FORCE MODIFICATION FACTORS AND CAPACITY DESIGN PROCEDURES FOR BRACED TIMBER FRAMES

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ABSTRACT

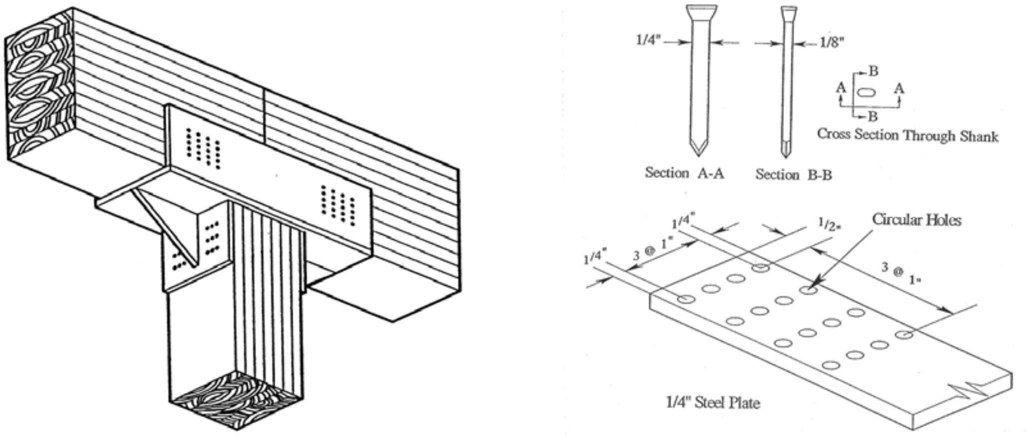
The work presented in this paper is part of a research project conducted at FPInnovations Forintek with objective to quantify the seismic behaviour of braced timber frames with timber riveted connections. Results are presented from monotonic and cyclic tests conducted on diagonal brace members with riveted connections on both ends. Based on the results from the cyclic tests, non-linear analytical models were developed for diagonal brace members as well as for the entire braced frames. Using the DRAIN 2DX computer program, a series of non-linear static and time history dynamic analyses were performed. The acceleration records used for the analyses were chosen to satisfy the seismic zonal parameters for Vancouver, BC, Canada. Parameters that influence the seismic behaviour of braced timber frames are discussed, including estimates of the newly introduced ductility-based force modification factors (R_d factors) in the 2005 edition of the National Building Code of Canada (NBCC). Finally, the paper provides some guidelines on implementation of capacity design procedures for braced timber frames.

KEYWORDS: Braced timber frames, Timber rivets, Capacity design, Seismic modification factors, Timber rivet connections

1. INTRODUCTION

Braced frames are very often the simplest and most economical structural systems used to resist lateral loads in timber construction. In concentrically braced frames it is assumed that there is no eccentricity in the joints and the lateral forces are resisted by almost pure axial loads in the braces. The seismic response of a braced timber structure involves many different interacting factors, which need to be understood and quantified. One of the most important considerations is to provide a system able to absorb relatively large amounts of energy and thus lower the earthquake-induced forces, while maintaining adequate stiffness to avoid excessive deformations. To satisfy these requirements, the seismic design process should include a careful balance of strength, stiffness and ductility. In braced timber frames significant deflection of the frame is dependent on the deformations of the connections at both ends of the braces. The stiffer nature of braced frames represents an advantage in the case of low to medium intensity earthquakes. Less induced deformation translates into less damage to non-structural elements and the serviceability requirements are easily met. On the other hand, it also means less potential for energy absorption, leading to higher forces and lower system redundancy (Popovski et al., 1999).

Timber rivets, are high-strength steel nails developed originally in Canada for glued-laminated timber construction. Timber riveted joints are typically used on truss, purlin-to-beam, beam-to-column (Figure 1a), column-to-base or column to diagonal brace connections, or as base connections for arches. Timber rivets are especially suited to field fabrication where plates may be attached to members on the ground before erection. Timber rivets have an oval shank and a wedge-shaped head. The hot-dip galvanized rivet that has an ultimate tensile strength of at least 1,000 MPa, is driven through a pre-drilled mild steel side plate until the tapered head deforms the hole and wedges tightly (Figure 1b). The wedging action provides a certain degree of fixity and allows the rivet to behave as a cantilevered beam on an elastic foundation (Karacabeyli et al., 1998). Timber rivets have proven to be very efficient connectors for the transfer of large static loads in heavy timber construction (Popovski et. al. 2002; Popovski and Karacabeyli, 2004).



