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DUCTILITY BASED FORCE REDUCTION FACTORS FOR SYMETRICAL CROSS-LAMINATED TIMBER STRUCTURES

Marjan POPOVSKI[1], Shiling PEI[2], John W. van de LINDT[3], Erol KARACABEYLI[4]

ABSTRACT

Cross-laminated timber (CLT) as a structural system has not been fully introduced in European or North American building codes. One of the most important issues for designers of CLT structures in earthquake prone regions when equivalent static design procedure is used, are the values for the force modification factors (R-factors) for this structural system. Consequently, the objective of this study was to derive suitable ductility-based force modification factors (Rd-factors) for seismic design of CLT buildings for the National Building Code of Canada (NBCC). For that purpose, the six-storey NEESWood Capstone wood-frame building was redesigned as a CLT structure and was used as a reference symmetrical structure for the analyses. The same floor plan was used to develop models for ten and fifteen storey buildings. Non-linear analytical models of the buildings designed with different Rd-factors were developed using the SAPWood computer program. CLT walls were modelled using the output from mechanics models developed in Matlab that were verified against CLT wall tests conducted at FPInnovations. Two design methodologies for determining the CLT wall design resistance (to include and exclude the influence of the hold-downs), were used. To study the effects of fastener behaviour on the R-factors, three different fasteners (16d nails, 4x70mm and 5x90mm screws) used to connect the CLT walls, were used in the analyses. Each of the 3-D building models was subjected to a series of 22 bi-axial input earthquake motions suggested in the FEMA P-695 procedure. Based on the results, the fragility curves were developed for the analysed buildings. Results showed that an Rd-factor of 2.0 is appropriate conservative estimate for the symmetrical CLT buildings studied, for the chosen level of seismic performance.

INTRODUCTION AND PREVIOUS RESEARCH

With two producers already in operation in Canada and one in the US, the use of cross-laminated timber (CLT) is gaining popularity in North America. Since CLT is not fully introduced as a structural system in the building codes and material standards, one of the most important issues for designers of CLT structures in earthquake prone regions when Equivalent Static Design Procedure is used are the values for the force modification factors (R-factors) for this system. The R-factors in building codes in North America account for the building over-strength and its capability to undergo ductile nonlinear

[1] Principal Scientist, FPInnovations, Advanced Building Systems, Vancouver, BC, Canada; E-mail: marjan.popovski@fpinnovations.ca
[2] Assistant Professor, Department of Civil and Environmental Engineering, Colorado School of Mines, Golden, CO, USA; E-mail: spei@mines.edu
[3] George T. Abell Professor in Infrastructure, Civil and Environmental Engineering, Colorado State University, Fort Collins, CO, USA. E-mail: jwv@engr.colostate.edu
[4] Research Manager, Advanced Building Systems, FPInnovations, Vancouver, BC, Canada
E-mail: erol.karacabeyli@fpinnovations.ca

response, which dissipates energy and increases the building period. In the 2010 edition of the National Building Code of Canada (NBCC 2010), the elastic seismic load is reduced by two types of R-factors, Ro-factor which is related to the over-strength of the system and Rd-factor that is related to the ductility of the structure (Mitchel et. al. 2003). In the major model codes in the United States, the International Building Code (IBC) and the ASCE 7 (ASCE 7, 2010), there is only one R-factor, called the response modification coefficient to reduce the seismic design force. Eurocode 8 (EN 1998-1:2004), the European seismic model code, also uses only one factor (q-factor) for reduction of the seismic design force.

