IMPROVED MOMENT-RESISTING TIMBER FRAMES FOR EARTHQUAKE-PRONE AREAS PART II: SHAKING TABLE TESTS

MEJORA DE PÓRTICOS DE MOMENTO DE MADERA PARA ZONAS SÍSMICAS PARTE II: ENSAYOS EN MESA VIBRADORA

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Abstract

This investigation comprises the development of innovative multi-storey timber buildings based on post-beam moment-resisting frames that can withstand moderate to intense earthquake loads. This is the second paper of a two-series and presents the shaking table tests results of two timber structures that mounted two of the structural connections presented in the first part. The first of the tested structures was a full-scale one-storey frame whose connections were based on hardwood blocks with self-tapping screws. The second was a 1:3 scaled three-storey timber building that mounted one frictional connection. Both structures were subjected to a series of dynamic tests including noise excitation, pullout, sinusoidal sweep and seismic tests of increasing intensity in where strains, accelerations and drifts where measured with multiple devices. The first structure resisted without appreciable damage up to a peak ground acceleration of 1.5g, beyond which the first natural frequency decreased significantly after each increasing seismic excitation indicating significant structural degradation. The second structure showed larger drifts but resisted without noticeable damage up to 2g peak ground acceleration. The results suggest that the frictional connection could be promising for timber structures.

Keywords: structural connection; earthquake; post-beam frame; damping device; energy dissipation; cyclic loading

Resumen

Esta investigación comprende el desarrollo de edificios de varias plantas de madera innovadores basados en pórticos de momento tipo poste-viga de modo que éstos puedan resistir cargas sísmicas de alta y moderada intensidad. Esta es la segunda parte de una serie de dos artículos y presenta los ensayos en mesa vibradora de dos estructuras que montaron dos de las uniones que se presentaron en la primera parte. La primera de las estructuras ensayadas fue un pórtico de una planta a escala real que montó las conexiones basadas en el uso de bloques de madera de frondosa. La segunda estructura fue un edificio de 3 plantas escalado a 1:3 que montó las conexiones friccionales. Ambas estructuras fueron sometidas a una serie de ensayos dinámicos que incluyeron excitación por ruido, pullout, carga sinusoidal y carga sísmica de intesidad creciente registrando en cada caso las deformaciones, aceleraciones y desplazamientos con múltiples dispositivos de medición. La primera estructura resistió una aceleración pico del suelo de 1.5 g sin mostrar daños significativos, pero a partir de esa intensidad la frecuencia natural decreció significativamente indicando daños importantes en la estructura. La segunda estructura mostró drifts mayores pero resistió sin daño aparente cargas sísmicas con picos de aceleración del suelo de hasta 2g. Los resultados sugieren que la conexión friccional podría ser prometedora en estructuras de madera.

***Palabras clave**: conexión structural; sismo; estructura post-beam; amortiguador; disipación de energía; carga cíclica*

1. INTRODUCTION

Post-beam moment resisting timber frames do not need bracing or walls and thus offer expansive floor space, design freedom and a greater flexibility for building utilization. A paramount aspect of these structures is however the design of the beam-to-column connection (BTC), as it plays a principal role in withstanding extreme events such as earthquakes as well as determining the structural drift.

Most pervious research of BTC in shake table was focused on 2D plane frames in were the brittle failure risk was mitigated via plastic yielding of multiple small metal connectors, along with the inherent degradation of the wood. The degradation on the BTC is not only favorable because of the elastic energy release, but also due to the decreasing of the natural frequencies of the structure, thus moving away from resonant frequencies and minimizing the destructive potential of a seism. Some examples of plane moment-resisting frames on the shaking table can be seen in [1-3].

The previous design and testing of plane moment resisting frames presents however several inconveniences. First, in very seismic prone areas the yielding of the BTC may happen even at relatively moderated earthquakes so that the building can remain irreversibly damaged after small excitations. Second, as presented in the previous part of this series, a critical aspect of the BTC design is the decrease of the effective cross section of the column. This decrease is much larger in 3D than in 2D BTC, so it may happen that failures in real frames are much more brittle than expected. Third, the drift is still large even for stiff BTC so that it is very difficult to fulfil with standard requirements. And finally, research regarding low damage BTC concepts is lacking in timber structures subjected to seismic events [4]. Despite the lack of research, the passive damping devices such as frictional connections could be very promising if specifically designed for timber structures as they are much more flexible than other conventional materials and therefore can offer superior energy dissipation characteristics while keeping the building undamaged and maintaining the self-aligning capability. Indeed, such frictional devices have proofed improved seismic performance for steel structures [5-7], even when the elastic deformations are much smaller than those of the timber.

