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DAMAGE ASSESSMENT OF CROSS LAMINATED TIMBER CONNECTIONS SUBJECTED TO SIMULATED EARTHQUAKE LOADS

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ABSTRACT: Wood-frame is the most common construction type for residential buildings in North America. However, there is a limit to the height of the building using a traditional wood-frame structure. Cross-laminated timber (CLT) provides possible solutions to mid-rise and high-rise wood buildings. CLT offers many advantages such as improved dimensional stability, a quicker erection time and good performance in case of fire. In order to introduce the cross-laminated timber products to the North American market, it is important to gain a comprehensive understanding of its structural properties. This paper focuses on the seismic performance of CLT connections. Over the last few years FPInnovations of Canada has conducted a test program to determine the structural properties of CLT panels and its application in shear walls. The test program comprised of more than 100 connection tests which followed the loading procedures of CUREE and ISO test protocols as specified in ASTM Standards ASTM E 2126-09 (2009). These tests were performed parallel and perpendicular to the grain of the outer layer, respectively. The impact of different connections on the seismic performance of CLT walls was investigated in a second phase on full size shearwall. CLT panels are relatively stiff and thus energy dissipation must be accomplished through the ductile behaviour of connections between different shear wall elements and the connections to the story below. A literature review on previous research work related to damage prediction and assessment for wood frame structures was performed. Different approaches for damage indices were compared and discussed. This paper describes how the energy-based cumulative damage assessment model was calibrated to the CLT connection and shear wall test data in order to investigate the damage under monotonic and cyclic loading. Comparison of different wall set-up provided a deeper insight into the damage estimation of CLT shear walls and determination of the key parameters in the damage formulation. This represents a first published attempt to apply the damage indices to estimate the seismic behaviour of CLT shear walls.

KEYWORDS: Cross laminated timber, Connections, , Seismic performance, Damage index, Damage prediction, Shearwall

1 INTRODUCTION

Throughout North America, timber framing is used for most residential buildings. Recently a new material called Cross-laminated timber (CLT) was developed in Austria and Germany. CLT found various applications in residential and non-residential buildings. It can be produced in sizes with maximum dimensions of 4.8m x 20m. Panels can be pre-cut and prepared with all openings on CNC- machines. Floors and walls can be prefabricated and erected within small tolerances in a short time. Benefiting from the solid structure of such a wood structure, good thermal insulation values as well as fire performance can be achieved. European architects and engineers have designed mid-rise buildings in CLT up to 9 storeys height with very good performance. The North American market could benefit from this development and implement this product into their code as prospective alternative to traditional timber-framing.

FPInnovations has started several testing series to investigate the material properties of CLT. The researchers focussed on the connection to the foundation, connection between wall and deck as well as wall to

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wall, and the behaviour under seismic loading. Using the data from individual connection and shear wall tests, in this paper, Kratzig's damage accumulation principles are utilized [7]. The calculated damage stages will be compared with visual observations.

2 PREVIOUS RESEARCH

Traditional timber-frame structures used numerous connections between studs and sheathing that are ductile and provide a variety of load paths [1]. Ductility in CLT shear walls is achieved through attachments to the foundation or floors. The CLT panel itself is very stiff. European research facilities have conducted CLT tests with various objectives. Tests with different set-ups were conducted at the Tree and Timber Institute of Italy (IVALSA) [2] at University of Ljublijana, Slovenia [3], and at University of Karlsruhe [4]. Blass and Uibel considered connections in the narrow side as well as the face side of the CLT panel [4], calibrated and modified Johansen's theory [5] accordingly. At University of Ljubljana, the main focus of all the tests was on the connector performance under monotonic and seismic load. Previous tests in Ljubljana and at IVALSA showed a number of different failure modes in the connection. Failure occurred in the bracket, in the fasteners and in the wood. At University of Karlsruhe a test series was conducted with individual and groups of fasteners under various loading conditions [4]. In nearly all test configurations commercial brackets and fasteners were used in the tests. The Trees and Timber Institute tested a custom made hold-down [2]. When a wall was connected to foundation the failure occurred in the CLT panel, in the bracket, and in the fasteners. The most common failure mode was the pull-out of fasteners which is the desired failure mode as the most energy can be dissipated by forming plastic hinges in the fasteners. The research projects in Italy [2] and Ljubijana [3] were interested in the entire behaviour of the shear walls less on individual bracket connections.

