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LATERAL STABILITY OF VERANDAS BY MEANS OF THE GLASS PANELS

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Abstract: In recent years, verandas have become increasingly larger and are more often built separately from the main building. Hence, the stability of windows submitted to horizontal loading has become a problem. In this paper, a new fabrication concept is tested. It uses support blocks ensuring a uniformly distributed pressure zone contributing to a redistribution of the horizontal load in the window's plane and an increase of the whole system's stiffness. On the basis of separate tests on the connection elements, on the support blocks and on the complete frames, the stiffness of a window was evaluated. The possibility of using this concept to ensure the lateral stability of verandas was then investigated using a finite element model.

1 Introduction

Verandas are an important part of the Belgian building culture. In the past, verandas were relatively small constructions made of three façades and a roof, attached to the main building. These constructions were made by a limited number of specialized firms, and built on the base of expertise more than on proper design rules. In the past decade, verandas became increasingly larger and, more importantly, they became constructions independent of the main building. Therefore, the overall stability against horizontal loading turned out to be a problem. The size of verandas and the aesthetic aspects imply that new concepts are needed to ensure the stability without visual changes.

2 General concept

In Belgium, the placement of glass in aluminium frames is prescribed in TV 221 [1]. The placement method is a dry method, where wooden or plastic support blocks distribute the weight of the glass onto the aluminium frame, Fig. 1. In this research [2], these blocks are placed in each of the four corners. In this way, a compression diagonal can be activated in the glass panel to transmit the horizontal load. To reduce the stresses in the corners, Fig. 2, the support blocks are placed at a distance equal to their own length.

This research concentrates on the instability phenomenon of the glass sheet and the lateral stiffness of the whole system composed of the aluminium frame, the support blocks and the glass panel.

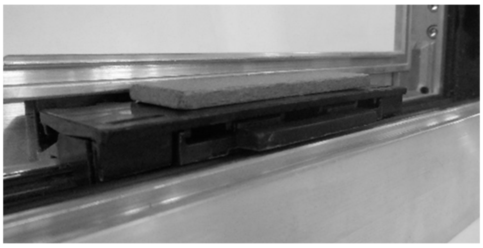


Fig. 1: Support block on aluminium profile

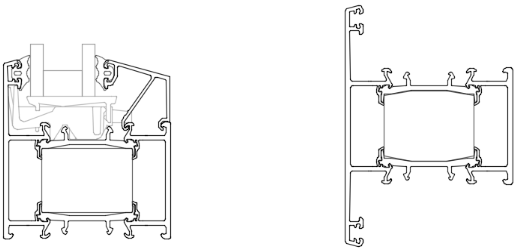


Fig. 3: Cross-section aluminium profiles

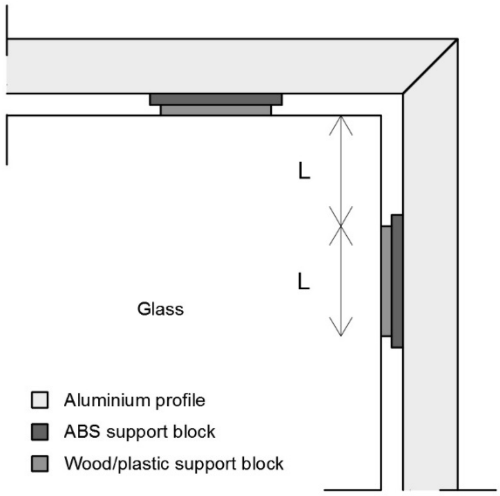


Fig. 2: Placement of the support blocks

3 Stability of the glass panels

Belgian rules for the use of glass sheets are listed in NBN 23-002 [3]. The rules are based on technical documents edited by producers and research institutes such as BBRI (Belgian Building Research Institute), [1] and [4]. In [5], the BBRI describe the rules included in Eurocode 0, 1, 3, 5 and 9 that are applicable for joinery.