a) b)

Figure 1. a) Column-to-beam riveted connection; b) Parts of a typical riveted connection

2. QUASI-STATIC TESTS ON DIAGONAL BRACES WITH TIMBER RIVETS

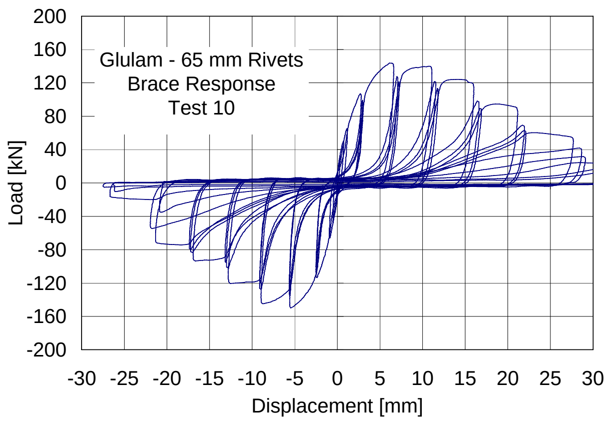
Since the seismic response of braced timber frames largely depends on the brace connections, the main objective of the experimental program was to characterize the behavior and failure modes of diagonal braces with riveted connections subjected to monotonic and cyclic loading.

2.1 Materials and Methods

Displacement controlled monotonic tension and cyclic tests were conducted on a total of 48 brace specimens with four different wood products, Spruce-Pine (SP) Glulam, Laminated Veneer Lumber (LVL), Parallel Strand Lumber (PSL) and Laminated Strand Lumber (LSL). The diagonal brace members consisted of a main wood member and double-sided riveted connections on both ends. Each connection utilized 6.4 mm steel side plates and 20 rivets (4 rows of 5) on each side of the wood member, for a total of 40 rivets on each end of the brace. Two lengths of rivets were used in the research program, 40 mm and 65 mm. After fabrication, riveted connections were conditioned in a dry laboratory environment for a minimum of three weeks to allow for the relaxation of wood fibres around the rivets. The spacing between rivets was 25 mm in all directions while the end distance was 75mm. Three brace specimens of each configuration were tested in monotonic tension tests, while five replicates were tested using the ISO cyclic testing protocol (ISO, 2003). The brace was connected to a bolted fixture at the top and bottom. The top of the brace was also attached to the load cell and the servo-controlled actuator. In addition, two rotational hinges (pins), one at the top and one at the bottom, were introduced to minimize the influence of secondary bending moments and ensure almost pure axial state of loading for the specimens.

2.2 Results and Discussion

During the monotonic tension tests, glulam riveted connections yielded in a ductile single shear mode. The early behaviour was almost completely governed by yielding of the fastener, while the failure mode was characterized by partial fastener pullout from the wood. It was observed that the top and bottom brace connections experienced significantly different deformations. Once non-linear deformations started to develop in one of the connections, the reduced stiffness of that particular connection would result in an increase of the deformation demand. This connection will be referred to as the weaker connection of the brace. Regardless of which connection in a brace is weaker or stronger, this finding is very important for understanding the seismic behaviour of braced timber frames. It shows that the deformation capacity of a brace is not equal to the capacities of both connections. This fact will be used later when developing the analytical models for braced timber frames.



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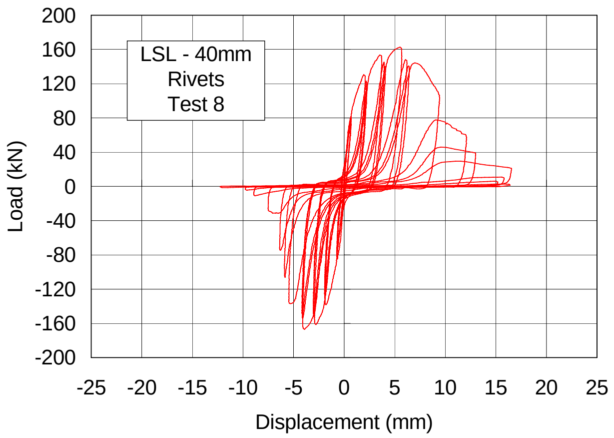


b)