Efforts have already been made to quantify the q-factor for CLT structures in Eurocode 8 using incremental non-linear dynamic analyses on analytical models of three-storey structures verified by testing of components. As there is no specific methodology for determining the q-factor in Europe, two approaches are common in the timber engineering community: the acceleration-based approach (Ceccotti and Sandhaas 2010), and the base shear approach (Pozza et al. 2009). In the acceleration-based approach the q-factor was calculated as a ratio of the peak acceleration of an earthquake record that causes 'near collapse" condition in the structure and the design acceleration in the code for the location for which the building was designed (Ceccotti et al. 2006, Ceccotti 2008, Schädle & Blaß 2010). In the base shear approach the q-factor was calculated as the ratio of the base shear force obtained from linear elastic analysis and the base shear force at "near collapse" state of the non-linear analysis for every input ground motion (Pozza et al. 2009). This method also takes into consideration the influence of the input ground motion on the elastic response of the structure. Using the acceleration-based approach (Ceccotti 2008) the average q-factor was found to be q=3.4 while q=3.8 was reported in Pozza et al. 2009. According to the base shear approach the average q-factor was found to be q=3.15 (Pozza et al. 2009, 2013).

An initial estimate for the R-factors in North America was conducted using the AC130 equivalency criteria (Popovski & Karacabeyli 2011). According to the criteria, assigning an R-factor for a new wood shearwall assembly can be made by showing equivalency of its seismic performance in terms of maximum load, ductility, and storey drift (all obtained from quasi-static cyclic tests), with respect to corresponding properties of wood-frame nailed shearwalls that are already in the code. Based on the experimental tests conducted at FPInnovations (Popovski et al. 2011, 2012), it was found that although not every single CLT wall configuration satisfied the response parameters as defined in AC130, the average values for the set of CLT walls did satisfy the AC130 criteria. Consequently, one may assume that the CLT walls tested can share the same seismic modification factors with regular wood-frame shearwalls in the US, which means using an R-factor of 6.5. This corresponds to having the product of RdRo equal to 5.1 in Canada (Rd=3.0; Ro=1.7) which are the factors used in NBCC for nailed wood-frame shearwalls. However, at this early stage of acceptance in the design practice, the authors recommended that a more conservative set of factors (Rd=2.0; Ro=1.5) be used for CLT structures with ductile nails or screws and hold-downs. It was also recommended that further studies such as the analyses presented in this paper and analyses according to the FEMA P-695 procedure (FEMA 2009) be considered. As a result, a comprehensive study on force reduction factors for the ASCE-7 has been initiated in the US (Amini et. al. 2013).

OBJECTIVES AND APPROACH

The research presented in this paper contributes further to determining the R-factors for seismic design of CLT structures. The objective of the research presented in this paper is to: (a) develop and verify analytical models for prediction of the seismic response of CLT walls as main lateral load resisting elements in CLT structures, (b) apply the verified models in a design of a mid-rise CLT residential building, and (c) recommend the appropriate Rd-factors for seismic design of CLT mid-rise buildings in NBCC. The NEESWood wood-frame building, also called Capstone Building (Fig. 1), was used as a reference building for this study.

The Capstone wood-frame building was tested on the E-Defense shaking table facility in Miki, Japan. During the tests, the building satisfied all performance targets imposed during the design, with only non-structural damage present even at Maximum Credible Earthquake (MCE) level of shaking with probability of exceedance of 2% in 50 years (Pei et al. 2010). For the purposes of this study the

wood-frame Capstone building was redesigned as a CLT structure and was used as a typical mid-rise CLT structure in all analyses. The results from the quasi-static tests on CLT walls performed at FPInnovations (Popovski et al. 2011) were used as input information for design and modeling of the CLT walls, the main lateral load resisting elements of the CLT structure.

