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LAMINATED GLASS - TIMBER PANEL AS A DISSIPATIVE BRACING STRUCTURAL COMPONENT FOR EXISTING FRAME STRUCTURES

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Abstract

The idea for development of innovative multipurpose structural component appeared following the recent decade rapid development of laminated glass structural elements as well as popularity of cross laminated timber elements. The need and introduction of innovative structural elements is running beyond the existing code guidance and influences their further enhancement. The collaborative research of University of Zagreb and University of Ljubljana has been launched in order to develop a hybrid structural component that can be used for various purposes in structures located in earthquake prone areas: as an independent load-bearing component for construction of prefabricated timber structures, as a temporary or even permanent support and bracing element in heritage buildings, as an external or internal bracing element in retrofitted or newly built frame structures or as a versatile element for creation of the multi-functional and adaptive building envelope. It is constructed of cross-laminated timber lintels and studs with glued-in rod joint fasteners and laminated glass infill. Due to CLT frame, the hybrid component can be easily connected with surrounding structural elements made of any structural material by semi-rigid ductile steel fasteners or other types of fasteners that can be fixed to CLT frame. The laminated glass infill is placed inside the timber frame in the way that can carry vertical and lateral load, provides high energy dissipation along the glass-to-timber contact and does not influence structural ductility of surrounding timber frame.

Full-scale specimens were tested by combined constant vertical load and displacement controlled cyclic horizontal load (racking load). Several series of equal specimens were tested varying boundary conditions (shear cantilever, shear wall) and intensity of vertical load. Both vertically loaded by assumed dead load of upper structure and vertical load equal to weight of panel were applied. The case of vertical load equal to own weight corresponds to cases of installing of panels in existing frame structure. Specimens were repaired after attaining large deformations and damages of frame joints and retested.

The purpose of racking test was to obtain data for development of computational model of tested type of structural element that can be used for prediction of inelastic response of buildings with the glass-infilled CLT frames on seismic action. In the paper the results of testing are presented in form of hysteresis loops and backbone curves. Data on stiffness degradation and energy dissipation is given in quantitative way as well as in diagrams showing the changing of those parameters with increasing of horizontal displacements during testing

Keywords: timber-glass panel, racking test, hysteretic response, energy dissipation, ductility

1. Introduction

Laminated glass in construction sector created an opportunity for development of hybrid structural elements. Although the glass has been long time considered as a brittle material that is not welcome in earthquake prone areas, new technologies offered the applicable solutions. Different types of polymer interlayers offer different levels of safety, but in general the currently available products offer the adequate safety.

The need and introduction of innovative structural elements is running beyond the existing code guidance and influences their further enhancement. Development of new European codes for use of structural glass in civil engineering started in 2007 [1]. Currently it progresses within CEN TC250/WG3 [2]. In the same time further development of Eurocode 5 – Design of timber structures will, assumable bring more attention to new types of timber connectors. However, the new generation of Eurocode 5 will also need to properly address the cross-laminated timber (CLT) to provide standardized guidance on design and use of this product. Eurocode 8 does not cover the emerging laminated materials [3]. Therefore, the future cooperation between working groups dealing with new structural glass codes and new generation of timber structure design will be necessary for upgrade of Eurocode 8, as well.

The research presented in this paper is a collaborative research of University of Zagreb and University of Ljubljana. Research has been launched in order to contribute to above-discussed development of codes since the subject of investigation is a hybrid structural element. It is made of cross-laminated timber lintels and studs with glued-in rod joint fasteners and laminated glass infill. This type of structural element is already addressed in the "Guidance for European Structural Design of Glass Components" [2]. The hybrid element may be used as an independent load-bearing panel for construction of prefabricated timber structures, as a temporary or even permanent support and bracing element in heritage buildings, as an external or internal bracing element in retrofitted or newly built frame structures at earthquake prone areas or as a versatile element for creation of the multi-functional and adaptive building envelope [4].

Due to CLT frame, the hybrid element can be easily connected with surrounding structural elements made of any structural material by semi-rigid ductile steel fasteners or other types of fasteners that can be fixed to CLT frame. The laminated glass infill is placed inside the timber frame in the way that can carry vertical and lateral load, provides high energy dissipation along the glass-to-timber contact and does not block high structural ductility of surrounding timber frame.

The research progressed in several phases. The basic configuration of specimens (frame dimension, type of laminated glass) and test protocol were equal in all phases of research. The difference was in type of timber (glulam and CLT) and in configuration of timber frame joints [5]. Beside the cyclic testing of panels in laboratory of University o Ljubljana, the full scale box type structure was tested on shaking table of Institute of Earthquake Engineering and Engineering Seismology (IZIIS) in Skopje, Macedonia [6]. The main purpose of shaking table tests was to verify the cyclic tests by means of comparison of all over behavior and development of damages at ultimate loading.