This research evaluated two of the BTC presented in the first part of this series. The first was a one-storey full-scale 3D timber frame that mounted the hardwood bocks BTC. The main aim of this testing was to assess the behavior of the 3D BTC, specially with regards to drift control and resisting capacity. The second testing involved a 3-storey 1 to 3 scaled timber frame that mounted the novel frictional BTC. The main purpose of the testing was evaluating the 3D BTC under seismic event, especially its ability to keep integrity and dissipate energy during earthquake.

2. ONE-STOREY FULL-SCALE FRAME WITH HARDWOOD BLOCKS BTC

The BTC with self-tapping screws was incorporated into the full-scale, one-storey frame in order to test its performance on a shaking table. The materials of this frame were identical to those used for the connections presented on the first part. The columns were 2560 mm in length with a cross section of 220 × 220 mm. The beams measured 3600 mm in length, 340 mm in height and 220 mm in width. The cross laminated timber (CLT) deck mounted on the top of the beams had a footprint of 4400 × 4400 mm, with a density of 500 kg/m^3 and thickness of 200 mm. For this panel to be constructed, two layers of the CLT, 100 mm each were glued and screws were used to apply the necessary pressure. Screws were also used to attach the decks to the beams on 100 mm from the center. The full live load was simulated by placing four steel masses of 1000 kg each on the deck. The total mass of the structure was about 6100 kg, see Figure 1.

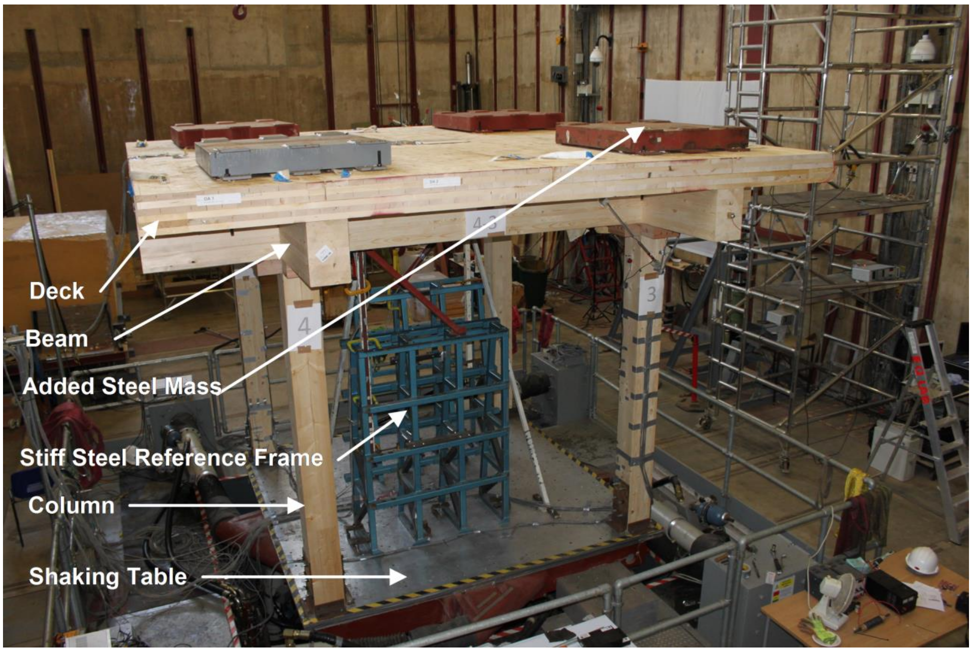

Figure 1: Typical testing layout of the BTC, based on [8]

The accelerations, drifts, strains and rotations of the connections were measured at multiple points. The acceleration recording set up was arranged by 14 accelerometers: three accelerometers on the shake table, four on the deck, six at the connections, and one attached to a beam. The drift of the frame was recorded by placing four string potentiometers on an extremely stiff steel structure placed beneath the timber deck, as seen in Figure 1. The relative rotation between beam and columns was measured in two connections at different directions with four LVDTs. The strain recording arrangement was composed by 16 strain gauges, four of them at the deck, four at the bottom of the beams, and eight at the columns. See an illustration of the measuring layout on Figure 2.

