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SEISMIC BEHAVIOUR OF CROSS-LAMINATED TIMBER STRUCTURES

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ABSTRACT: European experience shows that besides single family housing, Cross-Laminated Timber (CLT) can be competitive in mid-rise and high-rise buildings. Although this system has not been used to the same extent so far in North America, it can be viable wood structural solution for the shift towards sustainable densification of urban and suburban centres. FPInnovations has undertaken a multi-disciplinary project on determining the structural properties of a typical CLT construction, including quantifying the seismic resistance and force modification factors of CLT buildings. In this paper, some of the results from a series of quasi-static tests on CLT wall panels are presented as well as preliminary estimates for the force modification factors (R-factors) for seismic design of CLT structures. CLT wall panels with various configurations and connection details were tested. Wall configurations included single panels without openings with three different aspect ratios, panels with openings, as well as multi-panel walls with step joints and fasteners between them. Connections for securing the walls to the foundation included off-the-shelf steel brackets with annular ring nails, spiral nails, and screws; a combination of steel brackets and hold-downs; and custom made brackets with timber rivets. Results from two storey configurations that include two walls and a CLT slab in between are presented and discussed. Finally preliminary estimates and recommendations for the force modification factors (R-factors) for seismic design of CLT structures according to National Building Code of Canada (NBCC) are also made.

KEYWORDS: Cross-Laminated Timber, Seismic Performance, R-factors, Connections, Timber Rivets

1 INTRODUCTION

Cross-laminated timber (CLT) was first developed some 20 years ago in Austria and Germany and ever since it is gaining popularity in residential and non-residential applications in Europe. As the production of CLT has already started in Canada, this system can now be used as a viable wood-based structural solution for the shift towards sustainable densification of urban and suburban centres in North America. By using cross-laminated solid timber boards for pre-fabricated wall and floor panels, this system offers many advantages. The cross-lamination process itself provides improved dimensionally stability to the product that allows for prefabrication of long floor slabs and single storey walls. Openings for windows and doors can be pre-cut using sophisticated Computer Numerical Controlled (CNC) machines. CLT panels are easy to process and to assembly with ordinary tools. Quick erection of solid and durable structures is possible even for non-highly-skilled manpower. The good thermal insulation and a fairly good behaviour in case of fire are added benefits resulting from the massiveness of the wood structure. To gain the needed wide acceptance, the CLT as a structural system needs to be implemented in the North American codes arena. For these reasons FPInnovations has undertaken a multi-disciplinary research project on determining the structural properties of typical CLT construction. One of the important parts of the project is to quantify the seismic resistance of structures with CLT panels, including the development of the preliminary force modification factors (R-factors) for seismic design according to National Building Code of Canada (NBCC) [1] and the ASCE 7 in the United States [2]. In this paper, some of the results from a series of quasi-static monotonic and cyclic tests on CLT wall panels are presented, along with some estimates for R-factors according to NBCC.

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2 PREVIOIUS RESEARCH IN THE FIELD

A comprehensive review of the previous research in the area of seismic performance of CLT structures is given in [3] and [4]. A shorter summary of that review is given below, along with some new research in the area.

The most comprehensive study to quantify the seismic behaviour of low and mid-rise CLT construction was the SOFIE project in Italy. This project was undertaken by the Trees and Timber Institute of the National Research Council of Italy (CNR-IVALSA) in collaboration with National Institute for Earth Science and Disaster Prevention in Japan (NIED), Shizuoka University, and the Building Research Institute (BRI) in Japan. The testing programme included in-plane cyclic tests on CLT wall panels with different layouts of connections and openings [5], pseudo-dynamic tests on a one-storey 3-D specimen in three different layouts [6], shaking table tests on a three-storey building under different earthquakes [7], and a series of full-scale shaking table tests on a seven storey CLT building conducted at E-Defense facility in Miki, Japan.

Results from quasi-static tests on CLT wall panels showed that the connection layout and design has strong influence on the overall behaviour of the walls [5]. Hysteresis loops were found to have an equivalent viscous damping of 12% on average which makes the system suitable for implementation in high seismic zones. Shaking table tests on the 3-storey house showed that the CLT building survived 15 destructive earthquakes without any severe damage [6]. Results from the analytical prediction of the response of the 3-story building using the DRAIN 3-DX computer program showed good correlation with the test results. Using the verified analytical models, an evaluation of the behaviour factor *q* for seismic design according to Eurocode 8 was conducted [8] [9]. Full-scale shaking table tests on 7-storey CLT building showed that the structure withstood all tests without any significant damage.

A comprehensive study to determine the seismic behaviour of 2-D CLT wall panels was conducted at the University of Ljubljana. Numerous monotonic and cyclic tests were carried out on walls with different aspect ratios and boundary conditions from the cantilever type all the way to the pure shear [10]. Influence of vertical load and type of anchoring systems were evaluated, along with wall deformation mechanisms [11]. In addition, influence of openings on the shear properties of CLT wall panels was studied and formulae for calculating the wall stiffness were suggested [12]. Analytical prediction of the dynamic response of the 7-storey massive CLT structure tested during the SOFIE project was also carried out [13].

