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Structural contribution of glass in Saint-Hubertus Galleries

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Abstract

Many of the glass covered iron and steel frames from the nineteenth and twentieth centuries now require renovation. During rehabilitation, the question arises of how to preserve built heritage while fulfilling modern standards on safety and structural integrity. In a traditional recalculation, the glass panels are considered to be dead load on the iron frame, leading to the conclusion that the original frame requires strengthening. This paper presents a calculation that takes the contribution of the glass plates into account while assessing historical glass coverings. The overall structural behaviour of the iron and glass roof of the Saint-Hubertus Galleries was simulated and a parameter study was performed in a finite-element software package. The original structure comprises a wrought-iron frame clad with single glass panes connected to the glazing bars with traditional linseed-oil putty. The behaviour of the original structure, the influence of the application of modern adhesives and sealants, and the replacement of single with laminated glass plates is investigated. Although building such models is time consuming, including the glass panes in the model leads to lower stresses, deflections and a better buckling behaviour, even if connected with linseed-oil putty.

1. Introduction

During the renovation of nineteenth-century iron and glass roofs, both the heritage value and modern standards on safety and structural performance have to be taken into account. An integrated approach is necessary in which both historic and modern aspects are considered.

Modern standards require the application of laminated glass to limit the risk of falling glass fragments onto people walking underneath glazed roofs. As laminated glass is composed of two bonded glass panels, this implies an increased weight on the structure. In this case, a structural assessment of the roof is necessary. Other factors might also introduce the need for a structural calculation. For example, a change of function of the building could change the live loads, an adjustment of the glass cladding to modern requirements (e.g. energy performance) can increase the self-weight of the cladding, a variation of the boundary conditions (e.g. differential settlements) could change the geometry of and load transfers in the structure and extensive corrosion damage could reduce the structural sections of the iron components.

The focus of this paper is on the structural contribution of glass cladding to the load-bearing structure of the Saint-Hubertus Galleries (Figure 1). The goal of simulating structural behaviour is often to assess the structure's safety level. At the same time, the heritage value of the roof and its components defines the boundary conditions in which a restoration proposal has to be made. Incorporating glass cladding into a structural model so that it can have a structural contribution might limit the necessary interventions to fulfil the modern requirements for structural integrity.

In this paper, the contribution of the glass cladding to the strength, stiffness and stability of the galleries is investigated. A parameter study was performed to study the influence of the glass plate thickness as well as the connection stiffness between the iron components and the glass plates under different load cases.

Figure 1: Queen’s Gallery, April 2010

2. The Saint-Hubertus Galleries

The Saint-Hubertus Galleries were built in 1846-1847 by architect J.-P. Cluysenaar as a new pedestrian connection and shopping gallery in the centre of Brussels. They were part of a larger project to upgrade the centre of Brussels as the narrow and twisting alleys from the Middle Ages were not considered healthy and hygienic and accessibility of the Grand-Place from the north was insufficient for both carriages and pedestrians.

Cluysenaar's design was the first to break with a list of characteristics of galleries.

- It was the first passage that was built with both public and private funds (compared with solely private funding previously) (Geist 1985, p.199).
- It was unique in combining a commercial space (including retail, culture, leisure and dwelling function) with a public road in a monumental building (Plevoets & Cleempoel 2011, p.141).
- By detailing the facades as they were external facades and making the glass roof very slender and transparent, the gallery is a fine example of a gallery with an exterior street atmosphere (Geist 1985, p.113; Reis et al. 1998, p.28).
- The increased height on which the iron and glass roof was installed emphasized the impression of the street (Geist 1985, pp.100, 106; Reis et al. 1998, p.55).