(a)

(b)

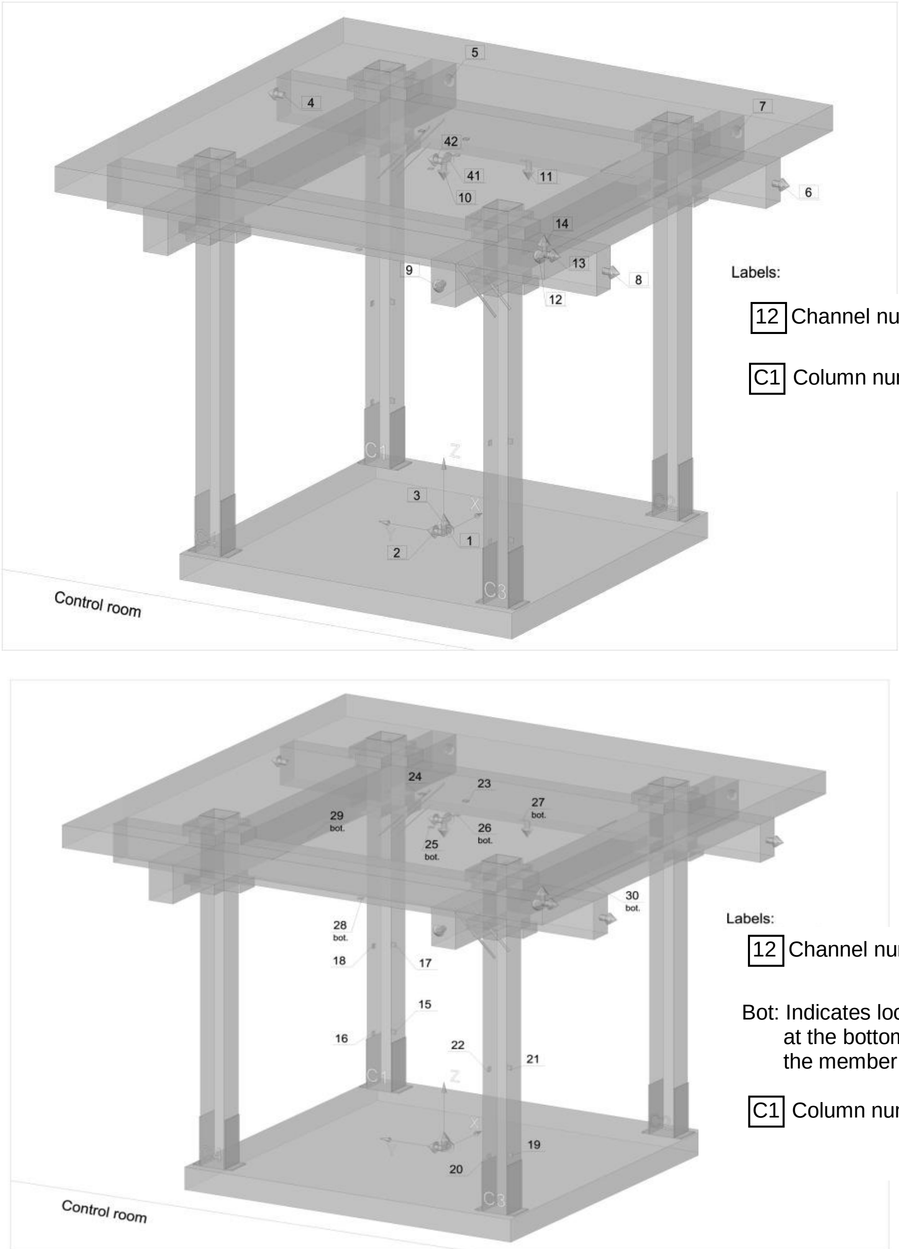

Figure 2: Locations of (a) accelerometers and (b) strain gauges, based on [8]

