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Behavior of cross-laminated timber diaphragm connections with self-tapping screws

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ABSTRACT

Monotonic and cyclic tests were carried out to determine strength and stiffness characteristics of 2.44 m (8 ft) long shear connections with 8 mm and 10 mm diameter self-tapping screws. The goal of this research is to compare test values of cross-laminated timber (CLT) diaphragm connections in seismic force-resisting systems to the design values calculated from formulas in the National Design Specification for Wood Construction (USA) and the Eurocode. Understanding and quantifying the behavior of these shear connections will provide structural engineers with increased confidence in designing these components, especially with regard to the seismic force-resisting systems. Ratios of the experimental yield strength (from the yield point on the load-deflection curve) to factored design strength were in the range of 2.1–6.1. In the ASCE 41-13 acceptance criteria analysis, the m-factors for the Life Safety performance level in cyclic tests ranged from 1.6 to 1.8 for surface spline connections and from 0.9 to 1.7 for cyclic half-lap connections. The half-lap connections with a unique combination of angled and vertical screws performed exceptionally well with both high, linear elastic initial stiffness and ductile, post-peak behavior.

1. Introduction

Cross-laminated timber (CLT) is an engineered wood product that is playing a major role in the worldwide push for wood buildings taller than the conventional limit of 5–6 stories for light-frame wood construction. As a majority of residential (and other low-rise) buildings are light-frame wood construction, the mid- and high-rise sector present new markets where wood can make a significant sustainable, cost-effective impact. The higher strength, stiffness, and solid wood volume of CLT, compared to conventional light frame construction, are the specific characteristics enabling the increased building heights of wood structures. However, the seismic behavior and analysis of a new building system like this requires additional research to accompany the existing knowledge of conventional wood construction. Floor and roof diaphragms, the horizontal components of the lateral force-resisting system of a building, are designed to resist earthquake and wind loadings. The in-plane shear forces in both light-frame wood diaphragms and CLT diaphragms are resisted by steel connections, such as nails and screws, and these provide most of the energy dissipation in the diaphragm during seismic loading.

1.1. Background

Several research programs seeking to quantify the quasi-static and cyclic performance characteristics of cross-laminated timber and their joints were carried out in Europe in the 2000s [1–3]—the infancy of the new engineered wood product—and later in North America [4]. Cyclic loading tests were conducted to investigate and quantify the ductility, energy dissipation, strength, and stiffness characteristics of the CLT components. Conventionally, the vertical elements of lateral force-resisting systems (LFRS), such as shear walls, are the primary building components designed to provide ductility in a building. Much of the initial testing that took place was on CLT shear walls and their associated connections. Performance characteristics of CLT shear walls and diaphragms (under lateral loads) are controlled by the ductile steel connections, as the CLT panels themselves are significantly more rigid. Yielding of steel nails and crushing of wood provide much of the ductility of wood systems for both shear walls and diaphragms; in addition, light-frame wood diaphragms are known to sustain much less damage compared to the similarly constructed shear walls during earthquakes [5]. To date, the authors have not encountered a building project where

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inelastic CLT diaphragm performance was assumed during design [6,7], nor has consensus been reached in design recommendations regarding this aspect [7,8].

1.2. CLT diaphragm design

Fundamental research is needed on CLT diaphragms and quantification of design parameters to make these innovative building projects cost efficient for owners. However, CLT structures are being built effectively in the U.S. and receiving local building code approval through alternate means after thorough engineering analysis; in some cases, testing programs are carried out to demonstrate performance. Diaphragm shear forces between panels, connection strengths, and diaphragm deflection can be calculated [9–15] and test data to support the calculation are available [2,3,16–20].

One option for developing the design methodologies for CLT diaphragms is to reference a research program completed for precast-concrete diaphragms [21,22]. The work contributed to the development of a new section for seismic diaphragm design in ASCE 7-16 [23] and the new diaphragm force reduction factor, R_s [24]. CLT panels in diaphragms, like pre-cast concrete slabs, effectively remain elastic during heavy loading and are substantially more rigid than the connections between the panels, which contribute to the ductility desired in a LFRS. In many structures, ductility of connections is not used for connection design directly. Instead, vertical components of lateral force-resisting systems account for the overall ductility in the R-factors in the equivalent lateral force method [32] (ASCE 7-10, Eq. 12.8-2). The new ASCE 7-16 diaphragm design force reduction factor, R_s, accounts for, among other characteristics, ductility in the diaphragm specifically. Currently, R_s factors are available only for cast-in-place concrete, precast concrete, and wood sheathed diaphragms. Ductility data compiled in these tests can be some of the data used to create R_s factors for CLT diaphragms with half-lap and spline connections.

Input characteristics for CLT diaphragm computer models can be obtained from monotonic and cyclic tests of inter-panel shear connections. Connections of CLT butt joints using 8 × 180 mm, fully-threaded, self-tapping screws installed along the shear plane at a 30-degree angle-to-grain in one direction and a 45-degree angle-to-grain in the other direction, exhibited average monotonic load-based strengths (peak load) of 6.8 kN (1.5 kips) per screw and an initial stiffness of 3 kN/mm (17.1 kip/in) per screw [17]. Tests with 19 mm (3/4 in) plywood splines, 105 mm (4–1/8 in) 3-ply CLT, and vertical, 8 × 80 mm self-tapping screws in shear produced strengths (yield load) of 2.0 kN (0.45 kip) per pair of fasteners (load is transferred from one CLT panel, through one screw, across the plywood spline, to the second screw, and back into the second CLT panel) and stiffness values of 0.4 kN/mm (2.3 kips/in) per pair of fasteners [18]. Additional, U.S.-based research [20] investigated similar shear connections and compared experimental testing to the predicted design strengths in the National Design Specification for Wood Construction (NDS) [13], finding estimation indices both less than and greater than one, for different connection types.

1.3. Objectives

The goal of this research is to compare test values of cross-laminated timber (CLT) diaphragm connections in the seismic force-resisting systems to the design values calculated from formulas in the NDS (USA) [13] and the Eurocode [43]. Specific objectives are:

1. Using monotonic and cyclic tests, determine strength, stiffness, and load-deflection behavior for CLT inter-panel shear connections using SWG/Wuerth ASSY self-tapping screws.
2. Compare strength and stiffness results from specimens with varying characteristics, such as screw spacing and screw diameter.
3. Compare experimental strengths to design strengths calculated using the NDS [13] and the Eurocode [43].
4. Determine m-factors for use in acceptance criteria for the CLT connections in ASCE 41-13— Seismic Evaluation and Retrofit of Existing Buildings [25].
5. Characterize failure modes for the different connection types and loading protocols.

Table 1
Wood properties.

		V1 Grade CLT		DF-L #2	DF-L #3
		Characteristic test value[a]	Allowable design value [18]	Allowable design value [8]	Allowable design value [8]
$f_{c,0}$[b]	(MPa)	17.7	9.31	9.31	5.34
	(psi)	2565	1350	1350	775
E_0[b]	(MPa)	11,034	11,034	11,034	9655
	(psi × 10^6)	1.6	1.6	1.6	1.4
$f_{c,90}$[b]	(MPa)	10.1	5.34	4.31	4.31
	(psi)	1470	775	625	625
E_{90}[b]	(MPa)	9655	9655	–	–
	(psi × 10^6)	1.4	1.4	–	–
G[b]	(MPa)	–	–	3448	3448
	(psi × 10^6)	–	–	0.5	0.5

[a] Characteristic test values defined as population mean for stiffness properties and 5th percentile with a 75% confidence for strength properties, Table 1 in [18].
[b] $f_{c,0}$ = compressive strength parallel to grain or major direction. E_0 = modulus of elasticity parallel to grain or major direction. $f_{c,90}$ = compressive strength perpendicular to grain or major direction. E_{90} = modulus of elasticity perpendicular to grain or major direction. G = specific gravity, Tbl. 4a in [8].

2. Materials and methods

2.1. Test specimens

Six different specimen constructions were used in this research. Each specimen consisted of three separate CLT panels (each 2.44 m × 0.610 m) connected side-by-side along the long edge. CLT panels were three-ply, Douglas fir-Larch (DF-L) layups, with DF-L #3 in the core layer and DF-L #2 in the outer layers [26]. See Table 1 for properties of the wood and CLT grades. The panel producer has been given certification by the APA (Engineered Wood Association) for their V1-designated CLT product, defined as "No. 2 Douglas fir-Larch lumber in all parallel layers and No. 3 Douglas fir-Larch lumber in all perpendicular layers" [26]. At the time of procurement, the manufacturer was certified to only produce V1-grade [26] panels with DF-L #2 in all layers; however, the panels in this study were manufactured with DF-L #3 in the core layer and DF-L #2 in the outer layers only. This layup was chosen to match the PRG-320 [26] definition.

Half-lap and surface spline connection types were tested with both 8 mm and 10 mm self-tapping screws at spacings of 152 mm (6 in) and 305 mm (12 in). Table 2 shows the details of the screws used in this project. See [27] for the Würth/SWG ASSY self-tapping screw ICC ES technical report and [28] for the European Technical Approval.

Table 2
Self-tapping screws used in specimens.[a]

Fastener Type	Brand	Model	Nominal Diameter (d_{thread})	Fastener Length (L)	Thread Type
Self-tapping Screw	Würth/ SWG	ASSY Eco	8 mm	100 mm	Partial
			10 mm	100 mm	Partial
		ASSY VG CSK	8 mm	120 mm	Full
			10 mm	140 mm	Full

[a] Technical data sheet in [27,28].

Table 3
Test matrix.

Construction #	Test #	Load type	Connection type	Thread type	Spacing	Screw dia.	Δ_{ref} (mm)
1	1	Monotonic	SS	Partial	152 mm	8 mm	–
1	2	Monotonic	SS	Partial	152 mm	8 mm	–
1	3	Cyclic	SS	Partial	152 mm	8 mm	24
1	4	Cyclic	SS	Partial	152 mm	8 mm	81
2	5	Monotonic	SS	Partial	152 mm	10 mm	–
2	6	Monotonic	SS	Partial	152 mm	10 mm	–
2	7	Cyclic	SS	Partial	152 mm	10 mm	41
2	8	Cyclic	SS	Partial	152 mm	10 mm	41
3	9	Monotonic	SS	Partial	305 mm	10 mm	–
3	10	Monotonic	SS	Partial	305 mm	10 mm	–
3	11	Cyclic	SS	Partial	305 mm	10 mm	41
3	12	Cyclic	SS	Partial	305 mm	10 mm	41
4	13	Monotonic	SS	Full	152 mm	8 mm	–
4	14	Monotonic	SS	Full	152 mm	8 mm	–
5	15	Monotonic	HL	Full	152 mm	10 mm	–
5	16	Monotonic	HL	Full	152 mm	10 mm	–
5	17	Cyclic	HL	Full	152 mm	10 mm	15
5	18	Cyclic	HL	Full	152 mm	10 mm	30
6	19	Monotonic	HL	Full	305 mm	10 mm	–
6	20	Monotonic	HL	Full	305 mm	10 mm	–
6	21	Cyclic	HL	Full	305 mm	10 mm	15
6	22	Cyclic	HL	Full	305 mm	10 mm	15

Notes: 1. "SS" = surface spline, "HL" = half-lap.

SWG ASSY 3.0 Eco model and VG CSK model wood screws were used. SWG ASSY Eco (partially-threaded and counter-sunk) screws were used for three of the four surface spline connections, and SWG ASSY VG CSK (fully-threaded) screws for one surface spline connection and all the half-lap connections. Edge and end spacing of 7D and 22.5D, respectively, were determined using the Douglas-fir (*Pseudotsuga menziesii*, specific gravity, SG > 0.42) values in the ICC-ES Report ESR-3179 for SWG ASSY 3.0 Wood Screws [27]. Although ESR-3179 [27] is applicable for sawn lumber and not CLT or plywood, results from these tests show that the plywood side member, which is less than the 45 mm minimum thickness that ESR-3179 [27] requires, performs satisfactorily with no splitting of the plywood or CLT.

A complete test matrix is given in Table 3. Two replicates per test type were tested based on the requirements given in ASTM E455 [29] and ASTM E2126 [30]. Although the peak strengths of four of the eleven tests were not within 10% their replicate as the ASTM standard requires (or a third test must be completed), due to budget limitations and the desire to have a wider range in construction variation rather than higher precision in results, this third test was omitted. The last column, Δ_{ref}, shows the reference displacement (defined in [31]) used for the cyclic tests. Figs. 1 and 2 show perspective views of the surface spline and half-lap specimen types. Figs. 3a–3f contain detailed information on each construction type. Figs. 3e and 3f show the unique WSSW install pattern (**W**ithdrawal/**S**hear/**S**hear/**W**ithdrawal). The screws at either end of the joint are angled at 45°, contributing to the shear capacity of the connection either in screw withdrawal or compression, depending on the direction of the shear force, while the screws in the middle of the joint are in pure shear (installed vertically).

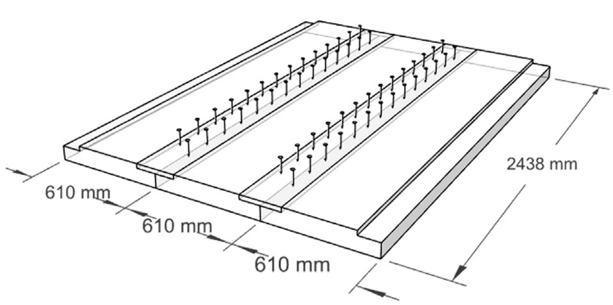

Fig. 1. Surface spline specimen (perspective view).

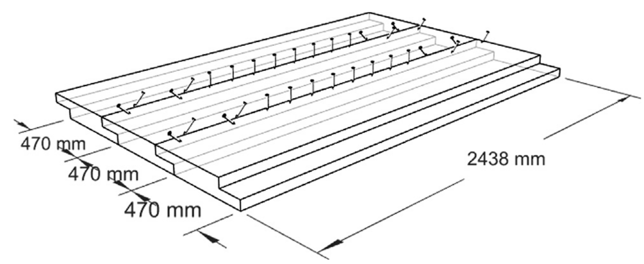

Fig. 2. Half-lap specimen (perspective view).