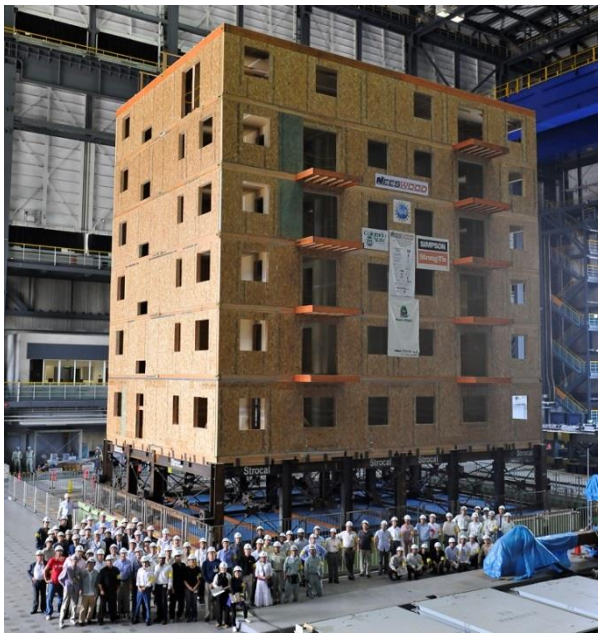

Figure 1. The Capstone wood-frame building tested during the NEESWood Project (Photo courtesy of Simpson String-Tie)

CLT WALL MODELING

Modelling of the CLT walls as main lateral load resisting elements of the structure was done using a kinematics model developed in Matlab (Fig. 2a). The model was calibrated for various input parameters such as size of the walls, gravity load level, number and type of brackets and location, number of hold-downs, number of step-joints (if present), number of fasteners per-bracket, etc., based on the CLT wall test data (Popovski et al. 2011), and using the basic kinematics formula shown in Equation (1).

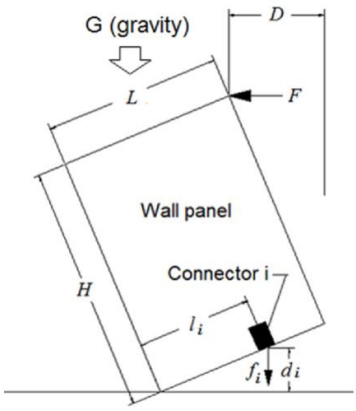

Figure 2. Basic kinematics used for developing of the simplified CLT wall models

$$F(D) = \sum_{i=1}^{n} \frac{l_i}{H} f_i(d_i) + \frac{L}{2H} G = \sum_{i=1}^{n} \frac{l_i}{H} f_i\left(\frac{l_i}{H} D\right) + \frac{L}{2H} G \qquad (1)$$

The fastener behaviour in the model was represented using the ten-parameter CUREE model, which is widely used for wood based shear wall and connection modelling. Example of the tested and the calibrated model hysteretic response of a 2.3 m long CLT wall, which uses brackets with eighteen 16d spiral nails per bracket and has step-joint with 8 4x70 mm (SFS1) screws in the middle, is shown in Fig. 3. A total of 19 different models were developed for CLT walls (assemblies) with 2 different bracket types and hold-downs, with each of the 10 parameters calibrated for every model (Pei et al. 2013a, b).

With the numerical model and connector parameters calibrated, the hysteresis curve for any given CLT wall configuration with different connectors can be estimated numerically using the kinematics (Matlab) model. Consequently, backbone curves and lateral load design values for various CLT walls using three different fasteners in the brackets were developed. Since at this point no design loads exist for CLT walls in Canada, the design levels were developed by dividing the ultimate load obtained from the hysteresis loop (by the Matlab model) by a factor of 2.5, thus obtaining approximately the same level of safety as with regular wood-frame walls.

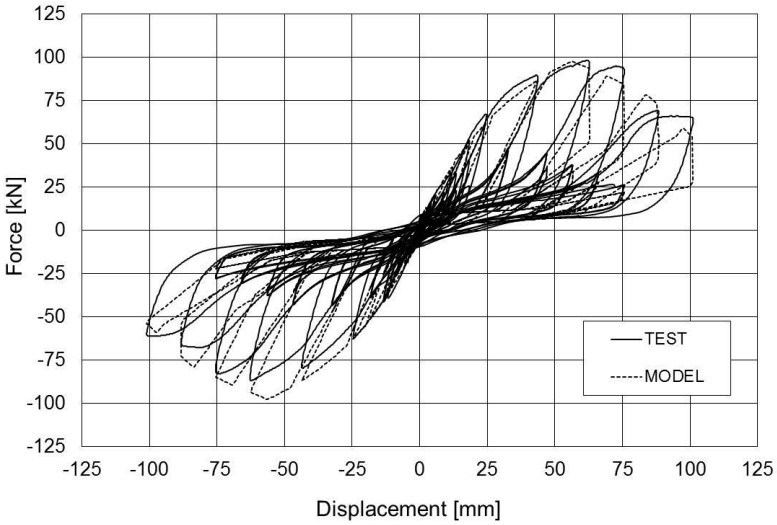

Figure 3. An example of tested and modelled (calibrated) response of 2.3 m long CLT wall with 16d spiral nails in the brackets and a step-joint in the middle