CLT wall tests were also carried out by the Karlsruhe Institute of Technology to compare the performance of such modern system vs. the traditional timber frame construction [14]. Based on the test results, non-linear analytical model of a three-storey 2-D CLT structure (frame) was developed using the DRAIN-2DX computer program. The model was subjected to a series of 20 earthquake records (10 from real earthquakes and 10 synthetic ones) to determine the behaviour factor q for such system using the acceleration based approach. The results showed that an average value of q-factor was 4.7, with the 5^{th} percentile value being 3.3 for all 20 earthquakes.

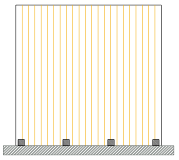
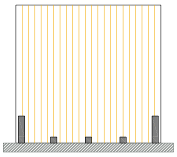
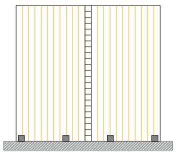
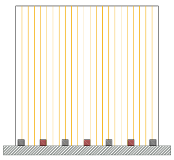
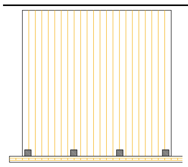
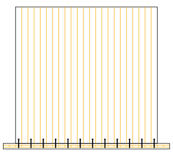
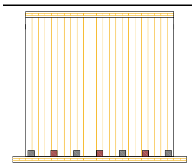
Upon completion of the SOFIE project, further research was carried out in Italy to determine the hysteretic behaviour of single CLT walls, bracket connections, wall half-lap connections with different fasteners, and hold-downs [15]. The test results confirmed that the layout and design of the joints is critical for the overall behaviour of the structural system. Analytical models for connections in CLT structures (brackets, hold-downs and connections between panels) were developed in Abacus software [16]. The connection models were calibrated against the test results and were implemented in models of entire CLT panels. Predicted analytical response of CLT walls subjected to lateral loads closely resembled the wall test results conducted at IVALSA.

3 CLT WALL SPECIMENS TESTED

In the testing program at FPInnovations a total of 32 monotonic and cyclic tests were performed. All walls were 3-ply CLT panels with a thickness of 94 mm. They were made of European spruce and manufactured at KLH Massiveholz GmbH in Austria. CLT walls with 12 different configurations were tested. The details about the testing matrix and the different wall configurations I to XII are given in Tables 1, 2 and 3. In table 1, walls with aspect ratio of 1:1 are shown (2.3m x 2.3m) while in Table 2 walls with aspect ratio of 1:1.5 are shown (2.3m high and 3.45m long). In Table 3, two storey assemblies of 2.3m x 2.3m walls are presented along with tall CLT walls that had a height of 4.9m and length of 2.3m (aspect ratio of 2.1:1).

Four different types of brackets (A, B, C, and D) were used to connect the walls to the steel foundation beam or to the CLT floor panel bellow. Bracket A, BMF 90mm x 48mm x 116mm (W x D x H), and bracket B Simpson Strong Tie (90mm x 105mm x 105mm), were off-the-shelf products that are commonly used in CLT applications in Europe. Brackets C and D were custom made out of 6.4mm thick steel plates to accommodate the use of timber rivets. The designations of the tests shown in Tables 1, 2 and 3 were developed to show the bracket type and the fastener type used in the tests. For example designation CA-SNH-08A means that the **C**LT wall had brackets type **A, S**piral **N**ails as fasteners, had **H**old-downs and it is test number 08A. The following acronyms were also used in the test designations: TR for **T**imber **R**ivets, RN for Annular **R**ing **N**ails, S1 for SFS screws 4 x 70mm, S2 for SFS screws 5x90mm, and WT for SFS WT-T type screws. Walls in the configuration I had four brackets spaced at 710mm o.c. In addition to 3, type A brackets spaced at 550mm o.c. nailed with 18 spiral nails (D = 3.9mm; L = 89mm), walls 07, 08 and 08A of the configuration II, had Simpson Strong Tie HTT-16 hold-downs at both ends.

Table 1: *Test matrix for 2.3m long and 2.3m high walls*

Wall Configuration	Test Designation	Fasteners	Loading
 I	CA-SN-00	Bracket A SN 16d, n=18 D=3.9mm L=89mm	CUREE
	CA-SN-01		Mono
	CA-SN-02		CUREE
	CA-SN-03		CUREE
	CA-RN-04	RN 10d, n=12	CUREE
	CA-S1-05	S1 n=18	CUREE
	CA-S2-06	S2 n=10	CUREE
	CC-TR-09	Bracket C, Rivets L=65mm, n=10	Mono
	CC-TR-10A	Bracket C, Rivets L=65mm, n=10	CUREE
 II	CA-SNH-07	SN 16d, n=18 same on HD	Mono
	CA-SNH-08	SN 16d, n=18 same on HD	CUREE
	CA-SNH-08A	SN 16d, n=18 12d, n=18 on HD	CUREE
 III	CA-SN-11	SN 16d, n=18 WT-T n=12	CUREE
	CA-SN-12	SN 16d, n=18 SFS1 n=12	CUREE
	CA-SN-12A	SN 16d, n=18 SFS1 n=12	ISO
 IV	CA-SN-20	Bracket A SN 16d, n=18 D=3.9mm L=89mm 3 brackets on the back side	CUREE
 V	CA-SN-21	Bracket A SN 16d, n=6 D=3.9mm L=89mm	CUREE
 VI	CS-WT-22	WTT-T n=18 D=6.5mm L=130mm	CUREE
	CS-WT-22B	WTT-T n=34 D=6.5mm L=130mm	CUREE
 VII	CA-SN-23	Bracket A SN 16d, n=6 D=3.9mm L=89mm 3 brackets on the back side	CUREE