Although fully-threaded screws were used in shear in the half-lap specimens, it may be more cost effective to use partially threaded screws instead. 25.4 mm (1 in) plywood was used for the surface splines. Moisture content was measured to be between 5% and 10% for the plywood and CLT in all specimens within 30 min of testing.

The specimens tested in this study were designed based on the following considerations:

(1) CLT connection types chosen were expected to be frequently used panel-to-panel shear connections in CLT diaphragms. Ease of fabrication, efficient utilization of raw material, speed of on-site installation, and realistic screw spacing were considered.
(2) The half-lap installation pattern (**W**ithdrawal/**S**hear/**S**hear/**W**ithdrawal) was chosen due to its unique structural performance (high initial stiffness, high strength, and sustained post-peak resistance with vertical screws in shear, and lack of previous test data. It was recommended for testing based on previous smaller scale tests [18].
(3) Splines, in general, were chosen because of their extensive use in CLT diaphragm applications.
(4) A 25 mm (1-inch) surface spline was chosen over thinner, more commonly used plywood options for increased connection strength, which was found to be largely dependent on the thickness of the plywood [18].

2.2. Test apparatus

All tests were conducted using the set-up shown in Figs. 4–6. The apparatus was designed and built specifically for this testing program.

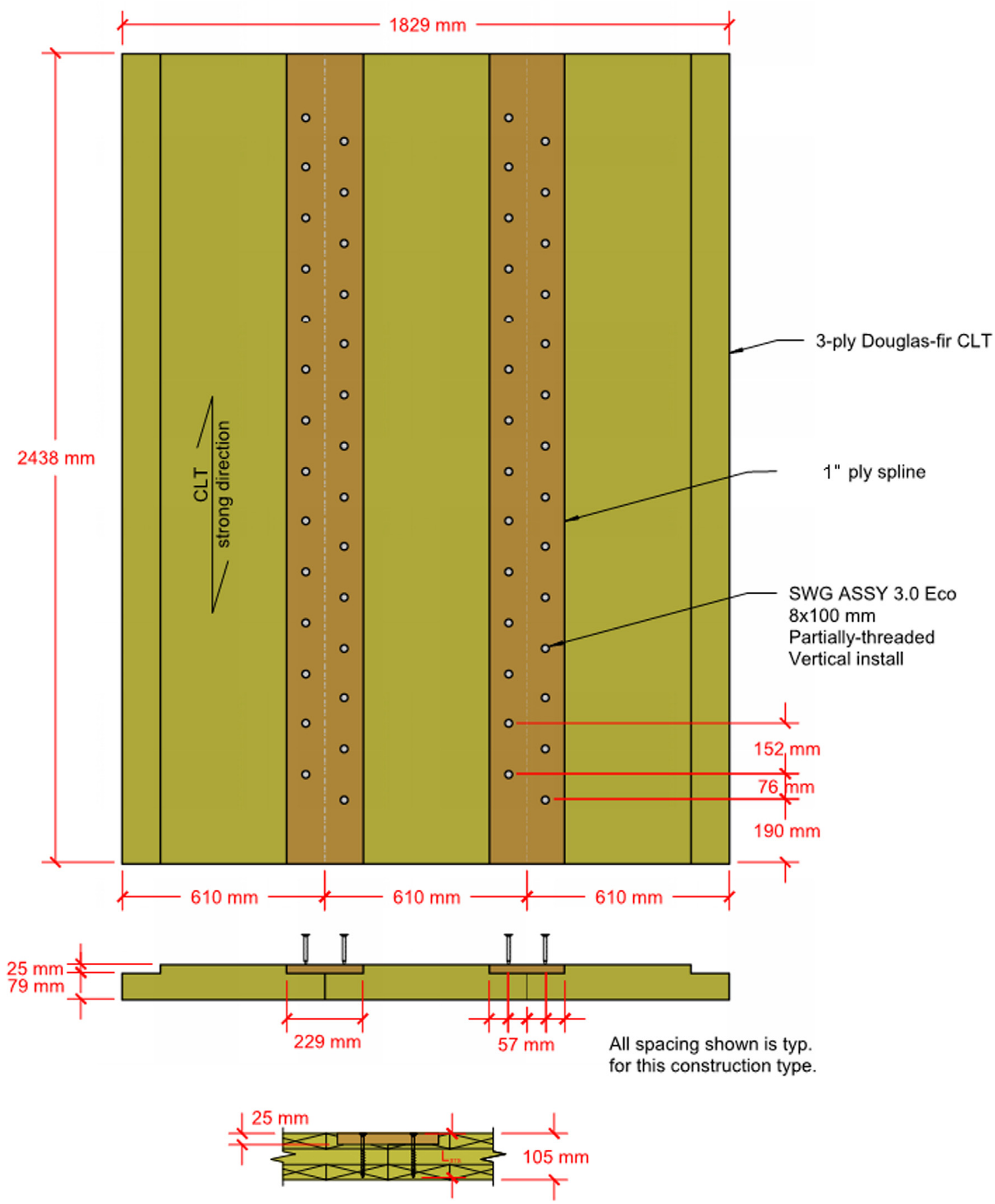

Fig. 3a. Construction 1 details.

See Table 4 for a label legend of the apparatus components.

CLT test specimens were positioned flat, parallel to the strong-floor (1) underneath. A 489 kN (110 kip) hydraulic actuator (9) with a stroke capacity of 152 mm (6 in) was used to load the center panel (4). Wood blocking (7) between the steel pipes (13) and the specimen was installed under each separate panel of the specimen to support the specimen vertically and to mitigate out-of-plane movement of the specimens. Although vertical restraints of the hydraulic cylinder knuckle (8) were in place for all tests, lateral restraint (10) was added only after all the monotonic tests were completed and was in place for all the cyclic tests. For several of the monotonic tests, sideways buckling of the apparatus occurred at the end portion of the tests, as shown in Fig. 7, on the scale of 25–50 mm, but had no apparent effect on the results of the data analysis. Most of this movement occurred after the peak load of the test specimen was reached.

Linearly variable differential transducers (LVDTs) were used to measure displacements at four locations on the spline specimens (Fig. 4a-#6, 5-#6) and two locations on the half-lap specimens (Fig. 4b-#6). The hydraulic cylinder used an externally-mounted LVDT for both controlling and measuring displacement. The four LVDTs on the spline connections measured relative displacement between the CLT panel and plywood spline, near the center of the specimen. Measurements taken from the hydraulic cylinder LVDT were used for the displacement data analysis. Measurements from the LVDTs located on the CLT specimens were used for verification of the hydraulic cylinder LVDT measurements.

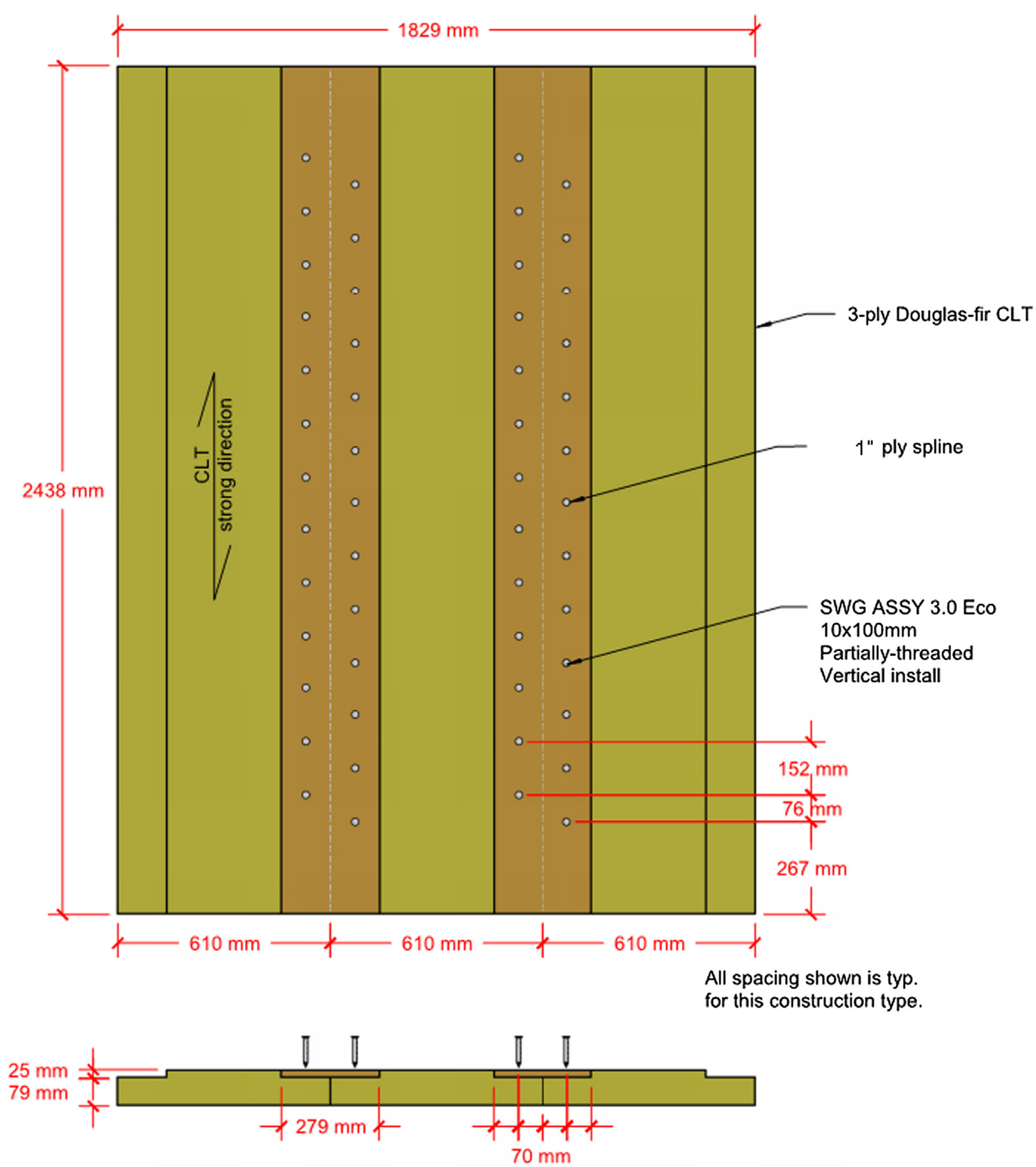

Fig. 3b. Construction 2 details.

2.3. Loading protocols

The monotonic test series was based on ASTM E455 [29]. Tests were displacement-controlled at 5 mm (0.2 in) per minute and stopped when the load reached approximately 50% of peak load.

ASTM E2126-11 [30] and the CUREE protocol (Fig. 8) [31] were followed for the reverse-cyclic tests. Loading for cyclic tests consisted of a total of 37 cycles including seven primary cycles and lasted approximately seven minutes. A reference displacement (denoted as Δ in Fig. 8, denoted elsewhere as Δ_{ref}) was chosen for each of the two connection types (surface spline, half-lap) based on the results of the monotonic tests. For each monotonic test, 60% of the displacement at 80% of peak load, post-peak, ($\Delta_{ref} = 0.6*\Delta_{P_{80\%}}$) was calculated. Then, for each connection type a Δ_{ref} was chosen based on: (1) average deflection calculated for all monotonic tests of that type; (2) 152 mm maximum extension of the hydraulic actuator; and (3) deflection required for sufficient specimen failure. Δ_{ref}'s for each cyclic test are shown in Table 3. A Δ_{ref} of 41 mm was chosen for the surface spline connections and 15 mm for the half-lap connection specimens. Δ_{ref} was modified for three cyclic tests to study the effect.

2.4. Data analysis

Two separate idealized curves were created from both the monotonic and cyclic load-deflection plots.

Fig. 9 shows an Equivalent Energy Elastic-Plastic (EEEP) curve and the respective performance parameters following ASTM E2126 [30]. The initial elastic portion of the graph passes through 40% of peak load, rising to the yield load. The location and extent of the horizontal plastic plateau is defined such that the area under the idealized EEEP curve is equal to the area under the monotonic load-deflection curve and terminates when 80% of peak load (post-peak) is reached.

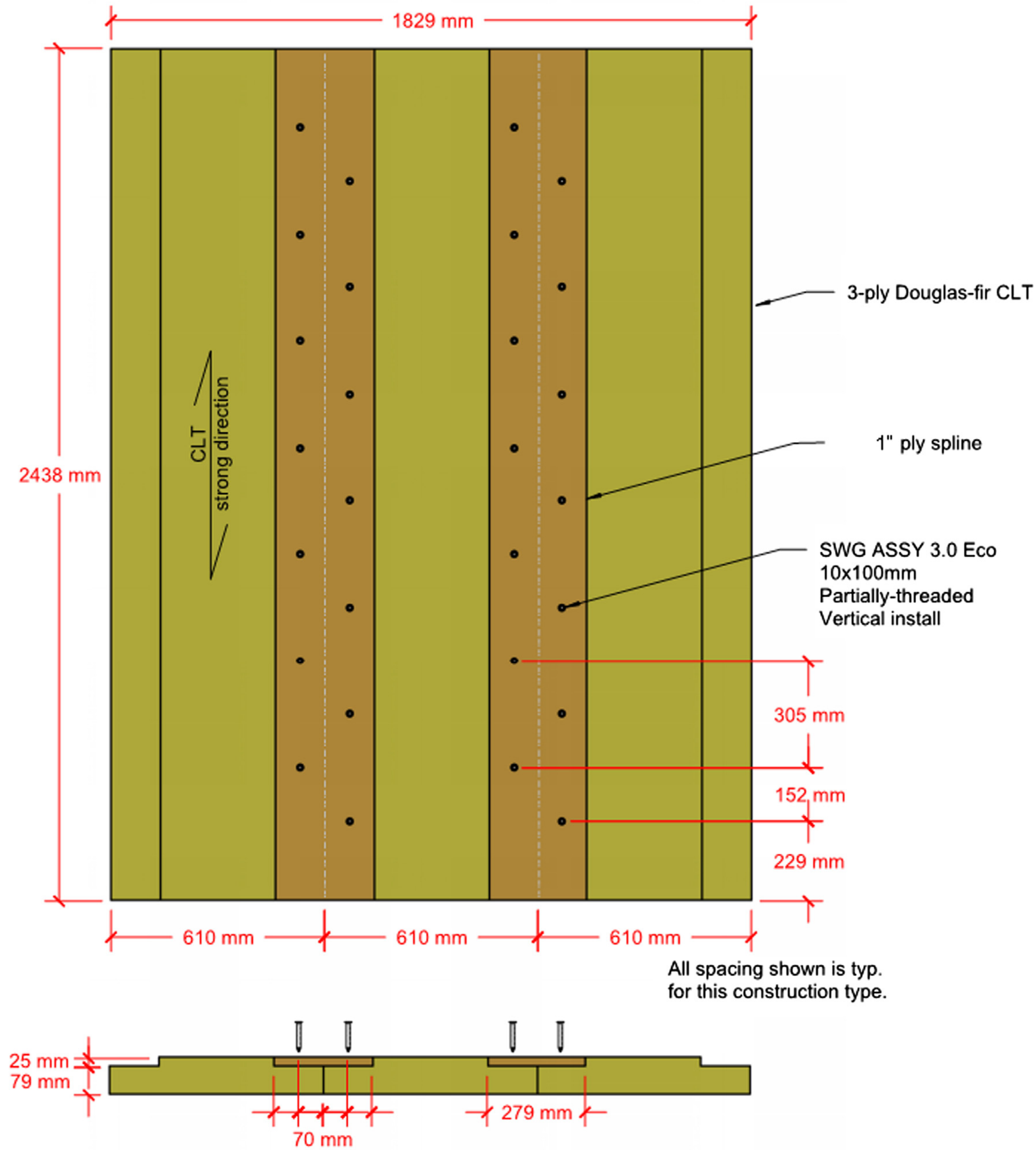

Fig. 3c. Construction 3 details.

