Timber Design Guide 2020-19

TIMBER FRAME ENGINEERING COUNCIL

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Title:	Cross-Laminated Timber (CLT) Diaphragms		

Introduction

Cross-Laminated Timber (CLT) panels are becoming increasingly common as a roof or floor deck system in timber frame buildings. On larger timber buildings, the roof and floor deck systems need to be carefully engineered and detailed to serve as diaphragms resisting wind and seismic loads. The diaphragm transmits lateral loads to the vertical lateral load resisting elements – usually shear walls or braced frames.

It is common for CLT floor decks to have a concrete topping slab for sound isolation. The concrete topping slab can be detailed to serve as the lateral load resisting diaphragm, but it is often more practical to engineer the CLT deck to serve as the diaphragm. There is currently little guidance in the *AWC National Design Specification for Wood Construction (NDS)* or in the *AWC Special Design Provisions for Wind & Seismic (SDPWS)* on the engineering of CLT diaphragms.

Diaphragm Flexibility

In most instances, it is reasonable to consider CLT floor and roof decks to act as rigid diaphragms with lateral loads distributed to the vertical resisting elements based on their relative stiffness. It is recommended that the aspect ratio (L/W) of the rigid diaphragm not exceed 3:1 if there is a non-composite concrete topping (2 ½” minimum thickness) or 2:1 if there is no concrete topping. If the aspect ratio exceeds these limits, but is not more than 4:1, the diaphragm may be modeled as semi-rigid.

Alternatively, rather than performing a semi-rigid diaphragm analysis, which can be tedious, the lateral analysis may be run twice – once assuming a rigid diaphragm and once assuming a flexible diaphragm, and designing for the envelope of the two. If the lateral force resisting elements are similar in stiffness and well distributed throughout the footprint, the two solutions will often not be far apart.

The CLT decks can be considered flexible diaphragms only if calculations demonstrate that the in-plane deflection of the diaphragm is more than twice the average drift of the vertical resisting elements per the provisions contained in *ASCE/SEI 7-16 Minimum Design Loads and Associated Criteria Buildings and Other Structures* section 12.3.1.3. In no case should the diaphragm aspect ratio exceed 4:1.

Diaphragm Strength

Table 1 - CLT in-plane Shear ASD Reference Design Values (lbf/ft of width)

CLT Layup	F_v (psi)	4 1/8" (3-ply)	6 7/8" (5-ply)	9 5/8" (7-ply)
E1	135	2385	4769	7154
E2	180	3180	6359	9539
E3	110	1943	3886	5829
E4	175	3091	6182	9274
E5	150	2650	5299	7949
V1	180	3180	6359	9539
V2	135	2385	4769	7154
V3	175	3091	6182	9274
V4	135	2385	4769	7154
V5	150	2650	5299	7949
CLT Layup	**F_v (psi)**	**4 1/2" (3-ply)**	**7 1/2" (5-ply)**	**10 1/2" (7-ply)**
S1	130	2496	4992	7488
S2	150	2880	5760	8640
S3	115	2208	4416	6624