The dynamic tests were performed at the shaking table of the Earthquake Engineering Research Centre (EERC), at Department of Civil Engineering of the University of Bristol, Bristol, UK. The shaking table's footprint was 3000 × 3000 mm, and had a payload capacity of 17 tones with a maximum testing frequency of 100 Hz. The maximum displacements, velocities and accelerations of this table range respectively from ±150 mm, ±1100 mm/s, and ±60 mm/s^2. The series of dynamic tests included sinusoidal sweep, noise excitation, snap-back and seismic loads. The sinusoidal sweep consists of a sinusoidal excitation of the building at increasing or decreasing amplitudes so that the natural frequency can be estimated by examining the recording of accelerations, velocities or displacement on the frequency domain. The sinusoidal sweep presents the disadvantage that the frequencies occur one after another and that some of them have much higher amplitude than others, however on a natural seismic event the different frequencies are convoluted therefore a random excitation like the white noise test is a better reflection of the reality for determination purposes. The white noise (low amplitude random signal) overcomes this disadvantage and in addition it can be performed after any seismic test to evaluate the change on the natural frequencies without risk of damaging the structure any further. However, it presents on the other hand the disadvantage that frequencies are continuing and of very small amplitude so that it can be difficult to filter out of the noise. The snap-back consists of applying a certain displacement to the building and then releasing it into free-vibration so that not only the first natural frequency but also the damping and the initial stiffness of the building can be determined. Finally, the seismic signal in this research was design synthetically so to contain the most amount of the energy close to the frame's first natural frequency to ensure worse-case scenario. This approach has the advantage that the seismic signal can be recalibrated after each test according to the current building's natural frequencies.

Overall we performed three different seismic tests of increasing peak ground acceleration (PGA) from 0.08 g through 0.78 g and 1.5 g, and before and after each seismic tests some additional white noise, sinusoidal sweep and snap-back measurements were performed to determine the change of the natural frequencies due to damage on the frame. It was found that the natural frequency of the frame decreased with increasing earthquake amplitude, so that after the first seismic test the first natural frequency was 2.57 Hz, however this value was reduced to 2 .01 Hz and 1.56 Hz after the second and third seismic testing, respectively indicating significant damage on the BTC.

The frame behaved as anticipated and resisted until the third seismic test when failure was produced at a PGA of about 1.5 g. Before failure, the BTC experimented rotations of less than 1° which lead to maximum drift of 56 mm. This drift is less than 60 mm, which was the corresponding limit design value according to ASCE 7-10. An illustration of the drift and rotation of the joint can be observed Figure 3a for the third seismic test. As shown by the hysteretic curve the frame behaved rather elastically showing low energy dissipation capabilities. The frame showed self-aligning capabilities as the residual strain after seism was negligible. This is illustrated in the Figure 3b showing the typical recording of the strain at one critical point on the columns.

Although the frame resisted up to a PGA of 1.25g without critical damage, a brittle failure by shear-off of one of the columns close to the BTC occurred during an excitation of 1.5g (PGA). We attributed the reason for such brittle failure due to a 60 percent reduction of the column's cross-section and local stress concentration due to the presence of multiple screws (reducing of the cross-section), as the column had at that point a strain of 0.2 % which is far from the anticipated strain yield limits of about 0.5 %. However, the PGA at failure is considered to be unrealistically high for most of the cases.

