SEISMIC ASSESSMENT OF A TIMBER-STEEL STRUCTURE USING HYBRID SIMULATION

S. Miller¹, J. Woods¹, J. Erochko², and D. Lau³

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The system level performance of a seven-story combined timber-steel structure with a steel friction bracing device is validated through pseudo-dynamic hybrid testing. In the substructured hybrid simulation, the first story of the structure is physically tested in the laboratory while the rest of the structure is modeled in OpenSees. Using the hybrid simulation method, an incremental dynamic analysis is conducted and fragility curves are derived to assess the seismic performance of the prototype structure at various performance levels, including operational, design, and maximum credible hazard levels. The incremental analysis is also performed separately using a computer model without hybrid simulation. The results of the hybrid simulation are shown to be in good agreement with purely analytical simulations and the results of the IDA demonstrate that the seven-story combined timber-steel structure with steel friction braces satisfies performance limits specified in the National Building Code of Canada.

Introduction

The use of heavy timber in medium-rise to high-rise construction in the United States and Canada has increased significantly over the past decade. The demand for tall wood structures has led to the need to improve the earthquake resistance of conventional heavy timber structural systems. Traditional high-rise wood construction is typically relies on ductile wood or concrete shear walls as the primary seismic force resisting system (SFRS). The use of timber moment-resisting frames or braced frames is not common in tall wood structures because of the low force reduction factors (R-factor) that are specified in building codes for such systems. To overcome these challenges, an innovative structural system has been developed and tested at Carleton University that incorporates high-performance steel structural bracing systems into heavy timber frames. The proposed structural system includes structural steel at the beam-column interfaces to allow the incorporation of steel braces into a heavy timber structural system.

A previous experimental study of the combined timber-steel SFRS with a friction braces demonstrated excellent performance of a single-story scaled frame under cyclic, wind, and quasi-static earthquake loading [1]. Experimental results showed that the energy dissipation capacity was provided by the steel friction brace, while the surrounding timber-steel connections remained elastic. This study established the linear performance of the timber-steel connections, the nonlinear response of the friction brace, and the ability of the frame to satisfy building code requirements for strength and drift; however, they did not address the system-level performance of a complete structure with friction braces or the response of the system at varying seismic hazard intensities. To address these areas, the objective of the current study is to use hybrid testing to perform an incremental dynamic analysis (IDA) and derive fragility curves for the proposed seven-story timber-steel SFRS configuration. Results of the IDA demonstrate excellent seismic performance of the overall system under a range of ground motion intensities. The results improve the understanding of the behaviour of the timber-steel connections and friction brace under realistic seismic load.
Hybrid Timber-Steel Braced Frame System

The combined heavy timber-steel braced frame discussed in this paper was first developed and tested by Gilbert and Erochko (2016) [1]. The system combines traditional glue-laminated heavy timber elements with steel panel zone joints at critical beam-column connections. Figure 1 shows a typical beam-column interface in the timber-steel braced frame. The glulam and steel components are fastened together using glued-in steel rod connections. These connections are designed to transfer the forces to the timber elements in the strong parallel-to-grain direction. The combined timber-steel connection design allows the beam-column connections to transfer the high forces from a traditional steel brace or any type of high performance steel brace into the columns and beams without causing splitting or damage in the timber (e.g. buckling-restrained brace, self-centering energy dissipative braces, and friction brakes). Additional information on this type of structural system can be found in [2].

Figure 1. Beam-column interface in timber-steel braced frame.

Prototype Structure

The prototype heavy timber building for the hybrid simulation is a seven-story four-bay braced frame located in Victoria, British Columbia, Canada. The building is designed according to the National Building Code of Canada (NBCC) [3], the wood members are designed using Canadian Timber Design Manual (CSA O-86-14) [4], and the steel elements of the connection are designed according to the Canadian Design Standard for Steel Structures (CSA S16-09) [5]. Due to the absence of a Canadian approved design method for glued-in-rod connections, the pull-out resistance of these fasteners were evaluated using three different European design procedures. The pull-out resistances were determined from the German Design Code (DIN 1052) [6], GIRod Project Design Procedure (2002) [7], and the LICONS Report Design Procedure (2003) [8]. Additionally, experimental pull-out tests were conducted to verify design strength calculations, results for which are available in [2]. The braced frame uses the connection detail shown in Figure 1 to form a combined timber-steel SFRS. Figure 2 shows the floor plan and elevation view for the prototype structure and Table 1 shows the final design member sizes of the heavy timber beams, columns, and cross-laminated (CLT) slab, and the glued-in-rod connections. The beams and columns are Douglas-Fir-Larch glue laminated timber, while the slab consists of 7-ply cross-laminated timber (CLT) panels (245 mm thick) spanning 6.5 m.
Table 1. Prototype structure member sizes.

