CYCLIC AND SHAKING-TABLE TESTS OF TIMBER–GLASS BUILDINGS

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ABSTRACT

As a natural raw material timber shows indisputable environmental excellence and certainly represents one of the best choices for sustainable construction. The use of glazing in buildings has always contributed to openness, visual comfort and better daylight situation. The features of the both building materials lead to the development of a new type of highly attractive structures, the so-called timber–glass buildings. However, in a view to maximising the use of natural solar radiation gains, the most of the glazing is usually placed in the south facade of such buildings, which can lead to many structural problems, especially when the building is exposed to heavy horizontal loads. In such cases it is usually to assure a horizontal stability by using additional visible diagonal elements or by internal wall elements. In this study we are presenting another solution by using timber-frame wall elements with fixed insulating glazing placed on the external side of the timber frame where the glass pane is considered as a load-bearing element. It is presented that such timber–glass load-bearing wall element can significantly contribute to the overall horizontal resistance of the whole building. The behaviour of load-bearing timber–glass wall elements is additionally modelled with FE model where the bonding line is modelled with spring elements. With such developed mathematical model it is possible further parametrically to analyse many various parameters which significantly influence on the capacity, stiffness and failure mechanism of such composite elements.

Keywords: experiments, finite element modelling, glass, timber, structural stability.

1 INTRODUCTION

Designing nearly zero-energy buildings is a goal in many European countries; therefore, numerous studies have emerged to find a solution for designing buildings with high energy performance. As a natural raw material requiring minimal energy input into the process of becoming construction material, timber shows indisputable environmental excellence with very low CO_2 emissions. The use of glazing in buildings has always contributed to openness, visual comfort and better daylight situation. Although characterized by weak thermal properties in the past, glass has been gaining an ever greater significance as a building material due to its improved thermal, optical and strength properties, resulting from years of development.

The features of both building materials presented above lead to the development of a new type of structures, the so called timber–glass buildings (see Figure 1), suitable for the construction of energy-efficient buildings where an optimal proportion and appropriate orientation of the glazing surfaces play an important part due to exploitation of solar radiation as a source of renewable energy within the passive use of energy for heating. However, architects need to be careful when placing the large glazed areas to make the best use of natural solar incomes. The transparent areas have to be of appropriate size and orientation to transmit an adequate amount of solar energy into a building in order to assure natural lighting and heating. Respecting these facts, the largest area of the glazing in a building has to be orientated towards south (for buildings in the northern hemisphere), Zegarac Leskovar and Premrov [1]. Such placement of large glass areas enables better energy performance of a building, where the daily obtained solar gains through the glazing can be evidently higher than the transmission losses throughout the night.

ISSN: 2046-0546 (paper format), ISSN: 2046-0554 (online), http://www.witpress.com/journals
DOI: 10.2495/CMEM-V5-N6-928-939

Figure 1: Contemporary timber–glass house with enlarged size of glazing on the south side.

However, this kind of construction systems can be, despite their energy efficiency, very problematic from a structural view when a building is horizontally loaded (i.e. wind and earthquake). If the timber–glass wall elements are not considered as load-bearing bracing elements the rest of the walls (external and internal) without any openings should be able to transmit horizontal load actions to the basement. Another possibility is to insert visible diagonal steel or timber elements as main bracing elements.

In most cases the more problematic of these two loads is earthquake, which subjects a building to a high intensity dynamic load often resulting in catastrophic consequences. One of the basic principles when designing a building to resist seismic loads is trying to avoid plan irregularity. This means that the building's centre of mass and centre of stiffness should be close together. Unfortunately, this is an issue concerning energy efficient buildings that have large glazing areas predominantly placed on southern facades and evidently smaller glass areas especially on the north side, hence resulting in an uneven stiffness over their floor plan and an important dislocation between the centre of gravity and centre of rigidity. To avoid this fact, it is important to consider most of the external walls on the south facade as load-bearing elements, which means that the walls with fixed glazing areas (but not windows) should also be treated as resisting elements and will be treated as composite elements composed of a timber frame and a glass sheathing, which will be somehow able to transmit a considerable part of horizontal forces to the basement. Considering such approach, a racking resistance and stiffness of the whole analysed building can be essentially increased and thus can result in decreasing number of resisting internal walls or external diagonals to be used to assure a horizontal stability of the whole building.