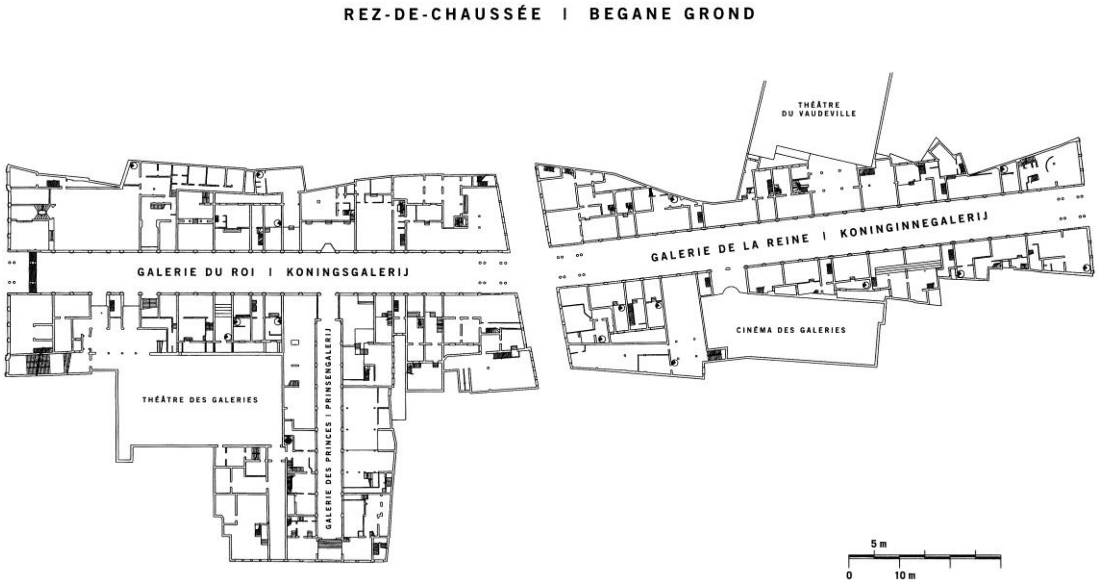

Figure 2: Ground plan of Saint-Hubertus Galleries (Reis et al. 1998)

The complex is made up of three galleries: Queen's Gallery (*Galerie de la* Reine) leads over into King's Gallery (*Galerie du Roi*) at a slight angle, with Princes Gallery (*Galerie des Princes*) as a side alley (Figure 2). Together, King's and Queen's Gallery are 213 m long. Queen's Gallery consists of 230 iron arches, while King's Gallery has 214 (A.2R.C et al. 1996, p.6). The glazed section is 18 m high and 8 m wide. The altering depth of the parcels is filled with the apartments, shops, theatres and a cinema (Figure 2).

King's and Queen's Gallery were built in 15 months from March 1846 to June 1847 (Reis et al. 1998, p.36). The iron structure was built by the ateliers of Le Grand Hornu between September 1846 and January 1847 and covered with glass from J.B. Capellemans and A. Deby from October 1846 to February 1847 (Reis et al. 1998, pp.37, 53).

The Saint-Hubertus Galleries were protected as a whole by *Brussels Hoofdstedelijk Gewest* (Brussels Capital Region) in 1986. The first major renovation campaign was carried out from 1993 until 1997 by architects A.2R.C under the supervision of the Royal Commission of Monuments and Sites, to celebrate the 150th birthday of the galleries. Several studies were performed for the restoration campaign, covering interior and exterior aspects. The renovation of the glass roof included interventions on the iron structure, the glass plates and the connection details.

3. The geometry of the Saint-Hubertus Galleries

The iron and glass roof spans 8 m and consists of a series of wrought iron circular arches (Figure 3). Each arc is, at the central part with a low inclination, topped-off with a small pitched roof called a “lanterneau", which is raised from the arches via "columns". All arcs are supported with a hinge at both ends on a cast-iron support strip that rests on top of the masonry walls. The arc and columns have a rectangular cross-section of 50x7 mm with a radius of 4.23 m, while the lanterneau has a section of 40x4 mm. The entity of the arc, the lanterneau and the columns in between will be called the "arch" in the following text. One arch is constructed every 40 cm.

Perpendicular to the arched main frame, rectangular sections are fixed at the ridge and supports of the lanterneau. At the centre of the arcs, the same function is taken up by freestanding round bars. Discontinuous L-sections are placed in between the arches at the top of every glass plate. The nomenclature of all iron components is given in Figure 4.

The arches (except the central arc part and the columns) are clad with glass plates of size 40x46 cm. Originally, the glass plates were put on two L-sections (glazing L-bars in Figure 4) that were connected with the arch section (Figure 5(a)). According to the restoration report (A.2R.C et al. 1996), the connection between the glazing L-bars and arches would have been accomplished with small angle sections, however this could not be verified since the glazing L-bars were replaced during the renovation. The glass was placed on the glazing L-bars and sealed with putty. In the restoration report, the drawing of the transverse connection shows the glass plates directly in contact with the iron glazing L-bars. This drawing could not be verified on site because the connection is now altered, but the connection was probably not installed as drawn, but with a zone of putty underneath the glass plates. In the longitudinal direction (Figure 5(b)), no additional measures were taken to hold the glass plate in position. The overlap between two glass plates stayed open.