<table>
<thead>
<tr>
<th>Floor</th>
<th>Column size (mm)</th>
<th>Column GIR details</th>
<th>Beam size / GIR details</th>
<th>Floor slab</th>
<th>Brace Size</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-2</td>
<td>465x465 D-Fir-L 16c-E</td>
<td>16 – 19 mm Steel B7 Rods</td>
<td>265 x 494 mm D.Fir-L 24f-E</td>
<td>245 mm 7-ply CLT SPF No.1/No.2</td>
<td>HSS 152x203x13</td>
</tr>
<tr>
<td>3-4</td>
<td>342x342 D-Fir-L 16c-E</td>
<td>9 – 19 mm Steel B7 Rods</td>
<td>10 – 19 mm Steel B7 Rods</td>
<td>infeldl</td>
<td></td>
</tr>
<tr>
<td>5-7</td>
<td>265x265 D-Fir-L 16c-E</td>
<td>4 – 19 mm Steel B7 Rods</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*GIR = glued-in-rod connection to steel assemblies.

The lateral load resistance is provided using a brass-stainless steel friction bracing system. A friction damping device was selected as the optimal bracing element because it is able to replicate the probable forces and displacement demands of an advanced steel bracing system consistently over a number of tests, allowing the same physical setup in the lab to be tested repeatedly. The friction brace is designed to slip at a specific load, at which point the friction plates slide relative to one another, dissipating seismic energy and protecting surrounding structural elements. Figure 3 shows the theoretical hysteresis of the friction device and the observed hysteretic performance of the friction brace in the laboratory. The friction slip force is calibrated using 167 N-m of torque on each of four steel bolts which provide the normal force to the friction interface. The timber-steel connections and glulam beams and columns are capacity designed relative to the friction brace slip force to prevent damage. Additional details on the design of the prototype structure and the capacity based design approach for the connections and timber members can be found in [2].
Hybrid Simulation

The use of hybrid simulation provides a cost-effective and accurate method to capture the overall behaviour of full-scale structures. In the hybrid testing technique, the portions of a structure that are expected to experience significant nonlinear behaviour are tested in the lab, while the rest of the structure is modelled numerically in a finite element program. The process of splitting the structure into experimental and numerical components is referred to as substructuring. The portion of the structure that is tested in the lab is referred to as the experimental/physical substructure, while the remaining portion of the building that is modelled in a finite element software is referred to as the numerical/analytical substructure. The benefits of hybrid simulation stems from its ability to accurately incorporate the nonlinear behaviour of structural elements that are difficult to model analytically by physically testing only those elements in the laboratory, reducing the need for expensive full-scale tests.

The first story of the prototype braced frame is selected as the physical substructure. Figure 3a shows the physical and analytical substructures for the multi-story timber structure. In this configuration, one actuator is used in the lab to apply the lateral deformation to the first-story braced frame. OpenFresco is used as the interware in the hybrid simulation to connect the physical and analytical substructures together [9]. OpenFresco is responsible for performing the linear transformation between actuator and global degrees of freedom and sending commands and receiving feedback between the substructures. The experimental substructure is represented in OpenFresco using a number of beam column elements, whose initial stiffness are also provided according to the global degrees of freedom. Since the primary nonlinear behaviour in the frame comes from the braces, the physical substructure has been simplified by not including the axial load on the column. As a result, to maintain model stability and to maintain realistic rotations at the top of the first story column, an identical second column is modelled in the analytical model in the same location of the physical column. Future studies will include the effects of axial load on the seismic response of the timber-steel braced frame.

OpenSees is used to model the remaining six stories of the structure for the hybrid simulation [10]. The OpenSees finite element model of the heavy timber building is modelled in two-dimensions, considering only a single bay of the SFRS. Figure 4a shows a schematic of the
finite element model of the timber braced frame. The glulam beams and columns in the frame are assumed to remain linearly elastic and are modelled using elastic beam-column elements. A past study demonstrated that the glulam beam and columns in this type of frame remained elastic up to large story drifts of at least 4.4% [1]. The nonlinear friction braces are modelled using a 2-node link element, which is assigned a uniaxial material model in the direction longitudinal to the brace. The behaviour of the friction brace is modelled using the bilinear stress-strain relationship (Steel01) shown in Figure 4a, which is calibrated to meet the design target slip force of the friction brace. Rigid joint offsets are used to simulate the rigidity of the intermediate steel connection/panel zone. The CLT slabs at each level in the prototype structure are assumed to act as rigid diaphragms.