This paper deals with the experimental analysis of such composed timber–glass wall elements which are finally tested as main vertical resisting elements in »box-house models« on a shaking table. Simple mathematical models using FEM formulation are presented at the end of the study as a possibility how further parametrically investigate many different parameters which have a significant impact on resistance of such timber–glass elements.

2 TIMBER–GLASS WALL ELEMENTS

Prefabricated timber construction systems differ from each other in the appearance of the structure and in the approach to planning and designing a particular system. As presented in [1, 2], timber houses can be classified into six major structural systems: log construction, solid timber construction, timber-frame construction, frame construction, balloon- and platform-frame construction and frame-panel construction. Log construction and solid timber construction can also be classified as massive structural systems since all load bearing elements consist of solid elements. Other construction systems consist of timber-frame bearing elements and are therefore classified as lightweight structural systems. According to the load bearing function they can be subdivided into classical linear skeletal systems where all the loads are transmitted via linear bearing elements and planar frame systems where sheathing boards take over the horizontal loads. The main focus of the paper will be laid on the frame-panel construction system only whose detailed analysis in combination with the glazing is the subject matter of our research.

2.1 Frame-panel structural system

Advantages of the frame-panel construction system over the above mentioned traditional timber-frame construction systems were first noticed at the beginning of the 80' of the previous century and made a significant contribution to the development of such timber construction. Furthermore, the transition from the single-panel construction system (Fig. 2a) to the macro-panel construction system (Fig. 2b) means an even higher assembly time reduction and higher stiffness of the entire structure due to a lesser number of joints. The wall elements with a total length of up to 12.5 metres are now entirely produced in a factory. Prefabricated timber-frame walls functioning as the main vertical bearing capacity elements, whose single panel typical dimensions have a width of $b = 1250\ mm$ and a height of $h = 2500 – 3100\ mm$, are composed of a timber frame and sheets of board-material fixed by mechanical fasteners to both sides of the timber frame (Fig. 2a).

There are many types of panel sheet products available, which may have a certain level of structural capacity such as wood-based materials (plywood, oriented strand board, hardboard, particleboard, etc. or fibre-plaster boards). Experimental and numerical analysis of the influence of the sheathing board type on the load-bearing capacity and racking stiffness of the single-panel wall elements are presented in many studies [3–5].

Each wall assembly at individual levels consists of separate wall segments acting as individual cantilevers, where every segment is determined with the width b of the sheathing board (usually 1250 mm). The lateral forces acting at the top of the element are considered to be uniformly distributed to each segment, the horizontal force acting on a single wall element

a.) b.)

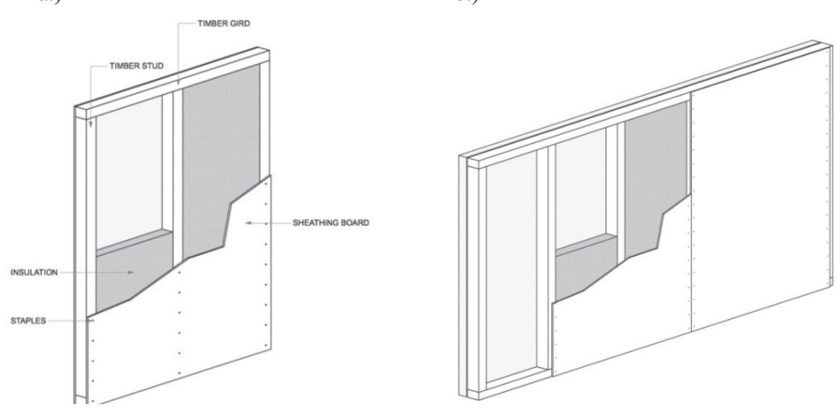

Figure 2: Single panel construction system (a) and macro-panel construction system (b).