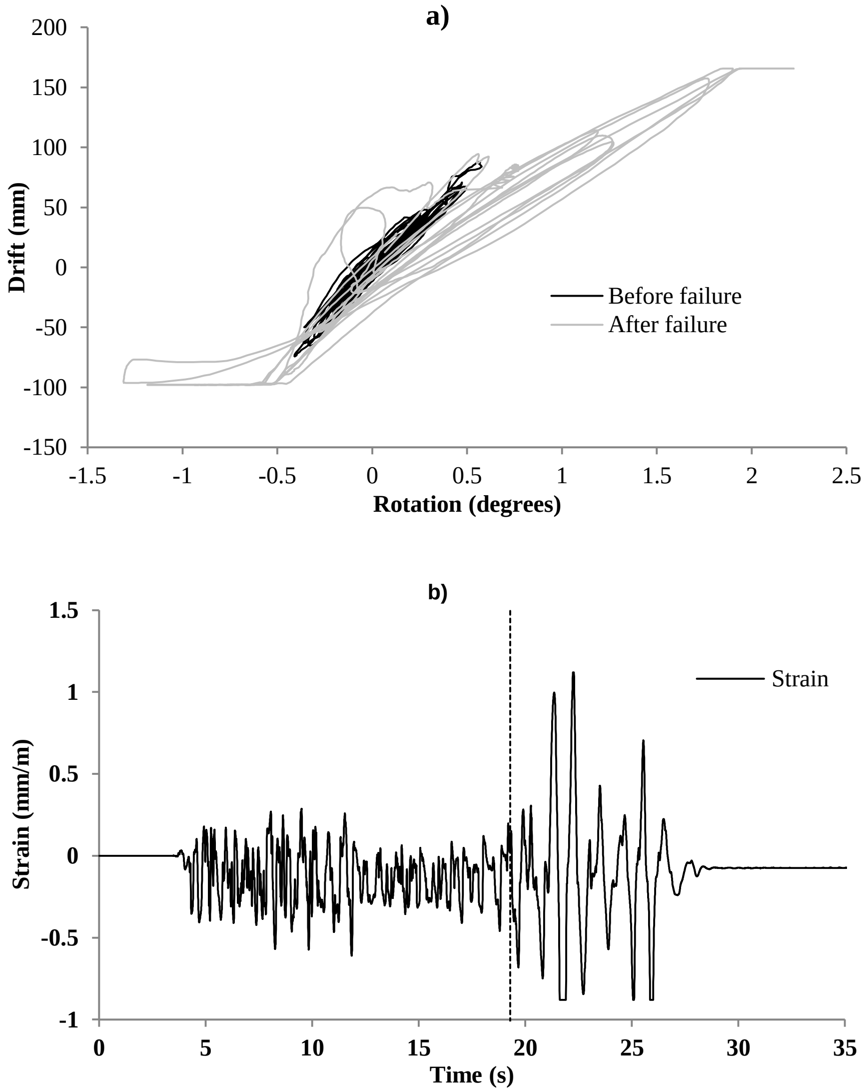

Figure 3: (a) Rotations versus drifts, and (b) strains in the one-storey frame during the third seismic test (1.43g PGA), based on [8]

3. THREE-STOREY 1 TO 3 SCALED FRAME WITH FRICTIONAL BTC

The second experiment comprised the dynamic testing of a 1 to 3 scaled three-storey frame that mounted the frictional connections presented on the first part of this series, see Figure 4a. The materials used in the frame were the same as those used in the previous structure. The columns were 1680 mm in length with a cross section of 140 × 140 mm for the first storey. The lengths of the column were then decreased to 1200 mm for the following two stories. The beams measured 2160 mm in length, 200 mm in depth and 100 mm in width. The length of the overhang beams was 825mm and were located only on one corner of the structure. The purpose of these overhangs was to simulate adjacent bays which provided the frame more realistic attributes as those found in real structures. The beams and columns were fiber reinforced at each of the BTC connections. Each floor was made with CLT deck which had a footprint of 3000 × 3000 mm. The density of the deck was 500 kg/m3 and had a thickness of 50 mm, see Figure 4a for clarification of the entire structure ready for testing. Steel masses of 150 kg total per floor, were placed on the deck next to one column to simulate the combination of dead and live loads, the Figure 4b shows a detail of the additional mass locations. The total mass of the structure with loads was approximately 1750 kg. The BTCs and column-to-foundation (CTF) connections details are shown in Figure 4c and 4d, respectively.