Fig. 10 qualitatively shows the second idealized backbone, which originates from ASCE 41-13 [25]. ASCE 41-13 provides performance acceptance criteria for a variety of construction types and structural components for use in seismic retrofits. These criteria involve m-factors for linear analyses and deformation ratios for nonlinear analyses. m-Factors can be calculated for components or connections that aren't already included in ASCE 41-13 [25] and used in evaluating acceptance criteria for existing structures. m-Factors developed for a certain structural component or connection and incorporated into ASCE 41-13 [25] can be used to increase the strength of the component in building rehabilitation design to take advantage of the post-yield residual strength in a linear analysis. In this project, m-factors and deformation ratios are developed for CLT diaphragms with the tested connection types. The m-factors for CLT connections will be beneficial for designers to have a better understanding of their ductility and potentially be able to undertake a retrofit of an existing building using CLT. ASCE 41-13 [25] provides a framework for performance-based retrofit design of existing structures while ASCE 7-10 [32] does not yet include such provisions for new or existing buildings. The m-factors are useful in a performance-based design context and reflect the ductility of the components/connections at various performance levels. Several theoretical and experimental studies have been completed regarding various aspects of timber floor retrofits in historical buildings [33–41].

A linear or nonlinear analysis of a building can be performed using ASCE 41-13 [25]. Four options are available: a linear static procedure (LSP); a linear dynamic procedure (LDP); a nonlinear static procedure (NSP); and a nonlinear dynamic procedure (NDP). Linear procedures are allowed for buildings that do not contain irregularities such as weak stories, in-plane/out-of-plane discontinuities, or torsional strength irregularities. LSP is the simplest method with a pseudo-seismic force used to calculate internal forces and system displacements for regular buildings. However, the NSP ("push-over analysis") is generally more

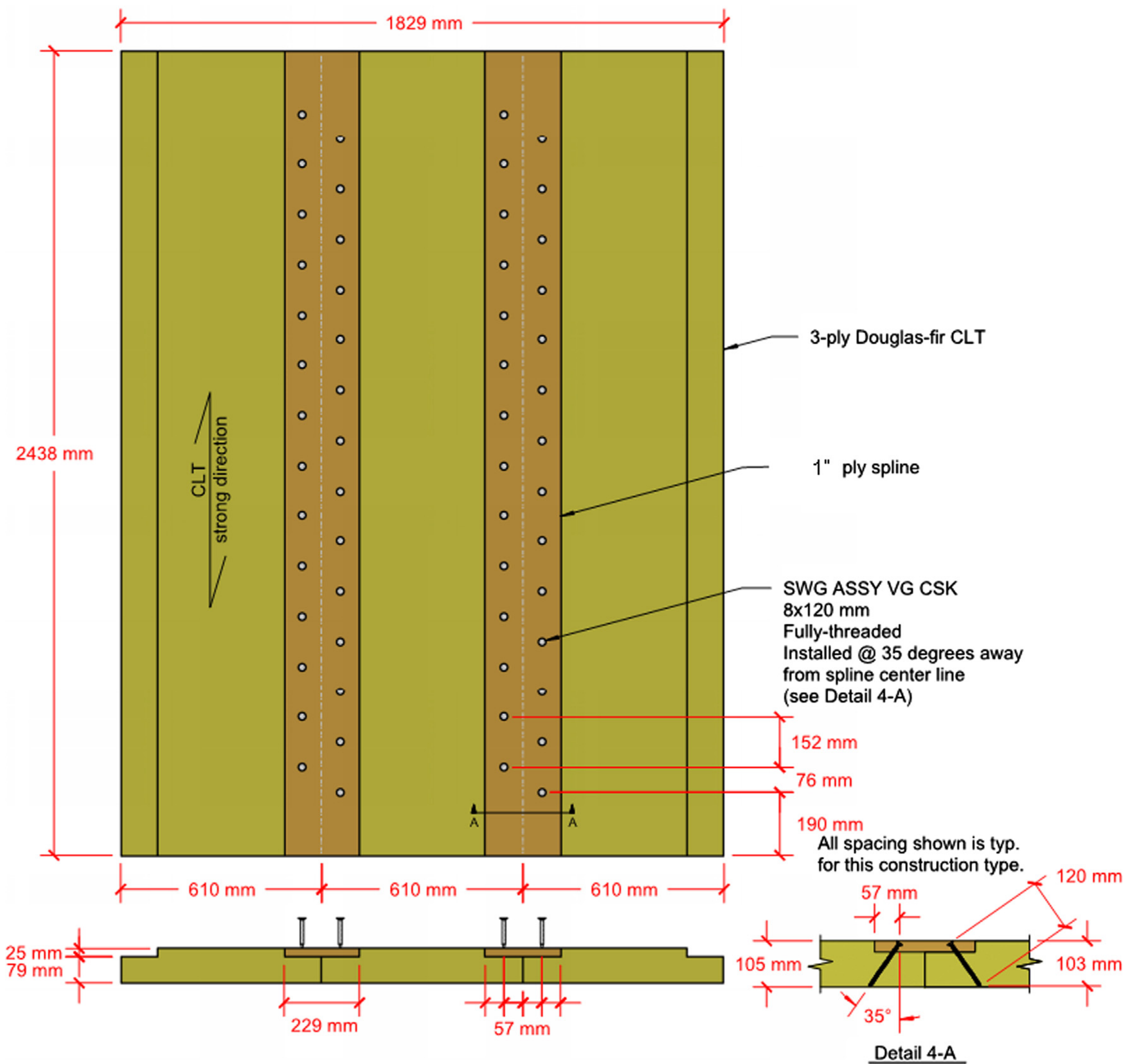

Fig. 3d. Construction 4 details.

reliable yet cannot accurately include changes in stiffness in the dynamic response of the structure as the strength degrades or for higher mode effects. The NDP is the most complex and permitted for all buildings but requires considerable engineering judgment and experience. The m-factor is "an indirect measure of the nonlinear deformation capacity of a component" and is used in the acceptance criteria for linear procedures. Acceptance criteria for nonlinear analyses involve expected deformation capacities, or deformation ratios. They are directly proportional to the m-factors that appear in Eq. (1) below, where the m-factor adjusts (increases) the strength of the component used in design to take advantage of the post-yield residual strength in a linear analysis.

$$m\kappa Q_{CE} > Q_{UD}$$
(Eq. 7-36, [25]) (1)

m is defined as the "component capacity modification factor (and accounts) for expected ductility associated with this action at the selected Structural Performance Level." Q_{CE} is the "expected strength of the component deformation-controlled action of an element at the deformation level under consideration." The knowledge factor, κ, reflects uncertainty in the collection of structural data. Q_{UD} is the "deformation-controlled action caused by gravity loads and earthquake forces."

The first step in determining m-factors from experiments is to create an idealized backbone curve from the cyclic test data. Once the backbone is created, key deflection and load values can be obtained from the multi-linear curve. The m-factors at each Performance Level are calculated as the associated deflection divided by the deflection at yield load and multiplied by a reduction factor of 0.75. Deformation ratios used for nonlinear analysis are defined as Δ/Δ_y, where Δ is the displacement at each performance level and Δ_y is the displacement at yield. Fig. 10 includes three "Performance Levels": Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP). Deformation for CP occurs at peak load. Deformation for LS is 75% of the deformation at CP. Deformation for IO is 67% of the deformation for LS.

The m-factors are used in calculating the "pseudo lateral force", V, on a structure, for the LSP procedure. Member forces determined from V, Q_{UD} (in Eq. (1)), are compared to the expected component strengths, Q_{CE}, in Eq. (1). For connection hardware, Q_{CE} is "taken as the average ultimate test values from published reports" [25].

3. Results and discussion

Results from all tests are summarized in Table 5 and raw data from representative tests can be found in Figs. 11-1 to 11-17. The full set of raw data can be found in [33]. Although the relatively small sample size for each test prevents the values being recommended for design at this time, the strength and stiffness values provide the motivation for additional testing. They are also useful for structural engineers to know

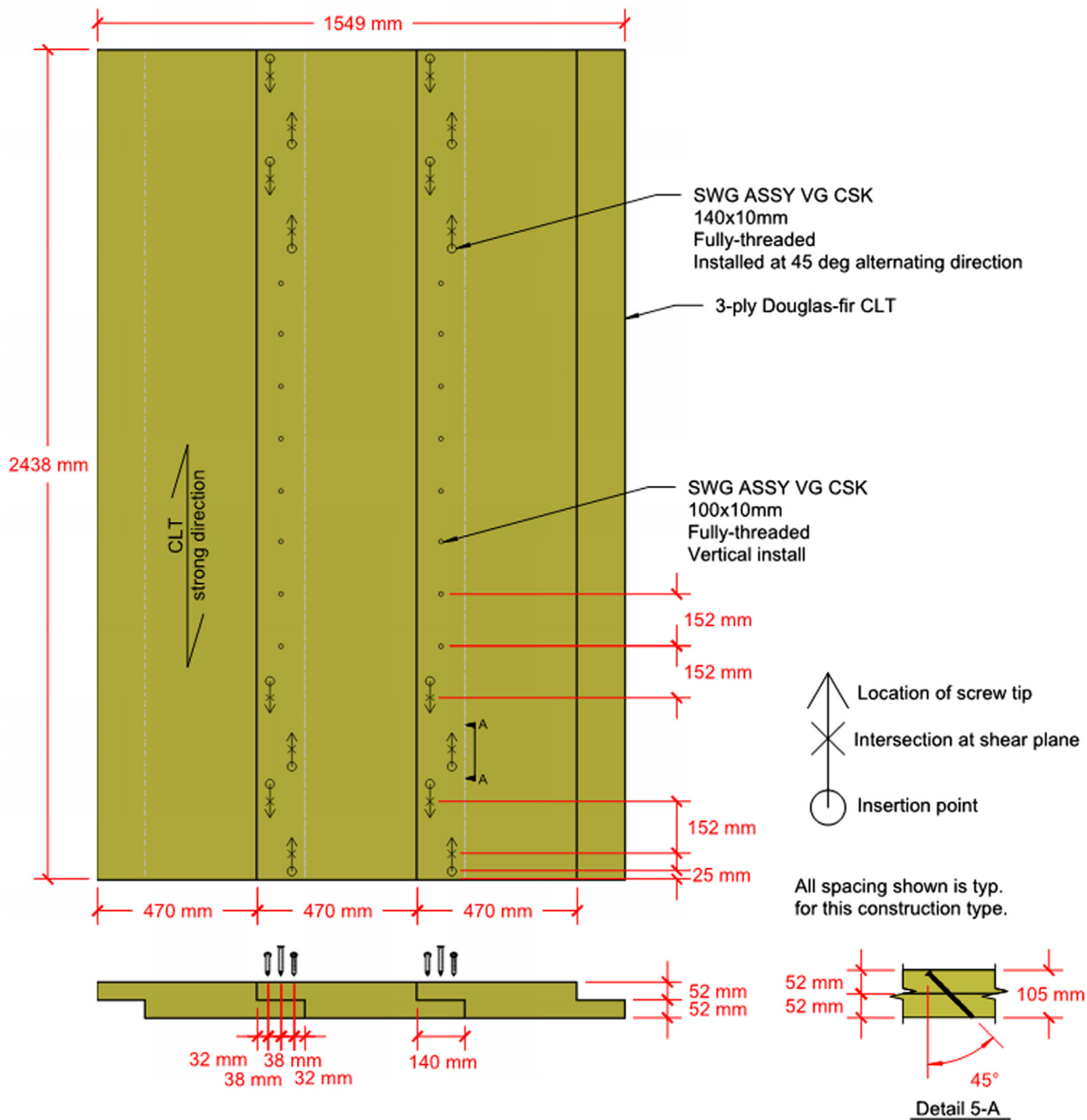

Fig. 3e. Construction 5 details.

the conservative nature of the methods used in designing connections like those tested in this research.

Definitions of variables in Table 5 are as follows:

- **Load Type** – "M" is monotonic, "C" is cyclic.
- # of fasteners
- The per fastener designation for the half-lap specimens considers a group of fasteners consisting of two vertical screws and two angled screws (one in tension and one in compression). The total number of fasteners in a specimen counts the total number of groups in each shear plane (ie. Construction 5, Fig. 3e, contains 8 total "fasteners", or fastener groups). For the surface spline specimens, the "per fastener" designation is defined as the sum of the number of "screw pairs" along the left joint and right joint (i.e. Construction 3, Fig. 3c, has seven screw pairs along the left joint and seven screw pairs along the right joint, summing to a total of fourteen fasteners for that construction type).
- **Screw diameter, D_n** – Nominal diameter of screw.
- **Test peak load, V_{max} –** Peak load that the specimen resisted during the test.
- **Peak unit shear load** – V_{max} divided by the length of the joint, 2.44 m.
- **EEEP yield load, V_y** – The yield load from the EEEP curve.
- **Displacement at peak load, Δ_{max}** – Displacement at peak load.
- **Displacement at yield, Δ_y** – Displacement at the yield point on the EEEP curve.
- **Ductility, μ –** $\frac{\Delta_{max}}{\Delta_y}$
- Stiffness, k
- Load at 40% of peak load (not tabulated) divided by displacement at 40% of peak load (not tabulated, as defined in Sec. 3.2.2 of [30]).
- **Stiffness per fastener –** Stiffness, k, divided by # of fasteners, as defined above.