In North America a typical design practice for CLT walls at this point is to assume that entire shear force along the wall is taken by the bracket connections, while the hold-downs are placed for vertical continuity and to take the wall uplift. In other words, the contribution of the hold-downs is ignored when determining the shear capacities for the wall panels. It was recognized from experimental tests, however, that due to rocking response of CLT walls, hold-downs also contribute to the shear wall strength. Consequently, the lateral resistances for CLT walls were derived with the two options considered with and without the hold-down contribution taken into account. The capacities derived without hold-down contribution are therefore conservative compared to the walls actually installed in practice that contain hold-downs.

Some of the configurations for which analytical models and design values were derived are shown in Fig. 4. The notation "**S**" stands for **S**ingle sided brackets for each location, "**DE**" stands for **D**ouble sided brackets at the **E**nd of the panel only, and "**DA**" stands for **D**ouble **A**ll, meaning all brackets are double sided. For a case of a CLT wall with 2 brackets only, configurations **DE** and **DA** are identical.

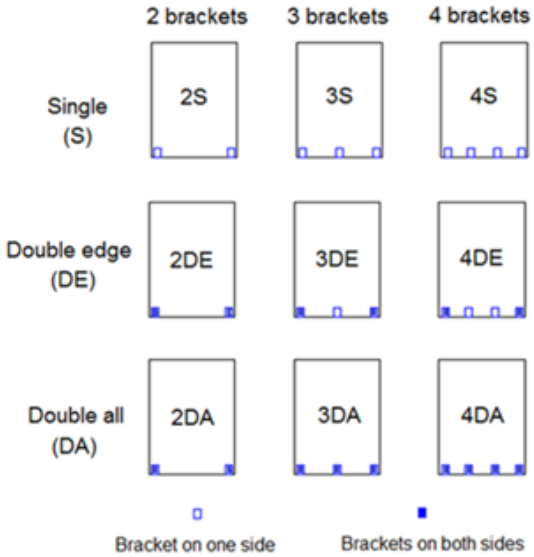

Figure 4. CLT panel configurations for deriving design values

CALIBRATION OF THE Rd-FACTOR FOR THE CLT BUILDING

To calibrate the appropriate ductility related force modification factor (Rd-factor) for the National Building Code of Canada (NBCC), the value for the over-strength related modification factor Ro that is mostly related to the over-strength, was chosen to be constant (Ro=1.5) throughout this process. This value is the same with the other heavy timber systems in NBCC. As mentioned the 6-storey Capstone CLT building was used as a reference structure for developing the different building models used in the analyses. The seismic demand for the CLT models (structures) was determined according to the Equivalent Static Force Procedure (ESFP) given in 2010 NBCC for 8 different values of the Rd factor (Rd=1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 5.0, and 6.0). It was assumed that the buildings were located in Vancouver, BC (maximum spectral acceleration of 0.96g at T=0.2s) with a design spectral acceleration Sa=0.743g at the building fundamental period of 0.4s. According to NBCC requirements, buildings located on firm soil with Rd ≥ 1.5 shall not be designed for a seismic force that is greater than the 2/3 of the force at T=0.2s. This force cut-off value for the design base shear was found to govern all designs, meaning that the buildings were actually designed for force equivalent to Sa=0.64g.