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**Figure 4.** (a) Physical and analytical substructures of the hybrid model; (b) Physical substructure in the laboratory; (c) overview of experimental test setup.
Figures 4b and 4c show the experimental test setup for the heavy timber frame hybrid simulation. The test is conducted on a one-half scale frame specimen. The base of the glulam column is attached to a rigid support using a true-pin connection to represent an ideal boundary condition. Out-of-plane supports prevent any out-of-plane translation or rotation of the glulam column and beam elements. An actuator face mount is used to ensure that the load is applied to the frame in the in-plane direction.

Hybrid Simulation Results and Discussion

Hybrid simulations were conducted using a suite of eleven earthquake records. Each record is applied multiple times at gradually increasing acceleration magnitudes. In total, 165 hybrid simulations were performed on the prototype structure. Table A1 in the Appendix lists the ground motion records and Figure A1 shows their respective spectral acceleration. Note that the eleven records were selected for the IDA based on recommendations from the NBCC 2015, which are in line with recommendations from ASCE 7-10 [11,12]. The ground motion records were selected such that their median response spectra closely match the uniform hazard spectra for Victoria, British Columbia from NBCC 2015 [11].

Figure 5 shows sample hybrid test results for the second story displacement of the structure when subjected to the Landers (1992) earthquake record. Results from a purely analytical simulation of the structure (the same model but with the first story modelled instead of tested) are also shown on the same plot for comparison. Results from the hybrid test are in generally good agreement with the analytical predictions. The differences between the responses is attributed to the fact that this system has effectively zero post-yield stiffness, small variations in the model parameters (stiffness, mass, damping) can have a significant impact on the results for maximum displacement.

Incremental Dynamic Analysis and Fragility Assessment

Using the hybrid simulation setup discussed previously, an incremental dynamic analysis (IDA) is conducted to understand the seismic behaviour of the prototype structure throughout the full range of its dynamic response [13]. This is accomplished by applying the same ground motion record at
increasing levels of intensity until the structure reaches the maximum inter-story drift limit of 2.5% according to the NBCC (2015) [11]. In some cases, the response curve for an individual level will lead to collapse over a specific range of spectral accelerations, in which case the analysis is stopped at the initiation of collapse. Based on these results, collapse fragility curves based on maximum inter-story drift and maximum residual inter-story drift were derived to estimate the probability of collapse over a range of earthquake intensities.

Figure 6 shows the results of the IDA for three of the eleven records used in the hybrid tests in terms of both maximum inter-story drift and maximum residual inter-story drift. These results demonstrate the different observed behaviors for the timber braced frame, including weaving, hardening, and softening behaviors. Analyzing the IDA results, all curves exhibit a distinct elastic linear range that ends at a spectral acceleration of approximately 0.2g, when the
frame begins to behave nonlinearly. Figure 6a displays clear weaving behaviour, in which the structure weaves around the elastic slope, displaying successive segments of softening and hardening under different earthquake intensities. Figure 6b showcases a typical hardening behavior in which the structure initially follows the elastic slope and then hardens under larger ground motion intensity. Although this appears counter-intuitive, studies have found that this behavior occurs as a result of the pattern and timing of the record, as opposed to intensity [14]. For example, a strong ground motion may cause earlier slip of a friction brace at a single story relative to the same record at a lower intensity. This story then acts as a fuse relative to other stories in the structure. Finally, the softening behavior in Figure 6c occurs after initial slip of the friction brace followed by sharp softening of the structure, that accelerates towards large story drifts.

Comparing maximum inter-story drift and maximum residual inter-story drift results from the IDA, the results show that there is a large variability depending on not only the earthquake record but also the intensity of the earthquake. For example, in Figure 6b, large maximum drifts at spectral accelerations between 0.5g and 0.75g produced small inter-story residual drifts, while at larger spectral acceleration values the maximum inter-story residual drift was much closer to the maximum inter-story drift. In Figure 6c, the results show consistently that the maximum inter-story drift and maximum inter-story residual drift were very similar. Much of the variability in these results can be attributed to the fact that the system has virtually zero post-yield stiffness.