Surface spline capacities ranged from 4.8 kN (1.08 kips) to 10.1 kN (2.27 kips) per fastener, and half-lap connection capacities ranged from 30.6 kN (6.89 kips) to 43.2 kN (9.72 kips) per fastener. Stiffness values for surface splines ranged from 0.36 kN/mm (2.1 kips/in) per fastener to 0.87 kN/mm (5.0 kips/in). Stiffness values for half-lap connections ranged from 4.3 kN/mm (25 kips/in) per fastener to 5.5 kN/mm (31 kips/in). See definition for the "per fastener" designation in table variables list above.

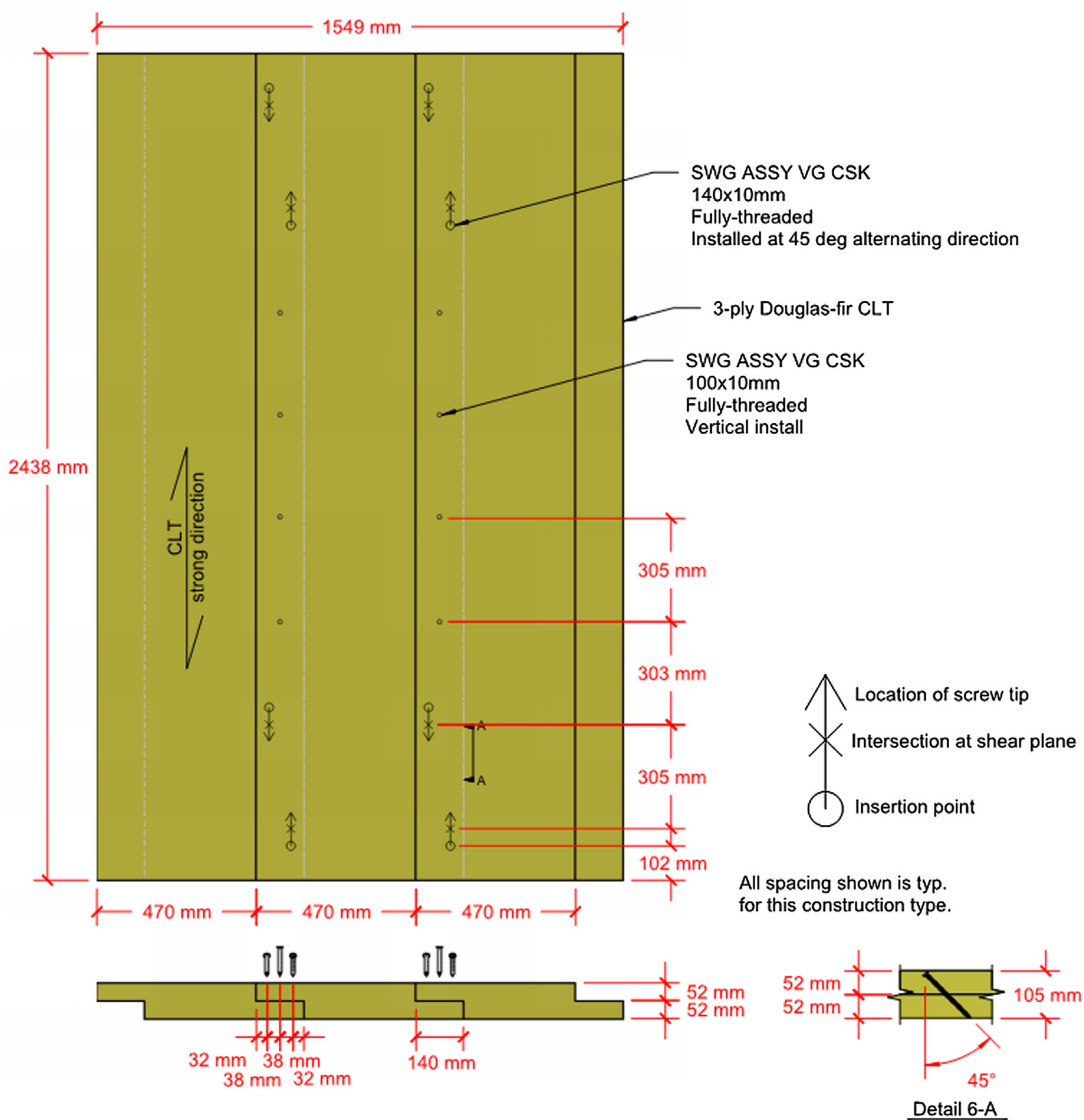

Fig. 3f. Construction 6 details.

The spline specimens exhibited a lower initial stiffness and resisted the peak load for a greater displacement (high ductility) (i.e. Fig. 11-1) compared to the half-lap specimens. The half-lap connections have higher initial stiffness values than the surface spline connections due to the axial engagement of the angled screws. The WSSW combination of shear and withdrawal screws in the half-lap connections creates a two-part load-deflection curve: the withdrawal screws create the initial high stiffness, high strength behavior while the vertical screws in the half-lap connections contribute to the ductile post-peak behavior (i.e. Fig. 11-15).

Due to the higher stiffness of the screws in withdrawal, the strength of the half-lap connection is conservatively calculated using only the number of angled screws while the vertical screws are ignored. Using the stiffness values of the surface spline and half-lap connections listed above, it is apparent that the surface spline connection had stiffness values that were roughly 10% that of the stiffness values of the half-lap connections. It could be argued that at least 10% of the strength of the half-lap connection was contributed by the vertical screws in shear, but likely more, as the main member in the half-lap was 52 mm thick while the plywood spline was 25 mm thick. Table 7.1 in EN-1995 [43] provides an equation to calculate stiffness (joint slip) for screws in shear based on wood density and diameter of screw, with no dependency on thicknesses of wood members. The slip modulus, K_{ser}, for an 8 mm screw in Douglas-fir lumber (*Pseudotsuga menziesii*, density of 500 kg/m^3) is $\frac{\rho_m^{1.5} d}{23}$ = 3890 N/mm, is significantly higher than the experimental stiffness values of the surface spline, which were in the range of 390–560 N/mm per fastener. This value would be halved if the total number of screws in the specimen were used to determine the per fastener stiffness, instead of the previous definition of "per fastener" which counts fastener pairs in the surface spline connections.

For the half-lap connections, the axial stiffness, K_{ax}, is calculated to be 6440 N/mm^2 for one 10 mm dia. screws at a 45 deg. angle with 62 mm penetration length of the screw (Eq. 1.4 of A.1.3 in the Würth ETA [28]). Accounting for the full "unit" connection of two oppositely angled screws with an approximately equal thickness side and main member, the singular screw's axial stiffness is converted to the shear stiffness of the full connection as follows: $K_{ser} = K_{ax} * \cos^2(45\text{deg}) = 3220$ N/mm [42]. This K_{ser} is now the value for a half-lap "fastener" (fastener group) as defined in the definitions list for Table 5, which consists of two angled screws and two vertical screws. This stiffness is relatively close to the measured stiffness for the

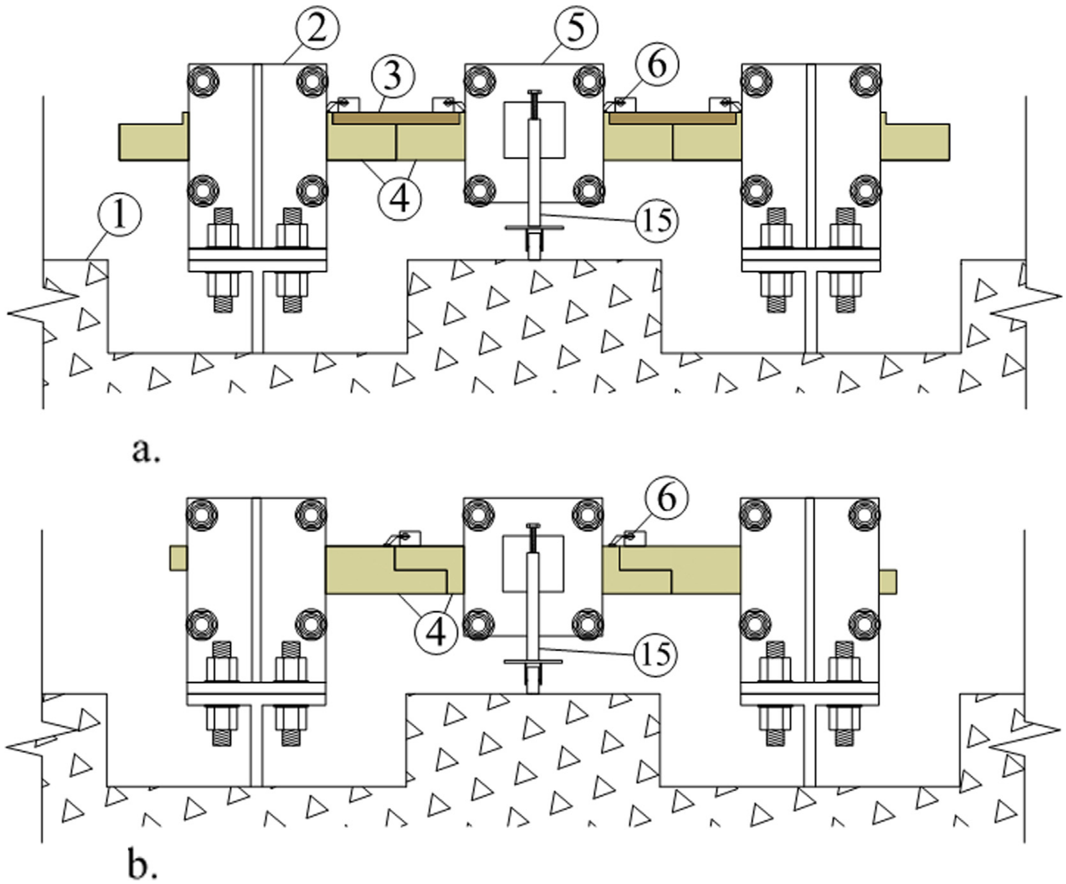

Fig. 4. Test apparatus (a. front view spline specimen, b. front view half-lap specimen).

monotonic tests seen in Table 5 for Construction 5–3710 and 3810 N/mm^2. The results for the two monotonic tests are 70% higher at 5500 and 5240 N/mm^2. Construction 6 stiffness values are 2.5 times higher than the values for Construction 5 (average of both monotonic and cyclic tests).

Section 8.1.2-(3) of EN-1995 [43] reminds designers that, “when a connection comprises different types of fasteners, or when the stiffness of the connections in respective shear planes of a multiple shear plane connection is different, their compatibility should be verified.” Section 11.1.4 in the NDS [13] reminds the designer that, “methods of analysis and test data for establishing reference design values for connections made with more than one type of fastener have not been developed.

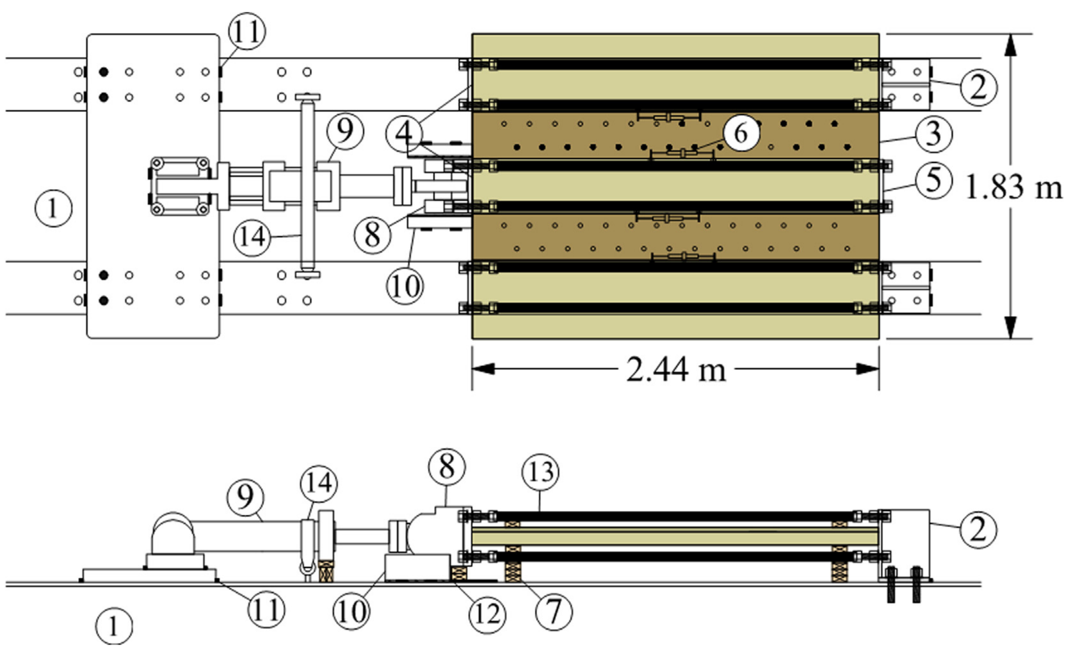

Fig. 5. Test apparatus (top and side view).

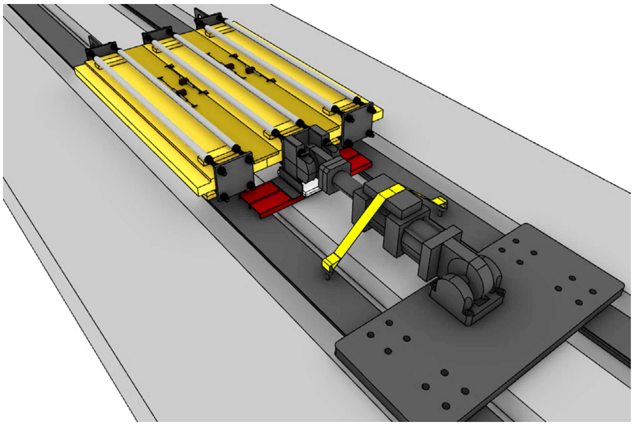

Fig. 6. Rendered perspective view of test apparatus.