In this paper the results of the recent series of testing are presented. Since the detailed analysis of experimental results and development of computational model is still under progress only the main results are presented. The main purpose of herein reported investigation was comparison of behavior of vertically loaded panels and those loaded only with own weight. The first ones represent the load bearing panel in timber building while the second ones are representing the panels installed in existing or new built frame structure where vertical load is carried by frame structure. As it is presented in the paper, higher vertical load increases the energy dissipation capacity due to higher rate of friction between glass sheet and timber elements. Therefore, in practice is possible to induce the additional vertical load in panels by restressing or specially designed fasteners. The presented results can serve for estimation of influence of different vertical loads in range between those applied in presented cases. Both, timber frames with single glass sheeting and double glass sheeting have been tested and to examine their load bearing characteristics and hysteretic response. In practice, double glazed panels are suitable for bracing of the external parts of frame structure while the single glazed panels might be used for internal parts of structure.

2. Program of investigation

As mentioned above, in order to examine load-bearing characteristics of elements used for different purposes the extensive research program has been launched. In the first phase different bolted timber frame joints were examined [5]. Two bare timber frames and twelve timber frames with double laminated glass infill were tested. Six CLT and eight GLULAM timber frames and three types of bolted frame joints were varied. Specimens were tested in set-up that enables applying of three different boundary conditions.

Learning from behavior of the first series of specimens the improved ones were constructed for further examinations. It has been decided to test only CLT timber frames with glued-in rod fasteners used in frame joints. Infill was made in double or single glazing configuration. Each glass sheet was 20 mm thick and made of two-ply semi-tempered laminated glass with EVA interlayer. The length of infill was 2900 mm and height 2400 mm. The dimensions of frame were 3220 mm in length, 2720 mm in height and in cross-section b/h = 90/160 mm. The steel glued-in rods were 10 mm in diameter with exception of rods in two specimens where they were 14 mm in diameter.

In Table 1 below the list of recent series of specimens is presented. Specimens were tested by combined constant vertical load of 25 kN/m' (estimated loading by four stories of timber building above the panel) or vertical load equal to their weight and cyclic horizontal load (racking load) up to reaching of serious damages in joints. Eleven out of twenty-three specimens were repaired and retested. Simple method of repairing has been used. In addition, two repaired specimens that were previously tested in-plane were tested out-of-plane. Along each side of upper and lower lintel steel threaded rod 8 mm in diameter was placed and tightened on both ends by nut over 10 mm thick steel plate. By tightening of rod the timber frame was returned to original geometry (Fig.2).

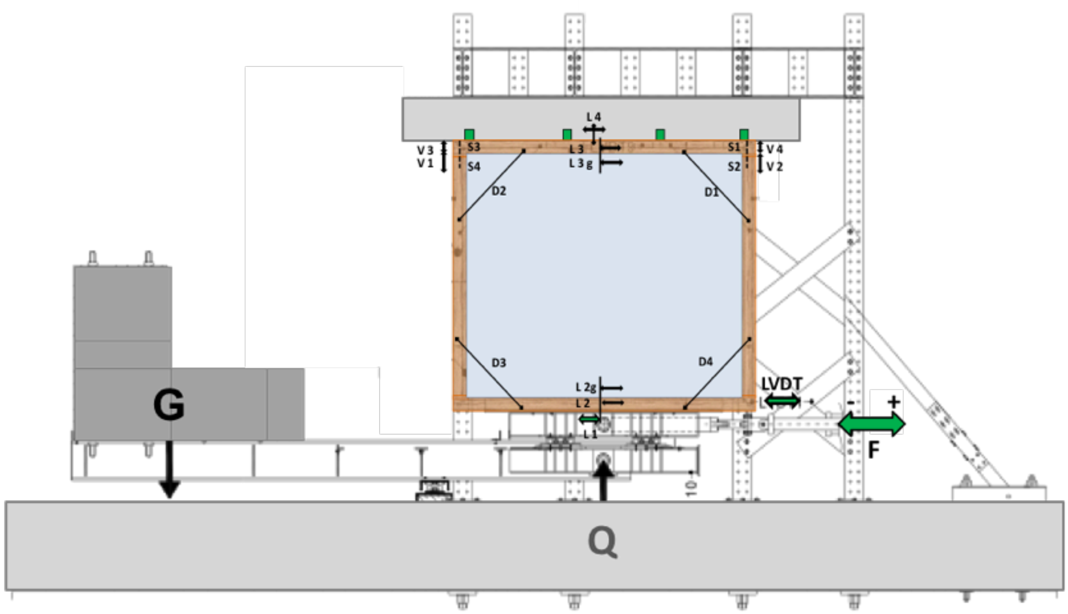


Fig. 1 – Racking test set-up

The test-setup (Fig. 1) has been constructed for testing of panels by simulation of three different boundary conditions:

1. **Shear cantilever:** one of horizontal edges of panel is supported by the firm base while the other can freely translate and rotate
2. **Constrained rocking:** one of horizontal edges of panel is supported by the firm base, the other can translate and rotate as much as allowed by the ballast; ballast can translate only vertically without rotation.
3. **Shear wall**: one of horizontal edges of panel is supported by the firm base while the other can translate only in parallel with the other edge while it's rotation is fully constrained

Learning from the test results of the initial series of investigation behavior of specimens exposed to the second and third above described boundary condition did not significantly defer. Therefore, only the first and second boundary conditions were applied in the here reported testing. The response of the grey-shaded specimens in table below is discussed in the continuation of paper because they are considered as typical cases from which the main characteristics of here in reported structural elements could be derived.

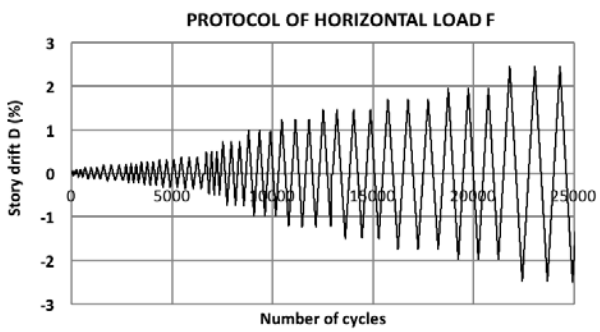
Table 1 – List of tested specimens

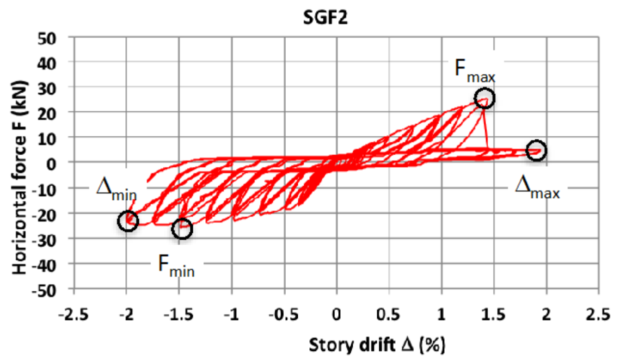
Name	**Horizontal load**				**Boundary condition**		**Vertical load Q**	
	Original specimens		**Repaired specimens**					
	Monotone	**Cyclic**	**Monotone**	**Cyclic**	**1**	**2**	**Panel weight**	**80kN (25kN/m')**
Single glazed – in-plane								
SGF1	X				X		X	
SGF1S				X	X		X	
SGF2		X			X		X	
SGF2S				X	X		X	
SGF3	X					X	X	
SGF3S				X		X	X	
SGF4		**X**				**X**	**X**	
SGF4S				**X**		**X**	**X**	
SGF5		X			X			X
SGF5S				X	X			X
SGF6		**X**				**X**		**X**
SGF6S				**X**		**X**		**X**
Double glazed – in-plane								
DGF1	X				X		X	
DGF1S				X	X		X	
DGF2		X			X		X	
DGF2S				X	X		X	
DGF3	X					X	X	
DGF3S				X		X	X	
DGF4		**X**				**X**	**X**	
DGF4S				**X**		**X**	**X**	
DGF5		X			X			X
DGF6*		**X**				**X**		**X**
DGF6S*		**X**				**X**		**X**
Double glazed – out-of-plane								
DGF3-S						**X**	**X**	
DGF4-S				**X**		**X**	**X**	

3. In-plane cycling response

In the following Table 2 are presented the load protocol diagram and key data defining the load bearing capacity and maximal story drift attained during the testing until failure of timber joints.

Table 2 – Load protocol, definition of hysteresis extreme points and values of extreme points





Name	Extreme points in negative direction		Extreme points in positive direction	
	Δ_{min} (%)	F_{min} (kN)	Δ_{max} (%)	F_{max} (kN)
Single glazed - in-plane				
SGF1	-1,97	-15,74	1,97	25,93
SGF1-S	-1,72	-16,19	1,73	20,33
SGF2	-2,00	-25,79	1,94	25,33
SGF2-S	-1,78	-34,75	1,77	8,64
SGF3	-2,08	-24,44	1,97	23,98
SGF3-S	-1,72	-26,24	1,73	17,41
SGF4	**-1,48**	**-23,40**	**1,48**	**26,82**
SGF4-S	**-1,73**	**-19,73**	**1,72**	**31,95**
SGF5	-1,48	-32,26	1,48	40,64
SGF5-S	-0,98	-51,71	0,98	48,80
SGF6	**-1,48**	**- 45,76**	**1,48**	**38,06**
SGF6-S	**-1,23**	**-52,48**	**1,23**	**54,97**
Double glazed – in-plane				
DGF1	-1,67	-21,02	2,10	29,98
DGF1-S	-1,73	-52,48	1,73	54,97
DGF2	-1,72	-24,39	1,72	19,30
DGF2-S	-1,48	-24,78	1,48	23,51
DGF3	-1,83	-14,07	1,84	27,27
DGF3-S	-1,98	-15,66	1,97	43,65
DGF4	**-2,47**	**-34,82**	**2,47**	**20,64**
DGF4-S	**-2,46**	**-39,60**	**2,47**	**27,37**
DGF5	-2,47	-44,72	2,47	46,77
DGF6*	**-2,47**	**-50,37**	**2,47**	**49,83**
DGF6-S*	**-1,72**	**-51,29**	**1,72**	**54,75**