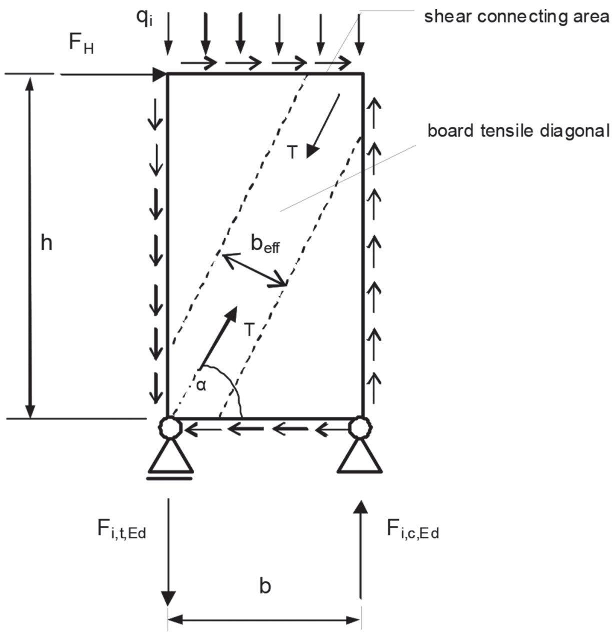

Figure 3: Scheme of the force distribution in a timber-frame wall element.

can be calculated as $F_H = F_{H,tot}/n$. Since only the segments with the full wall height having no window and door openings are usually taken into account for the calculation, the value n represents the number of single-panel wall elements without any openings or fixed glazing. Stress distribution with a horizontal load action in a single-panel wall element is schematically presented in Figure 3. It is shown that basically three possible criteria exist to determine the wall racking resistance:

- The shear stresses in shear connecting area between the sheathing board and the timber frame elements reach the yielding point of the fasteners (Eurocode 5, Method A).
- The tensile stress in the sheathing board reach the tensile strength of the boards (composite wall model).
- The tensile stress in the timber frame elements reach the tensile timber strength.

It was experimentally and numerically presented in many studies [3–5] that the first criteria usually appear by the wall elements with the boards with a high tensile strength (OSB) and the second by the boards with relative low tensile strength (fibre-plaster boards). The last criteria practically never appears.

2.2 Glass as a load-bearing material in timber-frame wall elements

The concept of using glass for the main load bearing elements, i.e. beams, columns and shear walls in contemporary timber construction is rare. This is due to several reasons, from the lack of building codes on one end to the psychological effect of perceiving glass as a fragile material on the other. The demand for the use of glass in timber architecture is increasing though and several studies [6–9] have been performed over the past decade to investigate the possibilities of using glass for load bearing elements. The main idea of using fixed glazing in

Figure 4: Timber–glass prefabricated walls - replacing the classical sheathing boards with the glass panes, [1].

prefabricated timber-frame panel wall elements is to replace the classical sheathing boards with glass panes, as seen in Figure 4. The glass pane in this case has to assume the role of the classical sheathing board to transform tensile stresses in diagonal direction and the adhesive the role of the fasteners in the connecting area. Thus, according to the already presented stress distribution in subchapter 2.1, three possible failure modes exist if the glazing is considered as a load-bearing sub-element:

- The shear stresses in the bonding line cause rupture of the adhesive; adhesive failure mode.
- The tensile stress in the glass pane reaches the tensile strength of the glass; glass failure mode.
- The tensile stress in the timber frame elements reaches the tensile timber strength; timber failure mode.

3 EXPERIMENTAL STUDY

Three systematic steps (monotonic, cyclic and seismic) of experimental testing will be presented with a basic aim to investigate the behaviour of load-bearing timber–glass wall elements under different types of horizontal load action. The timber-frame wall elements with an insulating three-layered glazing directly bonded to the timber frame (Fig. 5) were subdivided into two special groups (TGWE-1 and TGWE-2). The glass panes were on the external side, directly bonded to the timber frame without using any special substructure. The tested timber–glass wall elements (TGWE) consisted of a timber frame with the outside edges measuring 2.4 × 2.4 m. The dimensions of the timber stud cross sections were 160/160 mm. The dimensions of the top post cross section (width/height) were 80/280 mm and the dimensions of the bottom post were 160/120 mm. TGWE-1 represents a wall element with one large insulating three-layer glass pane in one piece. TGWE-2 represents a wall element of same dimensions, but with two smaller glass panes divided by an additional stud in the middle. The polyurethane adhesive with the end-joint type according to Niedermaier classification [8] was used. Adhesive layer was made of 5.0 mm thick one-component polyurethane adhesive and it was applied circumferentially around the glass panel into a groove in the timber frame.