With the seismic demand determined for each storey, CLT walls were selected for each storey to satisfy the demands. Since design values for CLT walls with 3 types of connectors were developed (16d spiral nails, 4x70 mm SFS1 screws, and 5x90 mm SFS2 screws), for each of the two hold-down design considerations (with and without hold-downs considered), a total of 6 building designs were developed for each chosen Rd-factor. Since eight different Rd factors were considered, a total of 48 CLT Capstone building designs were generated. SAPWood models for every building design configuration were developed. The building models were developed to reflect the realistic as-built system, which includes the impact of gravity load and presence of the hold-downs. On the other hand, the resistances were generated without hold-downs considered in the resistance which will lead to conservative results. FEMA P-695 (FEMA 2009) suggested earthquake records were used as input ground motions in the non-linear dynamic analyses. Because all FEMA P-695 ground motions are biaxial, first the stronger ground motion component of each pair was scaled at the building natural period (Ta=0.4s) to match the Sa=0.743g from the Vancouver design spectrum. The other component was then scaled with the same scale factor so that the PGA ratio between two components was not altered. Fig. 5 shows the response spectra for the un-scaled and scaled ground motions.

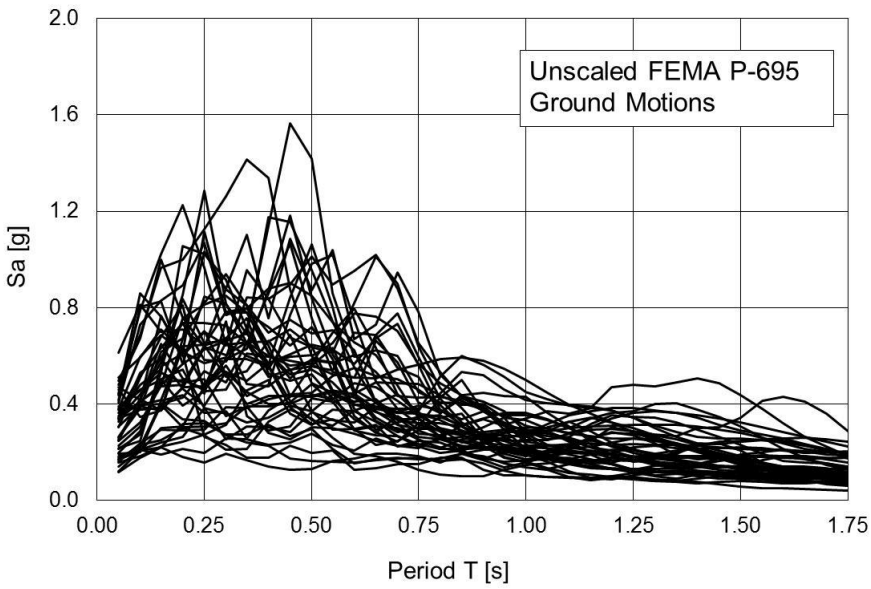

Figure 5. Response spectra for the unscaled FEMA P-695 earthquake ground motions

A series of 44 bi-axial nonlinear time history analyses was conducted for each of the 48 building designs (models) developed. For each analysis, the absolute maximum inter-storey drift from the building non-linear dynamic response at any storey and in any direction was recorded and rank-ordered. The distribution of these maximum drift values represents the performance of each particular design under the chosen hazard for site as Vancouver, BC. Examples of the cumulative distribution functions (CDFs) for the maximum inter-storey drifts for building configurations with 16d nails in the brackets and different Rd-factors are shown in Figures 6 and 7.