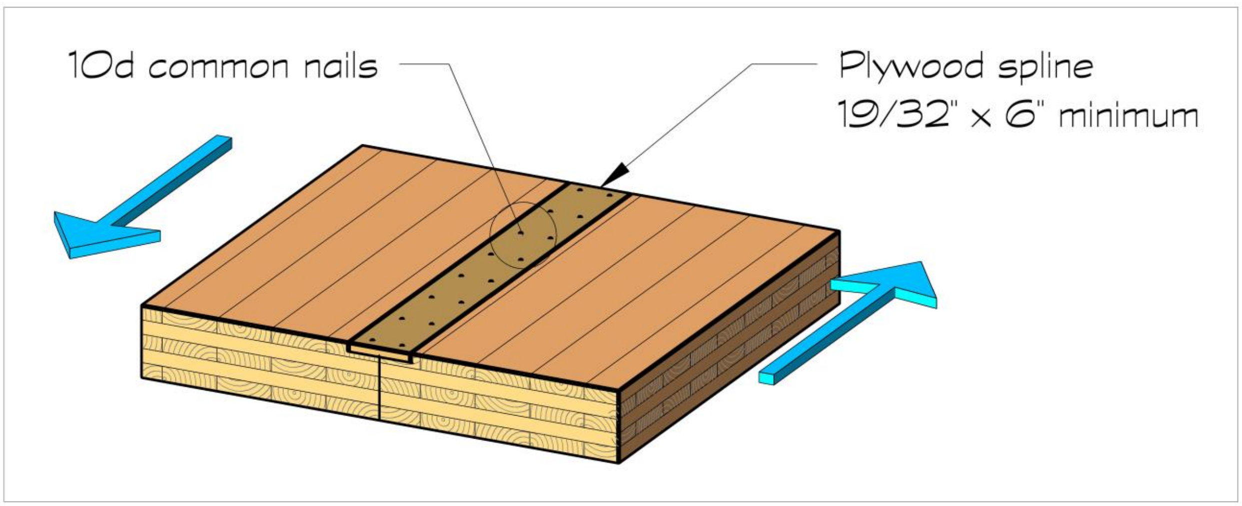
V (in-plane shear strength) = t (thickness of cross plies) x 12" x F_v x 1.6 (C_D) / 1.5

The in-plane shear strength of a CLT panel is controlled by the shear strength of the transverse cross laminations. The allowable in-plane shear values are shown in Table 1. To preclude a non-ductile shear rupture, CLT panels should be designed to resist 2.0 times the induced shear associated with seismic loads.

The in-plane shear strength of a CLT deck is typically limited by the strength of the connections between panels and the connections at boundary elements rather than by the strength of the CLT panels. Consequently, careful detailing of the panel connections and fasteners is crucial.

CLT Connections

Longitudinal side joints between CLT panels are most commonly achieved with plywood splines fastened with common nails. Spline joints deform and dissipate energy during a seismic event, lending ductility to the diaphragm. If thicker splines are used, or if they are fastened with screws rather than common nails, some ductility is sacrificed. The allowable unit shear capacity for plywood splines are shown in Table 2.



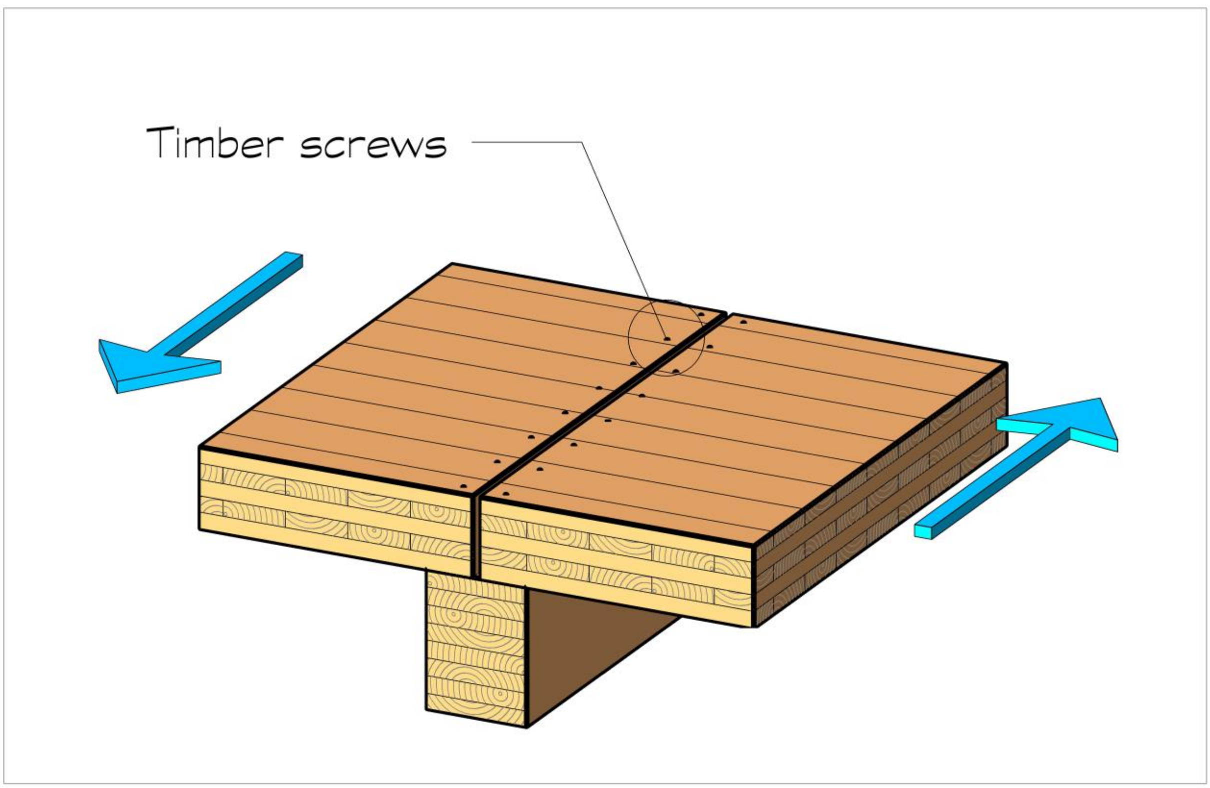
There has been some dynamic testing performed of spline joints with screw fasteners at the University of British Columbia. The research suggests that spline joints with inclined screws loaded axially typically exhibit high initial stiffness and ultimate static capacity, but are prone to non-ductile failure. Spline joints with screws installed at 90 degrees and loaded in shear, exhibited lower initial stiffness and ultimate static capacity but failed in a more ductile fashion.