Table 4
Test apparatus component legend.

ID #	Apparatus component
1	Concrete strong-floor
2	Bump-stop
3	Plywood spline
4	CLT panel
5	End plate
6	LVDT
7	2 × 4 wood blocking
8	Knuckle
9	Hydraulic actuator
10	Lateral restraint
11	Welds
12	Friction-reducing plastic plates
13	Steel pipes with nuts welded to ends
14	Hold-down ratchet straps
15	End plate roller

Reference design values and design value adjustments for mixed fastener connections shall be based on tests or other analyses (see 1.1.1.3)." These should be considered when analyzing stiffness and strengths of the WSSW half-lap connections with half of the screws in shear and half of the screws in withdrawal at 45-degree angles.

A method of estimating the shear resistance contribution from the vertical screws is to obtain the resisted load of the shear screws from the surface spline tests at the yield displacement location of the half-lap tests. For example, the half-lap connection with 10 mm diameter screws at a 150 mm spacing yielded at approximately 7 mm (average of all four tests, two monotonic and two cyclic) (Table 5). Figs. 11-4 and 11-5 show that the surface spline connection with 10 mm diameter screws at 150 mm spacing resisted a total load of approximately 50 kN for the monotonic tests and 100 kN for the cyclic tests at a displacement of 7 mm. Dividing this total resisted load at 7 mm displacement by the number of fasteners (26) gives a contribution of 2 kN per screw of resistance for the monotonic tests and 4 kN per screw of resistance for the cyclic tests at the yield point of the half-lap tests. The half-lap tests (Tests #15-18) resisted, on average, 32.7 kN for the monotonic tests and 34.6 kN for the cyclic tests. Thus, the vertical screws contributed $\frac{2\text{ kN}}{32.7\text{ kN} - 2\text{ kN}} = 5.6\%$ of the yield load in the monotonic tests and $\frac{4\text{ kN}}{34.6\text{ kN} - 4\text{ kN}} = 13\%$ of the yield load in the cyclic tests. This is in line with the estimated 10% contribution described above and again, this actual value is likely higher due to the thicker side member in the half-lap connection (52 mm vs 25 mm). That said, no further experimental studies have been carried out to investigate the effect on strength from this combination of screw orientations.

With regards to resistance from friction between wood components, although the angled screws in tension will develop increased frictional resistance in the connection (pulling the panels together), the angled screws in compression (pushing the panels apart) will counteract this effect. Further discussion of the friction from shear connections using self-tapping screws can be found in [10,15].

The failure mode for the monotonic surface spline tests was consistent across specimens and consisted of Mode III_s [13], which characterizes behavior up to the yield point. This matched the predicted failure mode calculation using the NDS [13]. Mode III_s [13] involves bending/yielding of the screw in one location in the side member (plywood) and crushing of the wood in the side and main member (CLT). Post-yield failure behavior is pull-through of the screw through the plywood spline. The cyclic surface-spline tests appeared to produce failures due to bending fatigue of the screws at the shear plane (between the CLT and the plywood spline) as the force on the screw put the screw into bending and caused associated sounds in the last few cycles of the tests. The monotonic half-lap failure was consistent for all specimens and was characterized by pull-through into or push-out from the CLT panel of the angled screws and bending (Mode III_m [13]) of the vertical screws. Yield mode calculation for the vertical screws was not completed. Most of the cyclic half-lap tests did not fail suddenly and were still physically connected at the end of the tests. Energy dissipation occurred through wood-crushing and incomplete withdrawal of the angled screws.

For an increase of 41% in cross-sectional area of the screw (based on root diameter), peak and yield loads per fastener for the monotonic tests both increased by 34%, while the cyclic tests saw increases of 62% and 54%, respectively. Stiffness per fastener decreased by 17% for monotonic tests but increased 43% for cyclic tests. Further, yield deflection increased 74% for monotonic tests, but only increased 6% for cyclic tests.

Screw spacing appeared to affect per fastener strengths in monotonic tests but not in cyclic tests. Load capacity per fastener in

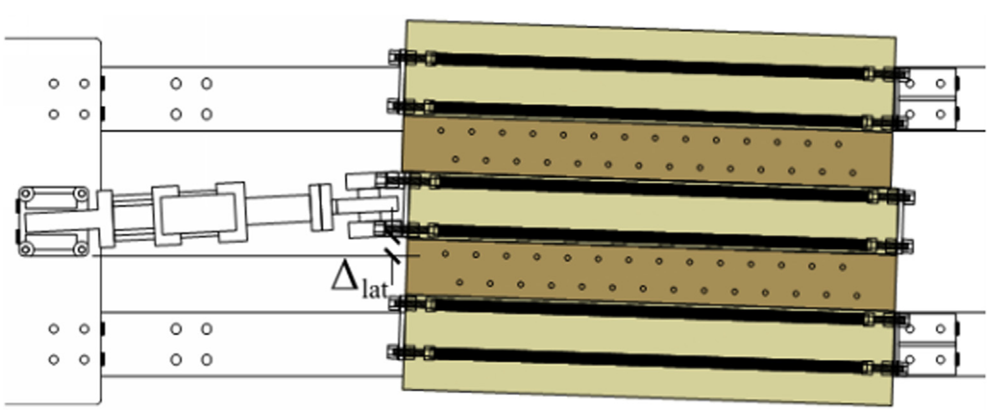

Fig. 7. Lateral movement of test apparatus occurring at end of monotonic tests.

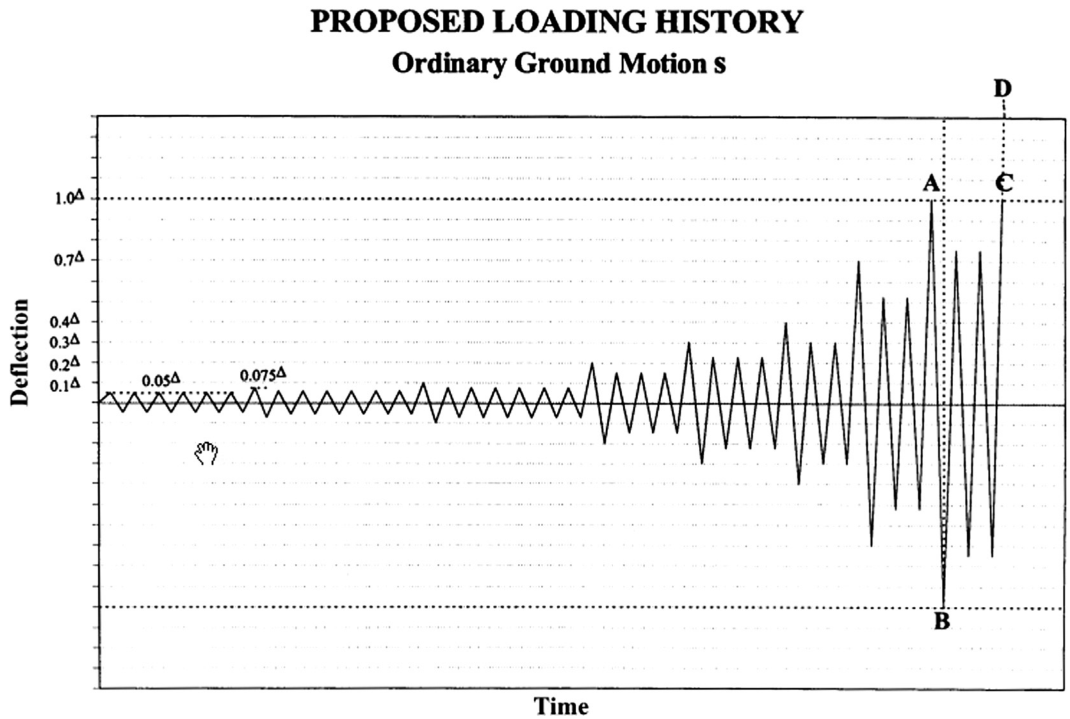

Fig. 8. CUREE load protocol [31]

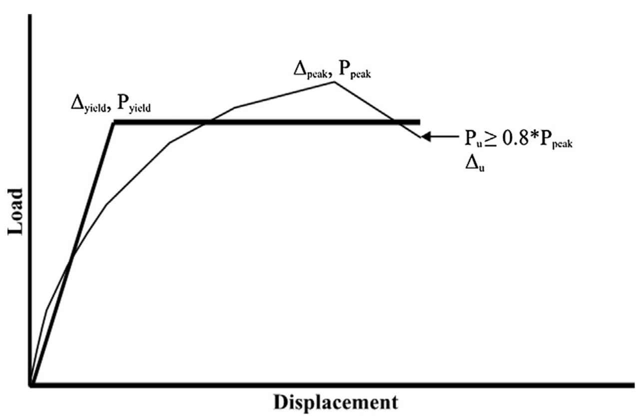

Fig. 9. EEEP Performance parameters [30]

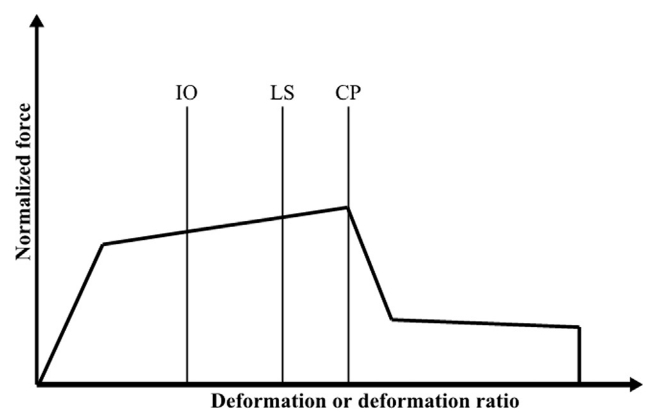

Fig. 10. ASCE 41-13 Performance Levels [25]

connections with less screws per joint were 20% higher than the capacities of the connections with double the number of screws per joint. There was no change in the load capacity per fastener for the cyclic tests of both constructions, indicating no group action. Stiffness per fastener were also 20% higher for spline connections with less screws compared to the specimens with double the screws, 15% higher for the monotonic half-lap tests, and 36% higher for the cyclic half-lap tests. As the number of fasteners per joint increased, deflection at yield increased by 15% on average for spline tests and 50% on average for half-lap connection tests.

The Eurocode [43], the New Zealand timber code [44], and the NDS [13] all discount the strength per fastener if multiple fasteners are used in a row parallel to grain. These factors exist for two reasons: (1) to reduce the chance of splitting in the timber, and (2) to account for the group action, common in bolted connections [13]. This reduction factor accounts for the fact that the load on each connector is not equal; the end connectors in a row parallel to the direction of the applied force have larger loads than the ones in the center of the row. Thus, adding connectors to the row does not appreciably increase the strength of the connection since the end fasteners control the strength.

The NDS [13] uses the group action factor, C_g, for adjustment of reference lateral design values of dowel-type fasteners with diameters between 6.4 mm (1/4 in) and 25.4 mm (1.0 in). The root diameter of the fasteners in these tests were 5.3 mm (8 mm nominal, outer thread diameter) and 6.3 mm (10 mm nominal, outer thread diameter), which are close to the 6.4 mm (1/4 in) cut-off in the NDS [13]. C_g uses the modulus of elasticity, the load-slip modulus of the fastener type, the dowel quantity, spacing, and diameter, and the gross cross-sectional areas of the side and main members to determine the reduction factor for multiple fasteners. C_g varies from 0.80 to 0.90 depending on diameter size and values of the other inputs.

The New Zealand timber code [44] tabulates the reduction factor for multiple bolts or "coach" screws (lag screws) in a connection. For 5–9 screws, the factor is 0.95, for 10–15 screws the factor is 0.80, and for 16 or more screws, the factor is 0.62.

The Eurocode [43] reduces the number of screws or bolts to an "effective number" of fasteners with an equation that uses dowel diameter, spacing, and quantity (Eq. 8.34, [43]).

Table 5
Results summary.

Type	Construction #	Test #	Load Type	# of fasteners	Screw diameter	Test peak load	Peak load per fastener	Peak unit shear load	EEEP yield load	Displacement at peak load	Displacement at EEEP yield	Ductility	Stiffness	Stiffness per fastener
					D_n (mm)	V_{max} (kN)	(kN)	(kN/m)	V_Y (kN)	Δ_{max} (mm)	Δ_y (mm)	μ	k (kN/mm)	(kN/mm)
Column ID	1	2	3	4	5	6	7	8	9	10	11	12	13	14
Spline	1	1	M	28	8	181	6.5	74	161	51	13.5	3.8	10.9	0.39
↓	1	2	M	28	8	179	6.4	73	161	47	12.3	3.8	11.1	0.40
	1	3	C	28	8	135	4.8	55	117	24	7.3	3.2	15.3	0.55
	1	4	C	28	8	164	5.9	67	151	23	9.6	2.4	15.7	0.56
	2	5	M	26	10	219	8.4	90	196	77	26.4	2.9	6.8	0.26
	2	6	M	26	10	229	8.8	94	203	50	18.7	2.7	10.2	0.39
	2	7	C	26	10	209	8.0	86	176	28	9.2	3.0	18.5	0.71
	2	8	C	26	10	240	9.2	98	206	28	8.8	3.2	22.7	0.87
	3	9	M	14	10	141	10.1	58	126	54	20.9	2.6	5.0	0.36
	3	10	M	14	10	144	10.3	59	127	73	19.9	3.6	6.1	0.43
	3	11	C	14	10	118	8.4	48	99	28	7.3	3.8	13.3	0.95
	3	12	C	14	10	116	8.3	48	103	28	8.0	3.5	12.7	0.91
	4	13	M	28	8	246	8.8	101	205	31	17.0	1.8	11.9	0.43
	4	14	M	28	8	225	8.0	92	190	50	17.3	2.9	9.2	0.33
Half-lap	5	15	M	8	10	279	34.8	114	263	16	9.2	1.7	29.7	3.71
↓	5	16	M	8	10	245	30.6	100	237	10	7.6	1.4	30.5	3.81
	5	17	C	8	10	272	34.0	111	248	9.4	5.6	1.7	44.0	5.50
	5	18	C	8	10	281	35.2	115	264	20	6.3	3.2	42.0	5.24
	6	19	M	4	10	138	34.6	57	128	10	7.9	1.2	17.3	4.34
	6	20	M	4	10	173	43.2	71	155	7.1	3.5	2.1	43.1	10.8
	6	21	C	4	10	136	33.9	56	125	5.7	3.5	1.6	35.7	8.93

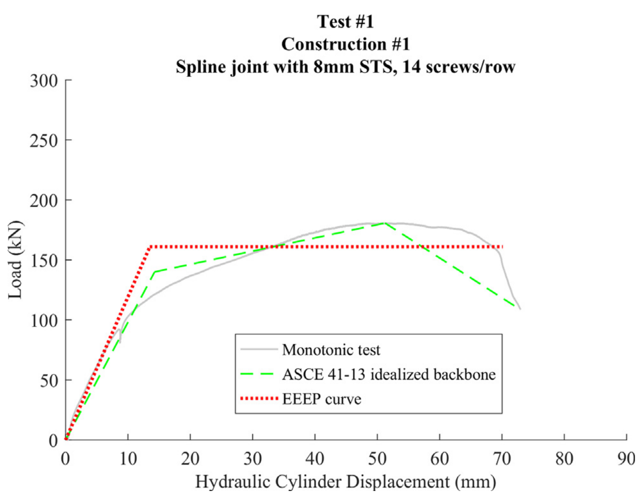

Fig. 11-1. Test #1 monotonic load-displacement, EEEP curve and ASCE 41-13 backbone curve (Replicate Test #2 similar).

