

Dynamic response of braced timber frames

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ABSTRACT

Braced timber frames are often used as a lateral load resisting system in buildings where large open spaces are required. In concentrically braced frames, there is essentially no eccentricity in the joints and the lateral forces are resisted by almost pure axial forces in the braces. For application in high-risk earthquake zones, however, the ductility of the system is a concern, since energy absorption is typically limited to the connection region. Static and cyclic tests were conducted on a variety of connections with different diameter bolts or high strength glulam rivets that are typically used in braced timber frames. Shaking table tests were also conducted on single storey models of braced frames. The resulting hysteresis curves obtained from tests were used in non-linear static and dynamic analyses to determine the response of braced frames to the input of different earthquakes. From these analyses, it was possible to determine the influence of different connection details on the seismic response of the frames. Based on the results, an estimate was made on the appropriate force reduction factors for earthquake resistant design of braced timber frames.

INTRODUCTION

Timber structures are generally recognized to perform well when subjected to strong earthquakes. This is attributed largely to the fact that traditional timber systems have a high degree of redundancy as well as sufficient ductility and energy absorption capacity. Favourable system geometry, e.g. symmetrical plan and elevation, also contributed to the desirable performance. Regardless of the demonstrated satisfactory performance, however, it was shown that timber construction in itself is not a guarantee for adequate structural performance during an earthquake. This was particularly evident during some of the recent strong earthquakes such as Northridge, California (1994) and especially Kobe, Japan (1995).

Braced frames are very often the simplest and most economical structural systems used to resist lateral loads in timber construction. In concentrically braced frames, there is essentially 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 in general is a complex issue, involving 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 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 must include a careful balance of strength, stiffness and ductility.

QUASI-STATIC CONNECTION TESTS

Based on results from previous studies (Yasamura 1990), it was evident that further research is needed in the field of seismic resistance of braced timber frames. A final stage of research project on braced timber frames is currently underway at the University of British Columbia (UBC) in collaboration with researchers at Forintek Canada Corp. The main objective of the project is to develop an understanding and determine the factors that influence the seismic behaviour of braced timber frames with different connection details through obtaining corresponding force reduction factors, as well as to provide designers and code officials with relevant information about the design details needed in support of those factors. It was expected that the seismic response of braced timber frames would largely depend on the connections, so a significant part of the experimental program was focused on the behaviour of different connection details. Displacement controlled monotonic and cyclic tests were conducted on different connections to determine their adequacy in resisting reversing loads such as experienced in seismic events. Two different connector types, namely mild steel bolts and glulam rivets (also known as griplam nails) were considered. Glulam rivets are high-strength oval-shaped nails with a tapered head and maximum shank diameter of 6.4 mm (1/4 in). The flat cross-section of the shank prevents splitting of the wood while the tapered head, after being driven into a 6.4 mm (1/4 in) round hole in the steel

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plate, provides fixity. This also ensures the formation of a plastic hinge at the head of the rivet under large deformations.

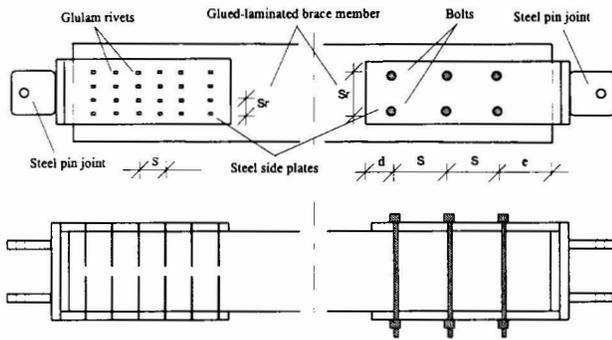


Figure 1 Typical details of riveted and bolted braced frame connection.

The test specimens were chosen to represent typical connection details used in braced timber frames (Fig 1), consisting of wood brace member, connectors (bolts or rivets) and steel plates on both sides. The brace member consisted of grade C, Spruce-Pine-Fir (SPF) glued-laminated timber, 130 x 152 mm (5 x 6 in) in cross section. Steel plates for bolted connections were 12.7 mm (1/2 in) thick, while riveted connections were built using 6.4 mm (1/4 in) plates. Mild steel ASTM A307 bolts were used with three different diameters: 9.5 mm (3/8 in), 12.7 mm (1/2 in) and 19 mm (3/4 in). The member thickness to bolt diameter ratio (bolt slenderness l/d) was 13.3, 10.0 and 6.7 respectively. For the riveted connections, 65 mm (2.5 in) long rivets were used. Three replicates were tested from each connection in both static and cyclic tests. The configuration of the connections tested is presented in Table 1.