3 DAMAGE INDICES

Damage accumulation index is a measure to evaluate damage to elements or buildings after earthquakes or other impact loads [6,7]. Cosenza and Manfredi [6], Williams and Sexsmith [7] have discussed various global and local damage indices. The local and global damage indices can be categorized into the following four categories [7]: i) non-cumulative indices; ii) deformation-based cumulative indices; iii) energy-based cumulative indices, and iv) combined indices. Non-cumulative indices neglect the effect of repeating loading cycles. Based on simplicity and ease of interpretation, however, non-cumulative indices are widely used [7]. Ductility and inter-storey drift are widely used to assess structures with non-cumulative indices [7]. Newmark and Rosenblueth [8] have proposed ductility ratio, defined as the ratio between maximum displacement and the yield displacement as a structural damage measurement. Park and Ang's model is the most widely accepted cumulative damage indices for reinforced concrete. Their model includes maximum deformation as well as the influence of repeated cyclic loadings [7]. Park and van de Lindt [9] has extended the Park and Ang index model for isolated shear walls based on sheathing perimeter nail spacing. In the past, rare attempts have been made to evaluate wood structures by applying damage indices [9,10]. The maximum displacement and the hysteretic energy absorption are the required parameters for the modified function by van de Lindt and Gupta [10]. The analyzed connection and shear walls tests in this paper will be focused on the development of the damage accumulation over time and will be validated with the on-going dynamic loading protocol and observed damage of the test specimen. In this paper only the energy-based cumulative index will be applied for the analysis.

3.1 ENERGY-BASED CUMULATIVE INDEX

Gosain et al. [14] defined energy absorption as a measure of damage.

$$D_e = \sum_i \frac{F_i\,\delta_i}{F_y\,\delta_y}; \quad F_i / F_y \geq 0.75 \tag{1}$$

where D_e = energy-related damage index, F_i = force in *i*-th cycle, δ_i = displacement in *i*-th cycle, F_y = force at yielding, δ_y= displacement at yielding.

Hysteresis loops when dropping below 75% of the yielding value after reaching yielding were considered to be negligible for the remaining capacity of the member. Kratzig et al. [15] developed a more complex energy formulation. Kratzig's formulation is based on following half-cycles. The first half-cycle of loading at certain amplitude is called *primary half-cycle* (PHC). The subsequent part of the cycle after peak load is called *follower half-cycle* (FHC). There are different cumulative formulations for the positive and negative part of the hysteresis. For the positive part of the response, the damage parameter is defined as:

$$D^+ = \frac{\sum E^+_{p,i} + \sum E^+_i}{E^+_f + \sum E^+_i} \tag{2}$$

where $E_{p,i}$ = energy in a PHC, E_i = energy in a FHC, E_f = energy absorbed in a monotonic test to failure.

The parameters for the negative deformation have to be calculated accordingly. The overall damage index is defined as:

$$D = D^+ + D^- - D^+ D^- \tag{3}$$

where D^+ = damage in positive cycle, D^- = damage in negative cycle, D^+D^- = interaction of D^+ and D^-

The inclusion of the FHC energy in the numerator as well as in the denominator limits the influence on a low level compared to the primary term. Both deformation and fatigue-type damage are taken into account. A similar high value of the damage index can be achieved by a single high-amplitude cycle or by repeating cycles at lower amplitude.

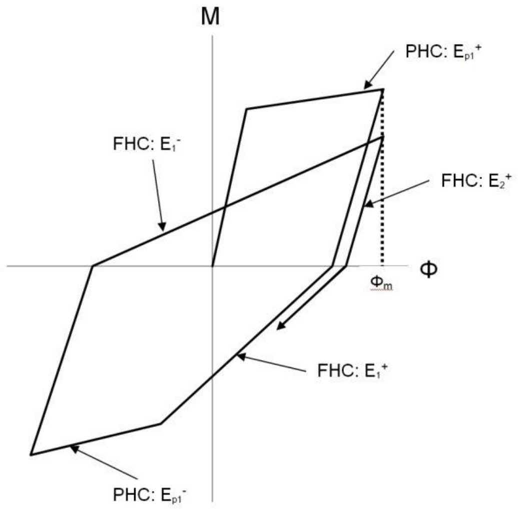


Figure 1: *(PHC) and follower (FHC) half-cycles* [7]

4 EXPERIMENTAL STUDIES

In the research reported here, individual CLT connection tests were carried out, as well as the shearwall tests using the same CLT connections.

4.1 CONNECTION TEST

The connection tests were conducted separately for loading parallel to grain and perpendicular to grain. The CLT samples were prepared to 7 × 14 inches. For this test series two commercial brackets in size 90 × 48 × 3.0 × 116 mm (Bracket A) and 105 × 105 × 3.0 × 90 mm (Bracket B) (Fig. 3) and five different fasteners (Fig. 4) were combined and tested. The table below shows the combinations and number of tests for each group.