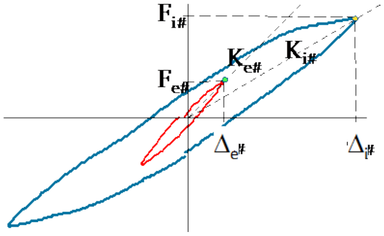
As it is presented in table 2 above, the load bearing capacity and ductility of panels are influenced by the level of constant vertical load. In the cases of vertical load equal to own weight the load bearing capacity is lower while the story drift capacity is higher than in cases with additional external vertical load. Mechanism of timber joint response to horizontal load determined the all over hysteretic behavior of tested panels. Joints in original configuration were flexible and rotated and slide freely around the glued-in rod in center of stud cross section. The repaired joints were stiffened by steel plates and stud-to-lintel relative displacement was prevented. However, in both configurations the integrity of panel was governed by glass infills that carried horizontal load and dissipated the energy by friction along the glass to timber contact surfaces. Therefore, the load capacity of panels was both higher due to additional vertical load and higher in the case of double glazing in comparison to single glazing.

During testing the load was transferred from the lower lintel via vertical studs of to the upper, fixed lintel by the glued-in rods. The rods were glued in the central layer of the CLT stud and beam in the manner that the horizontal force act perpendicular to the grain of the central layer and the axis of the glued-in rod. During cyclic loading rods compressed surrounding wood. When they reached ultimate embedment strength of horizontal timber elements (i.e. lintels) the shear failure occurred along the surfaces of inner layer of CLT lintel (Fig. 2 (left)) in direction of free edge. Failure mechanism and behavior of repaired specimens was different from behavior of original specimens. The load-bearing role of glued-in rods was replaced by load-bearing horizontal steel ties anchored in steel plates placed at the ends of horizontal frame elements (i.e. lintels). The ultimate load bearing capacity has been achieved at reaching the ultimate compression strength of wood perpendicular to the grain Failure has occurred by squeezing of the wood in the contact with anchor steel plate (Fig. 2 (right)).

Fig. 2 – Damages of frame joints at ultimate displacement: original (left) and strengthened joint (right)

Hysteretic response of tested specimens contents the information on ductility of structural element, deterioration of strength due to repeating of horizontal load to equal displacement, cycle to cycle stiffness degradation and energy dissipation to viscous damping of tested structure that passes different stages of gradual damaging of its parts. Tested type of structural element is highly dissipative, where the most significant amount of dissipation is caused by glass to wood interaction. Part of dissipation is also caused by glued-in joint rods and in some extends also by plastic deformations of frame anchoring elements to concrete base. Evaluation of stiffness degradation is explained in Fig. 3 and the evaluation of viscous damping in Fig. 4. The specimens containing “S” in their name (see diagrams below) are the repaired and retested specimens. For instance, specimen entitled SGF4 is renamed to SGF4-S after repairing and strengthening (see Fig. 2 (right)).

One of the main parameters describing the hysteresis response of structure is stiffness degradation. It can be calculated from the stiffness of the chosen loop and effective stiffness of the specimen in the early elastic stage. In the case of tested structural elements, the effective stiffness (Ke) was calculated from the inclination of the several first hysteresis loops in the elastic range of response (Fig. 3). From the coordinates of the subsequent hysteresis turning points (δi, Fi) the inclination (stiffness Ki) of corresponding loops was calculated. The diagram of stiffness degradation can be mathematically defined by Eq. (1) below, where the parameter Ck is named “stiffness degradation factor”. Since it’s values are calculated from hysteresis response of tested specimens, it can be considered as their own, unique characteristics. Ck is very useful parameter that can be well employed in process of validation of computational models, when the experimentally obtained and calculated hysteresis responses are compared

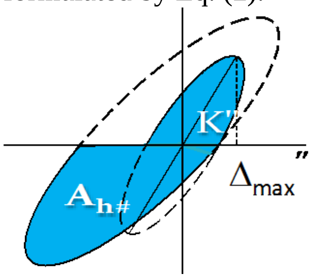


$$K_i = \frac{F_i}{\Delta_i} \qquad K_e = \frac{F_e}{\Delta_e} \qquad \frac{K_i}{K_e} = \frac{C_k}{\frac{\Delta_i}{\Delta_e}} \tag{1}$$