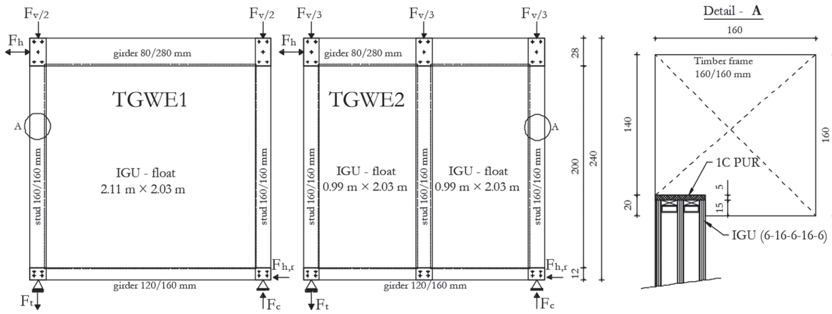

Figure 5: Geometry of the specimens from the testing groups.

Table 1: Mechanical characteristics of timber, glass and adhesives used in the experiments.

	$E_{0,m}$	G_m	$f_{m,k}$	$f_{t,0,k}$	$f_{c,0,k}$	ρ_k	ρ_m
	[N/mm^2]	[N/mm^2]	[N/mm^2]	[N/mm^2]	[N/mm^2]	[kg/m^3]	[kg/m^3]
Timber frame GL24h	11600	720	24	16.5	24	380	456
Float glass	70000	28455	45	45	500	2500	2500
	E	G_ν	ν	$f_{t,0,k}$	ρ_k	temp. resistance	
	[N/mm^2]	[N/mm^2]	[-]	[N/mm^2]	[kg/m^3]	min°C/max°C	
Polyurethane	1	0.454	0.49	2	1170	−30°C/+70°C	

Timber frames were made of wood with a strength grade GL24h, triple insulation glass panes were made of float glass and the adhesive used in the timber–glass joint was a one-component polyurethane adhesive, type Ködiglaze P produced by Kömmerling. Material properties of timber with a strength grade GL24h were taken from EN 1194:2003 [10], material properties of float glass were taken from EN 572-1:2004 [11] and material properties of adhesives were obtained from the producer's technical sheet. All material properties are listed in Table 1. Further detail information about the test specimens can be found in WP 6 Wood-Wisdom project report [9].

3.1 Racking tests

Besides the monotonous horizontal point loading (F) according to EN 594:2011 [12] an additional vertical load of $q = 25\ kN/m$ was applied onto the specimens to simulate the dead load of the roof and upper floors (Fig. 6). Maximal horizontal displacement (w) was measured at the top of the specimens.

The F–w diagrams for the both test samples groups are presented in Figure 7.

It is clear from the given results for the TGWE-1 and TGWE-2 test samples behaviour:

- The TGWE-1 test sample demonstrated a higher racking strength (for 16.6%) and especially essentially higher stiffness (for 78.3%) than TGWE-2.
- The TGWE-2 test sample demonstrated essentially higher ultimate deformation and con-

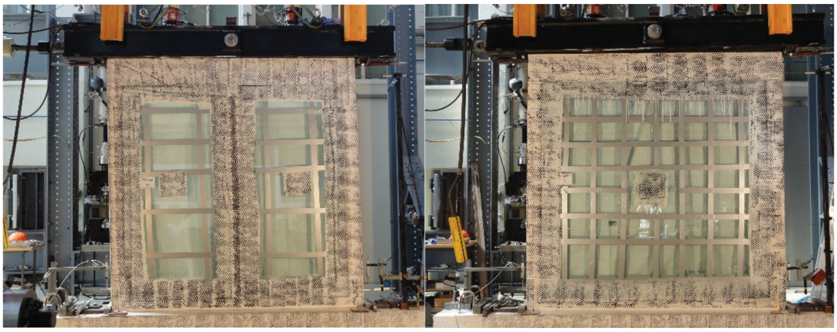

Figure 6: Photo of the test specimen in the loading machine subjected to horizontal point load and uniform vertical load.