Figure 2. Hysteresis loops from cyclic tests on diagonal braces with riveted connections in a) Glulam; b) LSL

Typical load-deformation relationships of the brace members in glulam with 65 mm long rivets and LSL with 40 mm long rivets obtained from cyclic tests are shown in Figures 2a and 2b respectively. As evident, significant pinching of the hysteresis curves had occurred. This is a very common feature for connections in timber structures and is a result of the irrecoverable crushing of the wood that leaves a gap at load reversals. During subsequent excursions through this gap, lateral resistance and energy dissipation occurs almost entirely in the metal connectors. The first loop in a cycle of three therefore is the widest and shows the highest resistance, while subsequent cycles are narrower and typically achieve lower resistance for a given displacement. Usually, this degradation of strength stabilizes after three cycles and the third cycle is therefore often considered to represent the actual connection (frame) resistance where reversible loading is expected. In riveted connections the pinching effect was most significant at higher deformation levels, while the hysteresis loops were thicker at lower deformation levels. Since the area inside the hysteresis loop for each cycle represents the amount of energy dissipated during that cycle, pinching in braced timber frames indicates a reduction in the hysteretic damping of the structure. However, the pinching is not the single most important parameter for adequate seismic behaviour of timber structures. The ability of the structure (connection) to sustain large deformations without significant strength deterioration is also very significant (Popovski, 2000). That is exactly the behaviour that riveted connections exhibited during the cyclic tests. They showed very ductile behaviour and were able to carry a significant portion of the load even at high deformation levels. Main properties of the riveted brace connections determined from the first envelope for the weaker connections with 40 mm and 65 mm rivets are presented in Table 1.

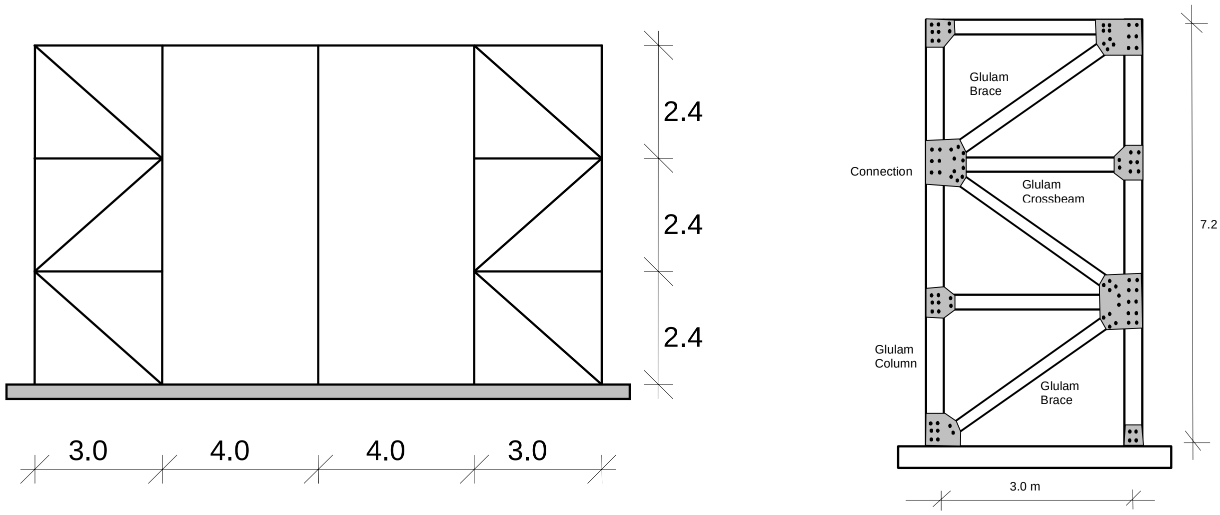
Table 1. Average properties from the first cycle envelope of the weaker brace connections