Walls from configuration III (11, 12 and 12A) consisted of two panels that were connected to the foundation in the same way as walls form configuration I. The two panels were connected together using a continuous 65mm long step joint (half-lap) with no gap, and one vertical row of screws. Twelve SFS WTT-T type screws with D=3.8mm and L= 89mm, spaced at 200mm were used in wall 11. Panels in walls 12 and 12A were connected with SFS screws with D=5.0mm and L=90mm. These walls were designed to investigate the effect of gaps in the walls on the overall wall performance under lateral loads.

Table 2: *Test matrix for 3.45m long and 2.3m high walls*

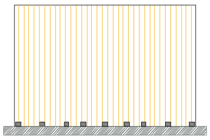
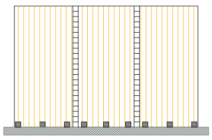
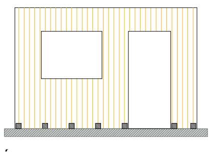
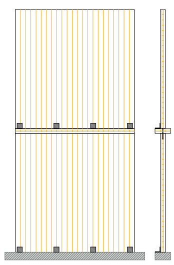
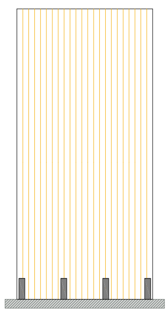
Wall Configuration	Test Designation	Fasteners	Loading
 VIII	CB-SN-13	Bracket B SN 16d, n=10 9 brackets	Mono
	CB-SN-14	Bracket B SN 16d, n=10 9 brackets	ISO
 IX	CB-SN-15	Bracket B SN 16d, n=10 9 brackets SFS1 n=8	Mono
	CB-SN-16	Bracket B SN 16d, n=10 9 brackets SFS1 n=8	ISO
 X	CB-SN-19	Bracket B SN 16d, n=10 D=3.9mm L=89mm	ISO

Table 3: *Test matrix for two storey assemblies and tall walls*

Wall Configuration	Test Designation	Fasteners	Loading
 XI	CA-SN-28	Bracket A SN 16d, n=6	Mono
	CA-SN-29B	Bracket A SN 16d, n=6	ISO
	CA-SN-29C	Bracket A SN 16d, n=8	ISO
 XII	CD-TR-24	Bracket D Rivets L=65mm n=40	Mono
	CD-TR-25	Bracket D Rivets L=65mm n=40	ISO
	CD-TR-26	Bracket D Rivets L=90mm n=40	ISO
	CD-TR-27	Bracket D Rivets L=90mm n=20	ISO

To investigate the effect of the foundation stiffness in a real case scenario, walls in configurations V, VI and VII were placed over a 94mm thick CLT slab with a width of 400mm. Wall 21 used 4 type A brackets spaced 710mm o.c., while wall 23 used a total of 7 brackets (4 in front and 3 back) in the same arrangement as in wall 20. The brackets were connected to the CLT floor slab using 3 SFS WFC screws with D=10mm and L=80 mm. Wall 22 used 9 pairs of SFS WT-T 6.5 x 130mm screws placed at an angle of 45 degrees to the slab spaced at 280 mm. Wall 22B used 17 pairs of the same screws with 5 pairs being closely grouped near each end of the wall (spaced at 40 mm) to simulate a hold-down effect. The rest of the screws were spaced at 320 mm.

Walls from configurations VIII, IX, and X were 3.45m long 2.3m high. Walls 13 and 14 (configuration VIII) were single panel walls that had a total of 9, type B brackets, each with ten, 16d spiral nails. Brackets were spaced at different spacing, varying from 320mm to 460mm. Walls 15 and 16 of configuration IX were 3-panel walls, with the same number and position of the brackets as the walls of configuration VIII. The panels were connected between them by step joints and fasteners. Walls 15 and 16 used 8 SFS screws 5x90mm spaced at 300mm. Configuration XI included 3 two-storey wall assemblies consisting of a lower an upper storey wall (2.3m x 2.3 m) with a 94mm CLT slab in between. Both walls were connected at the bottom using type A brackets, spaced at 710mm o.c. Finally configuration XII consisted of four single panel tall walls (2.3m x 4.9m) that were connected to the steel foundation beam using 4 type D brackets spaced at 710mm. Walls 24 and 25 had 40 rivets in each bracket (L=65mm), wall 26 had the same number of 90mm long rivets, while wall 27 had 20, L=90mm rivets. A tall two storey CLT wall specimen ready for testing in the testing apparatus is shown in Figure 1.