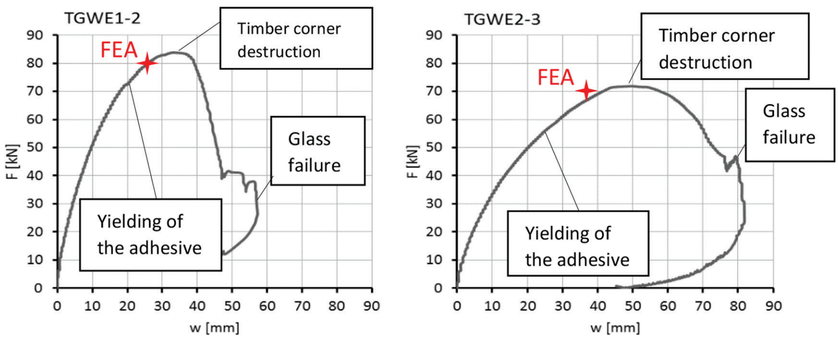

Figure 7: *F*–*w* diagrams for the TGWE-1 and TGWE-2 test samples.

sequently a higher ductility. Therefore, it could be concluded that the ductility is achieved with the flexibilty of the adhesive in the lateral area of the bonding line.

- The both TGWE test samples demonstrated similar kind of failure in the following stages:
- Yielding of the adhesive resulting in total destruction of the bonding line (Stage 1).
- Destruction in the corners of the timber elements connections – ductile failure (Stage 2).
- Brittle glass failure (Stage 3).

3.2 Cyclic tests

Two test samples of each testing group (TGWE-1 and TGWE-2) were tested in accordance with ISO 16670:2003 [13] with the horizontal point loading procedure with 10 steps. The standard is prescribed for testing joints with mechanical fasteners, however it could be reasonably adapted for timber–glass walls with adhesively bonded joints. In determining the loading protocol, ultimate values of displacements from the already presented monotonous static tests were taken into account. The results are presented in Figure 8.

It is evident that the both test samples from the same group demonstrated very similar behavior (deviation of the measured results was very small). The both TGWE groups demonstrated a very similar kind of failure as the samples with the monotonic loading procedure. The maximal measured horizontal point load and therefore the racking resistance of the wall elements of the group TGWE-1 is higher than by the TGWE-2 for 11.7%. It is also evident from the inclination of the hysteresis that the racking stiffness of TGWE-1 samples is essentially