(a) (b) (c) (d)

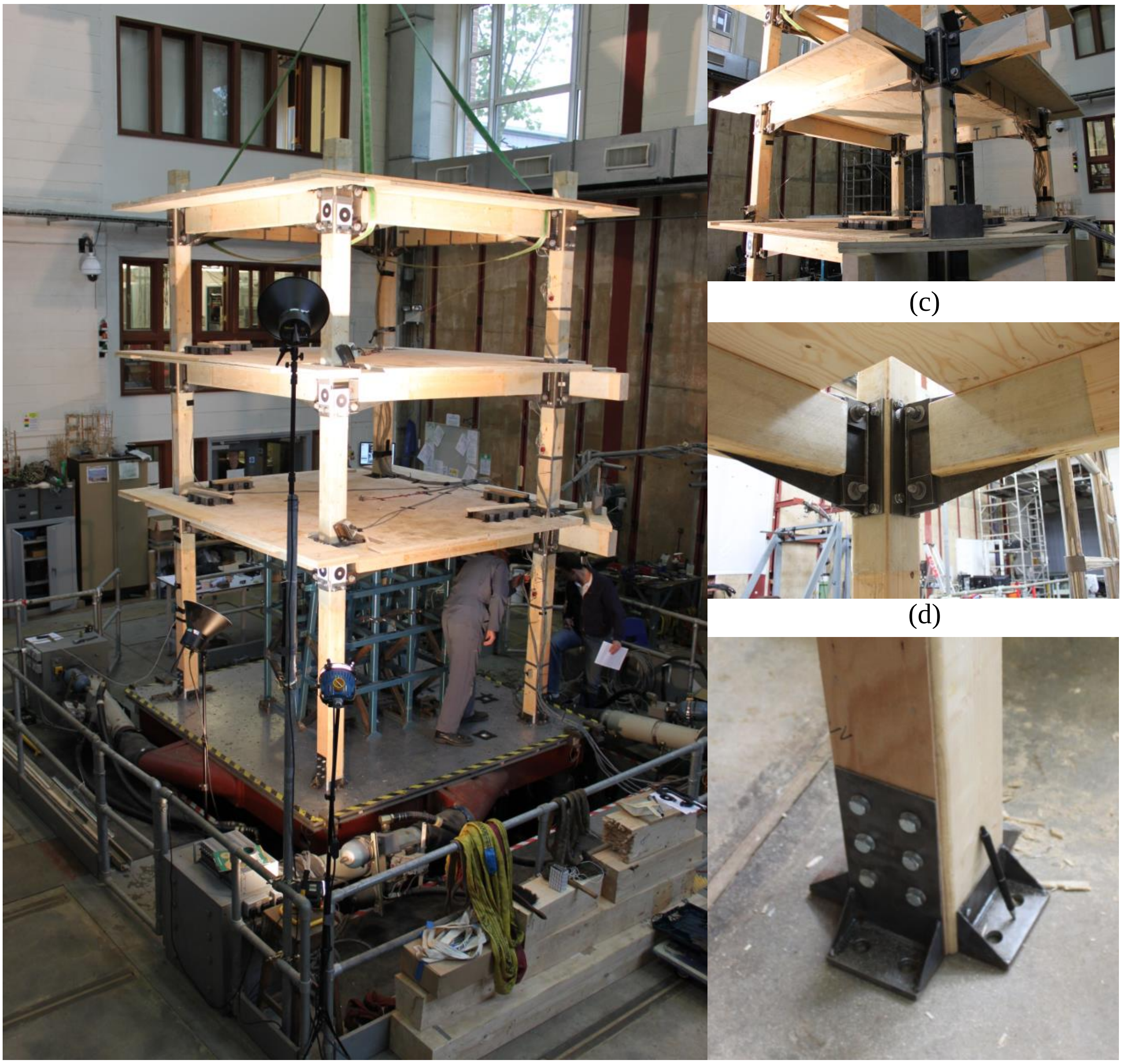

Figure 4: (a) scaled three-storey frame ready for testing, (b) detail of added steel masses, (c) detail of the BTC connection, (d) detail of the CTF connection

The dynamic testing was also performed at the EERC in Bristol and included a series of sinusoidal sweep, noise excitation, snap-back and seismic loading. The measuring device arrangement included a complex set of 62 channels corresponding to LVDT, potentiometers, accelerometers and strain gauges. 24 Accelerometers were installed: three on the shake table and seven at each floor. Potentiometers 1 through 8 measured the relative displacements between each BTC of one column, as well as the relative displacements between column and foundation. LVDTs 9 through 12 directly measured the inter-storey drift of the first floor by comparison with a reference extremely stiff steel frame (see Figure 4a). LVDTs 13 to 18 measured the inter-storey sway for the second and third storey. Finally, 8 strain gauges were placed on one column close to the BTC and CTF and 12 gauges measured the strain at the beam, see the layout of measuring devices in the Figure 5.