Glass can be placed single, double or even triple and each panel can be laminated, tempered, half tempered or float glass. The buckling behaviour of laminated glass panels can be assessed using the equivalent thickness method [5]. Using Eq. (1), the equivalent thickness for out of plane deformations can be calculated for laminated glass sheet, where the transmission coefficient ϖ is an indication of the shear capacity of the interlayer: 0 stands for two independent panels and 1 stands for perfectly connected panels. Research on the shear capacity of different interlayers can be found in [6] and [7].

$$h_{ef,w} = \sqrt[3]{(1-\varpi)\ x\ \sum_i h_i^3 + \varpi\ x\ \left(\sum_i h_i\right)^3} \qquad (1)$$

The stability of glass panels in case of pure compression or combined compression and shear can be found in [8] to [12]. However, in the present study, the load is concentrated at the level of the corners of the panel because of the presence of the blocks. Research on the behaviour of glass panels with concentrated loads is presented in [13]. In [14], [15] and [16], information on the contribution of the glass panel on the lateral stability of steel frames is provided.

The stability of glass panels for in-plane loading within this concept was studied using the finite element package Scia Engineer. First, a FE model was validated against the tests of Wellershof [13]. The relative difference between the model and the experimental results was lower than 4.6 %, on the conservative side. This FE model was then used to calibrate the reduction factors κ and ξ included in Euler's formula, Eq. (2).

This formula can only be used with the following boundary conditions:

1. The glass panel is supported by support blocks in each corner;
2. Only compression can be distributed between the glass and the support blocks;
3. The edges of the glass are supported perpendicular to the plane.

$$D_{crit} = \xi \, \frac{\pi^2 EI}{(\kappa L)^2} \tag{2}$$

In Eq. (2), D_{crit} is the resulting compression force in the diagonal, the moment of inertia around the weak axis I has to be calculated for the widest section perpendicular to the diagonal. For laminated glass, I can be calculated using $h_{ef,w}$. The critical buckling length is replaced by a fraction of the length of the diagonal L by means of a reduction factor κ, listed in Table 1. This factor is not constant but dependant on the width–to–length ratio of the glass panel. When the forces are perfectly located in the corners, it can approximate the numerically found buckling load with a relative difference lower than 1 %.

Table 1: Reduction factor κ

B/H ratio	**1.00**	**0.83**	**0.71**	**0.67**	**0.63**	**0.50**	**0.33**	**0.25**
Reduction κ	0.399	0.357	0.317	0.297	0.279	0.222	0.162	0.123

But, in this case, the forces are applied at a distance equal to 1.5 times the length of the support block, see Fig. 2. An additional reduction factor ξ for the critical buckling load has therefore been evaluated. This factor is depicted in Fig. 4 versus the relative position of the support blocks. For rectangular panels, the relative position is calculated using the shortest side. For example, when a support block of 100 mm is used, the resulting force will be acting at a level of 150 mm. When using a plate of 1 m wide and 2 m long, the relative position will thus equal to 15 %. For this plate, the width–to–length ratio is 0.5, κ is 0.222 and ξ is 0.85.

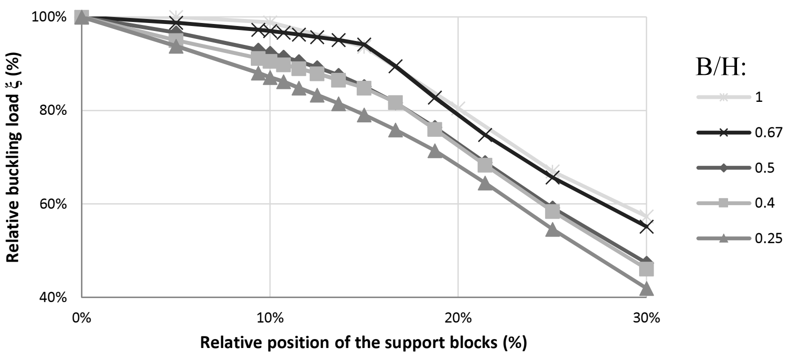


Fig. 4: Reduction factor ξ

The in-plane deformations of the glass panel are small. However, for panels with a width–to–length ratio smaller than 0.5, the deformations heavily increase. Very small width–to–length ratios should therefore be avoided because large deformations can lead to glass failure.

4 Materials characteristics and connections

The different materials that are used in the present research, are listed in Table 2. The cross-section of the aluminium profiles are depicted in Fig. 3. These sections are only used by Demasure, a Belgian fabricator. Most aluminium profiles used in windows will consist of closed box sections. As explained later, this will have a positive effect on the global stiffness.

The support blocks consist of two materials: a plastic block (reinforced Polyamide (PA) 6) and an additional block made of either wood or plastic. These blocks are available in different thicknesses. The wooden or plastic blocks have a maximum allowed stress in pure compression of 1.5 MPa [1].