Table 2 – Allowable Unit Shear Capacity of Splines (lbf/ft)

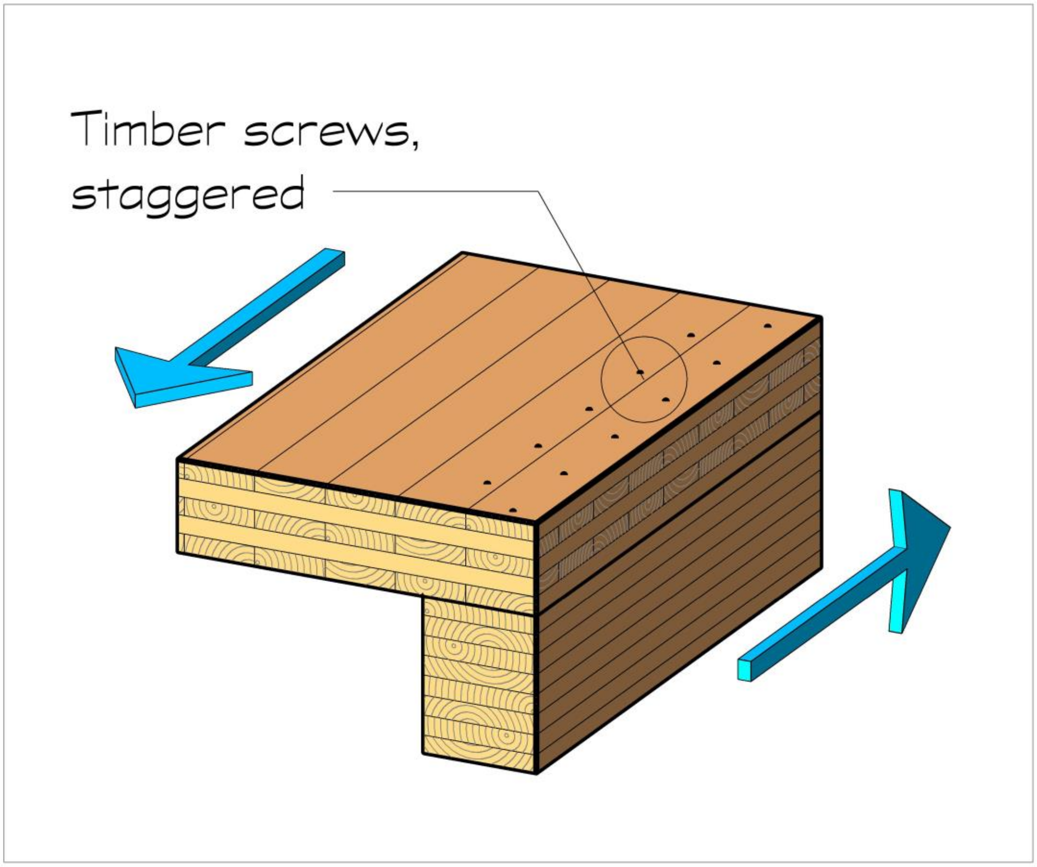
Spline Thickness	10d 6"o.c.	10d 4"o.c.	10d 2 ½"o.c.	2 rows 10d 4"o.c.	2 rows 10d 2 ½"o.c.
			Wind		
$^{19}/_{32}$"	505	672	1007		
$^{23}/_{32}$"				995	1427
			Seismic		
$^{19}/_{32}$"	360	480	720		
$^{23}/_{32}$"				710	1020