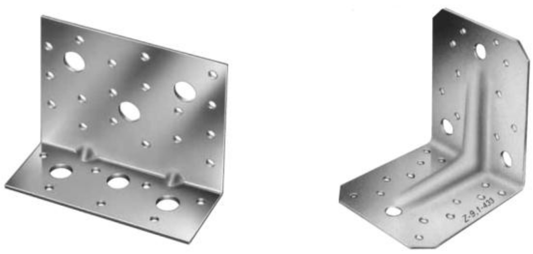


Figure 2: *Bracket A (90x48x3.0x116) and Bracket B (105x105x3.0x90)*

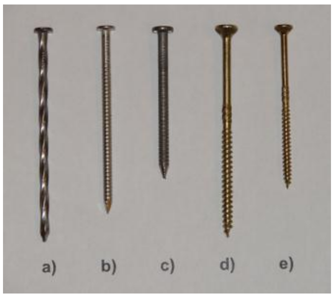


Figure 3: *Fasteners as used in the testing program*

Table 1: *Connection tests*

Combination		No. of Tests
Bracket A	18 Spiral Nail 16d × 3 1/2"	19
	12 Ring Shank Nail 10d × 3"	16
	12 Ring Shank Nail 16d × 60mm	16
	18 Screw 4 × 70mm	17
	9 Screw 5 × 90mm	16
Bracket B	10 Spiral Nail 16d × 3 1/2"	16

A universal testing machine was used to carry out the connection test, and parallel to grain and perpendicular to grain loading were set up differently.

4.2 SHEAR WALL TEST

All walls were tested on the same test set-up with minor changes for each different test. Each wall sat on a solid base beam, which provided holes for all wall set-ups. Figure 5 illustrates schematic of the test set-up. The wall length varied between 2.3m and 3.45m.. The brackets were connected to the base beam with bolts and washers either 1/2" (12.7 mm) or 3/8" (9.52 mm) depending on the bracket used, and connected with different fasteners to the CLT wall panel. The vertical load was applied to the wall panel by a top beam that was attached to the top edge side of the wall panel. The vertical load was chosen to 20 kN/m. This value is given by the Canadian code and represents the load on the bottom wall of a four-story building. Depending on length either two or four vertical actuators equipped with a load cell applied a constant force between 23 kN and 69 kN. To allow maximum freedom of rotation of the panel, the horizontal force was induced through a pinned connection on each end of the wall.In the starting position of the test, the centre of the actuator lines up with the top edge of the wall. The large actuator was connected to a tall braced tower. To prevent tipping out of plane of the wall panel, there were two rollers on either side of the top beam to provide lateral support at the top end of the wall.

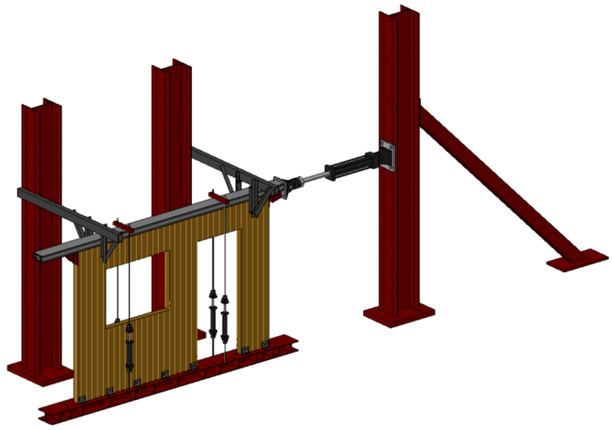
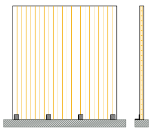
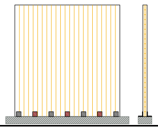
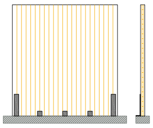
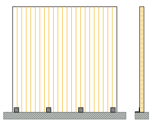
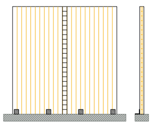
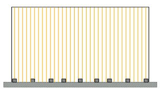
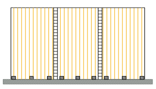
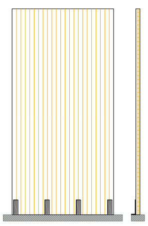
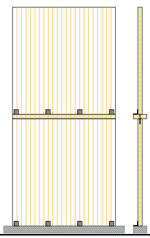


Figure 4: *3D sketch of wall test set-up*

Table 2 gives an overview of the tested walls which will be considered for the damage accumulation indices.

Table 2: Test matrix of shear walls

Wall Configuration		Test No.	Connections	Loading Protocol
1		00C	4 Bracket A SN 16d, n=18	CUREE
		01M		Ramp
		01C		CUREE
		03		CUREE
		04	RN 10d, n=12	CUREE
		05	S1 n=18	CUREE
		06	S2 n=9	CUREE
		20	7 Bracket A SN 16d, n=18	CUREE
2		07M	3 Bracket A 2 Hold-down SN 16d, n=18	Ramp
		07C		CUREE
		08		CUREE
3		09M	4 Bracket C TR 65, n=10	Ramp
		09C		CUREE
		10		CUREE
4		11	4 Bracket A SN 16d, n=18 in step joint WT-T, n=12	CUREE
		12		CUREE
		12ISO		ISO
5		13M	9 Bracket B SN 16d, n=10	Ramp
		13C		ISO
6		15M	9 Bracket B SN 16d, n=10 in step joint SFS 1, n=8	Ramp
		15C		ISO
		19	7 Bracket B SN 16d, n=10	ISO
7		24M	4 Bracket D TR 65, n=40	Ramp
		24C		ISO
		26	TR 90, n=40	ISO
		27	TR 90, n=20	ISO
8		28M	4 Bracket A /storey SN 16d, n=6	Ramp
		28C		ISO
		29	SN 16d, n=8	ISO

SN = Spiral nail 3.9 × 89mm, RN = Ring shank nail 2.4 × 76mm, S1 = Screw 4 × 70mm, S2 = Screw 5 × 90mm, TR65 = Timber rivet 65mm, TR90 = Timber rivet 90mm.