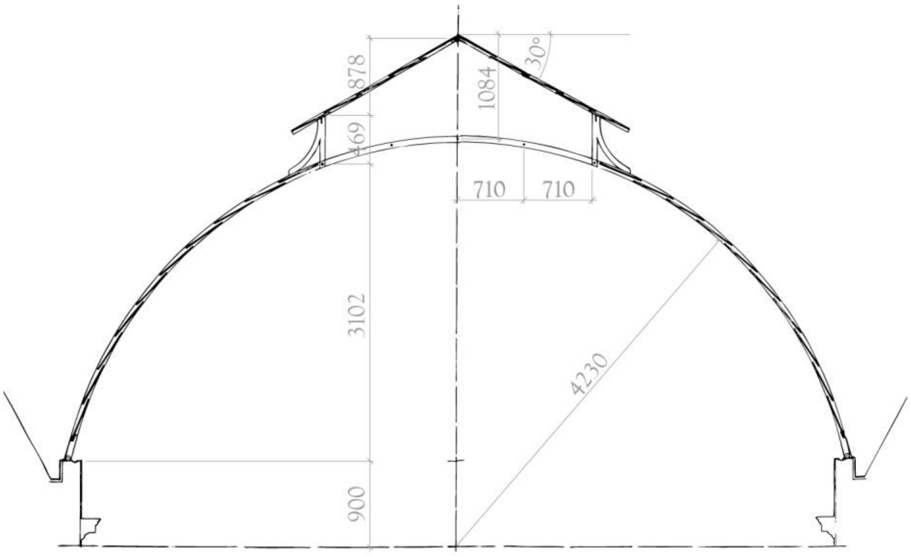

Figure 3: Cross-section of Saint-Hubertus Galleries arch (A.2R.C et al. 1996, p.7 with annotations by the authors) (dimensions in mm)

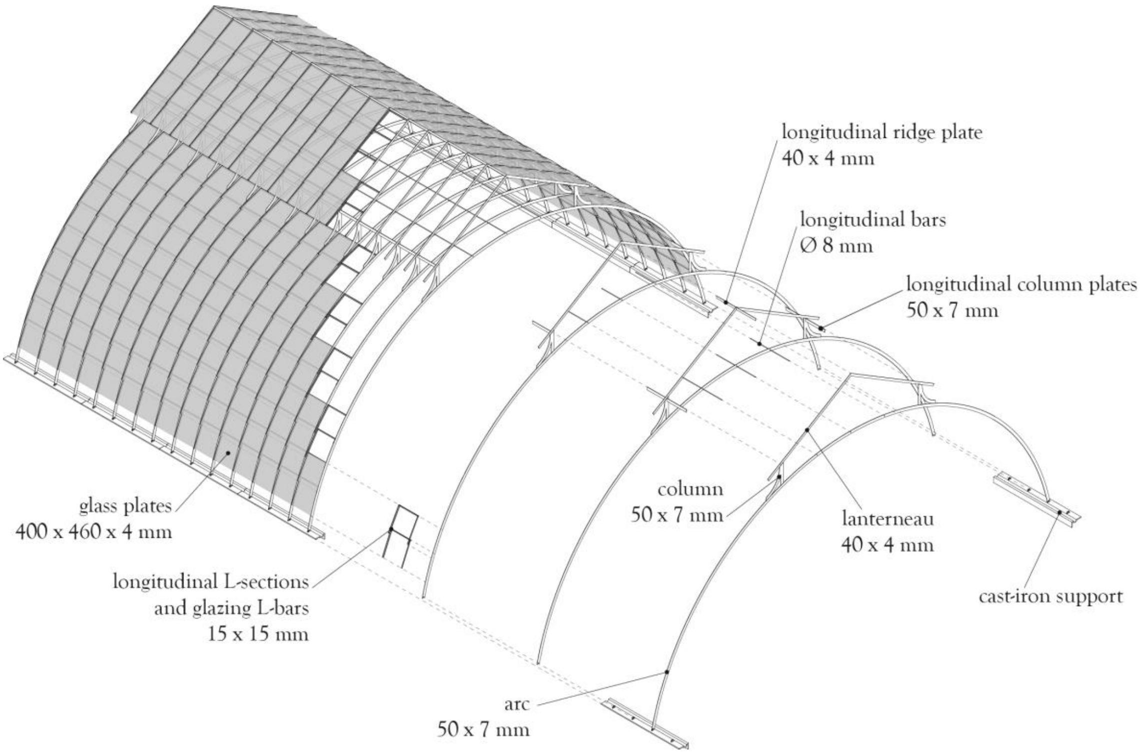

Figure 4: Nomenclature of the iron components

4. Methodology for modelling the galleries

A parameter study was carried out to investigate the circumstances in which the contribution of the glass cladding would be useful. Different parameters specific to the renovation of nineteenth-century iron and glass roofs that could affect the contribution of the glass cladding were investigated, as follows.