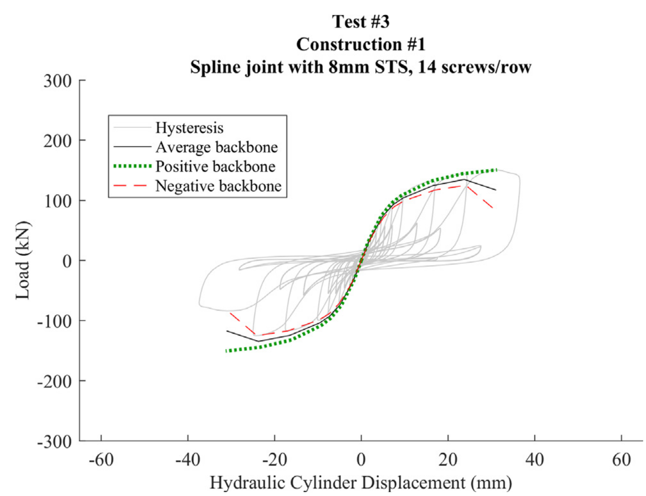

Fig. 11-2. Test #3 load-deflection hysteresis and experimental backbone curves (Replicate Test #4 similar).

$$n_{ef} = \min\begin{cases} n \\ n^{0.9} * \sqrt[4]{\frac{a_1}{13d}} \end{cases} \quad (2)$$

For $a_1 = 152$ mm, $d = 8$ mm, and $n = 14$, $n_{ef} = 11.8$, which is an effective reduction factor of 0.84.

The technical approval document for the screws [28] provides a similar formula to the Eurocode's:

$$n_{ef} = max\begin{cases} n^{0.9} \\ 0.9n \end{cases} \quad (3)$$

n_{ef} here equals the greater of 10.7 and 12.6 (0.9 ∗ 14 screws). 12.6 effective screws is an effective reduction factor of 12.6/14 = 0.90.

The specimens in this testing program showed no splitting in the CLT or in the plywood and therefore an adjustment of this type would not be needed. However, the results do show that the load capacity per fastener decreased when the number of screws increased and therefore an adjustment factor for this behavior would be beneficial to more accurately capture the behavior of the connection system. A group action factor of 0.80–0.85 is tentatively recommended based on the reduction in capacity per fastener in the monotonic tests completed. This is comparable to the reduction factors calculated in each of the three codes above.

3.1. Comparison to design values

Design strengths were calculated using the 2015 NDS [13] and EN-1995 (2008) [43] and compared to the test data. It is important to note that timber structures in the U.S. are most commonly, and historically, designed using Allowable Stress Design (ASD), and thus, all the tables and design formulas in the NDS have been created for this design methodology. The ASD design philosophy is generally characterized by non-factored loads and conventional safety factors in the range of 2–3

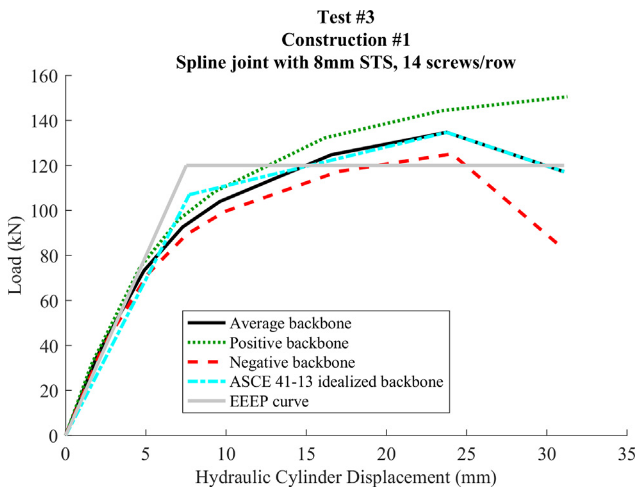

Fig. 11-3. Test #3 Experimental backbone curves, EEEP curve and ASCE 41-13 backbone curve (Replicate Test #4 similar).

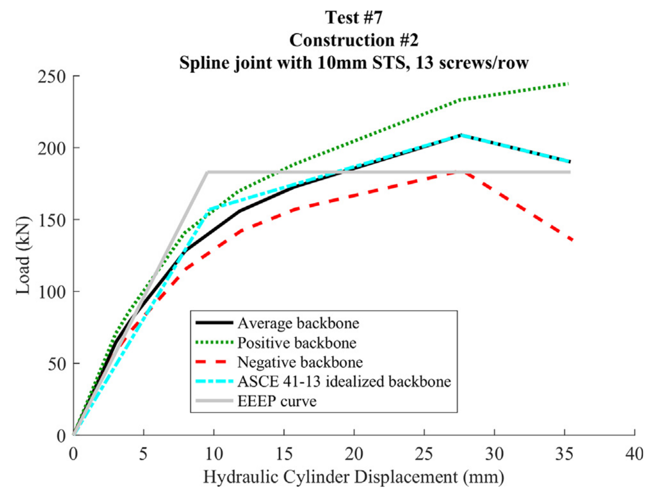

Fig. 11-6. Test #7 Experimental backbone curves, EEEP curve and ASCE 41-13 backbone curve (Replicate Test #8 similar).

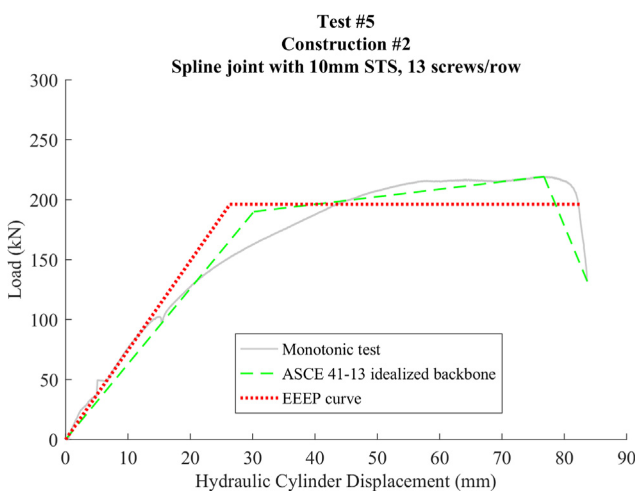

Fig. 11-4. Test #5 monotonic load-displacement, EEEP curve and ASCE 41-13 backbone curve (Replicate Test #6 similar).

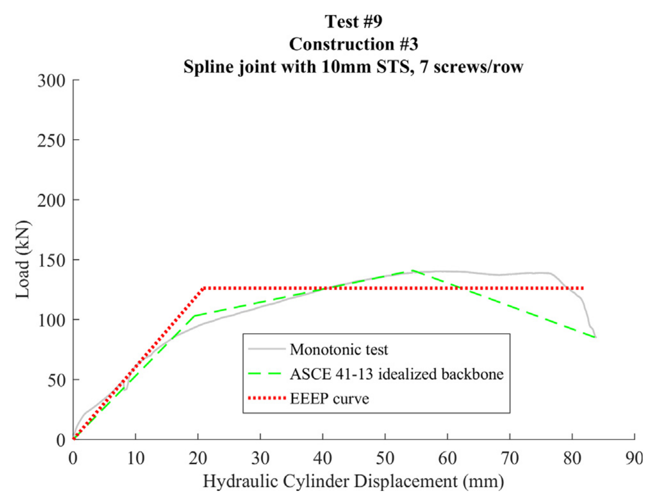

Fig. 11-7. Test #9 monotonic load-displacement, EEEP curve and ASCE 41-13 backbone curve (Replicate Test #10 similar).

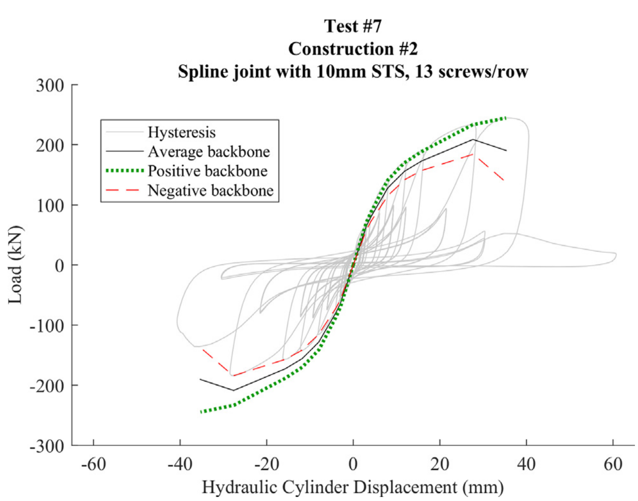

Fig. 11-5. Test #7 load-deflection hysteresis and experimental backbone curves (Replicate Test #8 similar).

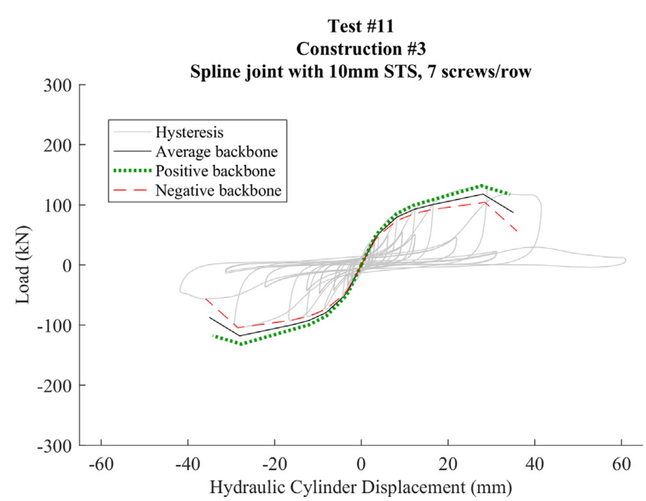

Fig. 11-8. Test #11 load-deflection hysteresis and experimental backbone curves (Replicate Test #12 similar).

that are built into design values. The Eurocode uses a design philosophy called Limit State Design (LSD) which uses the partial factor method [45] and is similar to the Load and Resistance Factor Design (LRFD) philosophy, also referred to as "Strength Design" in ASCE 7-16 [23], used in both concrete and steel design in the United States. LSD and LRFD design apply adjustment factors both to the characteristic

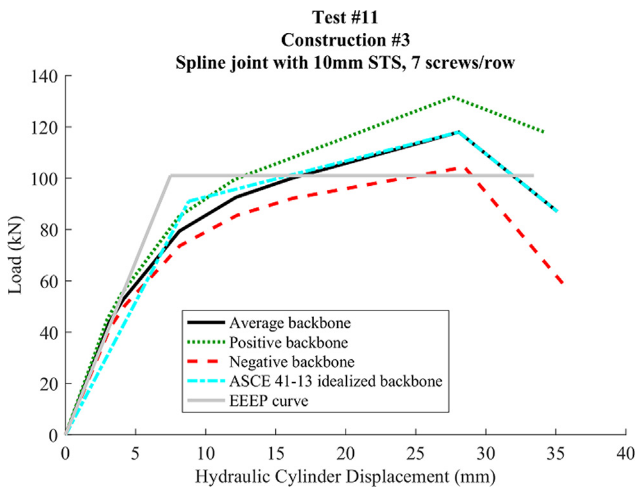

Fig. 11-9. Test #11 Experimental backbone curves, EEEP curve and ASCE 41-13 backbone curve (Replicate Test #12 similar).

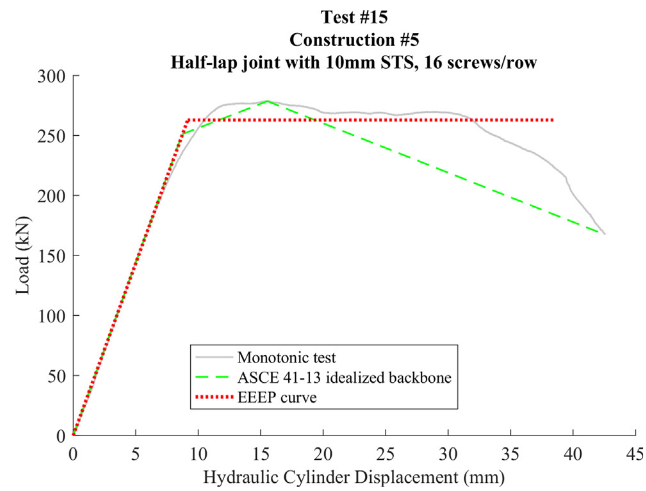

Fig. 11-12. Test #15 monotonic load-displacement, EEEP curve and ASCE 41-13 backbone curve (Replicate Test #16 similar).