	40 mm Rivets				65 mm Rivets	
	Glulam	PSL	LVL	LSL	Glulam	PSL
Yield load F_y (kN)	56.5	47.5	64.2	87.1	80.5	73.2
Yield displacement Δ_y (mm)	0.5	0.3	0.51	0.5	1.0	0.9
Maximum load F_{max} (kN)	112.5	99.3	127.2	155.8	144.0	135.0
Displacement at F_{max} (mm)	2.7	3.6	3.3	3.3	4.1	5.1
Ultimate deformation Δ_u (mm)	7.9	7.0	7.7	5.1	9.6	11.5
Initial stiffness (kN/mm)	144.6	163.0	152.9	221.4	98.4	102.2
Ductility (Δ_u / Δ_y)	16.2	19.9	15.0	10.8	9.4	12.7
Maximum load per rivet (kN)	2.8	2.5	3.2	3.9	3.6	3.4
St. Dev. for F_{max} (kN)	10.6	3.3	2.7	25.2	9.9	8.9
COV for F_{max} (%)	9.4	3.3	2.1	16.2	6.8	6.6

3. FORCE MODIFICATION FACTORS FOR BARCED TIMBER FRAMES

3.1 The Force Modification Factors in the 1995 and the 2005 NBCC

The force modification factors (R-factors) in building codes in North America account for the capability of the structure to absorb energy, and thereby reduce the seismic design forces. They also take into account the existence of alternate load paths, over-strength and redundancy of the structural system. Different R-factors are assigned to different types of structural systems in building codes reflecting their design, construction experience, and seismic performance during past earthquakes. Often there is little theoretical or experimental background for the numerical values of the R-factors given in codes. Consequently, the process of assignment of R-factors requires considerable individual judgment. In the 2005 edition of the National Building Code of Canada (NBCC, 2005), the elastic load is reduced by two types of R-factors, an 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. For braced timber frames with ductile connections, for example, the R_d-factor is 2.0, while the R_o-factor is 1.5. The product of both R-factors that can be refereed to as a combined force modification factor (one that reduces the elastic design seismic force) in this case is equal to 3.0. The previous 1995 edition of NBCC had only one R-factor that was related to the ductility of the system and a calibration *U* factor, the reciprocal number of which could have been interpreted as an over-strength factor with a value of 1.0/0.6 = 1.67. In this case the combined R-factor that reduces the elastic seismic force for ductile braced frames was 3.34. Other important changes in the 2005 NBCC with respect to the 1995 NBCC include the use of 2 % in 50 years seismic hazard maps (vs. 10% in 50 years previously) that changed the seismic design loads for most locations in Canada; the use of the design spectral accelerations at different periods (instead of velocity based coefficient *v*); and different formulas to calculate the period of the structure, that do not include the width of the structural system, but only its height.



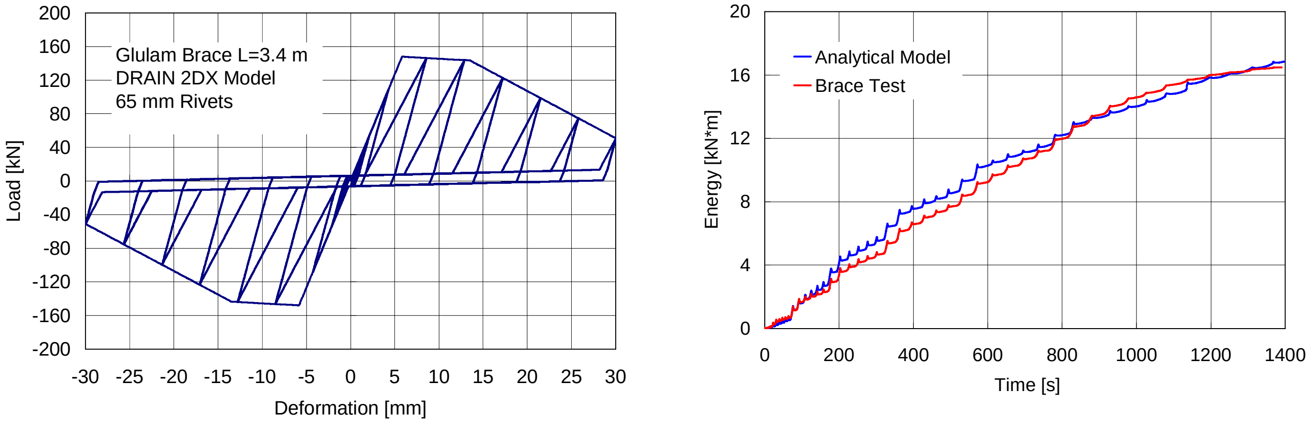
a) b)