Figure 1: *Two storey CLT specimen ready for testing*

4 RESULTS AND DISCUSSION

CLT wall panels behaved almost as rigid bodies during the testing. Although slight shear deformations in the panels were measured, most of the panel deflections occurred as a result of the deformation in the joints connecting the walls to the foundation. In case of multi-panel walls, deformations in the step (half-lap) joints also had significant contribution to the overall wall deflection. Selected average properties (based on both sides of the hysteretic loops) of the CLT walls tested are given in [3] and [4].

The value of axial load had some impact on the lateral resistance of the walls, although not as significant as in wood-frame shearwalls. Wall 00 with no vertical load had a maximum resistance of 88.9 kN while wall 02 with 10kN/m vertical load had a lateral resistance of 90.3 kN. When the vertical load was increased to 20kN/m (wall 03) the resistance increased to 98.1 kN, an increase of 10%. The maximum loads obtained from the static tests were greater than the corresponding values obtained from the cyclic tests for each of the two cyclic protocols. An example of this is given in Figure 2. It was also observed that during the static tests more deformation demand was induced on the brackets themselves than on the fasteners used to connect them. It is therefore suggested that cyclic be used for determining the properties of CLT wall panels under seismic loads.

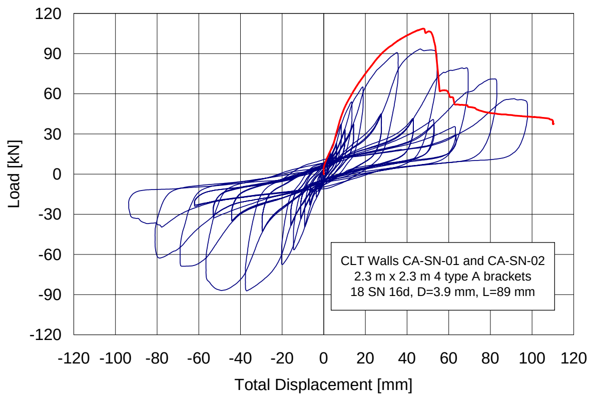


Figure 2: *Results from monotonic and cyclic tests on CLT walls with same configuration*

Wall 04 with twelve, 10d annular ring nails per bracket exhibited slightly higher resistance than wall 03 with eighteen 16d spiral nails per bracket. This was mainly due to the higher withdrawal resistance of the ring nails. The ductility of the wall 04, however, was slightly lower than that of wall 03. The failure mode observed at the brackets for wall 04 was also slightly different than that of wall 03. While spiral nails in the brackets exhibited mostly bearing failure combined with withdrawal of the nails, the ring nails in withdrawal had a tendency to pull out small chunks of wood along the way. The walls with screws (05 and 06) reached similar maximum loads as the walls with nails. The load carrying capacity for CLT walls with screws, however, dropped a little bit faster at higher deformation levels than in the case of walls with nails.

CLT wall panel with hold-downs (wall 08A) showed one of the highest stiffness for a wall with a length of 2.3 m. It also showed high ductility capacity and therefore this wall configuration can be recommended for use in regions with high seismicity.

By introducing a step joint in the wall i.e. creating a wall of two separate panels, the behaviour of the wall was not only influenced by the types of fasteners in the bottom brackets, but also by the type of fasteners used in the step joint. These walls (11 and 12) showed reduced stiffness by 32% and 22% respectively, with respect to the reference wall 03. Both walls were able to shift the occurrence of the yield load and ultimate load at higher deflection levels.

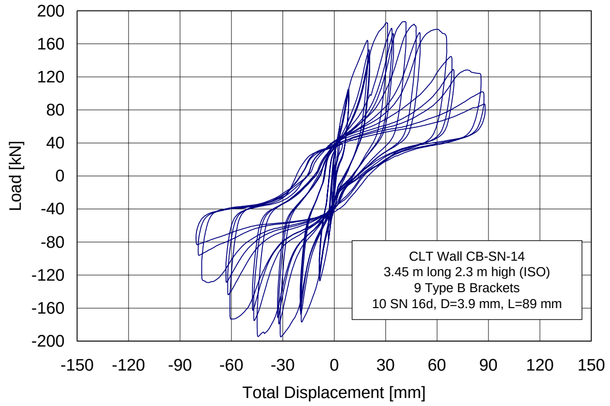


Figure 3: *Hysteresis loop for wall 14 consisting of one 3.45 m long panel*

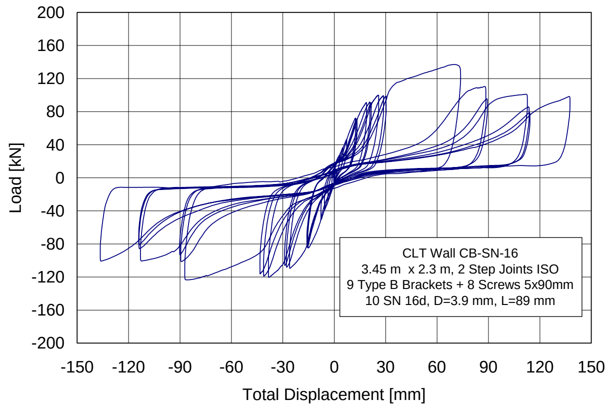


Figure 4: *Hysteresis loop for the 3-panel wall 16 where panels were connected with regular screws 5x90 mm*