Table 2: Overview of the materials characteristics

Material	Application	E-modulus (MPa)	Tensile strength (MPa)
Aluminium AW 6060 T66	Profiles	70000	215
Aluminium AC 46100D	Connections	70000	240
ABS	Insulating part	2340	51
PA6 15% glass fibre	Support block	3000	80

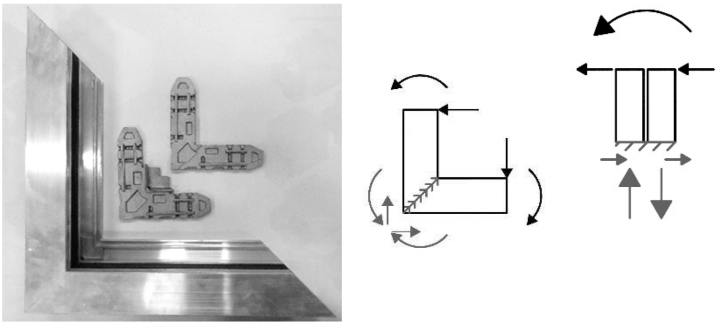


Fig. 5: L–connection

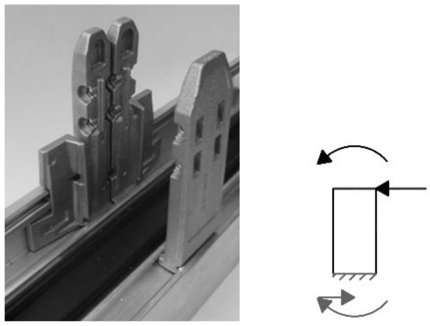


Fig. 6: T–connection

The elements connecting the aluminium profiles (denoted herein "connections") are made of casted aluminium (AC46100D). Because of their complex shape, Fig. 5 and Fig. 6, herein a simplification is made: the L–connections are considered as two rectangles and only loaded with a moment and a shear force. As shown in Fig. 6, the T–connection consists of 3 pieces, one that resists using a moment and the two others that work together as a couple. By use of basic resistance formulae, the stiffness and strength of the connections submitted to the forces depicted in Fig. 5 and Fig. 6 are calculated and the results are provided in Table 3.

Table 3: Stiffness and strength of the components

Component	Stiffness *H*	Ultimate moment (kNm)
T–connection	23.2 kNm/rad	0.38
L–connection	42 kNm/rad	0.80
Support (profile)	0.8 kN/mm	/

The support blocks are submitted to pure compression. Because they are supported by the flange of an open section, the stiffness can become a problem. This system is simplified as shown in Fig. 7. The unloaded part of the profile and the insulation will increase the calculated stiffness.

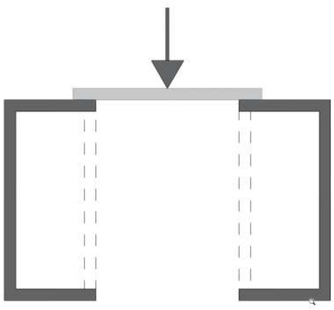


Fig. 7: Scheme support

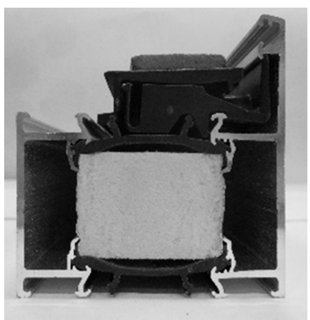


Fig. 8: Non-reinforced support

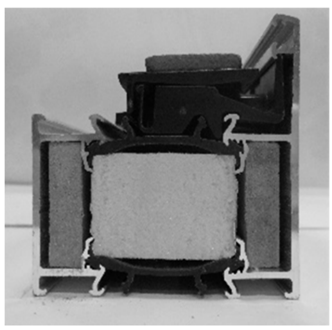


Fig. 9: Reinforced support

5 Test program

5.1 Corners

The stiffness of the AC 46100D connections is determined by a test according to NBN EN 514 [17] , Fig. 10, even though this standard controls the welds for plastic corners. One leg of the corner is clamped between two rigid profiles. The load is applied on the other leg. By using a load cell and LVDT's, the load and the corresponding displacements are measured. This procedure is used for L–connections and T–connections. Moreover, to investigate the effect of reinforcements on the whole structure, L–profiles (1 mm thick steel) were used, see Fig. 11. The test program consists of two non–reinforced and two reinforced specimens for each connection. Fig. 12 shows the moment–rotation curves for the T–connections, calculated with the centre of gravity of the fixed leg as reference. The connection failure mode involves fracture of the connecting piece at the level of the screws, as can be seen in Fig. 13. The ultimate loads equal 2.01 and 2.40 kN for the non-reinforced connections. The reinforced ones failed in the same way but at a higher load, 4.92 and 4.49 kN. For the finite element study, a trilinear model was fitted against the measured behaviour of the connections.