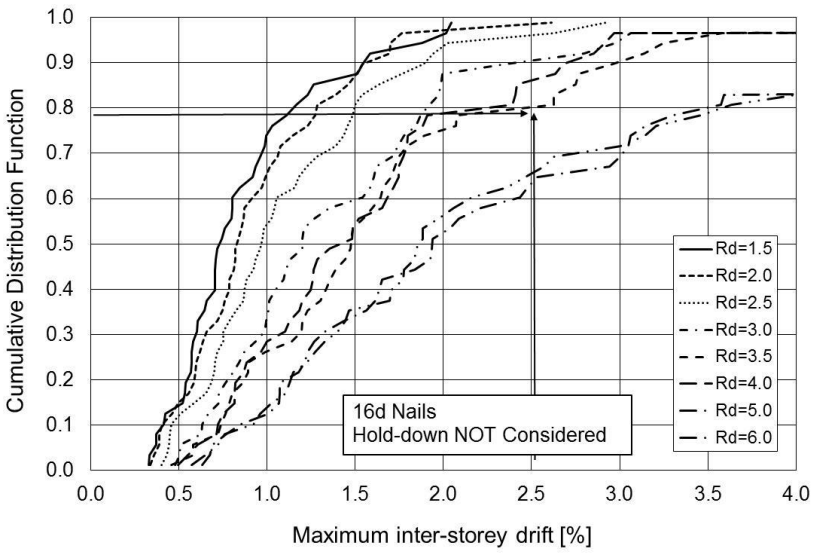

Figure 6. Fragility (CDF) curves for the building for various Rd-factors for case of 16d spiral nails in the brackets and the hold-down influence not included in the wall resistance values

Based on the performance of different building designs (different connectors in the brackets and Rd factors), a comprehensive evaluation of the appropriate Rd factors for achieving a prescribed performance target can be made. For example, if the acceptable performance level for the buildings is assumed as not to exceed 2.5% inter-storey drift in 80% of the cases (80% probability of non-exceedance), one just need to find the performance point corresponding to 2.5% drift on the X-axis

and 0.8 CDF value on the Y-axis of the plots. All the curves above that performance point will be able to satisfy the criteria. The Rd factor that corresponds to the curve that just satisfies the performance will be the most appropriate value for that performance objective. Based on this procedure, the Rd factors that satisfy several different performance objectives are shown in Table 1. The minimum values for Rd factors for each performance target are given in bold font. It should be noted that not all of the performance objectives need to be satisfied at the same time. It will be up to the design engineer, the jurisdiction of interest or the code committees to decide which performance level should dictate the Rd value used for the seismic design.

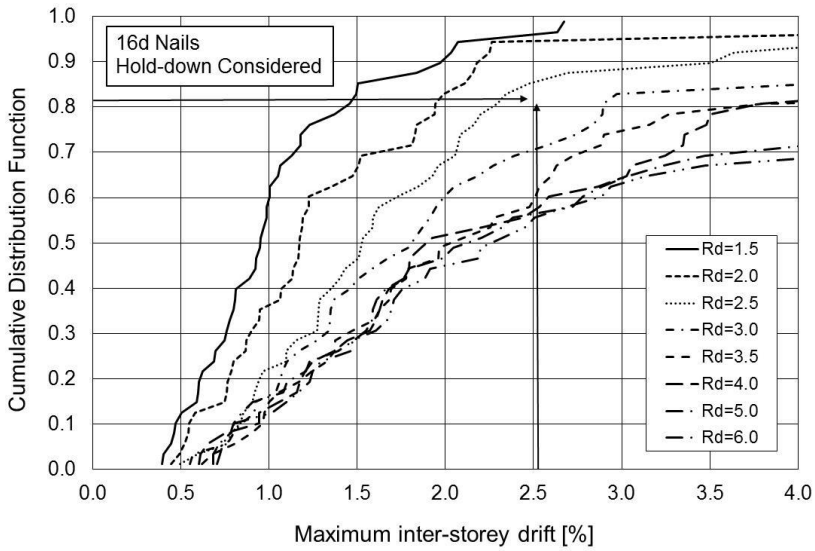

Figure 7. Fragility (CDF) curves for the building for various Rd-factors for case of 16d spiral nails in the brackets and the hold-down influence included in the wall resistance values

Table 1. Calibrated Rd factors for the CLT buildings for the 2010 NBCC equivalent static procedure