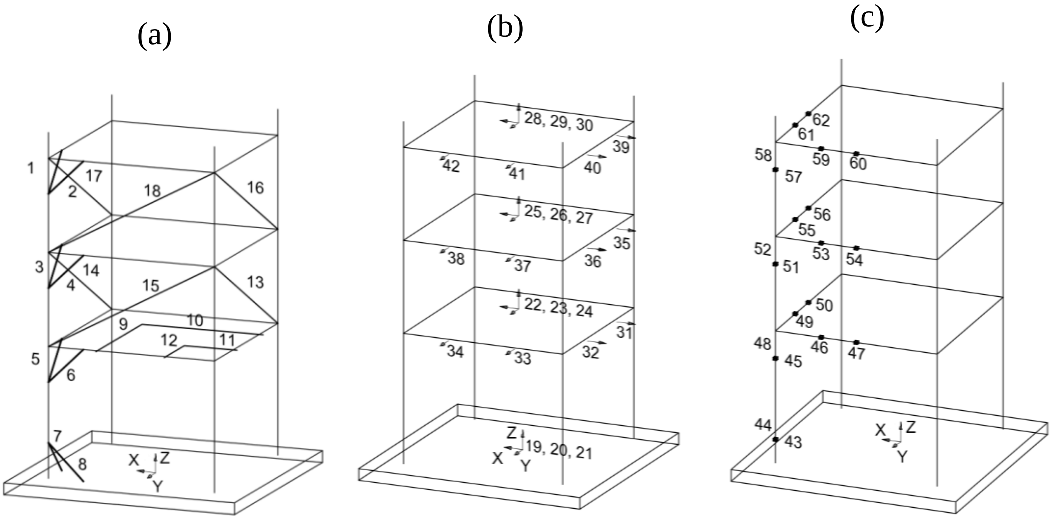

Figure 5: Schematic of the testing layout showing the location of (a) LVDT and potentiometers, (b) accelerometers and (c) strain gauges, based on [8]

Overall, there were five seismic tests of increasing PGA intensity corresponding to 0.1, 0.3, 0.5, 1.0, and 2.0 g for both the X and Y directions. The PGA on the Z (vertical) direction was set about 20 % of that of the X and Y directions. The seismic signal was also synthetically designed for this experiment to contain the maximum amount of energy close to the resonant frequencies of the building.

Results indicated that the PGA measured during testing always exceeded the expected PGA of each seismic test. No visible failures were observed for the seismic tests 1 through 4. The fourth seismic test already showed maximum PGAs of 1.4 g in the y-direction, meaning that the frame withstood at least an intensity of 1.4 g without visible damage. During the fifth seismic test one of the columns showed brittle failure following by a second column 10 seconds thereafter. The maximum PGA at that point was above 2g. Unfortunately, only the response of the second failing column could be recorded by a strain gauge, so that the strain at failure could not be recorded. However, the strains at the second failing column were far from elastic limits suggesting failure due to local transverse stress on the timber arising from the bolts fixing the metal plate at the column. Before the failure, the maximum measured strain was 0.12% which is far from the anticipated elastic limit of 0.4%. The strains at the columns were much smaller which is expected as the beams carried a relatively small gravity load, and also because the friction was expected to diminish the loading at the beams. The strains at the second and third floor was rather small. Fig. 6 shows the increasing strain in the x-direction during the different seismic tests. For seismic tests 1 through 4 the strain increased linearly for channels close to the BTC but decreased significantly for the last testing. The slope of the strain at the CTF however decreased already for the fourth and fifth test indicating that the CTF starting degrading before the BTC started the planned frictional mechanism. This change reflected on the natural frequencies of the building measured after each seismic test. During the first three tests there was no appreciable change, and after the fourth seismic test decreased by about 20%. This suggest that the building did not suffer degradation up to 1g, and even at this point the degradation should be produced by the CTF and not by the BTC. Also it was observed a negligible drift and strain after each test which confirms that the BTC worked as anticipated, decreasing the stress of the structural members and dissipating energy in a reversible way, without damage and keeping self-aligning capabilities.