Table 1. Connections used in quasi-static tests.

Fastener type Test group	9.5 mm Bolts		12.7 mm Bolts		19 mm Bolts	Rivets	
	I	II	I	II	I	I	II
Total connectors	10	3	6	2	2	48*	18*
Rows	2	1	2	1	1	4	3
Connectors in row	5	3	3	2	2	6	3
End distance (mm)	114		152		228	75	
Spacing (mm)	40		51		76	25	
Design load (kN)	64.4	24.2	67.6	28.1	56.9	90.7	34.0

* Number of rivets on both sides, i.e. half of that number is on one side.

Selected connections were divided into two test groups, based on their design load (factored resistance). Reduced design levels for the second test group had to be used due to limited load capacity of the shake table, used for dynamic tests on braced frame models later in the project. The end distance e was 12 times the bolt diameter ($12d$), which is larger than the minimum suggested distance of $10d$ according to the Canadian Timber Design Code (CSA-086.1). The spacing s between bolts in the row was $4d$, which is the minimum requirement. The spacing between rivets in the riveted connections was 25 mm (1 in) in all directions, while the end distance e was 75 mm (3 in).

Results and discussion

Two types of displacement controlled quasi-static tests were conducted: monotonic and cyclic. Properties such as initial stiffness, ultimate load, yield load, ultimate displacement and ductility were determined from these tests based on the recommendations in the European CEN standard. The connections in both monotonic and cyclic tests always failed in tension. The glulam rivets and bolts yielded in a ductile single shear and double shear mode respectively. It was found, however, that yield modes and failure modes for the same connection type could vary considerably. For glulam riveted connections, the early behaviour was almost completely governed by yielding of the fasteners, while the failure mode was fastener pullout from the wood. In the case of bolted connections, a double-shear-yielding mode was noticed with extensive wood crushing, but the failure mode was always splitting of the wood member. Splitting always occurred at the maximum load; however, most of the connections were able to carry a significant percentage of the maximum load for some time after splitting had occurred. Typical load-deformation relationships of the bolted (19 mm) and riveted connection from the first test group are shown in Figure 2.

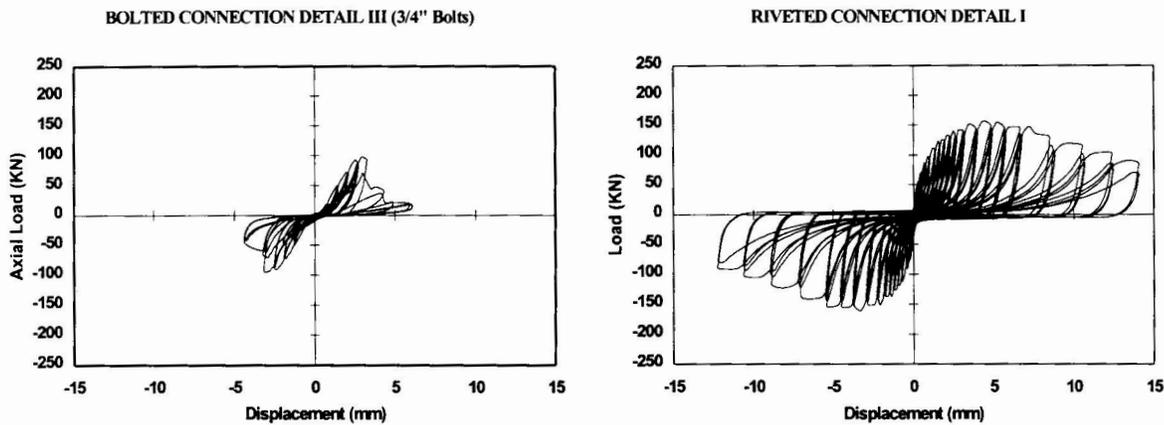


Figure 2. Typical hysteresis loops obtained from cyclic connection tests from the first test group.

It is evident that significant pinching of the hysteresis curves occurred, which is common for connections in timber structures. The pinching effect results in thinner loops in the middle compared to near the ends, which is typical for hysteresis loops where connector yielding and wood crushing occurs, forming a gap during the process. Since the area inside the hysteresis loop for each cycle represents the amount of energy dissipated during that cycle, pinching in timber braced frames reduces the hysteretic damping of the structure. In bolted connections with lower slenderness ratios (3/4 in bolts) rigidity of the bolts lead to more uniformly distributed bearing stresses in the wood member, which resulted in abrupt wood splitting and sudden loss of bearing capacity. Connections with higher slenderness ratios (smaller diameter bolts) exhibited more desirable behaviour, although wood splitting was consistently the failure mechanism for all bolted connections tested.