Performance targets in terms of storey drifts and probabilities of non-exceedance (PNE)	Hold-downs **NOT accounted** in resistance values for CLT walls			Hold-downs **accounted** in resistance values for CLT walls		
	16d nails 3.9x89mm	4x70m Screws	5x90mm Screws	16d nails 3.9x89mm	4x70mm Screws	5x90mm Screws
1.5% drift & 50% PNE	**4.0**	4.0	4.0	**2.5**	3.0	2.5
2.0% & 80% PNE	3.5	3.5	**3.0**	**2.0**	2.25	2.0
2.5% & 80% PNE	**3.5**	4.5	4.0	2.75	2.75	**2.5**
4.0% & 80% PNE	6.0	5.0	**4.0**	4.0	3.0	**2.5**

It can be seen from Table 1 that for each performance level the calibrated Rd values are smaller in cases where the hold-down capacity was included in the CLT wall resistance than in the cases where it was neglected. This is logical since neglecting hold-down contribution automatically builds in extra safety level in the design. At this point when the methods for deriving design values for CLT walls are being developed and are not agreed upon, it is prudent to take the more conservative approach for determining the calibrated Rd factor for CLT structures. If the objective of the design is to effectively control the damage and prevent collapse (2.5% storey drift) during a 2% in 50 years event, it is recommended that an Rd factor of 2.0 is used for all connectors considered for the buildings analysed. Based on the test results on the single wall components as well on the tests on 3-D CLT structures conducted to date (Ceccotti 2008), the 2.5% inter-storey drift is achievable in CLT structures without inducing excessive building damage.

In continuation of the work, analytical models were developed for a 10-storey and 15-storey building with the same floor plans. The buildings were designed using the same procedure as for the 6-storey ones, and were subjected to same set of earthquake motions scaled at their own periods. Based on the results from the 6-storey buildings and to reduce the number of analyses, analytical models for buildings designed with Rd-factor up to 4.0 were considered. Also, since no significant difference was observed among the fasteners, the models were built for buildings that utilize 16d nails in the brackets only. The cumulative distribution functions (CDFs) for the maximum inter-storey drifts for 10 and 15 storey buildings with 16d nails in the brackets and different Rd-factors are shown in Figures 8 and 9, respectively.

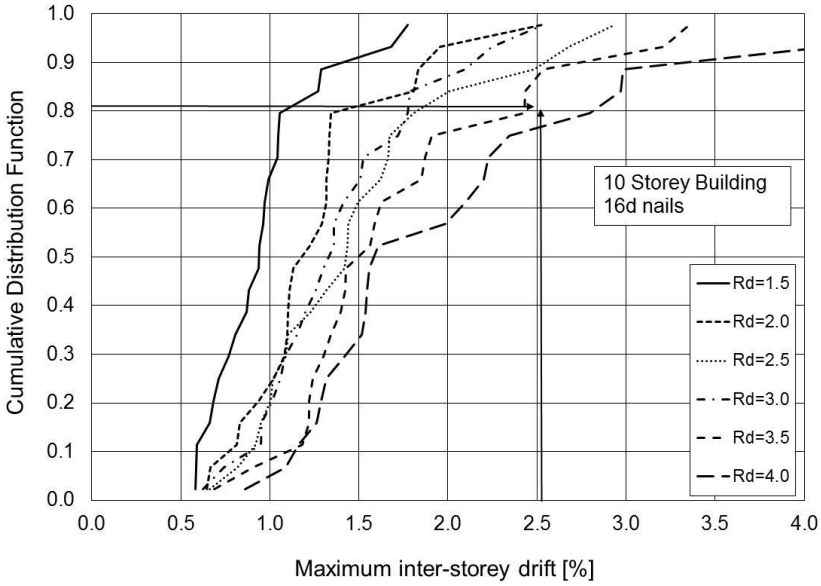

Figure 8. Fragility (CDF) curves for the 10 storey building for various Rd-factors for case of 16d spiral nails in the brackets and the hold-down influence included in the wall resistance values