The presence of the step joints and the type of fasteners used to connect them was found to have more significant influence on the overall wall behaviour as the length of the wall increases. For example by comparing the results from walls 14 and 16, with length of 3.45 m, we can see the significant change in stiffness and strength properties of the wall 16 with step joints (Figures 4) with respect to the reference wall without step joints - wall 14 (Figure 3). Introduction of step joints enabled wall 16 to carry a significant portion of the maximum load at higher deformation levels

Specimens 22 and 22B that were connected to the base CLT panel with WT-T type screws placed at 45^{o} showed lower resistance than any single storey wall in the program. Grouping the screws at the ends of the panels (wall 22B) created a hold-down effect and helped increase the wall capacity by about 30% compared to that of wall 22. Based on the test results, use of screws at an angle as a primary connector for wall to floor connections is not recommended for structures in seismic regions due to reduced capability for energy dissipation and the sudden pull-out failure of screws in tension (Figure 5).

Figure 5: *Failure mode of the Wall 22B – screw pull-out of the wood plate*

The CLT wall 10A with ten rivets per bracket exhibited by far the highest stiffness than any wall with a length of 2.3 m, with its stiffness being 220% higher than the wall 03 with 18 spiral nails per bracket. They were also able to carry more load per single fastener than any other fastener used in the program. In addition, the wall was able to attain high ductility levels. The hysteresis loop for wall 10A with timber rivets is shown in Figure 6.

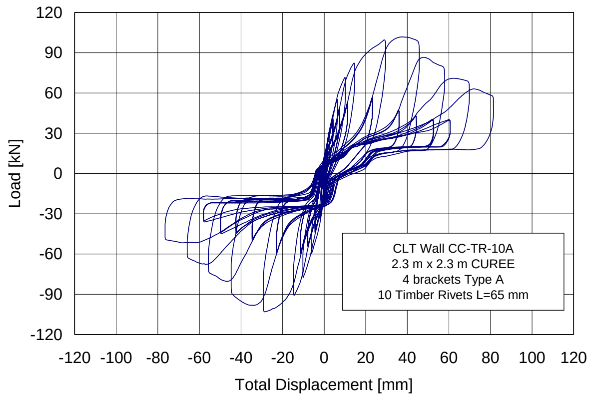


Figure 6: *Hysteresis behaviour of wall with timber rivets*

The behaviour of the tall walls specimens with riveted connections, however, was highly influenced by the number of rivets used in the brackets. Although the number of and spacing of the rivets in the brackets for walls 24, 25 and 26 was chosen to satisfy the rivet yielding failure mode according to the existing Canadian code specifications, that was not satisfactory for use in CLT. All tall walls, except wall 27, experienced fastener pull-out combined with wood shear plug failure mode. By increasing the spacing between the rivets in wall 27, the failure mode was changed to the desired rivet yielding mode.

Results from tests on two-storey assemblies (walls 28, 29B and 29C) showed that most of the deformation was concentrated at the connections at the bottom of the first storey wall. For example, for the East side of wall 29B, the maximum uplift between the floor slab and the upper storey was 4.2mm, while it was around 60mm at the bottom. This resulted in most of the damage during the tests to occur at the bottom connections (Figure 7a), while virtually no damage was observed in the connections on the upper storey (Figure 7b). No significant crushing of the slab or sliding at the top floor was observed.

a)

b)

Figure 7: *Deformation of the connections of the two storey specimen 29 B (a) at the bottom; (b) at the top*

Figure 8 shows the deflection of the wall 29C at the top of the both storeys at various stages of the testing; the nearly linear lines indicating that the deformation came mostly from rotation at the base. Also the shape of the lines are very similar indicating that the deformation in the inelastic range comes from rotation at the base and not shear deformation or rotation at the intermediate floor level.

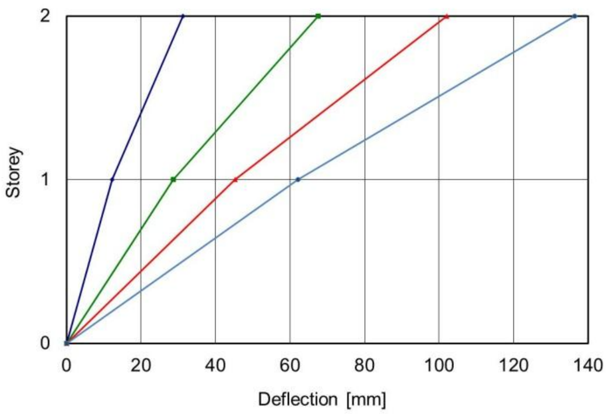


Figure 8: *Deflection of the wall 29C at the top of the both storeys at various stages of testing*

5 FORCE MODIFICATION FACTORS

The force modification factors (R-factors) in building codes in North America account for the capability of the structure to undergo ductile nonlinear response. This increases the building period and allows the structure to be designed for seismic forces smaller than the forces that would be generated if the structure remained elastic, without increasing the displacements from the seismic loads. In the 2010 edition of the National Building Code of Canada, the elastic seismic load is reduced by two types of R-factors, R_o-factor which is related to the over-strength of the system and an R_d-factor that is related to the ductility of the structure. In the major model codes in the United States, the International Building Code [17] and the ASCE7 [2], there is only one R-factor, called the response modification coefficient, which reduces the seismic design force. In Eurocode 8 [18], the European model code for seismic design, there is also only one factor used, the q-factor.