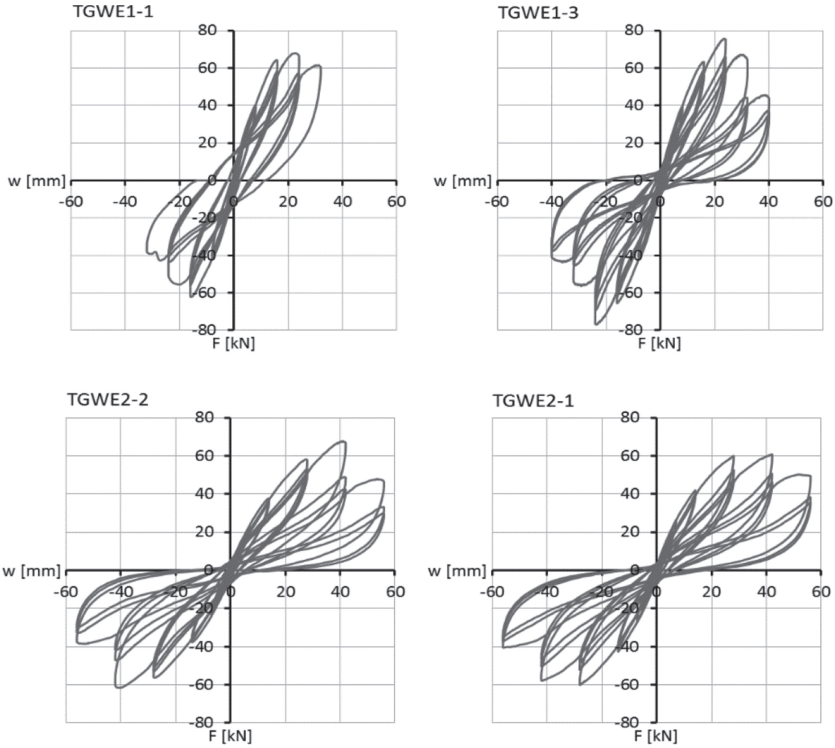

Figure 8: Results of the cyclic test for TGWE-1 and TGWE-2 testing groups.

higher than by TGWE-2 samples which is in good correlation with the conclusions from the monotonic test results. On the other hand, the ductility of the TGWE-2 group is evidently higher than by the TGWE-1 group. This is of the utmost importance for the seismic resistance of buildings erected with such timber–glass wall elements which will be further discussed.

3.3 Shaking-table tests

Our goal was not limited only to proving the load-bearing capacity of the wall elements under a monotonic static and cyclic point load, where the elements are tested in 2-dimensions only, but further to extend our research in behaviour of timber–glass wall elements incorporated into 3-dimensional full-scale timber box-house models which were tested on the shaking-table. The box model, schematically presented in Figure 9, consist of the already described timber–glass wall elements as well as of the timber-framed wall elements with classical OSB sheathing boards which proved essentially higher racking resistance and stiffness under the monotonic point load than the timber–glass wall elements. 100 mm thick cross laminated (CLT) timber floor slabs were used and an additional mass of 1600 kg was installed on each floor.

Four single-storey and four two-storey structures combining different types of wall elements with ground plane dimension of 2.4 × 3.4 m were tested on the shaking table. Single-storey setups had a total height of 2.5 m, two-storey setups reached exactly 5 m in height (Fig. 10).

The testing series was divided into two basic modules:

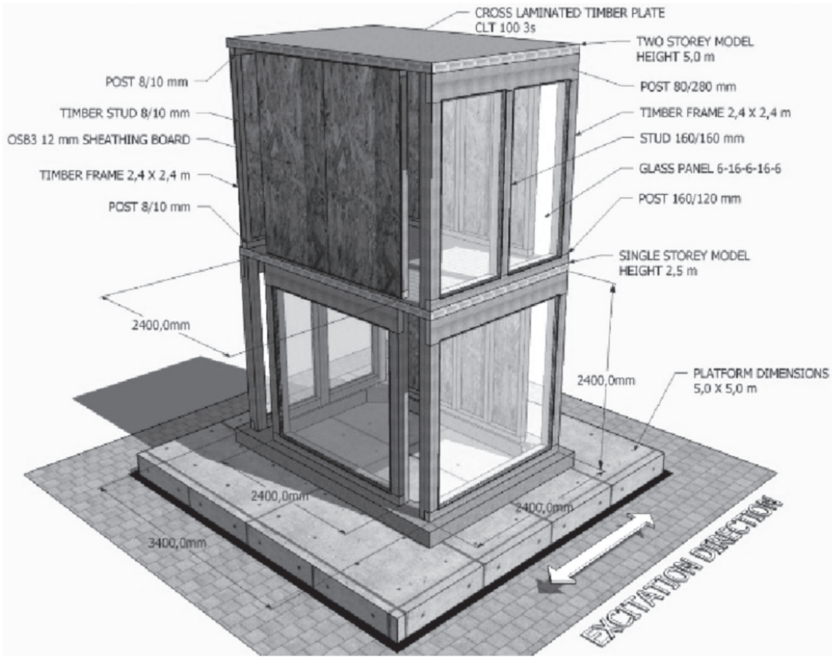

Figure 9: 3D box house model subjected to the seismic excitation on the 2D shaking-table.