4.3 LOADING PROTOCOL

The connection tests and shear wall tests were conducted under monotonic and cyclic loading. For the cyclic tests CUREE protocol as well as ISO was applied.

4.3.1 Loading protocol for connection tests

For the first set of connection tests the monotonic program was done with a unidirectional and downward loading at a rate of 6.35 mm (1/4") per minute. All tests were displacement-controlled. The machine stopped loading when 50% of the peak load was reached after passing peak load. In different sets of tests the load was applied parallel to grain and perpendicular to grain respectively. For cyclic loading tests CUREE and ISO protocol was followed ASTM E 2126 Standard [18]. Both protocols are displacement-controlled loading procedures. Each primary cycle in the CUREE protocol is followed by two cycles at 80% of the first one. ISO load protocol has three cycles of same magnitude in each loading level. Two different schedules were used to address the two different situations of displacement in the negative direction. In the first case the wall sample sat on the foundation and did not allow any displacement in the negative direction. The second case addressed the horizontal movement (perpendicular to grain). The two loading schedules are shown in Figure 6.

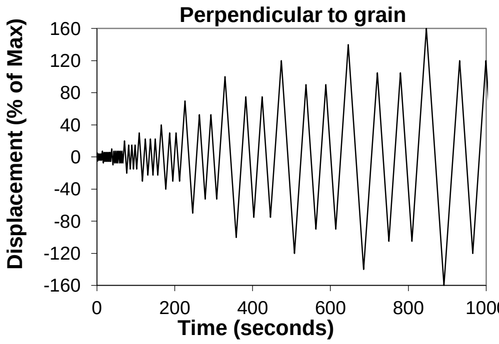
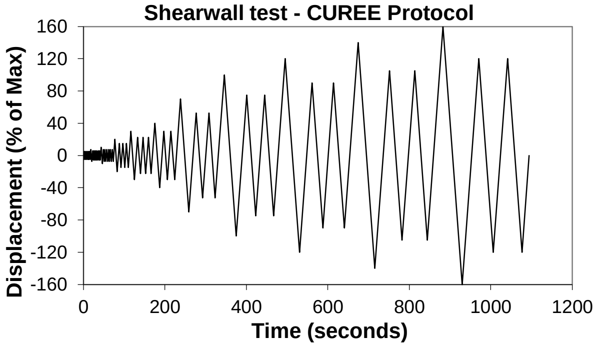


Figure 5: *Cyclic loading protocols for connection test parallel and perpendicular to grain*

4.3.2 Loading protocol for shearwall tests

For the shearwall tests the same type loading protocols as for the connection tests were used. However, in the full size wall test, some walls were tested with the ISO protocol, which is a method, accepted internationally and also included in the ASTM E 2126 Standard (2009).

The monotonic test was loaded horizontally unidirectional and in plane at a loading rate of 0.4" (10.16 mm) per minute. The displacement-controlled loading stopped at 60% of the peak load after the peak load was reached. Using the obtained data, the displacement at 80% of the peak load is calculated which is necessary for the cyclic tests. For the CUREE protocol a displacement rate of 0.2" per second (5.08 mm/s) was chosen. Figure 7 depicts an appropriate plot time vs. displacement of the cyclic loading. The ISO protocol schedule was followed with a displacement rate of 1 mm/sec. The amplitudes of the reversed cycles are a function of the mean value of the ultimate slip (v_u) obtained in the monotonic test.



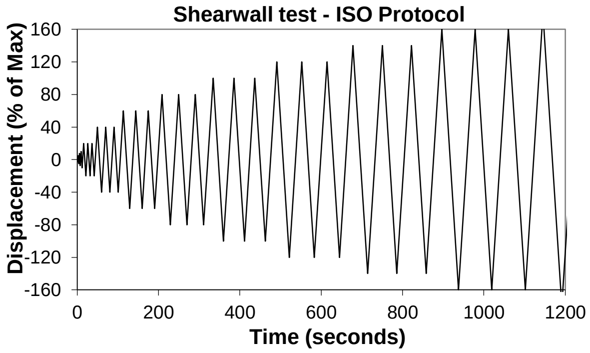


Figure 6: *Cyclic loading protocols for shear wall test CUREE (top) and ISO (bottom)*

5 EXPERIMENTAL RESULTS AND CALIBRATION OF DAMAGE INDICES

5.1 FAILURE MODES

Connection tests as well as full size shearwall tests showed similar failure modes. The failure occurred at the connections between the CLT panel and the base. Seven failure modes were observed within the test series: Pull-out failure of the fasteners, shear failure of the fasteners, edge break out of the CLT panel, extended wood crushing, CLT delamination, net tension failure of the bracket, and group tear-out of fasteners (Fig. 8).