Using the results from the IDA using hybrid testing, fragility curves were derived to assess the seismic performance of the proposed SFRS [14]. Figure 7 shows the fragility curves in terms of maximum inter-story drift and maximum residual drift for the prototype structure. Fragility curves were derived for different performance levels based on drift levels of 0.5%, 1%, 2%, and 2.5%, according to ASCE 7-10 [12]. These performance levels include collapse prevention (<2% drift), life safety (0.5% - 1.5%), and immediate occupancy (0.5%). The limit associated with the maximum credible earthquake (MCE) shown on the fragility curve plot for maximum inter-story drift is based on the spectral acceleration from the NBCC (2010) at the first mode period of the structure [3]. The first mode period of the structure in the hybrid test was 0.9 s. This resulted in a NBCC maximum spectral acceleration of 0.45 g. The design level earthquake (DBE) is assumed to be 2/3 of the maximum credible earthquake, resulting in a spectral acceleration of 0.3g.

![Fragility curves for (a) maximum inter-story drift and (b) inter-story residual drift.](image-url)
The fragility curves demonstrate excellent seismic performance of the structural system even at the MCE level hazard. The probability of reaching the life safety performance limit at the MCE hazard level is less than 15%, as shown in Figure 7. In terms of residual drift, the probability of exceeding a residual drift of 0.5%, the point beyond which the structure would generally not considered ready for immediate occupancy following an earthquake is approximately 65%, whereas the probability of exceeding a residual drift of 1% is less than 20% [12]. These results show the high seismic performance potential for the combined timber-steel braced frame with advanced structural bracing systems.

Conclusions

This study assessed the seismic performance of a combined timber-steel seven-story structure with friction braces using hybrid simulation. In the hybrid simulation substructuring approach, a first story beam, column, and friction brace were physically tested in the lab while the remaining six stories of the structure were modelled analytically in OpenSees. The hybrid simulation method was used to conduct an incremental dynamic analysis to derive fragility curves for the prototype structure in terms of maximum inter-story drift and maximum residual drift. In total, 165 earthquake hybrid simulations were carried out with no damage and minimal response deterioration, effectively demonstrating the resilience of the proposed SFRS. The results of the IDA and derived fragility curves show that the performance of the prototype structure met the MCE level performance limits from the NBCC while limiting residual drifts to within appropriate levels.

Acknowledgments

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Appendix

![Figure A1. Spectral acceleration for IDA ground motion suite.](image)
Table A1. IDA ground motion suite.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Eq. I.D.</th>
<th>Eq. Name &amp; Year</th>
<th>Station</th>
<th>PGA (g)</th>
<th>Duration (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>ATC-F02A</td>
<td>Northridge (1994)</td>
<td>Beverley Hills</td>
<td>0.34</td>
<td>25</td>
</tr>
<tr>
<td>2</td>
<td>ATC-F05A</td>
<td>Imperial Valley (1979)</td>
<td>Delta</td>
<td>0.46</td>
<td>105</td>
</tr>
<tr>
<td>3</td>
<td>ATC-F07A</td>
<td>Kobe (1995)</td>
<td>Nishi-Akashi</td>
<td>0.53</td>
<td>46</td>
</tr>
<tr>
<td>4</td>
<td>ATC-F09A</td>
<td>Kocaeli (1999)</td>
<td>Duzce</td>
<td>0.25</td>
<td>42</td>
</tr>
<tr>
<td>5</td>
<td>ATC-F11A</td>
<td>Landers (1992)</td>
<td>Yermo</td>
<td>0.24</td>
<td>49</td>
</tr>
<tr>
<td>6</td>
<td>ATC-F12A</td>
<td>Landers (1992)</td>
<td>Coolwater</td>
<td>0.48</td>
<td>33</td>
</tr>
<tr>
<td>7</td>
<td>ATC-F17A</td>
<td>Superstition Hills (1987)</td>
<td>El Centro</td>
<td>0.31</td>
<td>27</td>
</tr>
<tr>
<td>8</td>
<td>ATC-F19A</td>
<td>Chi-Chi (1999)</td>
<td>CHY101</td>
<td>0.18</td>
<td>95</td>
</tr>
<tr>
<td>9</td>
<td>ATC-F05B</td>
<td>Imperial Valley (1979)</td>
<td>Delta</td>
<td>0.46</td>
<td>105</td>
</tr>
<tr>
<td>10</td>
<td>ATC-F08B</td>
<td>Kobe (1995)</td>
<td>Shin-Osaka</td>
<td>0.26</td>
<td>46</td>
</tr>
<tr>
<td>11</td>
<td>ATC-F22B</td>
<td>Friuli (1976)</td>
<td>Tolmezzo</td>
<td>0.50</td>
<td>41</td>
</tr>
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</table>

References