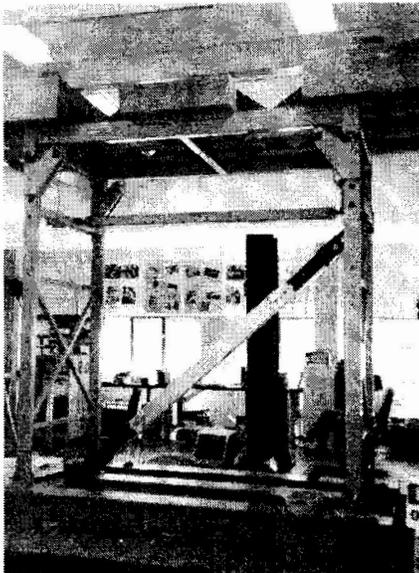


Figure 3. Single brace shake table setup.

displacements were typically possible before failure, which permits ample warning before structural failure.

SINGLE BRACE SHAKE TABLE TESTS

In the second phase of the experimental part, single brace shake table tests were conducted on a number of test specimens. The tests were conducted in the Earthquake Engineering Laboratory in the Department of Civil

Engineering, University of British Columbia. The main objectives of this part were to determine the influence of dynamic rate of loading on connection behaviour and compare the behaviour and properties with results from quasi-static tests, while verifying the analytical models developed on the basis of static tests.

A steel frame previously built for shake table tests of wooden shear walls was used as a basic test setup. Several modifications to the frame were done to enable it to accommodate wood brace members. The four-pinned steel frame that was mounted on a 3 x 3 m shake table (Figure 3) was designed to provide vertical support for an inertial mass on the top. The frame itself has no lateral resistance, so the only member that provides resistance to the generated inertial forces is the wooden brace. The axial load in the brace was measured directly from the installed load cell at the top end of the brace, while the connection slip was obtained from the displacement transducers mounted at both connections. To ensure a pure axial load state in the brace, pinned connections were introduced at the top and at the bottom of the brace. The frame was loaded with a total mass of 4,500 kg. (10,000 lbs.) on the top. Although the tested structure consisted of two materials, the whole test frame could be treated as a simple single story timber braced frame for two reasons. Firstly, braced timber frames are designed to resist the lateral loads only, while the vertical loads are resisted by some other system. Secondly, only the braces are expected to exhibit non-linear behaviour, while all other members are typically designed to remain in the linear elastic range. Regardless of the material of all the other elements, the non-linear behaviour of the brace would govern the system response.

Results and discussion

A total of eight braces (frames) were tested, utilizing four of the connections that were previously tested quasi-statically, i.e. all three from the second test group plus the 19 mm bolted connection. The Joshua Tree Station acceleration record, E-W direction, from the 1992 Landers, California earthquake, was used as the input ground motion scaled to different peak acceleration levels. Hysteresis loops obtained from cyclic and shake table tests on the same riveted connection from the second test group, are shown in Figure 4.

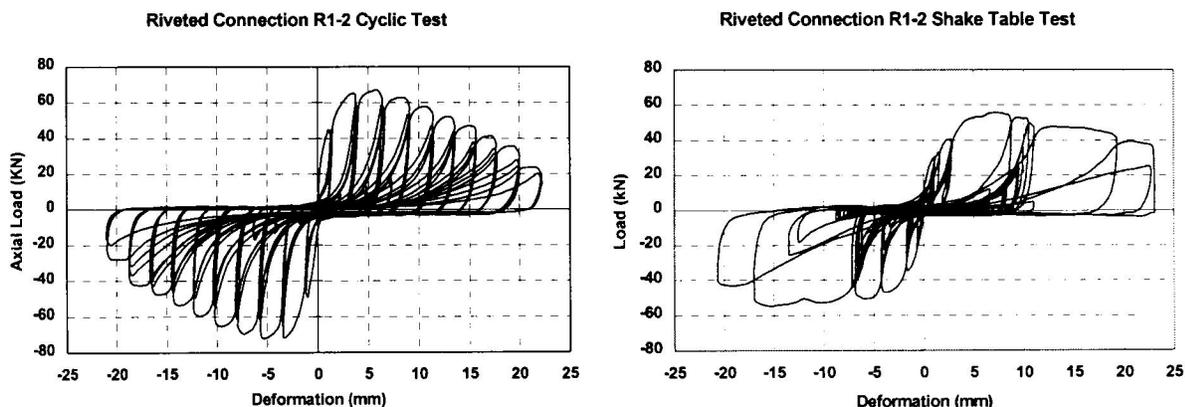


Figure 4. Comparison of cyclic versus shake table results for riveted connection detail.