Pull-out Failure

Shear failure

Edge break out

Wood crushing

Net tension failure

Group tear-out

Figure 7: *Failure modes in connection and wall tests*

5.2 CONNECTION TEST ANALYSIS WITH THE ENERGY-BASED APPROACH

The damage evaluation approach using the energy-based method of Kratzig [7] does not require a user defined factor. The function is only related to the absorbed energy of the monotonic and cyclic test. Figure 9 shows a typical load displacement curve of a conducted connection test.

The analysis was done separately for each connection and direction. Figure 10 shows test results of the group Bracket A with 18 Spiral nails 3.9 × 89mm. It can be shown that the loading direction (parallel or perpendicular) does not affect the slope of the curve which represents the damage increasing rate. Up to D = 0.75 the values of the individual test results are in a narrow range. Table 3 considers collapse at D = 0.75 which was proven in an earlier publication of the author. In all connection combinations the damage index D follows a nearly linear function until collapse is reached. In Figure 11 the average damage accumulation curves for the individual combinations are plotted. A lower slope will represent a more ductile behaviour of the connection since the CLT specimen can be considered as stiff.

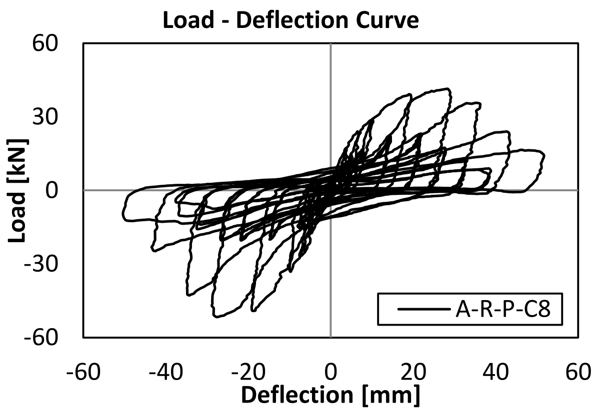


Figure 8: *Cyclic response for the connection test perpendicular to grain with 9 Ring shank nails*

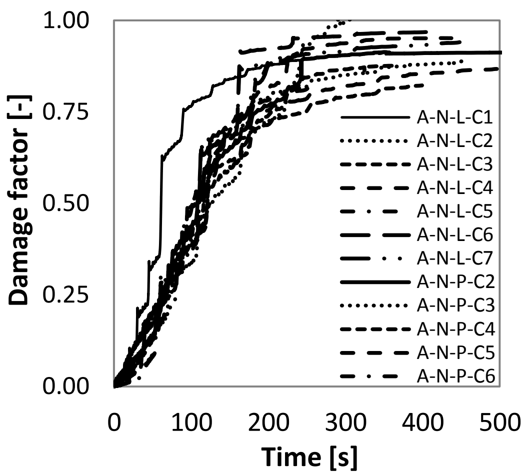


Figure 9: *Damage index for Bracket A with 18 Spiral nails*

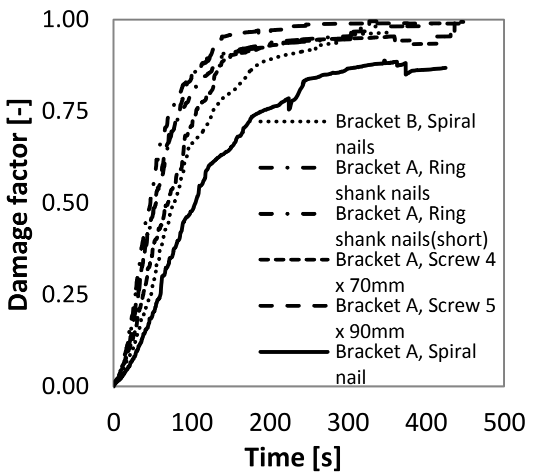


Figure 10: *Average values of all connection types*

5.3 SHEAR WALL ANALYSIS

The shear walls tests are divided into eight groups based on size and connections according to Table 2. The damage assessment was carried out based on the cyclic data sets for 22 walls. The damage-time curves of a wall with the size of 2.3m × 2.3m and various connectors are compared in Figure 12. The validation of the calculated value was done by visual observation of the test walls. The first group of wall tests shows a similar behaviour like the connection tests. The curves for walls with spiral nails can be found at the lower end of the curves (Figure 12; curve 00, 01C, 03).

It is interesting that in group 3 both tests performed identical. The timber rivets of wall named 09C separated gradually with the increasing loading protocol compared to test #10 (Fig. 13). This softer behaviour can be seen at the slope difference of the 2 tests.