Figure 3. a) Elevation of typical industrial building used for the analyses; b) Simplified braced frame model

3.2 The Analytical Methodology

Because of the changes in the 2005 NBCC, the intention of the research presented in this section was to validate the ductility-based force modification factor R_d for braced timber frames in the 2005 NBCC, using a simplified non-linear dynamic analysis procedure. A hypothetical structure representing an industrial or commercial type of building was chosen as basic model for the analyses (Figure 3a). Dimensions of the frame other than those of the braced frames alone are given for orientation purposes only. The lateral load is generally resisted by a discrete number of braced frames, the placement and number of which was determined by the lateral load generated by the tributary roof mass. To simplify the structure for computer analysis, only the concentrically braced frame was analyzed, representing the main lateral load resisting system of the building (Figure 3b). The frame was assumed to be constructed of 20f-EX Spruce-Pine (SP) glued-laminated timber. The cross section and the modulus of elasticity for the braces was chosen to be the same as those for the braces tested during the experimental program (an average MOE of 10,783 MPa).

Based on the results obtained from the cyclic tests on diagonal braces, non-linear models for the braces were developed using the “Florence” model (Ceccotti & Vignoli, 1989). The model subroutine was incorporated in the DRAIN-2DX computer package for two-dimensional non-linear analysis of building structures (Powell, 1993). The model reproduces the path of a typical hysteresis loop using nine parameters. Figure 4a shows the analytical response of a brace modeled with the Florence model subjected to same displacement history as that used in the testing (Figure 2a). Calibration of the brace model in each case was done not only in terms of strength and stiffness, but also in terms of energy dissipation. Model parameters were chosen so that the total hysteretic energy (area within the hysteresis loops) calculated from the analytical response of the brace model matches the hysteretic energy dissipated during the cyclic tests (Figure 4b). The developed brace models were than used in the DRAIN-2DX computer program when developing the analytical model for the entire braced frame. In the model (Figure 3b) the mass of the entire structure was concentrated at the roof level, equally distributed between the upper two nodes in the model. All brace and crossbeam connections in the model were assumed pinned, as were the base support connections. The non-linearity of the model was concentrated in the braces, while the vertical columns and the horizontal beams were assumed to remain linear elastic. The parameter that was varied for the analyses was the mass at the top, or in other words the tributary roof area for the braced frame.



a) b)

Figure 4. a) Analytical response of a diagonal brace obtained using the Florence model; b) Comparison of energy dissipation in the tested and modeled diagonal brace with riveted connections

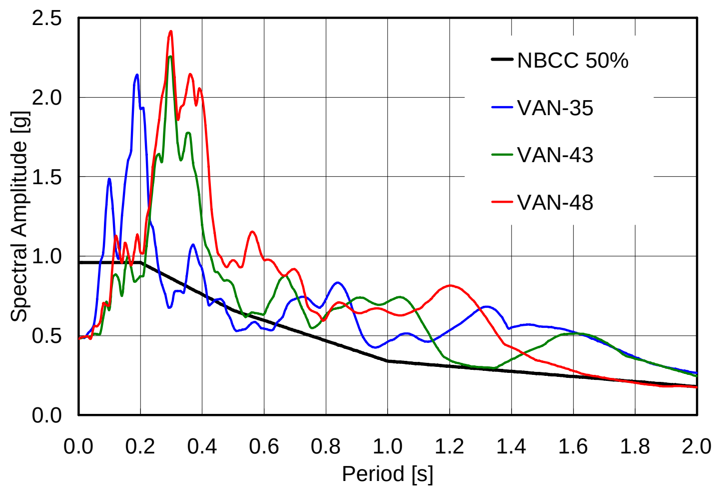


Figure 5. Pseudo acceleration response spectra for three records used in the analyses

To evaluate the R_d-factor a series of non-linear dynamic time history analyses were conducted using 12 different acceleration records from previous earthquakes around the world. Records were chosen to satisfy the seismic parameters for a locality such as Vancouver according to the 2005 NBCC, with peak horizontal ground acceleration of 0.46 g. The earthquake records were denoted as VAN-04, VAN-06, VAN-21, VAN-26, VAN-29, VAN-35, VAN-43, VAN-48, VAN-51, VAN-63, VAN-65 and VAN-72. The pseudo acceleration response spectra for three of these records for five percent damping, compared to the Vancouver design spectrum are shown in Figure 5. Besides satisfying the NBCC requirements for acceleration, the chosen records had different frequency characteristics throughout the spectrum.