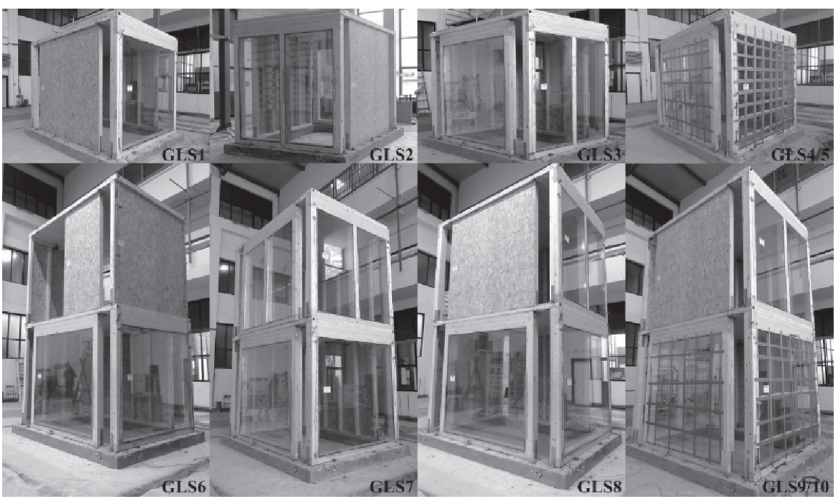

Figure 10: Configuration of one and two-storey test models.

- Low-intensity testing where the structure remained undamaged and in an elastic state of the material behaviour (including all the connections),
- High-intensity testing where the ground acceleration was scaled up enough to cause failure in the structure. Before and after each earthquake simulation a sine sweep test (frequencies in the range of 1–32 Hz, acceleration intensity of 0.01 g) was performed in order to clearly calculate the vibration period of the structure.

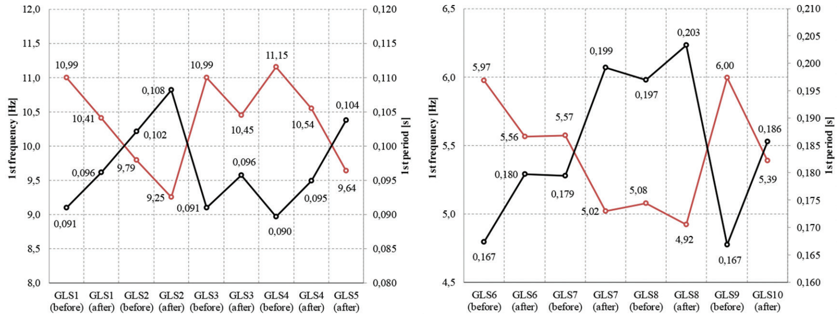

Figure 11: Diagram of first periods for each model before and after the earthquake simulation.

The sinus test was followed by a series of scaled modified accelerograms of the Landers earthquake. The accelerogram was modified in a way to excite a broad spectrum of vibration periods namely to affect all types of structures regardless of their stiffness as shown on the comparison of the accelerogram's elastic spectra (with 5% damping) to the standard Eurocode 8 elastic spectra. As already mentioned before and after each earthquake simulation sine sweep test with frequencies in the range of 1–32 Hz and intensity of 0.01 g was applied in order to clearly calculate the vibration period of the structure and to record the response of the building. The change of periods can also be treated as a measurment of stiffness decreasing. Diagrams of the measured first periods are shown in Figure 11.

It can be observed from the presented results that there is only a slight change in the measured 1st periods before and after excitation by all tested models. This can prove that there was only a small decrease in a horizontal stiffness of the tested models which can be the first indicator that the deformation range in the structural elements and connections was not essentially high. A visible type of the observed deformations in all structural elements as well as connections to the RC foundation and between the floor and wall elements were observed. The tested walls demonstrated a desirable rocking-type of behaviour without any residual deformations in the adhesive joint, glass panes and the timber frame. A ductile failure mechanism was established in the steel hold-downs. It should be noted that a low vertical load on the bracing walls had an influence on the development of a rocking mechanism. With a higher vertical load the shear behaviour of the glass panels would be activated, hence increasing the stresses in the shear brackets, the adhesive and the glass. Further results and analysis can be found in [9].