- Mechanical properties of the historic glass cladding. Nineteenth-century course books and manuals were examined and hardly any mention of the mechanical characteristics of nineteenth-century glass was found. The mechanical properties were thus considered to be unknown, and the mechanical properties of modern glass were used as a basis for this work (Table 1).
- Structural thickness of the glass plates. The thickness of a glass plate defines its structural stiffness and the glass weight. Both single and laminated glass plate compositions were investigated.
- Mechanical characteristics of the connection detail. The forces that can be transmitted between the iron and the glass are defined by the geometry and stiffness of the connection detail. During the nineteenth century, this connection was traditionally made with linseed-oil putty, which requires frequent maintenance. Modern sealants and adhesives could be an alternative to putty. The mechanical properties of these materials were experimentally defined (Lauriks 2011; Lauriks et al. 2011) and incorporated into the calculation.
- Condition of the iron structure. The iron components of the galleries were checked for corrosion and residual section properties before the start of the restoration campaign in 1993. In general, the condition of the iron components was good, but some local corrosion losses made some repairs necessary. In the parameter study, the sections of the iron components were thus modelled according to their original sections. The glazing L-bars, however, were heavily corroded and removed during the restoration campaign. They were thus not considered in the calculation model.

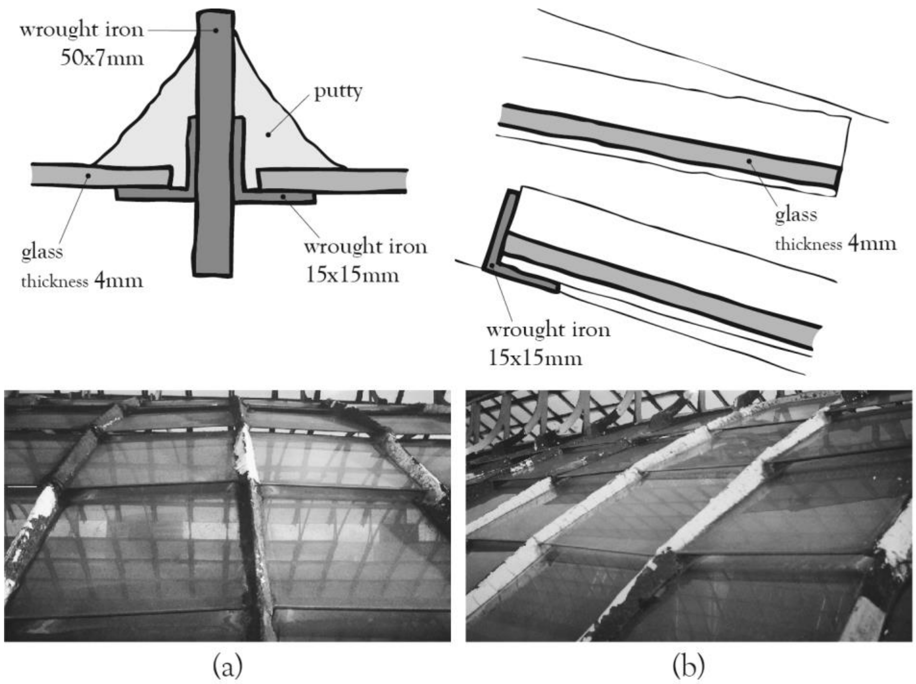

Figure 5: Transverse (a) and longitudinal (b) connections at the Saint-Hubertus Galleries: original detail before 1993-1996 renovation campaign (A.2R.C et al. 1996; LMMA 1993, p.2 with annotations by the authors)

The calculation model of the Saint-Hubertus Galleries isolates a limited number of segments out of the length of the galleries. The distributed loads are constant over all segments, but enough segments have to be modelled to isolate the impact of a concentrated load. A convergence study (evaluated on the dissipation of a central concentrated load) showed that 33 arches had to be modelled. The parameter study was then carried out on this limited number of segments.

At their ends, the masonry entrances of the galleries form a massive support for lateral displacements of the sequence of arches. However, in other nineteenth-century iron and glass roofs, the contribution of the glass cladding to the horizontal stiffness of the structure might be major. Therefore, the same geometry as the other models was extrapolated to a model where the horizontal displacements are free. The objective of this model is to calculate the contribution of the glass cladding to the longitudinal stiffness of the construction in a theoretical way, even if it does not directly relate to the real boundary conditions.