Source: SDPWS Tables 4.2A & 4.2B

Transverse end joints between CLT panels must resist the same in-plane shear as the longitudinal joints. In most instances, the ends of the CLT panels are supported on a timber beam and the shear is transmitted through the beam with timber screws.



Diaphragm boundary elements (chords and collectors) are often timber beams. Connections at the ends of timbers serving as boundary elements must be detailed to resist the axial forces induced into them to maintain the continuity of the chords. If the chord forces must be resisted by the CLT panels rather than by timber beams, steel straps are needed at the panel joints. To preclude non-ductile behavior, chord splices should be designed to resist 2.0 times the induced seismic chord force. The timber screws fastening the CLT panels to the boundary timbers need to be spaced to transmit the chord and collector forces.



The allowable shear capacity of timber screws fastening CLT panels to timber beams is shown in Table 3. The values in the table are based on timber screws with a minimum F_{yb} of 136.6 ksi, a CLT specific gravity of 0.42 and a timber beam specific gravity of 0.49. The load is assumed to be applied parallel to the grain of the timber beam. These values have been increased for a load duration factor, C_D of 1.6 and a diaphragm adjustment factor, C_{di} of 1.1

Table 3 – Allowable Shear Capacity of Timber Screws in CLTs (lbf)

Thickness	Fastener Diameter	Fastener Length	Major Strength Direction	Minor Strength Direction
$4\,{}^{1}/_{8}$”	${}^{1}/_{4}$” (6mm)	$7\,{}^{7}/_{8}$” (200mm)	348	348
$4\,{}^{1}/_{8}$”	${}^{5}/_{16}$” (8 mm)	$7\,{}^{7}/_{8}$” (200mm)	456	364
$6\,{}^{7}/_{8}$”	${}^{5}/_{16}$” (8 mm)	$11\,{}^{7}/_{8}$” (300 mm)	456	364
$6\,{}^{7}/_{8}$”	${}^{3}/_{8}$” (10 mm)	$11\,{}^{7}/_{8}$” (300 mm)	669	480
$9\,{}^{5}/_{8}$”	${}^{3}/_{8}$” (10 mm)	15” (380 mm)	669	480
$9\,{}^{5}/_{8}$”	${}^{1}/_{2}$” (12 mm)	$15\,{}^{3}/_{4}$” (400 mm)	961	676

Source: Heavy & Mass Timber Connections Handbook - MyTiCon

The timber screw values contained in Table 3 are controlled by Mode IV fastener yielding in accordance with NDS 12.3.1.

References

1. American Wood Council (AWC), *National Design Specification for Wood Construction (NDS)* 2018
2. American Wood Council (AWC), *Special Design Provisions for Wind and Seismic (SDPWS)* 2015
3. FPInnovations, *CLT Handbook* 2011
4. Structurlam Products, *Cross Laminated Timber Horizontal Diaphragm Design Example* 2015
5. MyTiCon Timber Connectors, *Update: Dynamic Testing on CLT Connections*
6. MyTiCon Timber Connectors, *Heavy & Mass Timber Connections Handbook* 2017

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