In this paper, preliminary estimates of the the R-factors for seismic design of CLT structures using three approaches was made: (a) based on research findings on this topic from Europe, (b) comparison with structural systems already in NBCC, and (c) the performance-based equivalency approach speciffied in the International Code Councli AC 130 criteria for assigning of an R-factor for new shearwall assemblies [19].

5.1 THE EUROPEAN EXPERIENCE

As mentioned in the section on previous research, first attempt to quantify the seismic response modification factors (q-factors) in Eurocode 8 was performed by Ceccotti and his collaborators [9]. An analytical model of the 3-storey house was developed using the DRAIN 3-DX computer program, and the model was verified against the observed behaviour during the shaking table tests. Using the verified analytical model, a number of non-linear dynamic time-history analyses were conducted using eight different earthquake records. Based on the results from the analytical studies, an evaluation of the behaviour factor q for seismic design according to Eurocode 8 was obtained. The behaviour factor q was defined as the ratio between the Peak Ground Acceleration (PGA) that caused near collapse the structure (the analytical model) vs. the design PGA. The near collapse peak ground acceleration was taken as the acceleration that caused uplift of 25.5mm at one or more hold-down positions in the walls, while the design PGA was taken equal to 0.35g. For seven out of eight earthquakes the *q* factor was greater than 3.0 and in two cases even greater than 4.0, with an average of 3.4. Although the results presented here apply to only one, 3 -storey CLT structure with a given configuration and initial period, still some observations can be made. It seems that the q factor of 3 is a reasonable conservative estimate for the CLT structure evaluated that is a representative of a typical 3 storey building that uses screws in the brackets and nails in the hold-downs.

Evaluation of the q-factor was also conducted at the Karlsruhe Institute of Technology [14]. A non-linear analytical model of a three-storey stacked shear wall structure in DRAIN-2DX was subjected to 20 different earthquake motions. Using a slightly modified version of the acceleration approach, it was found for the analysed structure that the average value for q was 4.7 and the 5th percentile value was 3.3.

Pozza et al. introduced a base shear approach to determine the q-factor for CLT structures [20]. There are two main differences in this base-shear based approach with respect to the PGA based approach. First, the q-factor is defined as a ratio of the base shear of the building during a linear elastic response and the base shear at near collapse level for each different record. This is a more common assumption as it defines the q-factor close to the terms of the well-known equivalent displacement rule. Second, since the response of the building subjected to different earthquakes is also dependent on its initial period of vibration, buildings with three different initial periods were used in the analyses. One building was the same three-storey model tested during the SOFIE project, while the other two were with the same geometry, but just different mass. In this study, the authors also used different analytical model to represent the behaviour of the CLT elements in the buildings. The model was verified against the test results from the shaking table tests on the 3-storey building that was part of the SOFIE project. Five artificially generated earthquakes that meet the spectrum compatibility requirements for the regions with highest seismicity in Italy were used in the analyses. The values of the q-factor calculated according to the base-shear method varied between a minimum of 2.75 and a maximum of 3.52, with the aveargae of 3.14. In general these values had a lower variability than those defined according to the acceleration based method (min. 2.86, max. 4.57, and avarage of 3.83). The base shear approach also showed lower values for the calculated q-factor that was on average 21% lower than using the PGA approach. The average values, however, showed again that a q-factor of 3 is acceptable for use in the seismic design of CLT structures in Europe.

A straight forward comparison of the suggested q-factor for CLT in Europe from both studies to the force modification factors in NBCC would correspond to a combination of Rd · Ro = 3.0. Several things should be noted, however, which may have an influence on such statement. First, unlike in the US and Canada, the Eurocode 8 uses design ground motions with probability of exceedance of 10% in 50 years (earthquake return period of 475 years) and that may have an effect on the findings for the seismic factors. Second, both approaches presented here used elastically designed structures as reference structures for determining the q-factor, which usually leads to more conservative values of the q-factor. An approach that is suggested for the future research should include analytical models of structures already designed with a certain seismic factor and conduct incremental non-liner dynamic analyses using a set of earthquake ground motions with increasing PGA or pseudo spectral acceleration (PSA) values.

5.2 COMPARISON TO EXISTING SYSTEMS IN NBCC

Although braced timber frames and portal moment resisting frames are completely different structural systems compared to the CLT construction, they have some common points, as the seismic performance of all these systems is mainly influenced by the behaviour of the connections. In braced frames it is the connection between the brace and the rest of the frame, while in portal moment frames it is the connection between the column and the beam, that governs the structural behaviour. These two structural systems have already been assigned an Rd-factor of 2.0 and Ro-factor of 1.5 in NBCC, when designed with ductile connections. Based on the research results from tests on connections used in heavy timebr systems [21] that largely influence the system behaviour, and on the tests results on braced frames [22], performance of CLT panels with ductile connections (such as nails or slender screws) is equivalent if not better than that of braced frames.