K_e = effective stiffness, K_i = stiffness of corresponding loop,
C_k = factor of stiffness degradation

Fig. 3 – Definition of the stiffness degradation factor C_k

Another important parameter that quantifies hysteresis response of structural elements is the equivalent coefficient of viscous damping ξ, which can be calculated from hysteresis response as explained below and formulated by Eq. (2).



$$\xi = \frac{A_h}{4 \cdot \pi \cdot K \cdot \Delta_{max}^2} \tag{2}$$

ξ□ = equivalent coefficient of viscous damping
A_h = surface of the corresponding hysteresis loop
Δ_{max} = maximal displacement of the corresponding loop

Fig. 4 – Definition of the equivalent coefficient of viscous damping ξ

The response mechanism of tested specimens depends on coupling of two load-bearing mechanisms. From the response diagrams two phases of behavior can be distinguished: the first phase until failure of frame joints and the second, post failure phase. In the first phase of behavior, the horizontal loading is carried both by in-glued rods in frame joints and by friction between the glass panels and frame elements. The vertical load is carried by timber struts and by glass panels. After joint, the vertical loading partly redistributes from studs to glass sheets what increase their contribution in carrying of horizontal load by glass-to-wood friction. Until failure of joints the load-bearing capacity is gradually increasing from cycle to cycle depending of the level of applied displacements. After failure the load bearing capacity depends mostly on friction and it is practically constant regardless of the level of applied displacements until the achievement of large deformations of specimen.

In Fig.5 are compared hysteretic responses of four tested specimens and curves derived from them. All specimens are composed of timber frame and single two-leaf glass laminated sheet. Two of them were tested in their original configuration (SGF4 and SGF6) and retested after repair and strengthening (SGF4-S and SGF6-S).

Hysteresis curves and other diagrams of original and repaired specimen are compared in the same graphs to clearly present the differences of behavior. The left side specimens were tested with vertical load equal to their own weight while the right side specimens were loaded with the external load of 25kN/m' as it is shown in Table 1 above. On the first sight the influence of vertical load on hysteretic response is visible. As mentioned earlier and presented in Table 2, the load bearing capacity of specimens with higher vertical load is significantly higher while the story drifts at the failure of original joints are practically equal. The interesting outcome of investigation is that after joint failure panels can be easily and inexpensively repaired and strengthened by steel plates simply attached to external surfaces of joints and fixed by steel rods. The repair method is interesting for real post-earthquake interventions because panels can serve their purpose after it. Moreover, as test results presented in Fig.5 shows, repaired panels have higher load bearing capacity than original ones without reduction in their ductility. The clear differences in load bearing properties and ductility are seen from the hysteresis back bone curves. While in the case of specimen with lower vertical load the initial stiffness of original panel is higher than stiffness of repaired one, in case of higher vertical load situation is reverse with slightly lover ductility of repaired panel.