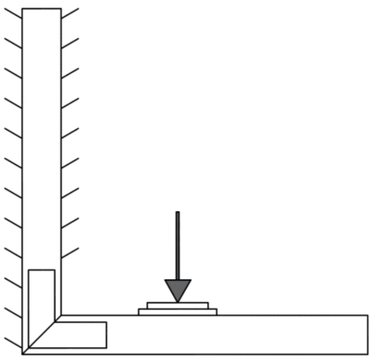


Fig. 10: Connection test

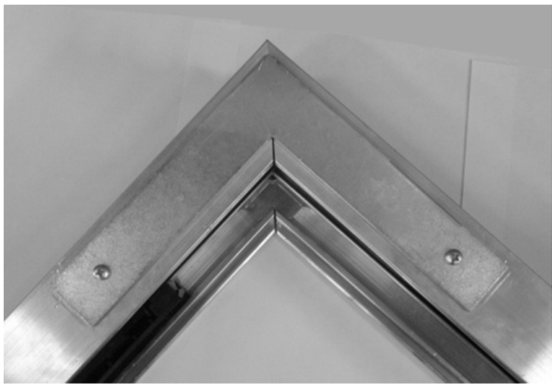


Fig. 11: L–reinforcement (for L and T–connections)

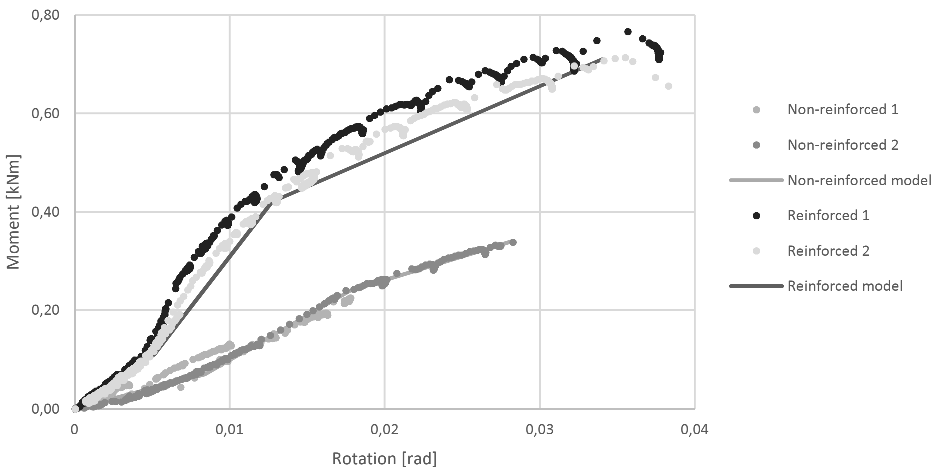


Fig. 12: Stiffness T–connection: Tests and model

The behaviour of the L–connection can be approximated by a bilinear course. The scatter on the results is small. The first crack originates in the inner corner at a moment of 0.50 kNm, Fig. 14. The reinforcements have a high influence on the stiffness with a factor 2 between the two connections.