3.3 Results and Discussion

Results from the non-linear dynamic analyses can be summarized in the graph shown in Figure 6. The maximum displacement in the diagonal braces was chosen as a basis for structural performance evaluation. Variation of the R_d-factor, as mentioned earlier, was achieved by changing the roof tributary area (and thus the mass) of the model. The graph represents the deformation demand (Y-axis) of the bottom brace of the braced frame designed with a certain R_d-factor according to NBCC (X-axis), for each of the 12 different earthquakes. The thick horizontal line represents the ultimate deformation capacity of the brace, determined as the deformation at which the load dropped to 80% of the maximum in the descending part of the envelope curve.

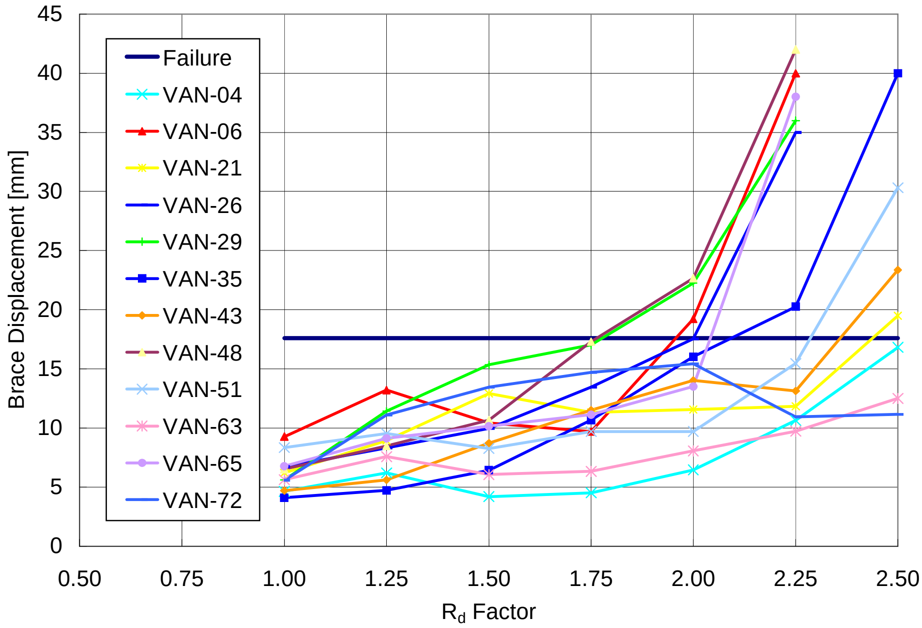


Figure 6. Deformation demands and R_d factors for the analysed braced timber frame with riveted connections

An appropriate value for the R_d-factor can be obtained based on the results shown in Figure 6 and assuming an acceptable level of seismic risk. As shown in Figure 6, the braced frame was able to “survive” all earthquakes if designed with an R_d-factor of 1.75, while the demand of three earthquakes VAN-06, VAN-29, and VAN-48 is higher than the failure deformation, if the structure were designed with an R_d-factor of 2.0. The value of 2.0 for the R_d-factor would be acceptable if one were to change the seismic risk criteria from no probability of failure to allowing the structure to have a 25% probability of failure (out of 12 earthquakes). Similarly, one may argue that although widely used to quantify failure of wood connections and assemblies, the failure displacement of the brace (defined as deformation of the brace at which the load drops to the 80% level), doesn’t actually cause a failure of the entire structure. Consequently if the failure criterion is set as deformation of the brace at which the load drops to 50% level (which is 25 mm), the braced frame designed with an R_d-factor of 2.0 will be able to survive all earthquakes. Based on the results from the analyses, it seems that an R_d-factor of 2.0 is appropriate in the analyzed case, especially if we have in mind the following assumptions that went into the modeling: (a) The seismic loads in the example were calculated based on the calculated period of the structure according to 2005

NBCC equation (T=0.18s), while the period of the modeled structure was found to be from 0.38s to 0.6s, depending on the R-factor used for the design; (b) The efficiency of the assumed design was 100% which is rarely obtained in the practice. It should be noted that these findings apply only to the case of the structure analysed and the methodology used. For a more robust approach, probably the methodology that will be developed as a part of the ATC-63 project may be used. Although no dynamic analyses were performed on braced frames with PSL and LVL, based on the performance of the single braces with riveted connections during the testing, braced frames in PSL and LVL may also be assigned an R_d-factor of 2.0 when used with timber rivets designed in rivet yielding mode.