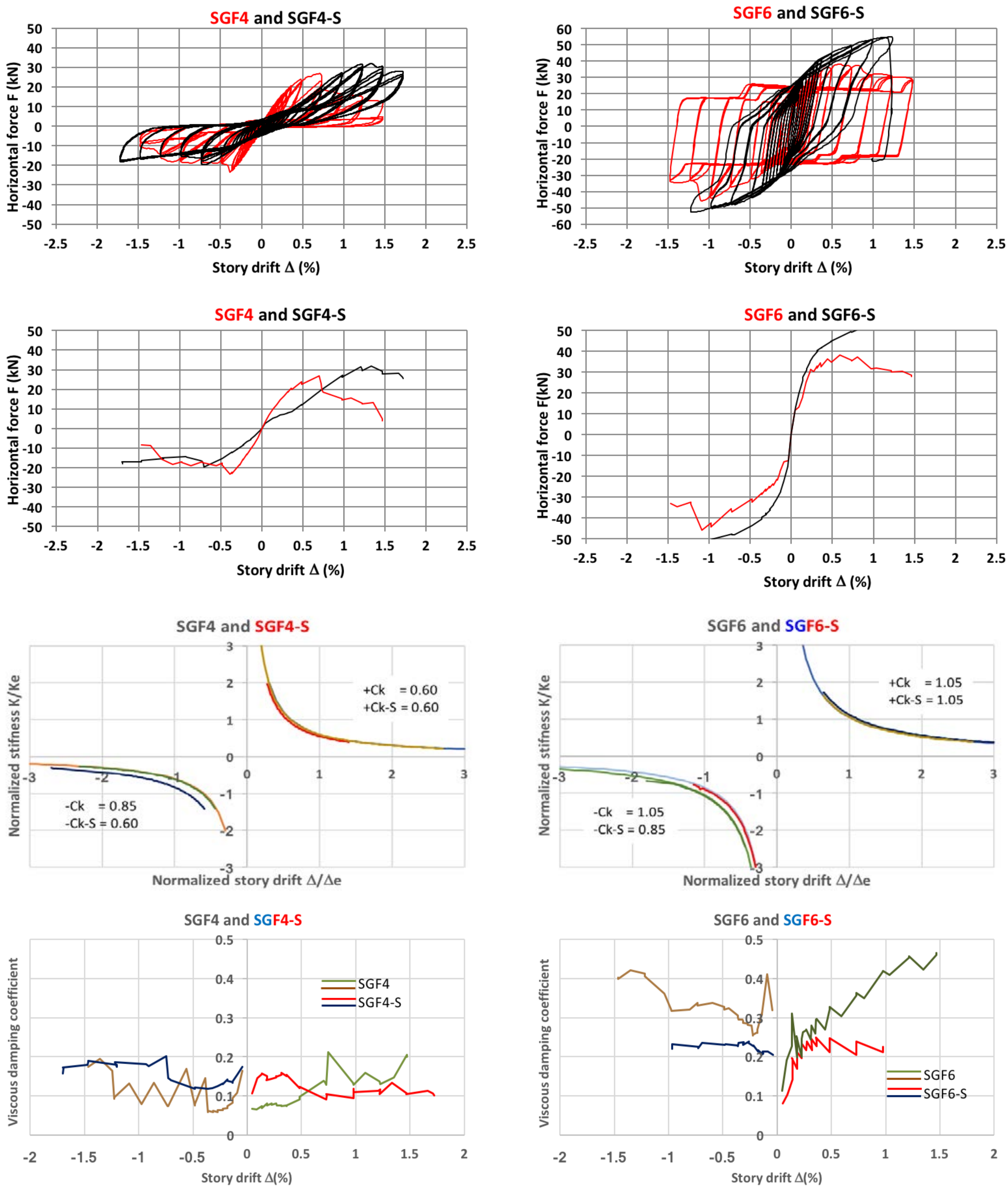


Fig. 5 – Hysteresis, backbone curves, stiffness degradation and viscous damping diagrams of in-plane tested single glazed cross-laminated timber frames

Described differences in behavior of panels can be, at least partly, attributed to influence of higher rate of friction between glass sheets and surrounding timber. The deterioration of stiffness from cycle to cycle as defined in Fig.3 is in all cases in the “positive” direction of loading equal in cases of original and repaired specimens and slightly different in “negative” direction of loading which reversely follows the “positive” one. Lover value of parameter Ck is associated with higher rate of stiffness degradation what is observed in the case of specimens carrying the lower vertical load.

Comparing diagrams at the bottom row of Fig.5 can be concluded that the viscous damping of original and repaired specimen vertically loaded only with its own weight is in the close range of values, while in case of vertically higher loaded specimens the differences between viscous damping of original and repaired panel are much higher. In general, the intensity of vertical load directly influences the higher rate of viscous damping due to higher friction between glass and timber. What also can be seen from diagrams is, that the original configuration of joint makes panels more energy dissipative than the configuration of repaired joint.

In Fig.6 are compared hysteretic responses of another four tested specimens and curves derived from them. All specimens are composed of timber frame and double two-leaf glass laminated sheet. Two of them were tested in their original configuration (DGF4 and DGF6*) and retested after repair and strengthening (DGF4-S and DGF6-S*).

Likewise, in Fig. 5, the hysteresis curves and other diagrams of original and repaired specimen are compared in the same graphs to clearly present the differences of behavior. Specimens DFG6* and DFG6-S* have different configuration of original joint because in the frame joints of DGF4 and DGF4-S are in-glued rods 10 mm in diameter while in other two specimens the rods are 14mm in diameter. As it can be seen from comparison of values of extreme points of specimen DGF5 (10 mm rod) and DFG6* (14 mm rod) the difference in load bearing capacity is 14%, while the ultimate story drifts were equal. The difference is load bearing capacity is the consequence of approximately 30% lower contact rod-to-wood stresses what resulted in higher lateral force needed for failure of joint.

Some observations from comparison of test results presented in Fig.5 and Fig.6 are similar, but the main differences in panel behavior are due to double glazing of frames. The impact of higher vertical load on load bearing capacity is much higher in case of double glazing comparing to single glazing. It can be easily explained by acting of double glass edge surfaces in friction. However, this did not influence the value of ultimate story drift. The effect of repair and strengthening was similar in the cases of single and double glazing. In both cases the ultimate story drift reached in cases of low vertical load was equal, while in the cases of high vertical load the repaired specimens failed at ultimate story drifts 23 to 30% lower than were story drifts of original panels (see Table 2 above).