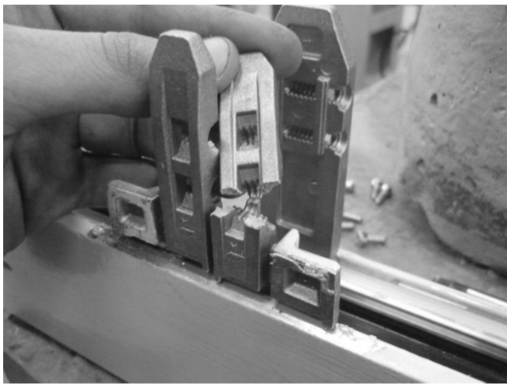


Fig. 13: Failure mode T–connection

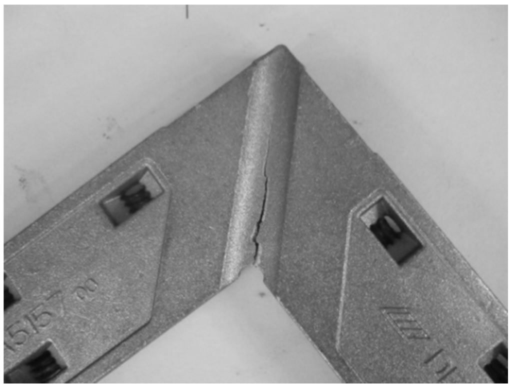


Fig. 14: Failure mode L–connection

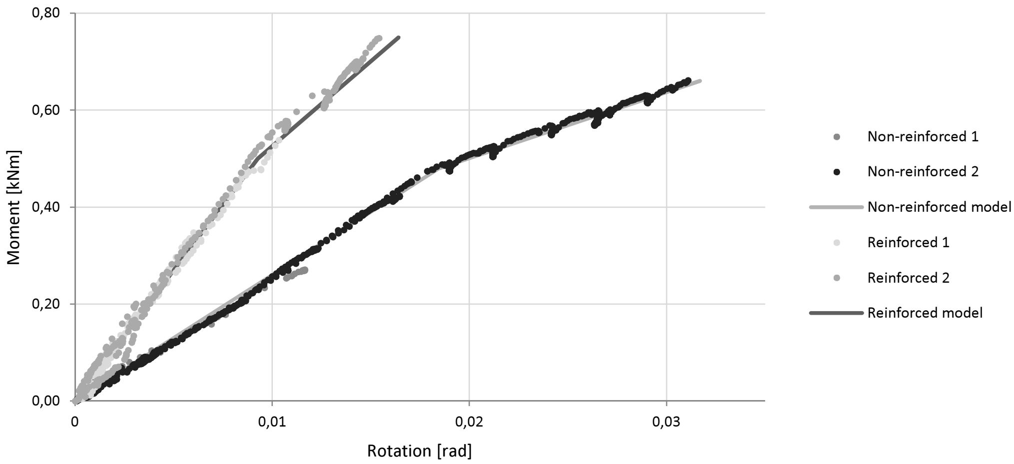


Fig. 15: Stiffness L–connection: Tests and model

5.2 Support blocks

The support blocks were also tested separately under pure compression. The main cause of deformation is not the support block itself but the combination of the support block with the aluminium profile lying underneath, Fig. 8. By applying concentrated loads on the support blocks, the profile will collapse as can be seen on Fig. 16. Using two closed box-sections (dotted lines in Fig. 7) can decrease the deformability of the whole. This effect was simulated using wooden blocks for the reinforced specimens, see Fig. 9. The tests showed an average stiffness of 1.28 kN/mm for the non-reinforced and 2.48 kN/mm for the reinforced supports. The failure load is on average 15.1 kN.

Fig. 16: Failure mode of the support blocks

5.3 Window frame

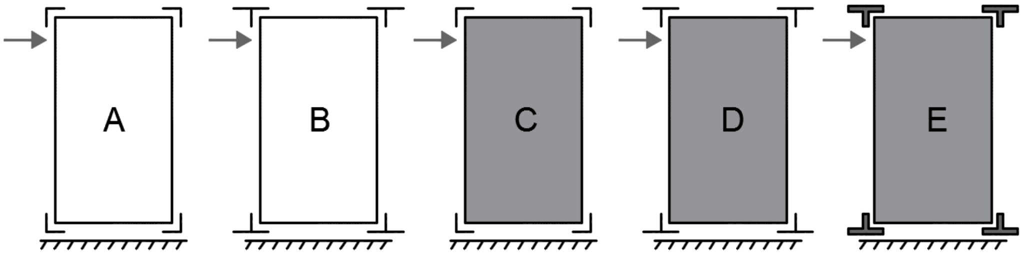


Fig. 17: Schematic overview of the window frames

During the tests, the lower element of the frame is fixed and the frame is stabilized laterally. On one top corner, the load is applied at the level of the support blocks. On the other side the displacements are measured on different heights. All tested frames are 1.1 m wide and 2 m long.

Firstly, two types of empty window frames were tested: one frame using L–connections (A) and one using T–connections (B), see Fig. 17. For empty frames the stiffness is depicted in Fig. 18.