The shake table test results showed that the average maximum load for all connections was from 10 to 13 % lower than the corresponding load obtained from the quasi-static cyclic tests. Although this is within the expected variability for wood structures, this finding is in contradiction with the fact that wood as a material and thus the wood joints have enhanced strength under short-term loading. It appears that obtaining lower loads in shake table tests might be due to other reasons such as loosening and impact effects during the shaking. In addition, a significant vertical component of motion noticed during the tests might have contributed as well. Since only two replicates from each connection were tested, however, no statistically supported conclusions can be drawn at this point. The initial stiffness for bolted connections obtained from shake table tests was found to be higher than the corresponding one obtained from cyclic tests. The opposite was found to hold for riveted connections. It was also noticed that the ductility for all bolted connections was higher for shake table tests compared to cyclic tests. For riveted connections again, the opposite was found. During the shake table tests, more deformation and energy dissipation was experienced in one of the connections (usually the upper one), while the other one remained less excited. The average energy dissipation of bolted connections with smaller diameter bolts was larger than in connections with larger diameter bolts, at the same

earthquake input level. As expected, riveted connections experienced the highest amount of energy dissipation of all connections. It was also noticed that the total energy dissipation in each connection during shake table tests was lower than the corresponding total energy dissipated during cyclic tests.

ANALYTICAL STUDIES

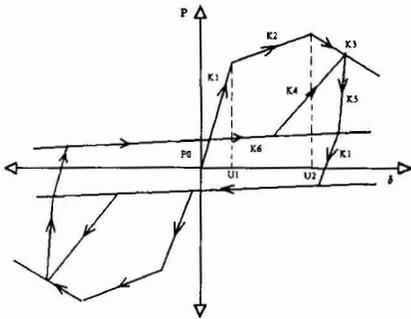


Figure 5. Analytical model for connection behaviour used in analyses.

Based on the results obtained from the cyclic and shaking table tests, non-linear mathematical models for all tested connections were defined. A non-linear model, developed by Dr. Ario Ceccotti and his collaborators from The University of Florence, Italy, was found to be most suitable for modeling the connection behaviour. The model (Figure 5) is defined by a total of nine parameters including six stiffness parameters (K1 to K6), two displacement parameters (U1 and U2), and one force parameter (Fo).

The mathematical model was incorporated into the well-known computer program DRAIN 2DX (Powel, 1993) for non-linear static and dynamic analysis of structures. Modeling of the connection behaviour in each case was done not only in terms of strength and stiffness, but also in terms of energy dissipation. The mathematical connection models were incorporated in a non-linear analytical model of a whole braced timber frame.

Using DRAIN 2DX, a series of non-linear static and time history dynamic analyses were performed to determine the seismic performance of this type of structural system.

A simple braced timber frame model, that represents the lateral load resisting system of a laboratory or warehouse type structure, was chosen as a basic model for analysis (Fig. 6). All connections in the model were assumed to be pinned, with horizontal and vertical members assumed to be linearly elastic. The mass of the structure was assumed concentrated at the roof level only. Dynamic time history analyses were conducted using five different acceleration records from previous earthquakes around the world. Records were chosen to satisfy the seismic zonal parameters from The National Building Code of Canada (NBCC) for a locality such as Vancouver, with peak horizontal ground velocity of 0.21 m/sec and peak horizontal ground acceleration of 0.23g, with probability of exceedance of 10% in 50 years. They also represent earthquakes with different frequency characteristics throughout the spectrum. The damping value used in analyses was 3% of critical damping for all cases.

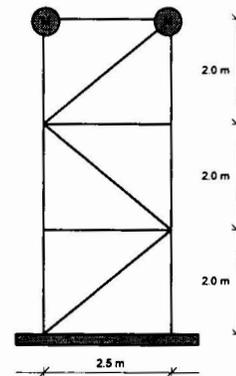


Figure 6 - Analytical braced frame model.