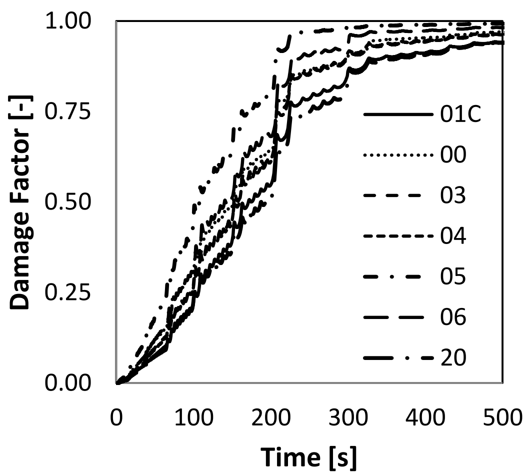


Figure 11: *Wall results for group 1 (2.3m x 2.3)*

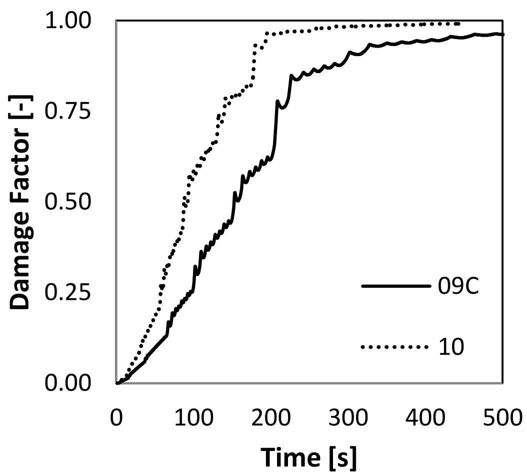


Figure 12: *Wall results for group 3 (2.3m x2.3m)*

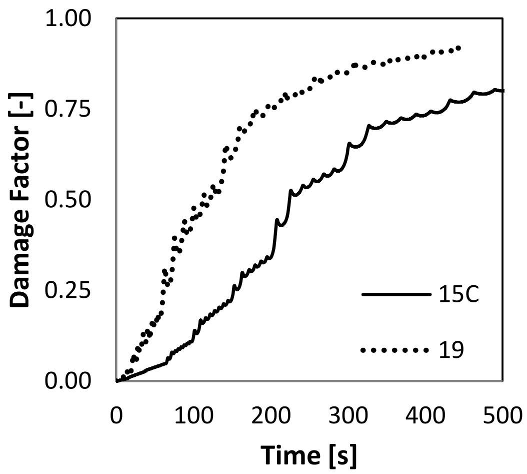


Figure 13: *Wall results for group 6 (3.45m x 2.3m)*

A comparison of group #1 with test #10 shows a significant difference in the damage accumulation where D = 0.75 was reached with timber rivets quite faster.

The results from group 6 show the influence of the wall panel on the damage accumulation (Fig. 14). Wall #19 consists of 3 separate panels connected with screws in the step joints. Even with a higher number of brackets the slope of the curve is very low. Wall 15C consisting of one panel with 2 openings loaded with the same

protocol shows a significant higher increase of the damage accumulation. It shows the necessity of having the same boundary conditions to be able to compare the damage results.
The visual observation of test 24C showed a group tear-out failure of the end brackets in an early stage of the test. This can be seen in the flat slope of the damage curve. In test #27 the wall was performing very strong until 3 brackets failed in tension in a proceeded stage of the loading as seen in the top end of the curve (Fig. 15).