Secondly, because glass and aluminium have the same Young's modulus, the frames were tested with two 5 mm thick aluminium panels as a replacement of glass for safety reasons. This is confirmed by performing both a test using an aluminium panel and a test using a glass panel. Both the reinforced (E) and non-reinforced (D) frames were tested for T–connection frame and, for reason of lack of aluminium frame, only the non-reinforced L–connection frame (C) was tested. All reinforcements for the frames consist of the ones used for the connections (Fig. 11) plus the wooden blocks placed under the support blocks (Fig. 9). The stiffness of filled frames is on average 8 times larger than the stiffness of empty frames.

It is worth pointing that, for all tests, the displacement perpendicular to the aluminium sheet (i.e. perpendicular to the window plane) was also measured to check that buckling of the infill does not occur in this range of load. The maximal measured displacement for all the frames was 2 mm and was lower than 1 mm for most of the tests.

Two frames were tested until failure. The failure occurs at the level of the internal connection components with a similar failure mode as the one occurring during the separate tests. Fracture takes place at a load equal to 5.6 and 6.6 kN for the reinforced T–frame and the non-reinforced L–frame respectively.

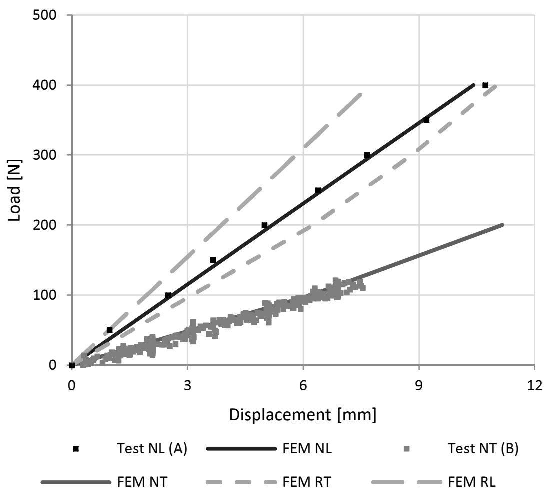


Fig. 18: Stiffness empty frames
N = Non-reinforced, R = Reinforced

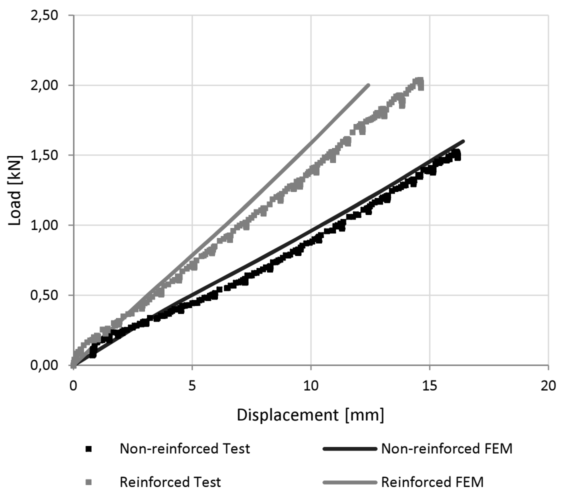


Fig. 19: Stiffness frame with T-connections

6 Finite element model

A finite element model was made using the SCIA Engineer software package. The materials characteristics as given in Table 2 were used. For the profiles, the aluminium cross-section is filled with insulation made of ABS, as shown in Fig. 3 and Fig. 8. For reason of simplification, the PIR-insulation in the centre was neglected. The connections were modelled using multi-linear springs placed between the profiles. The support blocks were simulated using bar elements with an extensional stiffness *EA/L* corresponding to the one of the blocks, hinged at both ends. The infill is made of 10 mm thick glass instead of two times 5 mm panels, since instability is presently not of concern. Like during the experiments, the bottom of the frame is fixed and the lateral stability of the panel and the frame is ensured.

Two models were made. The first one uses the theoretical values of the stiffness and strength of the connections and the second one uses the measured characteristics of the single connections. The second model clearly provided better results and is shown in Fig. 18 and Fig. 19.

The numerical failure loads are in the same range of the experimental ones, respectively 7 and 5.5 kN for the reinforced T–frame and the non-reinforced L–frame.