Table 1: Material properties used in the calculation model

	density [kg/m³]	Young's modulus [kN/mm²]	Poisson's ratio
Iron	7800	210	0.30
Glass	2500	70	0.22
Adhesive	1400	included in equivalent stiffness methodology (Section 5.3)	

In the course of the restoration studies, a tension test was carried out on one sample of wrought-iron extracted from the galleries and this showed a stress-strain curve with a linear elastic material behaviour similar to modern construction steel S235 (T.C.A. et al. 1996). However, the uncertainty on this one sample is high. Based on the results of this test as well as materials research on other nineteenth-century iron structures (de Bouw 2010), the decision was made to use the stiffness and density of modern construction steel (Table 1).

5. Parameter study definitions

5.1 The applied external loads

The parameter study was performed for different load cases. The load cases were conceived as simplified loading profiles that give an abstract representation of maintenance and climatic loads (snow load and horizontal and vertical wind load). The self-weight of the structure is considered in combination with one of the following live loads.

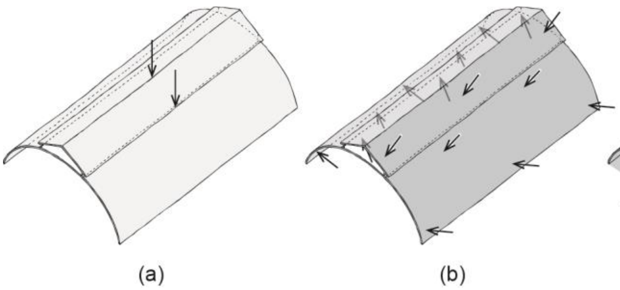

Figure 6: Loads considered in the calculation model: (a) concentrated maintenance load; (b) asymmetric wind load; (c) horizontal wind load

- A concentrated load at the attachment points of the maintenance ladder (both at the ridge of the arch and on top of the small columns supporting the lanterneau), simulating maintenance loads of 1 kN (both illustrated in Figure 6(a)).
- A vertical distributed load on the glass plates, simulating a snow load (both symmetrical over the whole structure and asymmetrical on only one half of the structure) of 0.5 kN/m^2.
- A distributed load perpendicular on the glass plates, simulating a wind load (both symmetrical on the whole structure and asymmetric with pressure and suction on each half of the structure) of 0.5 kN/m^2 (asymmetric load illustrated in Figure 6(b));
- A horizontal surface load for which a wind pressure of 0.5 kN/m^2 on the whole surface enclosed by the arc and lanterneau is transposed to a distributed load of 20.105 kN/m^2 on the side surface of the first arch (only considered for the model with free horizontal displacements, illustrated in Figure 6(c)).

The values of the loads are simplified assumptions of the loads prescribed by the Eurocodes: 1 kN is the advised concentrated maintenance load on roofs (NBN 2002); 0.5 kN/m^2 is the characteristic value of snow load on the ground in Belgium (CEN 2003; NBN 2007); 0.5 kN/m^2 is the maximum external wind pressure on cylindrical roofs in Belgium (CEN 2005; NBN 2010). The distributed loads were considered to be constant over all segments of the structure.

5.2 The glass plate thicknesses

The glass plate thickness parameter has an influence on the glass weight and the stiffness of the individual glass plates. In turn, the stiffness of the glass plate influences the buckling resistance of a single glass plate (Haldimann et al. 2008). The influence of glass thickness on the global behaviour of the roof was analysed by using the following four glass plate compositions.

- Original glass plate thickness of 4 mm.
- A new glass plate composition of two panes of 2 mm glass laminated against each other; this is the glass plate composition that was applied at the galleries after the 1993-1997 renovation campaign (A.2R.C et al. 1996).

- A very stiff glass plate composition of two panes of 4 mm thick glass laminated against each other. This composition was added to the parameter study to study the effect of doubling the glass weight.
- Application of a new 4 mm thick glass plate onto which the original 4 mm thick glass plate is laminated using a resin technique. The new glass plate takes up all the loads. In this way, the original glass plate can be preserved when it has historic value, but only adds extra load without contributing to the glass plate stiffness.

All glass compositions were modelled, using a finite-element model, as a monolithic glass plate with an effective thickness calculated by the SLS method (with participation coefficient of the interlayer material for laminated glass (ω) set to zero) proposed in the technical report of the Belgian Building Research Institute (BBRI 2011), based on a draft European Standard (CEN 2008). The difference between the effective thickness and real thickness is added as extra self-weight to the loads.