In addition, although it is a platform type of structural system, the CLT construction is far less susceptible to development of soft storey mechanism than many other structural systems of the same type. Since the nonlinear behaviour (and the potential damage) is localized to the bracket and hold-down connection areas only, the panels that are also the vertical load carrying elements are virtually left intact in place, and well connected to the floor panels, even after a “near collapse” state is reached. Also, all walls in one storey in CLT construction contribute to the lateral and gravity resistance, thus providing a degree of redundancy.

The Ro factor for structures with CLT wall panels in NBCC can be determined according to the procedure specified in [23]. Based on the type of the connections used, a value of Ro of 1.5, which is currently used for most connections in heavy timber construction designed according to the CSAO86, is considered to be a reasonable conservative estimate.

5.3 EQUIVALENCY APPROACH (AC 130)

According to the ICC-ES acceptance criteria document AC130 [19], assigning an R-factor for new wood shear wall assembly (used in conjunction with wood-frame shearwalls) according to ASCE 7 in the US can be made by showing equivalency of the seismic performance of the new wall assembly (in terms of maximum load, ductility, and storey drift) obtained from quasi-static cyclic tests, with respect to the properties of lumber-based nailed shearwalls, that are already implemented in the code. Although CLT wall panels as a system differ from wood-frame shearwalls, the equivalency criteria given in AC130 can be used for assessing the seismic behaviour of CLT panels, since the criteria are performance-based. For example the AC130 criteria specify that for a new shearwall assembly to have the same seismic design response coefficients (R=6.5, C_d=4.0, Ω_0=3.0) used for regular shearwalls according to ASCE 7, the assembly shall have the response characteristics listed below:

(a) The lower bound on the ratio of the displacement at the post-peak load to the displacement at the assigned Allowable Stress Design (ASD) load level shall be equal or greater than 11 as shown in equation (1)

$$\frac{\Delta_u}{\Delta_{ASD}} \geq 11 \tag{1}$$

where Δ_{ASD} is the displacement at the ASD load level developed according to the International Building Code (IBC) and Δ_U is the ultimate displacement taken from the backbone curve corresponding to an absolute load having no more than 20 percent strength degradation of the post-peak load as given in ASTM E 2126 [24].

(b) The minimum post-peak displacement shall be in accordance with the requirements of equation (2),

$$\Delta_u \geq 0.028 \cdot H \tag{2}$$

where H is the height of the panel element.

(c) The ratio of the maximum load P_{max} obtained from the backbone curve of the panel to the assigned ASD design load P_{ASD} shall be in accordance with the requirements of equation (3)

$$2.5 \leq \frac{P_{\max}}{P_{ASD}} \leq 5.0 \tag{3}$$

We can use the requirements shown in equations (1) to (3) to assess the performance of CLT panels according the AC130 criteria. In order to do that we have to make the assumption that the future design values (lateral resistances) for CLT panels will have the same "safety margin" as that of regular wood-frame shearwalls according to IBC and CSAO86, the Canadian Standard for Engineering Design in Wood [25]. In such a case, we can assume that the design values for lateral loads for CLT panels can be derived in the same way as they were determined for wood-frame shearwalls. The specified strengths for shearwalls in Canada were soft converted from the Allowable Stress Design (ASD) values of the Uniform Building Code (UBC) in the US. The ASD values in UBC were derived using the average maximum load obtained from monotonic pushover tests divided by a safety factor of 2.8, or the average maximum load from cyclic tests divided by a safety factor of 2.5. In our case since we use the cyclic test results, we will use the second one. In addition, to be compatible with the AC130 criteria, only single storey walls tested under the CUREE protocol will be used for the analyses. Also, walls that included WT-T screws will be excluded as they showed undesirable failure modes. Finally, the influence of the vertical load is assumed not to have significant effect on the wall performance, as AC130 criteria do not require presence of a vertical load during the testing of the elements.

Main response parameters related to the AC 130 criteria obtained from average envelopes (first and third quadrant) of single storey cyclic tests on CLT wall panels are shown in Table 4. In the Table, P_{max} is the maximum load, P_{ASD} is the load that would be an equivalent to ASD design level (determined as P_{max}/2.5), Δ_{ASD} is the displacement at P_{ASD}, and the Δ_u is the ultimate displacement (displacement at which the load has dropped to 80% of the maximum). As can be seen from the results, although the avarage of every single wall cannot satisfy the criteria, the avarage from all walls can satisfy the performance levels required by the AC 130 criteria. The average ductility ratio (as defined in AC130) is 11.8, which is greater than the required minimum of 11, and the average ultimate storey drift is 3.0%, which is greater than the required 2.8%. The minimum value for the maximum drift was about 2% while the minimum ductility was approximately 6. The average values can further improve if we take into consideration only the set of CLT walls with nailed connections only (excluding the walls 5 and 6 that used screws).