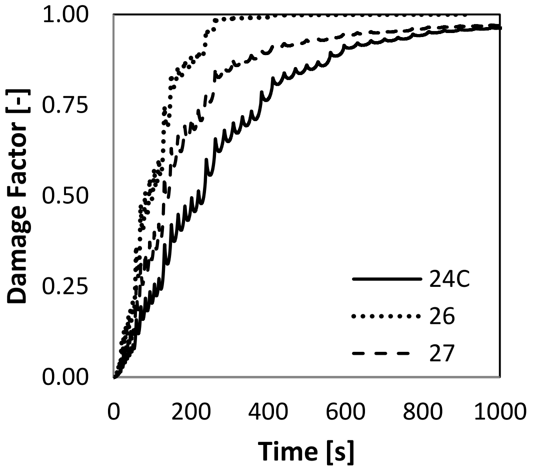


Figure 14: *Wall results for group 7 (4.8m x 2.3m)*

5.4 COMPARISON OF CONNECTION AND FULL SIZE WALL TEST RESULTS

For a representative comparison between connection and wall tests #7 walls were picked. Six walls were of size 2.3m x 2.3m. One wall has the dimensions of 3.45m x 2.3m. To compare the results the average values of the connection tests were used. All walls were subjected to a vertical load of 20 kN/m. The Comparison of Figures 16 to 20 shows that the wall results do never exceed the damage level of the connection tests before D= 0.75 is reached. The connection test set-up restricts movements to the particular direction which will be tested. Movements out of plane as well as bending of test samples are prevented. Compared to the connection tests the test walls were only guided at the top end to keep them in plane. The movement at the bracket was free in all directions. The walls for this comparison were generally connected with 4 brackets to the foundation. Through rocking of the wall panel these brackets experienced a considerably bigger displacement than the inner brackets. However, the calculated damage index considers the entire wall. This is the reason of the different slopes of the damage index curves between connection and wall test.
Walls in Figures 16 to 19 had a size of 2.3m x 2.3m and were connected to the foundation with 4 brackets. Wall 13C (Fig. 20) had a size of 3.45m x 2.3m and was hold in place with 9 brackets. Wall 13C shows a good correlation between connection and wall test. However, walls with a different size and number of connectors cannot reach this correlation although Figure 16 shows a close result.
The comparison of the energy-based damage accumulation index of connection and wall test indicate that size and number of brackets have a significant influence on the result. For all conducted wall tests the damage index stayed at any point below the damage index of the isolated connection test.

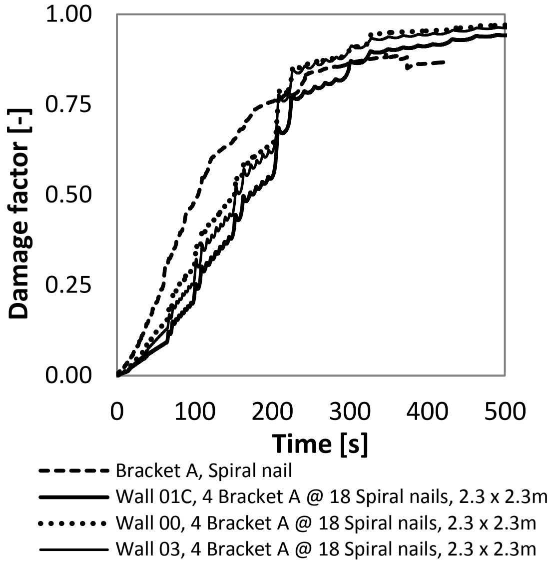


Figure 15: *Comparison between connection and wall test, Bracket A with Spiral nail 3.8 x 89mm*

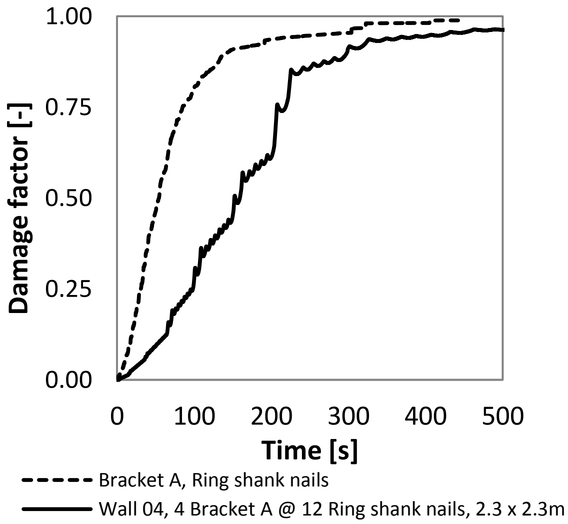


Figure 16: *Comparison between connection and wall test, Bracket A with Ring shank nail 3.4 x 76mm*

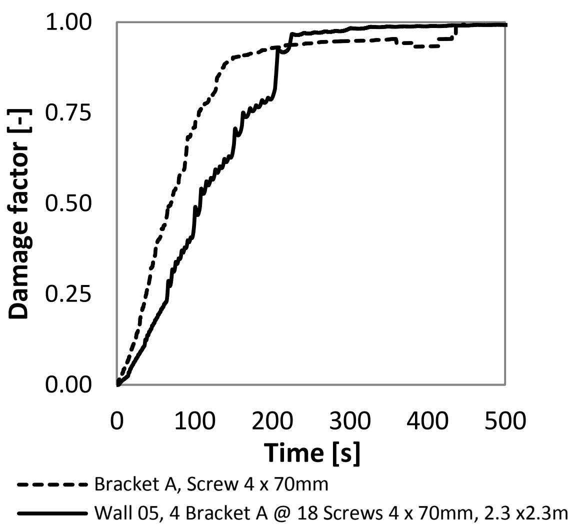


Figure 17: *Comparison between connection and wall test, Bracket A with Screw 4 x 70mm*