4. CAPACITY-BASED DESIGN OF BRACED TIMBER FRAMES

The concept of capacity design is of major importance in seismic design. This design approach is based on the simple understanding of the way a structure sustains large deformations under severe earthquakes. By choosing certain modes of deformation, we can ensure that the brittle elements have the capacity to remain intact, while inelastic deformations occur in selected ductile elements. These "dissipative zones" act as dampers to control the force level in the structure. In steel structures the members are typically designed to yield before the connections. Beam failure mechanisms are preferred since they provide sufficient structural ductility without creating undesirable mechanism of collapse. In timber structures, however, the failure of wood members in tension or bending is not favourable because of its brittle characteristics. Consequently, all non-linear deformations and energy dissipation in case of concentrically braced timber frames should occur in the connections. The dissipative zones should be located in the connections connecting the braces to the rest of the frame. They should be able to produce yielding by combination of wood crushing and fastener bending. All other connections shall be designed to remain linear elastic, with a strength that is slightly higher than the force induced on each of them when neighbouring dissipative zones reach their probable strength.

Detailing in case of braced timber frames can be done in two different ways, with and without a gap that is left between the diagonal braces and the rest of the frame (in the corners). The presence of a gap allows for the brace connections (dissipative zones) to deform and reach their ductility capacity. This is how the braced frame was modeled in the analytical study, and how the single braces were tested. When there is no gap between the end of the brace and the frame corners (a case of tight fit corners), the braced frame will be much stiffer, would probably be able to carry larger lateral loads, but also may probably be less ductile. A new research in this field is underway at FPInnovations-Forintek and results will be published when completed.

Eccentricities in all connections of a braced frame, and especially in the dissipative zones, should be minimized. All wood members should be designed to remain linear elastic at all times. The columns should be continuous and of constant cross section over the height of the structure, and should be able to carry the vertical load at all deflection levels, including the maximum allowable lateral drift. Wood diagonal braces should be designed not to buckle at any time. Structural integrity of the frame shall be maintained at all times while dissipative zones experience inelastic behaviour. Narrow braced frames, with aspect ratio (story height vs. frame width) higher than 1.0, should be avoided because of their cantilever (bending) type seismic response (Popovski, 2000). Wider frames have been shown to exhibit more of a shear type response and make better use of the braces and their connections. However, wide frames with aspect ratios lower than 0.67 should be avoided because the benefits of having a wider frame are usually outweighed by the drawbacks of having a long brace, susceptible to buckling at lower force levels.

5. CONCLUSIONS

The paper summarizes some of the results from a research project aimed to quantify the ductility-based force modification factor for braced timber frames with riveted connections. Because energy absorption capacity and overall ductility of braced frames is typically influenced by the connections used, adequate connection design is of particular importance when these frames are used in high risk earthquake zones. Timber riveted connections in rivet yielding mode showed good seismic performance. During quasi-static and cyclic tests, riveted connections in four different wood products were capable of resisting many load reversals without significant strength deterioration. In addition, large displacements were attained before failure, which permits warning before any potential structural failure.

The study has shown that during a seismic event the two connections in a single brace of a braced frame do not experience same deformation levels. Due to numerous factors, including the variability of the wood properties, one of the brace connections starts to experience higher initial deformations and wood crushing, which results in concentration of the deformation demand in that connection later in the response. This finding was important for understanding the seismic behaviour of braced timber frames, pointing out that the deformation capacity of a brace is not equal to twice the capacity of one connection. The study showed that braced timber frames with riveted connections designed in rivet yielding mode built in SP glulam can be assigned an R_d-factor of 2.0, in recognition of their high and consistent ductility capacity. It should be noted that these findings apply only to the case of the structure analysed and the methodology used. The results of it are also applicable to the limited sample sizes tested in the study. For a more robust approach, probably the methodology that will be developed as a part of the ATC-63 project may be used. Further research is needed, however, to study the effects of corner detailing, and response of X-braced timber frames.

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