Table 4: *Average properties of selected CLT walls according to AC 130 acceptance criteria*

Wall	P_{ASD} [kN]	Δ_{ASD} [mm]	P_{max} [kN]	Δ_u [%]	Ductility Δ_u/Δ_{ASD}
00	35.6	7.8	88.9	3.0	9.4
02	36.1	8.5	90.3	3.3	8.5
03	39.2	7.5	98.1	2.7	8.8
04	40.9	7.5	102.3	2.8	8.1
05	41.1	8.0	102.7	1.9	6.8
06	40.0	8.1	100.1	2.3	6.2
08A	42.8	4.9	107.1	2.7	13.7
10A	41.0	3.2	102.4	2.0	16.5
12	37.0	8.6	92.5	3.1	8.5
14	76.4	3.9	190.9	3.0	17.7
16	52.1	8.2	130.2	4.4	13.2
20	60.8	8.6	152.1	3.1	8.7
21	21.6	3.6	54.1	3.7	23.8
23	28.9	5.6	72.2	3.4	15.0
Average for all panels above				3.0	11.8
Minimum for all panels above				1.9	6.2

Based on the test results and the performance based AC130 acceptance criteria, some of the individual CLT walls tested (and the average of the entire wall set) can qualify as new structural wall elements and can share the same seismic response parameters with regular wood-frame shearwalls in the US, which means using an R factor of 6.5. This value would correspond to having the product of R_d R_o in Canada of 5.1 (R_d factor of 3.0 and R_o factor of 1.7), which is currently used in NBCC for wood-frame shearwalls.

Despite these encouraging findings, at this early stage of the system acceptance and pending results of more complex non-linear dynamic analyses, the authors are of the opinion that a bit more conservative factors should be assigned for CLT wall systems at this point. It is recommended that conservative estimates of R_o =1.5 and R_d = 2.0 should be used for force modification factors for CLT as a structural system.

5.4 NON-LINEAR DYNAMIC ANALYSES

Non-linear dynamic analyses on verified structural models of CLT structures designed with a certain R-factor are one of the best ways to verify the validity of the force modification factor used. Fpinnovations has joined forces with researchers from the South Dakota State University and the University of Alabama in conducting such analyses, including the use of Direct Displacement Based Design for CLT structures. Results from these analyses will be presented in the near future.

The best procedure for quantification of R-factors for seismic design is the procedure developed during the ATC-63 project (FEMA P-695 document) [26]. This document contains a procedural methodology where for first time the inelastic response characteristics and performance of typical structural systems could be quantified, and the adequacy of the structural system provisions to meet the design performance objectives could be verified. The methodology directly accounts for the potential variations in structural configuration of buildings, the variations in ground motion to which these structures may be subjected, and the available laboratory data on the behavioral characteristics of structural elements. The drawbacks of the FEMA P-695 procedure are that it is quite complex, very time consuming and therefore very expensive. A large number of non-linear dynamic analyses are required on a number of different building models with different configurations. In addition, the types of analyses required are sophisticated, and out of reach of the average design engineers, especially in the area of timber design. FEMA has also published a new procedure FEMA P-795 entitled "Quantification of Building System Performance and Response Parameters - Component Equivalency Methodology" [27] that is much easier to use and can be used to introduce CLT as substituting element in a structural system that is already implemented in the codes.

6 CONCLUSIONS

Based on the research work conducted around the world and the results from a series of quasi-static tests on CLT wall panels conducted at FPInnovations in Vancouver, CLT wall panels can be an effective lateral load resisting system. They can have adequate seismic performance when nails or slender screws are used with the steel brackets to connect the walls to the floors below. The use of hold-downs with nails on each end of the walls tends to further improve their seismic performance. Use of diagonally placed long screws to connect the CLT walls to the floor below is not recommended in high seismic zones due to less ductile wall behaviour and sudden failure mechanism. Use of step (half lap) joints in longer walls can be an effective solution not only to reduce the wall stiffness and thus reduce the seismic input load, but also to improve the wall deformation capabilities. Timber rivets in smaller groups with custom made brackets were found to be effective connectors for CLT wall panels. Further research in this field is needed to further clarify the use of timber rivets in CLT.

Although CLT construction is a platform type of structural system, it is far less susceptible to the soft storey mechanism than many other structural systems of the same type. Since the nonlinear behaviour (and the potential damage) is localized to the hold-down and bracket connection areas only, the panels that are also the vertical load carrying elements are virtually left intact in place, and well connected to the floor panels, even after a "near collapse" state is reached. In addition, all walls in one storey in CLT construction contribute to the lateral and gravity resistance, thus providing a degree of redundancy and a system effect.

Preliminary evaluation of the force modification factors (R-factors) for the seismic design of structures according to NBCC was also performed. Based on the experimental and analytical research work conducted in this field to date in Europe and at FPInnovations, the performance comparison to already existing systems in NBCC, and on the equivalency performance criteria given in AC130, values of 2.0 for the R_d factor and 1.5 for the Ro factor are recommended as conservative estimates for CLT structures that use ductile connections such as nails and slender screws.

Research results from non-linear dynamic analyses on models of CLT structures designed with a certain R-factor that are conducted at the time of submission of this paper, will provide more information on appropriate values of R-factors for CLT structures.

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