5.3 Connection stiffness

Loads are transferred between the glass plates and the iron arches via the putty connection. The stiffness of the putty material is one of the parameters defining the load transfer. However, the geometry of the joint will also influence the stiffness of the connection as a whole. The geometry of the whole connection was simplified and an equivalent stiffness of the putty, sealant or adhesive connection was calculated. More information on the connection geometry and stiffness methodology can be found elsewhere (Lauriks 2012, pp. 157-208).

The equivalent stiffness is an approximation for the connection stiffness that can be expected. The global model was built using the following three stiffness classes.

- The first stiffness class is a simulation of the glass plates sealed with traditional linseed-oil putty. From experimental research (Lauriks 2011; Lauriks et al. 2016), it was clear that this putty could transmit a relevant amount of compressive force but could not resist any shear or tensile forces.

- The second class simulates the behaviour of a connection sealed with a modern sealant with low-stiffness characteristics. It has the same filling capacity as the traditional putty, is less maintenance-intensive, but is not designed for load-bearing applications.
- The third stiffness class applies a modern adhesive with a higher stiffness, developed specifically for its better mechanical properties but still with sufficient filling capacity and elasticity.

The mass of the connection material was included in the simulation model via its physical properties (Table 1) and the equivalent geometry of the above described methodology.

5.4 Overview of parameter study

The parameter study was carried out by the finite-element calculation software Abaqus/Standard (version 6.11) (3DS Simulia 2011). The structure was analysed by linear calculations only, which limits the results to small deformations. Stress, deformation and buckling analysis were carried out.

Per load case, sixteen models were analyse: four glass thicknesses combined with three connection stiffness classes and a model without glass plates. Table 2 gives an overview of the parameters in the 16 models, along with model abbreviations that will be used in the following text.

Table 2: Overview of parameter matrix per load case

connection / glass	no glass	linseed-oil putty	low stiffness sealant	high stiffness sealant
original 4mm thickness	4a-none	4a-putty	4a-low	4a-high
new laminated, two 2mm thickness	4b-none	4b-putty	4b-low	4b-high
new laminated, two 4mm thickness	8a-none	8a-putty	8a-low	8a-high
new 4mm thickness + original 4mm thickness	8b-none	8b-putty	8b-low	8b-high

The iron and glass materials were simulated using the appropriate mesh elements and sizes (decided on after a convergence study). The iron arches and the glass plates were modelled using general shell elements (S4 and S3), while the longitudinal iron components (the longitudinal L-sections, ridge and columns plates and bars) were modelled with linear elements (B33).

Appropriate constraints were chosen for modelling the internal connections. The connection between the arches and the glass plates was modelled using an equivalent connection stiffness methodology (Section 5.3). Coupling constraints were used to simulate the fixed connections between the arches and the longitudinal iron components.

5.5 Evaluation of parameter study

The contribution of the glass plates was evaluated by simulating three possible interventions. The structural behaviour of the original structure (iron frame covered with 4 mm thick monolithic glass plates connected to the iron arches with traditional linseed-oil putty) was compared with the structural behaviour of

- the original iron frame (without glass plates); recalculations of the roof structure during the renovation studies took only the iron frame in account (Lauriks 2012), so this model simulates the same condition for comparison reasons;
- the original iron frame and glass plates but with an adjusted connection detail to obtain a higher stiffness so that larger forces are transmitted between the iron glazing bar and the glass plates
- the original iron frame but clad with laminated glass instead of the original monolithic glass.

The results of all the models were compared to study the influence of glass plate thickness and connection stiffness. For both parameters and under different loads, the reaction forces, deformation of the iron arches, deformation of the adhesive connections, stresses in all iron components and the buckling behaviour were evaluated. The results of all the calculation models were normalised to the value of that quantity in the model simulating the original structure (with 4 mm thick monolithic glass and a connection sealed with putty). The 100%

value will therefore be different per result graph, but the influence of the parameters can be read directly from the graphs.

6. Results

The most onerous load case for both deformations and stresses was found to be the self-weight combined with an asymmetric wind load (the scaled deformation is shown in Figure 7 and Von Mises stresses in iron components in Figure 8). In general, asymmetric load cases are more difficult for this structure to resists than their symmetric equivalents. For a complete description of all the nominal results, readers are referred to previous work (Lauriks 2012).

The results presented in the following will not be discussed for their nominal values, but the influence of the parameters will be illustrated. The results were normalized to the value of that quantity in the 4a-putty model (i.e. the original structure).