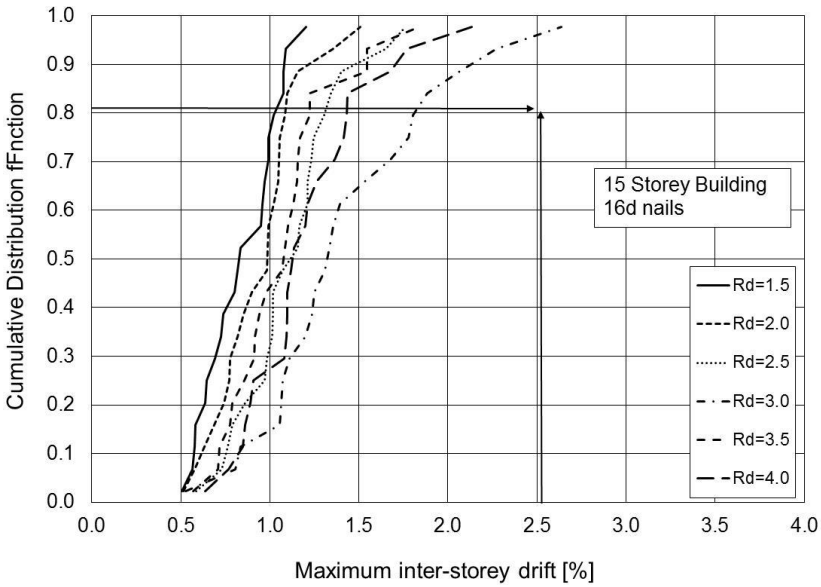

Figure 9. Fragility (CDF) curves for the 15 storey building for various Rd-factors for case of 16d spiral nails in the brackets and the hold-down influence included in the wall resistance values

As shown in the Figures 8 and 9, for a design objective of having 2.5% storey drift during a 2% in 50 years event, the appropriate Rd-factor for design of the 10-storey building can go as high as 3.5, while Rd=4.0 can be used for the 15-storey building. It should be noted that 2nd order effects were not included in the analyses and their inclusion may have some impact on the results of the taller

buildings. A summary of Rd-factors that satisfy several different performance objectives for CLT buildings with different heights is presented in Table 2.

Table 2. Calibrated Rd factors for the CLT buildings with different heights

Performance Target in Terms of Storey Drift and PNE	6-storey	10-storey	15-storey
1.5% drift with 50% PNE	2.5	3.5	4.0
2.0% drift with 80% PNE	**2.0**	3.0	4.0
2.5% drift with 80% PNE	2.5	3.5	4.0
4.0% drift with 80% PNE	2.5	4.0	4.0

It should also be noted that changes in the building configuration (unsymmetrical floor plans), the hazard characteristics of the location, and the influence of the boundary conditions (effects of the perpendicular walls and floor slabs), will have an influence on the calibrated Rd factors shown in Tables 1 and 2. However, such influences may either not make significant changes to the values of the Rd factors suggested or may add additional conservatism to the values (in case of the boundary conditions). Therefore the Rd values presented here are felt to be reasonable estimates for symmetrical CLT buildings located in high seismic region in Canada. The conservative values of Rd=2.0 and Ro=1.5 will be proposed to the NBCC Standing Committee on Earthquake Design for acceptance of CLT as a structural system in the upcoming edition of NBCC.

CONCLUSIONS

Based on the analyses conducted in this study, CLT as a structural system is a viable option for mid-rise buildings in moderate and high seismic regions. When adequately designed, CLT structures with symmetrical plans will have only limited damage under MCE earthquakes. By selecting appropriate R-factors, the Equivalent Static Design Procedures can meet the performance objectives selected. Although the type of fasteners in the brackets of the CLT walls considered in this study has effect on the R-factors, the impact was not as significant as expected for drifts up to 2.5%. The design methodology that is used for determining the lateral resistance for CLT walls was found to have more significant impact on the R-factors. For that reason, both cases, when CLT wall design resistances include and exclude the influence of the hold-downs, were used in this study. At this time when methods for deriving design values for CLT shearwalls are in the development stage and not agreed upon, it is prudent to take a more conservative approach by taking into account the influence of the hold-downs in deriving CLT wall design values. In such case, the recommended values for R-factors in Canada would be Rd=2.0 and Ro=1.5.

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