For the first floor, the drifts measured directly from the LVDTs were compared to those obtained by integrating the accelerometer response. The comparison yield to almost the same results which is important to verify the accuracy of the drifts measured in the second and third floors, as they can only be estimated via accelerometers. Drift results showed that the building was able to keep drift within limiting standards (ASCE 7-10) up to a PGA of about 1.45 g.

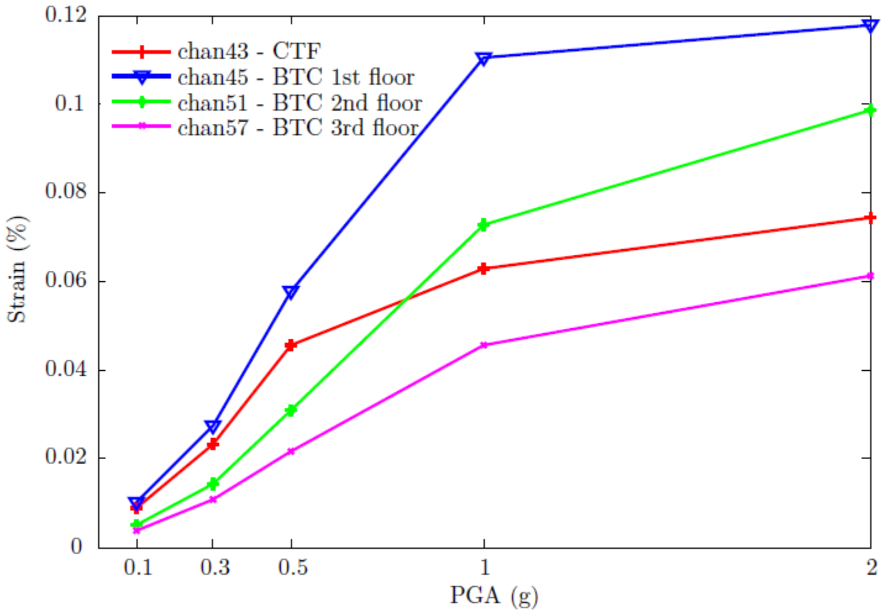

Figure 6: Stress increase on the different seismic tests at critical points of the structure, based on [9]

5. CONCLUSION

This paper presents the results of an experimental investigation in where two of the 6 connections introduced in the first paper of the series were tested under seismic excitations on a shake table. The tested connections were rather different and may symbolize two distinct concepts of wood design.

The first was high-strength, stiff and cost-effective connection based on hardwood blocks and self-tapping screws. This connection was mounted in a full-scale one-storey beam-post timber frame that withstood a peak ground acceleration up to 1.5 g and keeping drift limits close to current standards' limitations. The frame showed self-aligning capacities but the failure of the frame was very brittle and considerable degradation of the structure was recorded at relatively moderate seismic intensities.

The second was a low-damage damping, frictional connection that was mounted on a three-storey 1 to 3 scaled timber building that resisted accelerations up to 2g and kept the drift within standard limits up to 1.45 g. Unlike the previous structure, this building did not show any significant damage at moderate to intense seismic intensities thanks to the damping device, and the elastic energy was still enough for the structure to self-align. However, as the previous frame it failed rather brittle at very high seismic intensities.

These results suggest that it is very difficult to control the drift on post-beam timber structures to fulfil current standards. Also, it could be critical the design of earthquake resistant connections on 2D because in 3D there is an increase reduction of the cross-section in the column that usually followed by an intensification of transverse to the grain stresses in the wood, which may lead to brittle failure. The use of low-damage damping devices in timber seems however promising, because it can take advantage of the timber flexibility to dissipate large amounts of energy preventing members from failure, moving away structure from resonant frequencies and keeping structural damage to very small levels. Also, even with high energy dissipation devices, the energy stored in the timber seems enough for structural self-aligning. It is therefore suggested that future investigations should be performed to design specific damping devices for timber buildings.

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