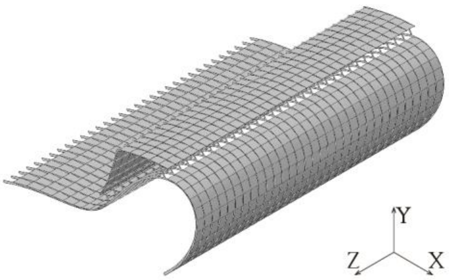

Figure 7: Deformed structure under asymmetric perpendicular load in 4a-putty model

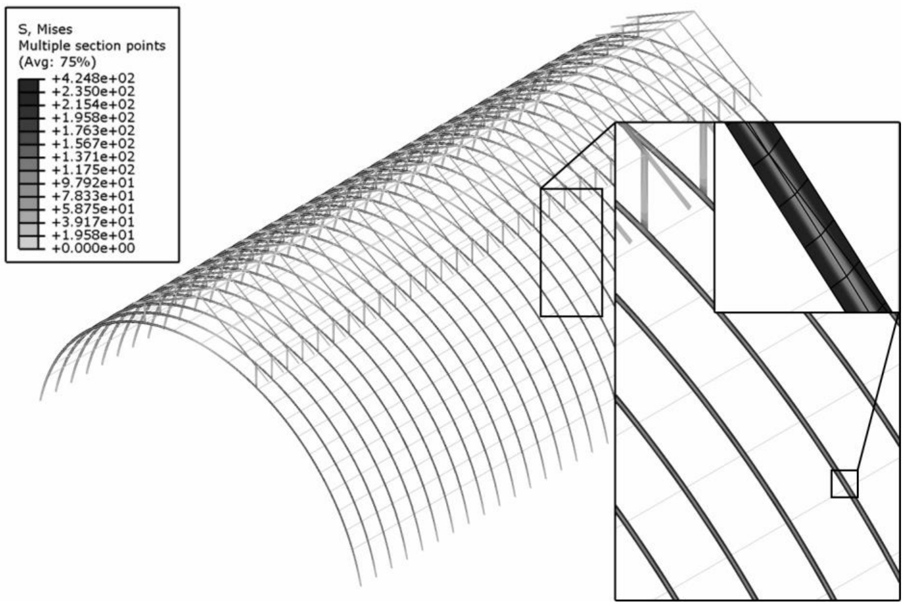

Figure 8: Von Mises stresses in iron components under asymmetric perpendicular load in 4b-low model

5.1 Influence of the presence of glass plates

The influence of including the glass plates in the calculation model is very clear. Table 3 shows the stresses, deflections and stability of the model without glass plates, in comparison with the original structure (e.g. the maximum stresses in the arches under asymmetric wind load in model 4a-none are 117 % of the same stresses in model 4a-putty). Figure 9 shows the maximum stresses in the arches under asymmetric wind load (e.g. the same 117 % can be read here at the 4a-none marker).

The influence of the glass plates on the most critical quantities (deflection of the arches and stresses in the arches) under the most onerous load case (asymmetric wind load) is relatively low, but still significant (e.g. 111 % for maximum deflections of the arches). The impact on the other quantities and for the other load cases sometimes is explicitly higher, as for example the horizontal wind load case (254 % for maximum stresses in the arches compared with the original model with 4 mm single glass panes, illustrated in Table 3 and Figure 10).

5.2 Influence of connection stiffness

In original structures, glass plates are often sealed to iron glazing bars with traditional linseed-oil putty. In a renovation, this connection can be adjusted and a modern adhesive or sealant could be used. The impact on structural behaviour of applying a modern adhesive with a relatively high stiffness was found to be only limited for most quantities (Table 4). The deflection of the arches and stresses in the arches are barely influenced (e.g. 99 % for the maximum stresses in the arches compared with the model with the original putty connection). However, the quantities out-of-plane of the cross-section of the Saint-Hubertus Galleries (e.g. the stresses in the longitudinal iron components) are significantly influenced.