Using this model, the effect of different adjustments was investigated. In Fig. 20, the influence of the parameters on the behaviour of a 1.2 by 2 m frame with a double 5 mm glass panel is shown. For example, if the connection is fortified and reaches a stiffness of 400 kNm/rad instead of 20 kNm/rad, the total stiffness increases to 120 N/mm. However, if the support block compression resistance increases to 20 kN/mm, the total stiffness increases up to 338 N/mm. It would therefore be beneficial to the frame rigidity to enhance the compressive stiffness of the support blocks first. This can be done by using closed Aluminium profiles instead of open cross-sections. When both fortifications take place, a total stiffness of 513 N/mm is reached.

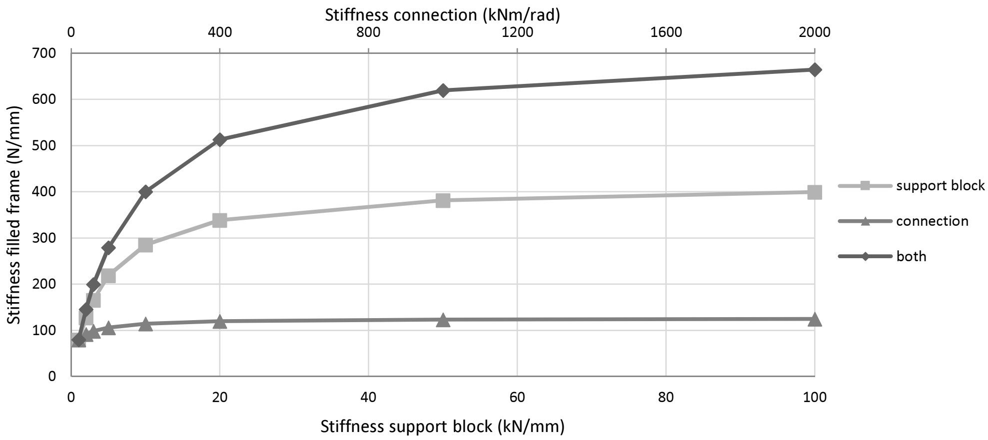


Fig. 20: Influence on the stiffness of two components on the stiffness of the filled frame

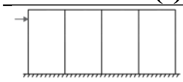
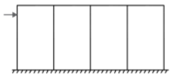
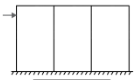
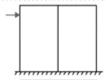
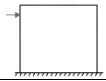
When multiple frames are placed next to each other, the summed stiffness of the separate frames is on average 5 % less stiff than the whole. For example, the stiffness of one frame with non-reinforced T–connections is 84 N/mm. Four frames together have however a stiffness of 350 N/mm. Hence, using a larger surface of windows improves the stiffness of the whole.

7 Conclusions

The main outcomes of this research are:

1. The infill highly improves the resistance to horizontal loads. Glass panels have a relatively high compression resistance and, hence, the instability (buckling) is the main restriction. However, this research shows that the critical buckling load of the window panels is greater than the applied loads as a result of the wind.
2. The in-plane deformations of the glass panel were checked with the same model. It can be concluded that for width-to-length ratios smaller than 0.5, the deformations of the glass panel exceeds multiple mm and this would lead to failure of the glass panel.
3. The stiffness of a standard frame with a glass panel is rather limited with 84 N/mm for T–connections and 101 N/mm for L–connections. Although, the stiffness can be increased by (1) using several frames next to each other; (2) increasing the stiffness of the connections and/or the support blocks. It was shown that improving the support blocks compressive behaviour (by using closed sections) should be the first action.
4. A veranda of 5 m x 5 m with a height of 2.5 m, has to resist a wind load of 1.4 kN/m² (Belgium). Therefore, the sidewall is loaded by a point load of 4.4 kN. A maximum displacement of 5 mm has to be ensured to secure the function of all components like doors and windows. Four non-reinforced T–frames are clearly not stiff enough, as can be seen in the first row of Table 4. This table gives in the other rows an overview of different options to secure the lateral stability of the veranda by using the concept described in this paper.

Table 4: Options to resist 4.4kN with a maximum displacement of 5mm

Frames (-)	**Size B x L (m x m)**	**Connection (kNm/rad)**	**Support block (kN/mm)**	**Displacement (mm)**
	1.1 x 2	12	1.3	12.18
	1,1 x 2	20	4	4.81
	1,1 x 2	60	5	4.94
	1,1 x 2	100	10	4.79
	2,2 x 2	100	5	5.01

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