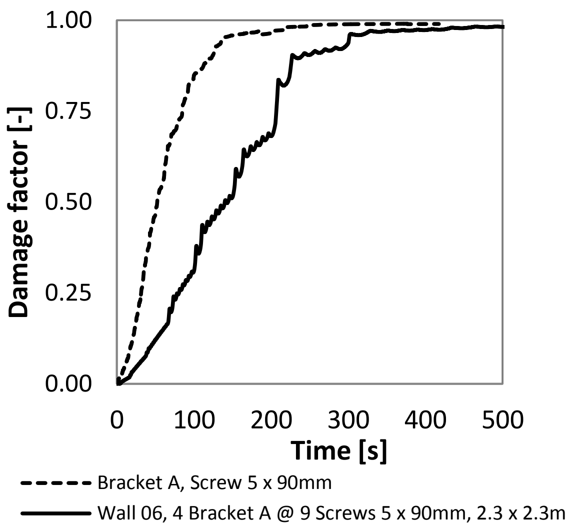


Figure 18: *Wall Comparison between connection and wall test, Bracket A with Screw 5 x 90mm*

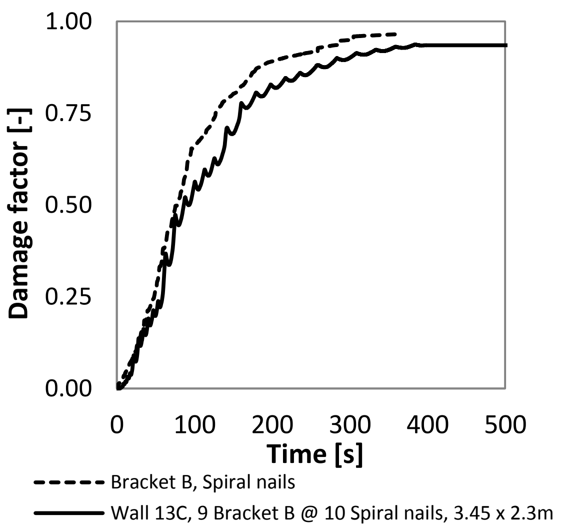


Figure 19: *Comparison between connection and wall test, Bracket B with Spiral nails 3.8 x 89mm*

5.5 DAMAGE PREDICTION

In this paper the connection and shear wall tests were analyzed using Kratzig's index (energy-based index). In order to make the proposed damage model useful for predicting and evaluating the damage state of CLT connections, the relationship between calculated damage index and observed damage needs to be established.

Table 3: *Classification of Damage for CLT connections*

Degree of damage	Damage description	Damage index scale
None	No visible damage observed	D < 0.20
Minor	Minor pull-out of fasteners; light plastic deformation of bracket; minor repairs are required	0.20 ≤ D < 0.35
Moderate	Visual permanent deflections of bracket; shear failure of small number of fasteners; extensive pull-out of fasteners;	0.35 ≤ D < 0.65
Severe	Major or complete failure of fasteners; severe crack in bracket; separation of bracket from CLT panel; requires replacement of bracket in different position at CLT wall to be serviceable again; severe wood crushing in outer layer of CLT	0.65 ≤ D < 0.75
Collapse	Total or partial collapse of connection	D > 0.75

Obtained hysteresis graphs (Fig. 15) and visual observations (Fig. 20) were used to calibrate the applied damage index. The classification consists of five damage limit states. To address the observed damage in the conducted tests damage states for CLT connections were defined accordingly. The results from the visual observations were validated with the calculated damage indices. Previous research considered the final stage after completing the loading protocol for the damage indices. The focus for CLT connections was placed on the development of the damage accumulation over time. The results were validated with the destructivity of the connection. The limit states were determined as *None*, *Minor*, *Moderate*, *Severe*, and *Collapse*. The relationship between damage index and observed damage is shown in Table 3

6 CONCLUSIONS

To use CLT in regions with a high earthquake hazard, it is necessary to evaluate the damage of such structures which may be subjected to ground motions due to a major earthquake. In this paper, Kratzig's energy-based damage accumulation index was utilized to evaluate connections and entire CLT walls subjected to cyclic loading.

The connection tests with nails and screws in CLT have showed adequate seismic performance. It is of interest to see that the direction of the applied load does not change the slope of the curve of an energy-based damage index. By comparing the calculated damage index over time with the visual observation of the individual connection tests it could be shown a good correlation with the proposed connection damage scale. The calculated damage indices for the full size shearwalls were validated according to the same principle as the connection tests. By analyzing the results it can be proven that the proposed connection damage scale can represent the wall behaviour satisfactorily.
The CLT wall panel in the conducted tests can be assumed as stiff. Thus, the ductility of the wall system comes from the connections. Among these different types of connections, bracket A and B with spiral nails agreed with the wall behaviour the best.
It can be summarized that the slope of the damage accumulation allows determining the stiffness of the connection and the degree of damage which will be reached at certain times. The calculated damage curves can be compared in a clear way with other calculated curves. For this comparison it is required that the CLT panels have the same size as well as the same loading protocol. For a comparison of individual connection and entire wall test result additional research has to be undertaken to determine the influence of size, vertical loading and number of connectors on the damage index.
The energy-based damage index is a reasonable alternative in evaluation of results from cyclic testing of connections and walls.

7 ACKNOWLEDGEMENT

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