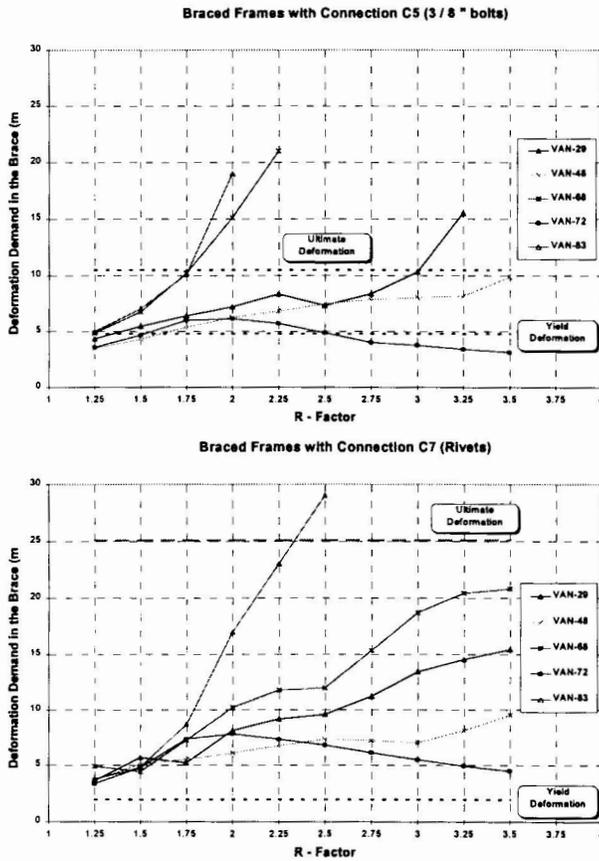


Figure 7. Deformation demand on braced timber frames with bolted and riveted connections.

The main emphasis of the analyses was on evaluating the force modification factor (R-factor) in NBCC. For

braced timber frames, when built with “ductile connections”, the assigned R-factor is 2.0, while other braced timber frames are assigned an R-factor of 1.5. Results from the non-linear dynamic analyses can be summarized in graphs such as those shown in Figure 7. Each graph presents analysis results for a frame that utilizes a certain connection detail. For example, Figure 7 shows results for frames that have connections with 9.5 mm bolts (3/8 in) and glulam rivets both from the second testing group. Each line on the graph represents the deformation demand (Y-axis) of the lowest brace of a braced frame structure designed with a certain R-factor (X-axis) according to NBCC, for each of the five different (VAN) earthquakes expected on a seismic site such as Vancouver. Two horizontal dotted lines represent the yield deformation and the ultimate deformation capacity of the brace, respectively. Because braced timber frames as a system typically have a low degree of redundancy, failure of the structure was assumed when the deformation demand of one or more earthquakes exceeded the deformation capacity of the structure.

As shown in the figure, the braced frame model with 9.5 mm (3/8 in) bolted connections would survive all five earthquakes if designed with an R-factor lower than 1.75. On the other hand, the same frame when built with riveted connections, could survive all the earthquakes if designed with an R-factor lower than 2.25. Similar analyses were done for frames with all other connections from both test groups. Values for R-factors obtained from analyses of the braced frames with all seven different connection details from both test groups are summarized in Table 2. Two different values of R-factors are presented for bolted connections, based on the failure criteria used. According to the CEN Standard, the ultimate displacement capacity is determined at a point where the load drops to an 80% level of the maximum load. In bolted connections splitting always occurred at the maximum load so this point can also be defined as ultimate displacement.

Table 2. Maximum R-factors for braced timber frames with different connections.

Fastener	9.5 mm Bolts		12.7mm Bolts		19 mm Bolts		Rivets	
	I	II	I	II	I	II	I	II
Test group								
R-factor (at 80 %)	2.0	1.75	2.0	2.0	1.5		2.0	2.25
R-factor (max load)	1.75	1.5	1.75	1.75	1.5		-	-

CONCLUDING REMARKS

From the results, it is evident that the seismic response of braced timber frames is heavily influenced by the behaviour of the connections. It is suggested that braced frames with different connections should be assigned different R-factors. Braced timber frames with mild steel (ASTM A-307) bolted connections, with slenderness ratios (l/d) of 10 or higher showed far more adequate seismic performance than those frames that utilized bolts with lower slenderness ratios. Further research is needed, however, to study the effects of other parameters such as end distance, spacing, number of rows, number of bolts in a row etc., for the development of general recommendations that will ensure ductile behaviour of braced frames with bolted connections. Until such research is undertaken, an R-factor of 1.5 appears to be reasonable for braced timber frames with slender bolts. On the other hand, glulam riveted connections showed promising results when used for braced timber frames. Braced frames with glulam riveted connections designed in rivet yielding mode may be assigned an R factor of 2.0, in recognition of their higher and more consistent ductility capacity.

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