)$$
$$= 13{,}581 \text{ lb-ft/ft}$$

Step 6: Compute flatwise shear capacity.

$C_{adj\_s} = C_M C_t = 1.0 * 1.0 = 1.0$ (Table 10.3.1 of [3] defines adjustment factors needed)

Use Equation (8) to compute flatwise shear strength:

$$V_n = 0.9 * SIF_s * DIF * C_{adj\_s} * V_s = 0.9 * 2.60 * 2 * 1.0 * 1{,}910 = 8{,}939 \text{ lb/ft}$$

Step 7: Compute ultimate resistance.

Use Table 3-4 of [2] to determine flexural ultimate resistance:

$$r_{u\_flex} = \frac{8M_{n\_axial}}{L^2} = \frac{8 * 13{,}581}{120^2} = 7.55 \text{ psi}$$

Similarly, use Table 3-4 of [2] to determine shear ultimate resistance:

$$r_{u\_shear} = \frac{2V_n}{L} = \frac{2 * 8{,}939}{120 * 12 \text{ in/ft}} = 12.42 \text{ psi}$$

The actual ultimate resistance is the lesser of r_{u_flex} and r_{u_shear}:

r_u = 7.55 psi

Using the stiffness and ultimate resistance, a resistance function capable for use with SDOF dynamic analysis can be constructed as shown in Figure D-1. The elastic displacement, X_E, is equal to:

$$X_E = \frac{r_u}{k} = \frac{7.55}{2.87} = 2.63 \text{ in}$$

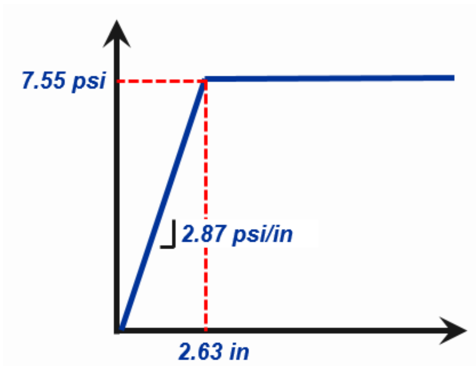


Figure D-1. Final Resistance Function for SDOF Dynamic Analysis.

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APPENDIX E
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