4 MATHEMATICAL MODELLING

With a goal to simulate the general response of TGWE subjected to a racking load, finite element models were built and calculated using SolidWorks Simulation software. Timber parts were modelled using 3D solid tetrahedral elements, while glass panes of the IGU were modelled as shells with a virtual thickness of 6 mm per each, as shown in Figure 12a. To save time and hardware resources the bond line and mechanically fastened joints of the timber frame were modelled with linear springs as it is schematically presented in Figure 12b. Axial stiffness of the spring defining adhesive (k_w) was calculated as a quotient of Young's modulus and thickness of the bond line, while shear stiffness (k_u) was considered as shear modulus divided by thickness of the bond line. Springs were distributed circumferentially between the timber frame and a glass panel at a distance of 100 mm. Each corner of the timber frame was

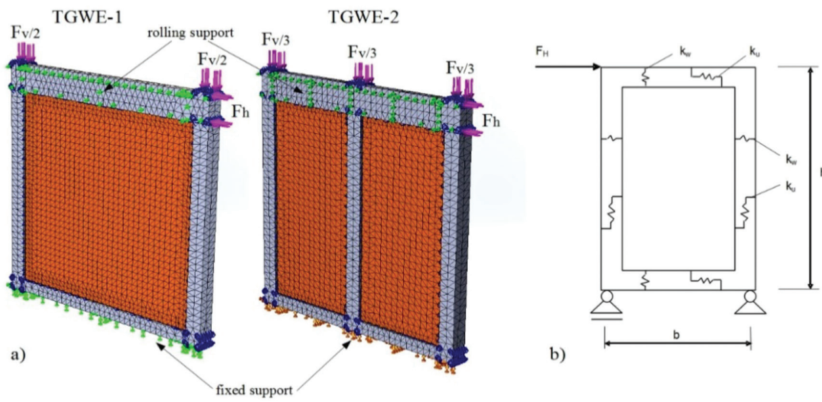

Figure 12: Scheme of the spring model (a) and boundary conditions of both TGWE (b).

joint using self-tapping screws 10 × 140 mm (Fig. 5). Each group of screws was modelled as springs with stiffness (K_u) which equals to ultimate slip modulus determined by the expression from Eurocode 5:

$$K_u = n \cdot \frac{2}{3} \cdot K_{ser} \tag{1}$$

Results of the FEA are added to force-displacement master curves of the racking tests (Fig. 7). For TGWE-1 a horizontal displacement of 26.1 mm was calculated at F_h = 80 kN, which exactly describes mechanically tested model TGWE1-2. However, TGWE-2 with w = 36.1 mm underestimates a horizontal displacement of TGWE2-3 at F_h = 70 kN.

5 CONCLUSIONS

To avoid a flor-plan asymmetry of timber buildings with enlarged areas of glazing placed on the south façade it is necessary somehow to ensure racking resistance and stiffness of timber–glass wall elements which can essentially contribute to the overall racking resistance of the whole building. It was demonstrated by the presented monotonic and cyclic tests that the bonding type by using a polyurethane adhesive with the end-joint type can be a good solution to assure a reasonable part of load-bearing capacity and ductility as well. It can be concluded that the TGWE-1 test samples (glass pane in one piece) demonstrated a higher racking strength and especially essentially higher stiffness than TGWE-2 test samples (glass pane in two pieces). On the other hand, the TGWE-2 test samples demonstrated essentially higher ultimate deformation and consequently a higher ductility. It is evident from the presented failure modes that this type of the bonding line condition with the polyurethane adhesive can demonstrate a quite ductile type of failure which finally results in s.c. ‚cradle behavior‘ of the glass pane in the timber frame under a dynamic horizontal load. Therefore, it could be concluded that the ductility is achieved with the flexibility of the adhesive in the lateral area of the bonding line and the number of the bonding lines therefore increase the ductility, but decrease the racking resistance and especially the stiffness of such timber–glass wall elements.

The tested walls on the shaking-table demonstrated a desirable rocking-type of behavior without any residual deformations in the adhesive joint, glass panes and the timber frame. A ductile failure mechanism was established in the steel hold-downs. Therefore, it is recommended to use this bonding type and polyurethane or silicone adhesive for the buildings located on heavy seismic areas. Due to a high range of ductility wall elements with glass panes in two pieces (TGWE-2) are more recommended as the walls with glass pane in one piece (TGWE-1).

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