Comparison of hysteresis backbone curves illustrates the differences in response of tested specimens. The curves show the amount of load bearing deterioration at repeating of deformations (three cycles to the selected displacement). The deterioration was insignificant until reaching the ultimate horizontal load. However, the load bearing capacity and stiffness of specimens is also influenced by material properties on local level especially in the frame joints. Therefore, it is difficult to distinguish the influence of structural properties (single vs. double glazing, dimension of in-glued rods) from influence of local wood properties especially when the differences are in the range of 10%. Therefore, additional information is needed to understand the mechanisms of behavior and its influence on structural characteristics of tested type of structural element.

The diagrams in the third row of Fig.6 shows that the stiffness degradation, similarly as in the cases of single glazed frames, did not highly differ in the cases of original and repaired panels. Higher difference was observed between vertically less and more loaded specimens, although it was lower than in the case of single glazed panels.

As it was observed in the Fig.5, the vertical load strongly influences the energy dissipation expressed in term of equivalent damping coefficient. The same was the case in panels compared in Fig.6. However, in case of panels DGF6* and DGF6-S* double glazing additionally contributed to energy dissipation.

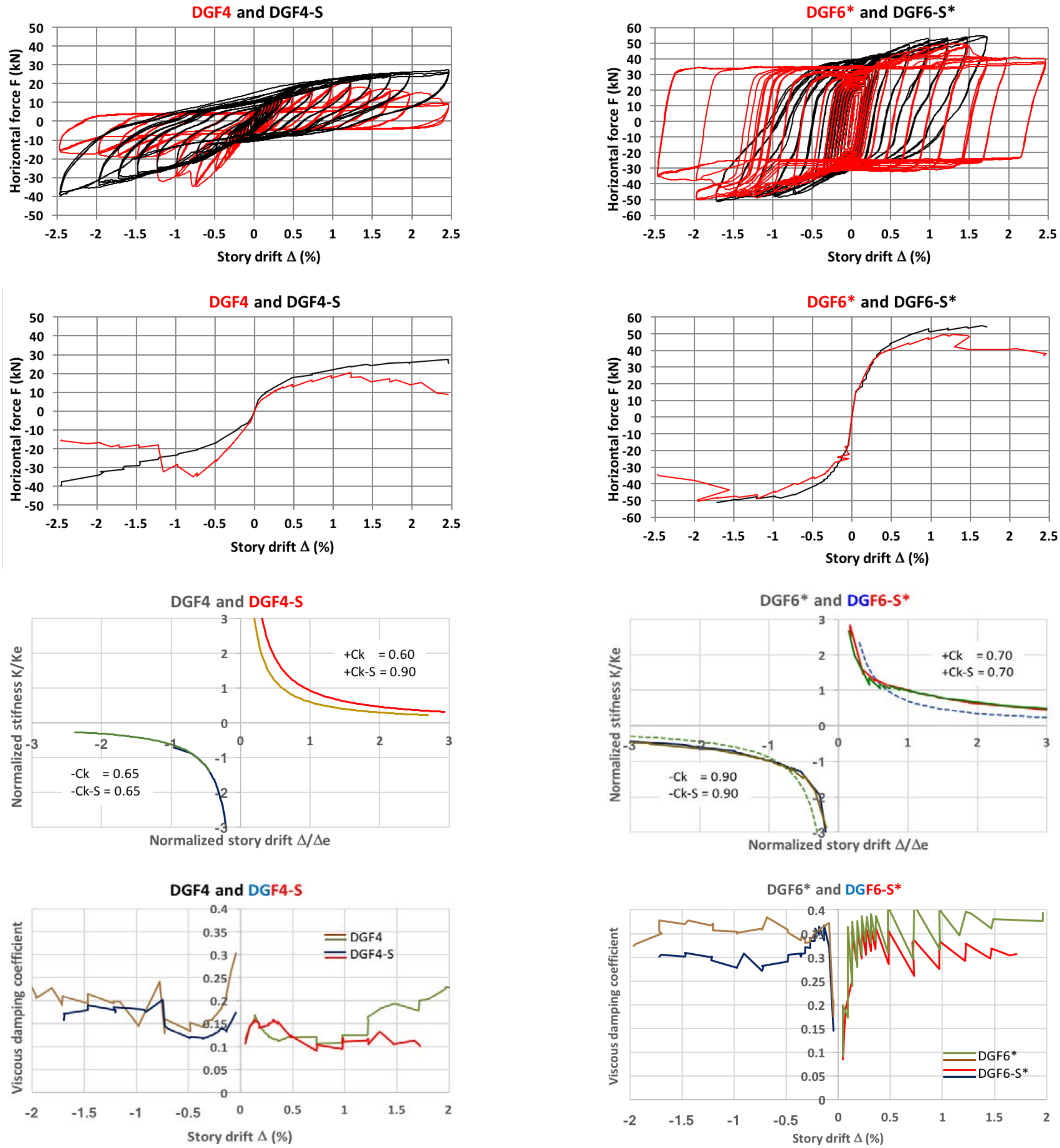


Fig. 6 – Hysteresis, backbone curves, stiffness degradation and viscous damping diagrams of in-plane tested double glazed cross-laminated timber frames

4. Out-of-plane cycling response

In order to examine out-of-plain behavior of panel integrated in flexible frame system two specimens were tested as presented in Fig.7. The out-of-plane tested double glazed specimens DGF3-S and DGF4-S were previously tested by lateral load in original and repaired configuration.