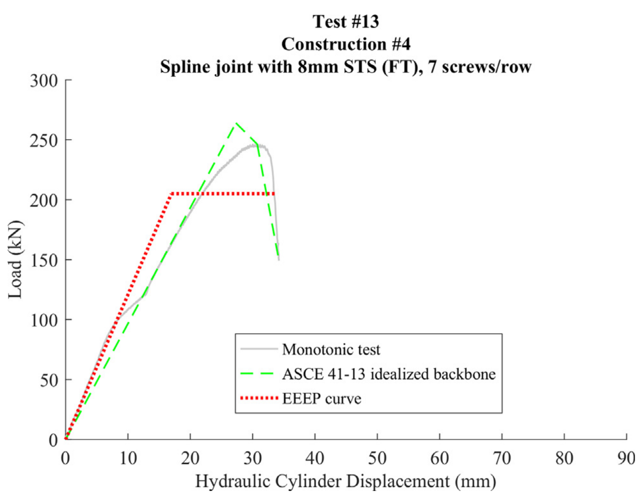

Fig. 11-10. Test #13 monotonic load-displacement, EEEP curve and ASCE 41-13 backbone curve.

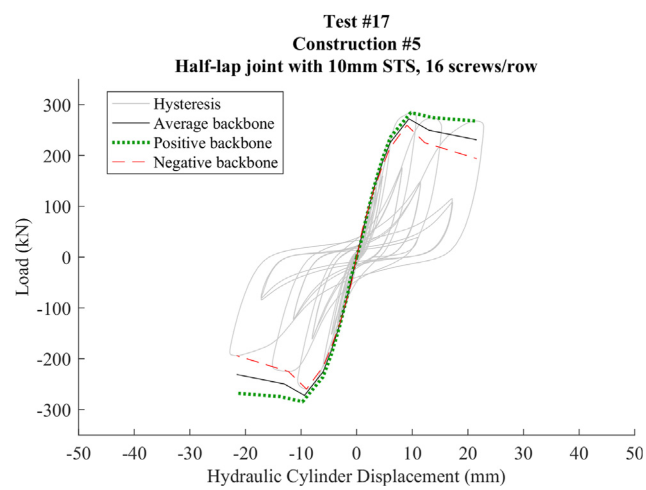

Fig. 11-13. Test #17 load-deflection hysteresis and experimental backbone curves (Replicate Test #18 similar).

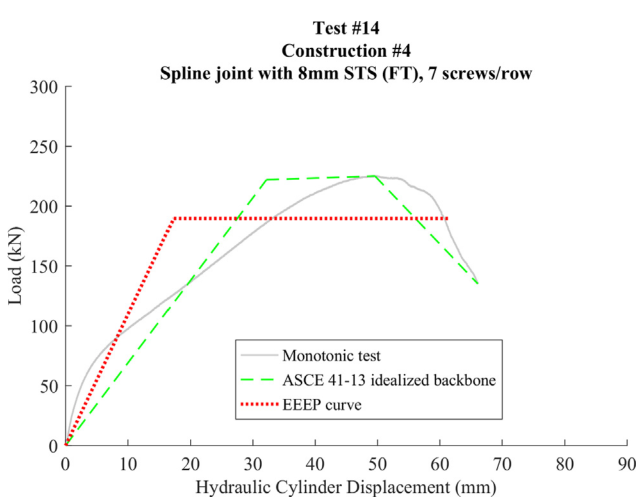

Fig. 11-11. Test #14 monotonic load-displacement, EEEP curve and ASCE 41-13 backbone curve.

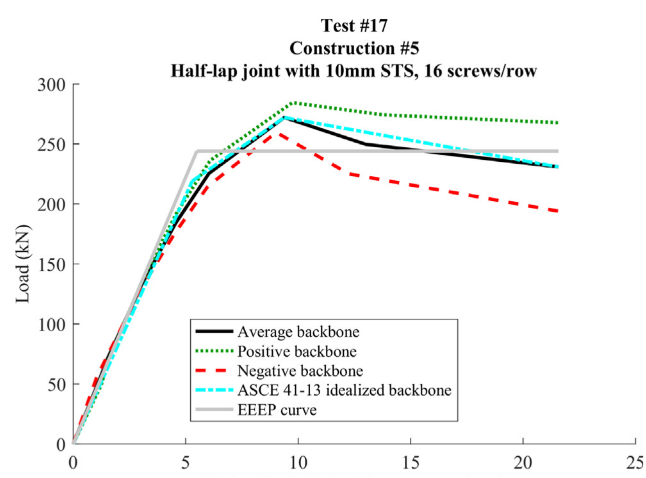

Fig. 11-14. Test #17 Experimental backbone curves, EEEP curve and ASCE 41-13 backbone curve (Replicate Test #18 similar).

strength of the material (typically decreased) and to the calculated loads (typically increased), which represent uncertainty in the original values. The NDS provides a factor to convert the calculated ASD design strengths into an LRFD value so that similar member sizing and connection designs would be reached with both design methodologies. LRFD strengths are compared to LRFD loads, which are higher than the

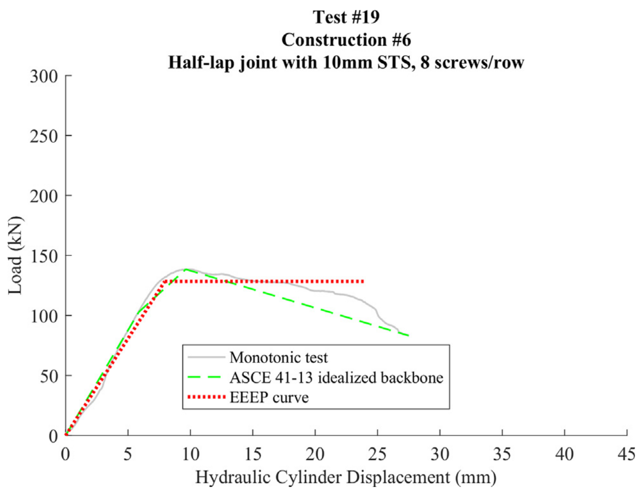

Fig. 11-15. Test #19 monotonic load-displacement, EEEP curve and ASCE 41-13 backbone curve (Replicate Test #20 similar).

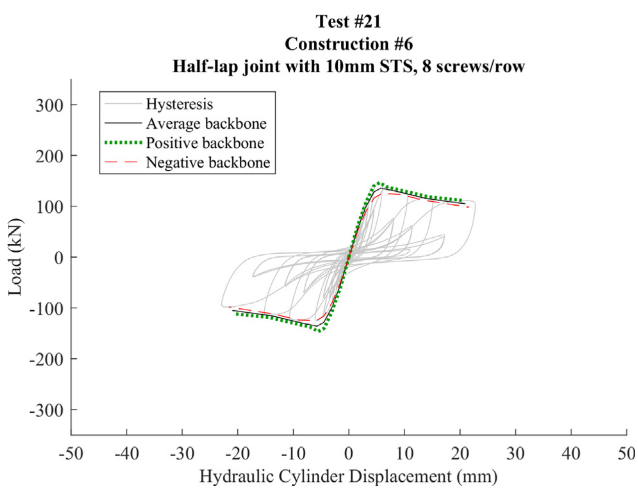

Fig. 11-16. Test #21 load-deflection hysteresis and experimental backbone curves (Replicate Test #22 similar).

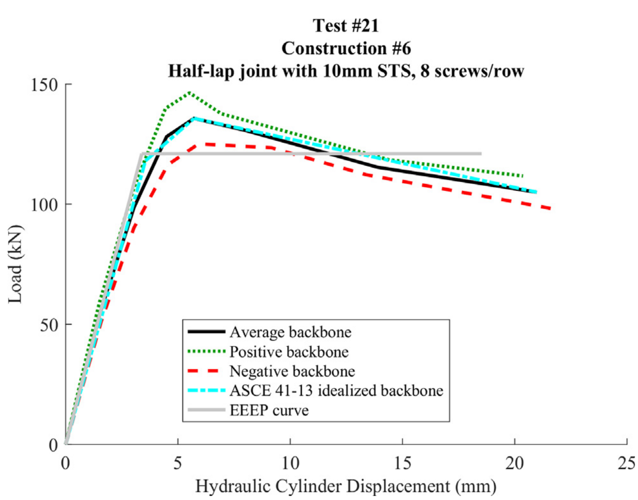

Fig. 11-17. Test #21 Experimental backbone curves, EEEP curve and ASCE 41-13 backbone curve (Replicate Test #22 similar).

calculated ASD loads due to the load factors. An effective conversion factor of 2.16 is used when the ASD to LRFD conversion factor from the NDS, $K_F = 3.32$, is coupled with the resistance factor $\varphi = 0.65$ for connections (Table 11.3.1 in [13]). The ASD factor that is not included in the LRFD calculation is the load duration factor, $C_D = 1.6$ (Table 2.3.2 in [13]) for load cases with durations under 10 min, such as wind and earthquake, the applications of the information from this research program.

The estimation indices in Table 6 used only the NDS design values but sample design calculations were completed for both the NDS and EN-1995 and the findings of those are explained below. In addition to the base Johansen Yield Theory equations, the NDS [13] uses an R_d factor (2.6 for the surface spline calculations) to reduce reference design strengths (or characteristic strengths, as termed in the Eurocode) to the historical, tabulated values in previous editions of the NDS [13], which are at ASD design levels, as well as a load duration factor (C_D) of 1.6 to increase strengths to match the approximate 10-min duration of the test.

Design values for the surface spline connection types were also calculated using timber design formulas in the Eurocode, EN 1995 [44], in conjunction with supplemental formulas found in the ETA report for the ASSY screws [28]. Unlike the NDS, the Eurocode takes into account the screw's axial withdrawal capacity of the rope effect, which increases the characteristic strength by 100% for Mode D (Mode IIIs in the NDS). The increase in shear strength due to the rope effect is limited to an increase equal to the base shear strength calculated using the Johannsen equations but for all cases the limit was not reached. The partial factor method includes $k_{mod} = 1.1$ and $\gamma_m = 1.3$, which decreases the strength to design levels by a factor of 0.84 (applied as k_{mod}/γ_m). Thus, the characteristic strengths are increased by an effective factor of 1.68 to the factored design strengths. Comparing factored LRFD design values calculated using the NDS to the factored design strengths using the Eurocode, it's apparent that the Eurocode values are 30% higher than the LRFD design values.

Estimation indices, defined as ratios of the experimental yield strength from the EEEP curve divided by the ASD design strength from the NDS [13], were compared for each test and are shown in Table 6. Estimation indices greater than one indicate that the NDS [13] design equations (ASD form) are conservative and underestimate the actual strength of the connection that would be called upon by seismic loads in a structural design. As the R_d factor is included in the calculation of both LRFD and ASD design formulas in the NDS, estimation indices for the NDS LRFD values can be calculated by dividing out $C_D = 1.6$ from the effective LRFD conversion factor of 2.16, calculated above. This shows that the LRFD design values are 35% higher than the ASD design values (Table 6) and thus the estimation indices for the LRFD design philosophy are 75% that of the ASD values below (Table 6).

The surface spline estimation indices ranged from 3.4 to 6.1. For the first surface spline specimen (Construction 1, Test #1), the yield load per fastener was 5.75 kN and the design strength per fastener was 1.24 kN. Thus, the estimation index for this specimen is $\frac{5.75\ \text{kN}}{1.24\ \text{kN}} = 4.6$. Construction 1 (surface-spline) had estimation indices ranging from 3.4 to 4.6. Increasing the screw diameter from 8 mm to 10 mm (Construction 1 to 2) increased the estimation indices by 21%. Doubling the number of screws (Construction 3–2) decreased the estimation indices by 7%. Spline specimens with fully-threaded screws tested monotonically (Construction 4, average of 5.7) had 36% higher estimation indices than spline specimens with partially-threaded screws (Construction 1, average of 4.2).

The estimation index increase for fully-threaded screws could be due to the "rope effect" of laterally loaded screws displaced far enough where they are engaged axially and begin to contribute to the lateral resistance of the connection. There is a twofold contribution from the rope effect: (1) as the connection is displaced the dowel will pull the side member towards the main member, increasing the horizontal friction force between the two components, (2) as the dowel bends, the axial resistance of the screw will have a horizontal (lateral) component, which will also contribute to the lateral resistance. The rope effect is not

Table 6
Estimation index results.

Type	Construction #	Test#	Load Type	# of fasteners	Screw diameter D_n (mm)	Yield load per fastener (kN)	Design strength per fastener (kN)	Estimation index	Displacement at yield Δ_y (mm)
	1	1	M	28	8	5.75	1.24	4.6	13.5
↓	1	2	M	28	8	5.74	1.24	4.6	12.3
	1	3	C	28	8	4.18	1.24	3.4	7.3
	1	4	C	28	8	5.39	1.24	4.3	9.6
	2	5	M	26	10	7.55	1.48	5.1	26.4
	2	6	M	26	10	7.80	1.48	5.3	18.7
	2	7	C	26	10	6.77	1.48	4.6	9.2
	2	8	C	26	10	7.92	1.48	5.4	8.8
	3	9	M	14	10	9.01	1.48	6.1	20.9
	3	10	M	14	10	9.10	1.48	6.1	19.9
	3	11	C	14	10	7.07	1.48	4.8	7.3
	3	12	C	14	10	7.36	1.48	5.0	8.0
	4	13	M	28	8	7.32	1.24	5.9	17.0
	4	14	M	28	8	6.77	1.24	5.5	17.3
	5	15	M	32	10	8.21	3.63	2.3	9.2
↓	5	16	M	32	10	7.40	3.63	2.0	7.6
	5	17	C	32	10	7.75	3.63	2.1	5.6
	5	18	C	32	10	8.25	3.63	2.3	6.3
	6	19	M	16	10	8.03	3.63	2.2	7.9
	6	20	M	16	10	9.67	3.63	2.7	3.5
	6	21	C	16	10	7.81	3.63	2.2	3.5
	6	22	C	16	10	8.13	3.63	2.2	4.1

considered in the NDS [13] but is found in the Eurocode's Johansen Yield Theory equations (Sec. 8.2.2) [43,9]. For most failure modes, a portion of the axial resistance of a dowel can be added to the lateral resistance. For all metal dowel-type fasteners, 25% of the dowel's axial resistance can be added to the base lateral resistance (the Johansen term [43]) but for screws specifically, only up to 100% of the base lateral resistance. Assuming this is allowed due to the threads on the screw, a partially-threaded screw with no threads in the side member would act more like a bolt, where the rope effect contribution is limited to 25% of the base lateral resistance. The design strength was calculated for the surface spline connection with 8 mm screws using EN-1995 and is fairly similar, with 20% higher strengths than the LRFD design strengths from the NDS (factors explained previously in Sec. 3.1) for the controlling failure mode (Mode IIIs [13] or Mode D [43]), which is characterized by bending of the screw and wood crushing in the side member (plywood spline). The half-lap connection estimation indices in this study ranged from 2.0 to 2.7. For the first half-lap specimen (Test #15), the yield load per fastener was 8.21 kN and the design strength per fastener was 3.63 kN. Thus, the estimation index for this specimen is $\frac{8.21\ \text{kN}}{3.63\ \text{kN}} = 2.3$. Doubling the number of screws (Construction 6 to 5) decreased the estimation indices by 4%. These results demonstrate the very conservative nature of the current design approaches and should motivate additional testing as well as provide designers with important insights into CLT diaphragm design, especially in the U.S. where the experience is more limited compared to Canada and Europe.