The out-of-plane hysteretic responses of tested specimens are presented in Fig.7 below. The ultimate story drifts were 2,4 and 3 times higher than in cases of lateral testing (see data in Table 2 and Table 3). In both cases the large displacements were achieved without development of damages that would influence integrity or stability of tested panels. It means that in the case of using glass infilled timber frames as bracing elements the in-plain loaded elements will govern the behavior of structural system. Story drifts of frame structural systems will be much lower than drifts which panels are able sustain out of their planes. But it has to be clearly stated that out-of-plane specimens were not vertically loaded and therefore the second order effect was not a case.

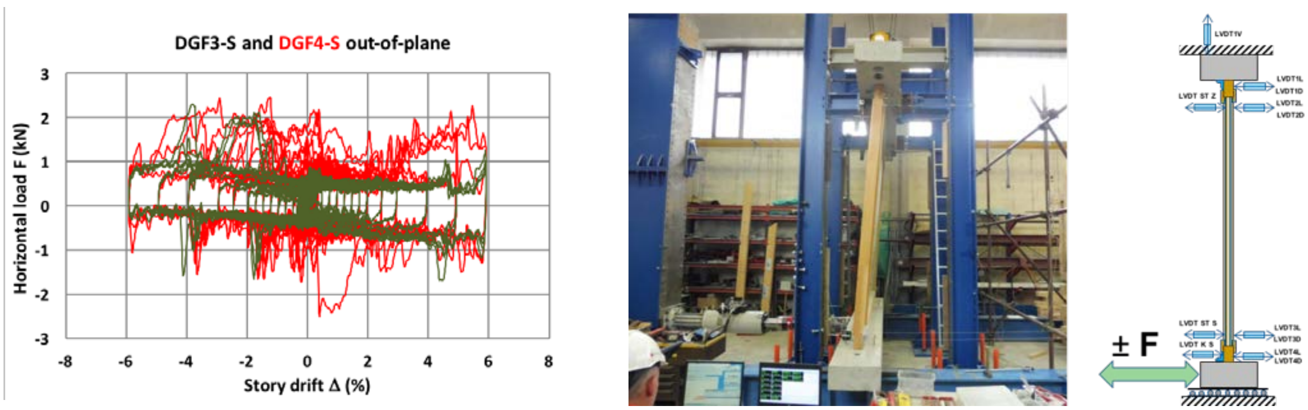


Fig. 7 – Hysteresis curves and test setup for out-of-plane testing of repaired double glazed cross-laminated timber frames

Table 3 – Hysteresis extreme points of out-of-plane tested repaired double glazed cross-laminated timber frames

Name	**Extreme points in negative direction**		**Extreme points in positive direction**	
	Δ_{min} (%)	**F_{min} (kN)**	**Δ_{max} (%)**	**F_{max} (kN)**
Double glazed – out-of-plane				
DGF3-S	-5,92	-2,50	5,92	2,45
DGF4-S	-5,92	-1,71	5,92	2,30

5. Conclusion

Glass is widely used in design of contemporary timber structures, but not yet in large extend as heavy load bearing material. One of reasons is in current costly or relatively complicated application in buildings as well as due to lack of codes. In order to contribute to wider introduction of structural glass in construction sector, the CEN/TC250 launched the preparation of Eurocodes for use of structural glass in buildings. In scope of this trend researchers from University of Zagreb and University of Ljubljana joint their efforts to develop a universal structural element that could be used in construction of new structures as well as an element for strengthening of existing structures including heritage buildings. The element is composed of CLT frame with glued-in rods in joints and filled with single or double-glazing made of semi-toughened two-ply laminated glass. Glass is installed in frame in way that during horizontal loading dissipates energy by glass-to-wood friction. Such an element is produced in factory and installed on site by steel connectors between CLT frame and surrounding structural elements made of any structural material.

The analysis of results of relatively extensive testing programme lead to the understanding of behavior of timber frames with laminated glass infills and open the possibility for near future development of design tools in form of simple formulas to be used in the future European codes and in the form of software for assessment of earthquake response of timber panel structures with load bearing glass-timber panels. The here presented results will be also a basis for development of mathematical models to be used in the commercial software for structural analysis and prediction of earthquake response of frame structures with added glass-timber panels as bracing and dissipative structural elements.

6. Acknowledgements

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