Calculated design strengths and estimation indices are shown in Table 6. Definitions of variables for Table 6 follow those defined for Table 5.

- **Yield load per fastener –** EEEP yield load for test divided by # of fasteners.
- **Design strength per fastener –** As calculated using the NDS [13] and explained further in Section 3.1.
- **Estimation index** – Test yield strength divided by design strength
- **Displacement at yield, Δ_y** – Displacement at the yield point on the EEEP curve

3.2. ASCE 41-13m-factors and deformation ratios

ASCE 41-13 [25] Tables 12-3 and 12-4 provide the linear and nonlinear acceptance criteria information, respectively, for use in evaluation and design of existing buildings. CLT may be used in rehabilitation of existing buildings and the criteria are useful in approaching and understanding performance-based design in general. For linear analysis, the standard uses "m-factors" for the acceptance criteria, and for nonlinear analysis, "deformation ratios." A factor of 0.75 is the sole differentiator between "m-factors" and "deformation ratios" for components like the CLT connections in this study (linear m-factors are 25% lower than the nonlinear deformation ratios).

Performance level displacements are proportional to the m-factors. A component is deflection-controlled if $\Delta_{peak}/\Delta_{yield} \geq 2$ and force-controlled otherwise. Deformation for CP occurs at peak load. Deformation for LS is 75% of the deformation at CP. Deformation for IO is 67% of the deformation for LS.

Table 7 is a summary of the ASCE 41-13 [25] m-factors and deformation ratios obtained for the two general types of connections (spline and half-lap) and two types of tests performed (monotonic and cyclic). Values published in ASCE 41-13 [25] for light-frame wood diaphragms and wood-to-wood screw connections are shown in Table 8 for comparison with the CLT tests. For the Life Safety performance level, the m-factors ranged from 1.58 to 1.80 for cyclic surface spline connections and from 0.89 to 1.66 for cyclic half-lap connections.

Since the experimentally-determined m-factors and deformation ratios at different performance levels are all proportional, it would suffice to list values for one performance level and either m-factors or deformation ratios. However, the published values for wood diaphragms and wood-to-wood screw connections are not proportional to each other, so each value is included in Table 7 for comparisons. The last two columns of Table 7 represent the controlling action of the components. ASCE 41-13 [25] specifies that if $\Delta_{peak}/\Delta_{yield} > 2$, then the component is deformation-controlled and if less than two, force-controlled. The acceptance criteria are deflection-based criteria for deflection-controlled and force-based criteria for force-controlled.

Most of the monotonic spline tests fall within the m-factor acceptance criteria range for IO for light-frame wood diaphragms (1.0–1.5). Both monotonic spline tests with 8 mm screws had IO m-factors greater than the m-factor of 1.2 for the wood-to-wood screw connection. Eight of the twelve spline tests with partially-threaded screws (Tests #1-#12) had IO m-factors less than 1.2 and four were higher than 1.2. At the LS level, only the two monotonic tests with 8 mm screws had m-factors

Table 7
m-factor and deformation ratio results.

#	Cxn type	Spacing (mm)	Screw dia. (mm)	Thread type	Load Type	Δ_{ref} (mm)	Linear Analysis m-Factor			Nonlinear Analysis Deformation ratios			Performance level displacements (mm)			$\Delta_{peak}/\Delta_{yield}$	Deformation or force controlled
							IO	LS	CP	IO	LS	CP	IO	LS	CP		
1	SS	150	8	P	Mono	–	1.35	2.02	2.69	1.80	2.69	3.59	25.7	38.4	51.2	3.59	D
2	SS	150	8	P	Mono	–	1.42	2.12	2.82	1.89	2.82	3.76	23.6	35.2	47.0	3.76	D
3	SS	150	8	P	Cyc	24	1.16	1.73	2.31	1.55	2.31	3.08	11.9	17.8	23.7	3.08	D
4	SS	150	8	P	Cyc	81	1.06	1.58	2.11	1.42	2.11	2.82	11.8	17.6	23.5	2.82	D
5	SS	150	10	P	Mono	–	0.96	1.43	1.91	1.28	1.91	2.54	38.6	57.6	76.7	2.54	D
6	SS	150	10	P	Mono	–	1.06	1.58	2.10	1.41	2.10	2.80	25.2	37.6	50.2	2.80	D
7	SS	150	10	P	Cyc	41	1.08	1.61	2.14	1.43	2.14	2.85	13.9	20.7	27.6	2.85	D
8	SS	150	10	P	Cyc	41	1.11	1.66	2.21	1.48	2.21	2.94	14.3	21.3	28.5	2.94	D
9	SS	300	10	P	Mono	–	1.06	1.58	2.10	1.41	2.10	2.80	27.4	40.8	54.5	2.80	D
10	SS	300	10	P	Mono	–	1.27	1.90	2.53	1.70	2.53	3.37	36.5	54.4	72.6	3.37	D
11	SS	300	10	P	Cyc	41	1.20	1.80	2.40	1.60	2.40	3.19	14.1	21.1	28.1	3.19	D
12	SS	300	10	P	Cyc	41	1.15	1.72	2.30	1.54	2.30	3.06	14.1	21.0	28.0	3.06	D
13	SS	150	8	F	Mono	–	0.42	0.63	0.84	0.57	0.84	1.13	15.4	23.1	30.7	1.13	F
14	SS	150	8	F	Mono	–	0.58	0.87	1.16	0.77	1.16	1.54	24.9	37.2	49.6	1.54	F
15	HL	150	10	F	Mono	–	0.67	1.00	1.34	0.90	1.34	1.78	7.8	11.7	15.5	1.78	F
16	HL	150	10	F	Mono	–	0.58	0.87	1.16	0.78	1.16	1.55	5.2	7.73	10.3	1.55	F
17	HL	150	10	F	Cyc	15	0.67	1.00	1.33	0.89	1.33	1.78	4.71	7.04	9.38	1.78	F
18	HL	150	10	F	Cyc	30	1.11	1.66	2.21	1.48	2.21	2.95	10.0	14.9	19.9	2.95	D
19	HL	300	10	F	Mono	–	0.62	0.92	1.23	0.82	1.23	1.64	4.84	7.22	9.62	1.64	F
20	HL	300	10	F	Mono	–	0.78	1.17	1.56	1.05	1.56	2.08	3.59	5.35	7.14	2.08	D
21	HL	300	10	F	Cyc	15	0.61	0.91	1.21	0.81	1.21	1.61	2.87	4.29	5.72	1.61	F
22	HL	300	10	F	Cyc	15	0.60	0.89	1.19	0.79	1.19	1.58	2.95	4.41	5.88	1.58	F

Notes
1. SS = Surface spline, HL = Half-lap.
2. F = fully-threaded screws, P = partially-threaded screws.
3. Mono = Monotonic test, Cyc = Cyclic test.
4. F = Force-controlled, D = Deformation-controlled.

Table 8
Light-frame wood m-factor comparison.

	Linear analysis m-Factor			Nonlinear analysis Deformation ratios		
	IO	LS	CP	IO	LS	CP
Light-frame wood structural components[a]						
Diaphragms	1.0–1.5	1.5–3.0	2.0–3.0	1.3–1.8	2.0–4.0	3.0–5.0
Wood-to-wood screw connections	1.2	2.0	2.2	1.4	2.5	3.0
CLT						
Spline monotonic	0.96–1.42	1.43–2.12	1.91–2.82	1.28–1.89	1.91–2.82	2.54–3.76
Spline cyclic	1.06–1.2	1.58–1.8	2.11–2.4	1.42–1.6	2.11–2.4	2.82–3.19
Half-lap monotonic	0.58–0.67	0.87–1.8	1.16–2.4	0.78–0.9	1.16–2.4	1.55–3.19
Half-lap cyclic	0.6–1.11	0.89–1.66	1.19–2.21	0.79–1.48	1.19–2.21	1.58–2.95

[a] Stick frame performance values taken from Table 12-3 (Numerical Acceptance Factors for Linear Procedures—Wood Components) and Table 12-4 (Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Wood Components) of ASCE 41-13.

higher than the 2.0 m-factor for wood-to-wood screw connections. At the CP level, seven of the twelve spline tests with partially-threaded screws had m-factors higher than the 2.2 value for wood-to-wood screw connections.

All half-lap tests (Tests #15-#22) had IO m-factors less than 1.2. At the LS level, none of the half-lap tests had m-factors higher than the 2.0 m-factor for wood-to-wood screw connections. At the CP level, one of the eight half-lap tests had an m-factor higher than the 2.2 m-factor value for wood-to-wood screw connections. There was no correlation with whether the test was a monotonic test or a cyclic test. The half-lap test with a reference deflection twice that of the other cyclic half-lap tests (Test #18) had much higher m-factors at each performance level than the other half-lap tests.

All the surface splines with partially-threaded screws (Tests #1-#12) are deformation-controlled, meaning the peak deformation to yield deformation ratio (Δ_{peak}/Δ_y) is greater than two. The surface spline construction with the fully-threaded screws (Test #13 and #14) had a peak displacement to yield displacement ratio less than two and was thus characterized as force-controlled. Six of the eight half-lap tests (Tests #15-#22) were characterized as force-controlled. This was expected as the load-deflection curves of the half-lap tests were stiffer than the surface spline specimens and the displacement at peak load was much closer to the displacement at the yield point.

4. Conclusions

The following conclusions can be drawn from this study:

(1) Peak loads for surface spline connections with 25 mm thick splines averaged 5.9 kN (1.33 kips) for 8 mm diameter screws and 8.9 kN (2.0 kips) per fastener for 10 mm screws. Half-lap capacities averaged 35.1 kN (7.9 kips) per fastener for 10 mm screws. The experimental strengths observed for these connections are useful for designers to know the conservative nature of the approaches

currently used in their designs.

(2) Stiffness values for surface splines range from 0.36 kN/mm (2.1 kips/in) per fastener to 0.87 kN/mm (5.0 kips/in) per fastener. Stiffness values for half-laps range from 4.3 kN/mm (25 kips/in) per fastener to 5.5 kN/mm (31 kips/in) per fastener. While formulas to estimate stiffness are not provided in the NDS [13], equations in EN-1995 [43] estimate 8 mm diameter screws in Douglas-fir (*Pseudotsuga menziesii*) lumber in pure shear to have a stiffness of 3.9 kN/mm.

(3) The load-deflection curves showed that the spline specimens exhibited high ductility as the screw heads bend and pull through the plywood spline. The Withdrawal/Shear/Shear/Withdrawal (WSSW) installation pattern for the half-lap connections exhibited high ductility due to the vertical screws in pure shear.

(4) As expected, strengths per screw increase with an increase in cross-sectional area of the screw and did so in an approximately proportional fashion. Also, higher ductility is exhibited by the 8 mm screws compared to the 10 mm screws. This information would be useful if considered in the development of a new diaphragm force reduction factor, R_s, for CLT diaphragms, as steel fasteners in wood are the main source of energy dissipation during earthquakes.

(5) Group action factors calculated using the Eurocode, the New Zealand Timber code, and the NDS, in the range of 0.80–0.85, were also observed in monotonic tests for both connection types, as indicated by the decrease in load capacity per fastener as the number of screws in the joint increases.

(6) Estimation indices (test yield strength divided by NDS [13] design strength) for all tests are in the range of 2.0–6.1. Design strengths calculated using EN-1995 [43] were approximately 20% higher than the NDS design strengths when converted to LRFD levels.

(7) All the surface splines are characterized as deformation-controlled in the m-factor analysis from ASCE 41-13 [25]. For the Life Safety performance level, the m-factors range from 1.58 to 1.80 for cyclic surface spline connection types and from 0.89 to 1.66 for cyclic half-lap connection types.

(8) The m-factors for the monotonic spline tests are mostly within the acceptance criteria range for light-frame wood diaphragms (1.0–1.5). Half of the tests have m-factors greater than those in the acceptance criteria for wood-to-wood screw connections.

9) Monotonic surface spline tests reach yield via Mode III_s [13] and then pull-through of the screw head through the plywood thereafter. Monotonic half-lap connection tests yield with screw withdrawal of the angled screws and subsequent Mode III_m [13] failure for the vertical screws in shear. Fatigue in the screws is the main mode of failure for the cyclic spline tests. The screws in the cyclic half-lap connection tests remain intact, and the failure presents itself as withdrawal of the angled screws and wood crushing in the vertical screws (Mode III_m [13]).

4.1. Recommendations

It is recommended that further testing be completed on larger scale diaphragm systems constructed of CLT and glulam beams to capture the boundary conditions seen in mass timber projects. It is possible that further research in this area will help in determining whether to design CLT diaphragms as elastic or inelastic (energy dissipating). CLT diaphragms and their accompanying panel-to-panel connections should be examined in a manner similar to the research done on precast concrete slabs using FEMA P795 – Quantification of Building Seismic Performance Factors: Component Equivalency Methodology [46]. CLT diaphragms are comparable to precast concrete slabs due to the rigid panel elements and ductile connections. Testing connections in tandem with developing a computer model of both the component performance and building system-wide performance would be beneficial for understanding the overall design process of a project with CLT diaphragms.

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