Table 3: Influence of the presence of glass plates: the quantity listed in the left-hand column using the 4a-none model expressed as a percentage of the quantity using the 4a-putty model

	asymmetric wind load	other wind loads snow and maintenance loads	horizontal wind load
maximum Von Mises stress in the arches	117 %	111 – 404 %	254 %
maximum Von Mises stress in the longitudinal L-sections	439 %	141 – 516 %	458 %
maximum vertical or horizontal deflection of the arches	111 %	84 – 135 %	401 %
stability of the structure	other buckling modes	other buckling modes	no difference in buckling behaviour

Table 4: Influence of changing the connection from a putty connection to a connection with an adhesive with high stiffness: the quantity listed in the left-hand column using the 4a-high model expressed as a percentage of the quantity using the 4a-putty model

	asymmetric wind load	other wind loads, snow and maintenance loads	horizontal wind load
maximum Von Mises stress in the arches	99 %	96 – 100 %	101 %
maximum Von Mises stress in the longitudinal L-sections	94 %	92 – 95 %	91 %
maximum vertical or horizontal deflection of the arches	99 %	98 – 99 %	99 %
stability of the structure	no difference in buckling behaviour	no difference in buckling behaviour	no difference in buckling behaviour

Table 5: Influence of changing from a monolithic to a laminated glass composition of the same total thickness: the quantity listed in the left-hand column using the 4b-putty model expressed as a percentage of the quantity using the 4a-putty model

	asymmetric wind load	other wind loads, snow and maintenance loads	horizontal wind load
maximum Von Mises stress in the arches	103 %	101 – 102 %	100 %
maximum Von Mises stress in the longitudinal L-sections	102 %	102 – 106 %	101 %
maximum vertical or horizontal deflection of the arches	103 %	103 – 104 %	100 %
stability of the structure	local glass buckling at load lower than asymmetric wind load	more chance for local glass buckling	no difference in buckling behaviour

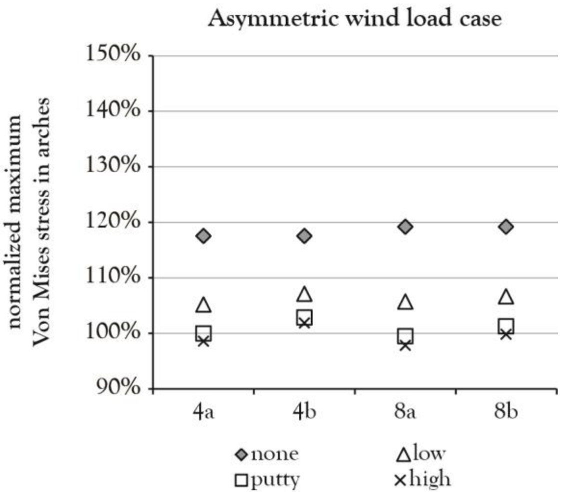

Figure 9: Maximum Von Mises stress in arches for the asymmetric perpendicular load case (normalized for 4a-putty model)

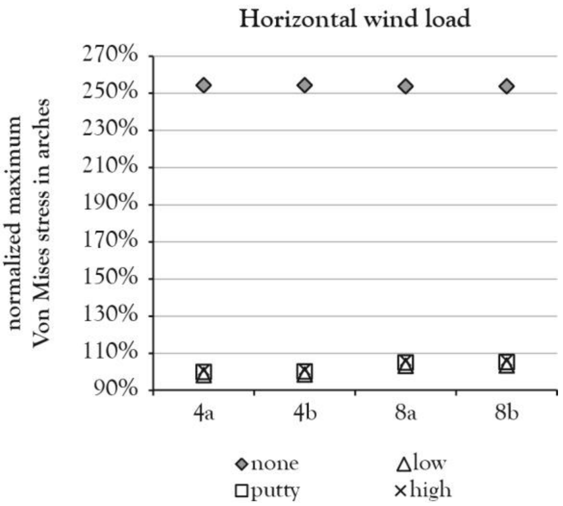

Figure 10: Maximum Von Mises stress in arches for the horizontal load case (normalized for 4a-putty model)

5.3 Influence of glass plate thickness: stiffness and weight

Changing the glass plate composition from monolithic to laminated glass was found to have a small influence on most of the structural behaviour of the Saint-Hubertus galleries (e.g. the maximum stresses in the arches are only 103% in the 4b-putty model compared with the 4a-putty model, as illustrated in Table 5 and Figure 9). However, it is important to note that a laminated glass composition is more vulnerable to buckling of the glass plate. For example, under an asymmetric wind load, the glass plates buckled before even reaching the total wind load.

However, the effective thickness of the laminated glass composition was calculated based on the assumption of no shear composition action between the two glass panes – this is conservative, especially for short-term loads such as wind load.

7. Conclusions

Taking only the iron frame into account is often the most time-efficient way to recalculate a nineteenth-century iron and glass roof. However, when such a calculation shows that the stresses in the iron components and the deformations of the structure are too high, a calculation with a refined model including the glass cladding might be advisable.

The overall structural behaviour of the Saint-Hubertus Galleries was simulated and a parameter study was performed. Including the glass plates into the structural model proved to be appropriate when only slight overloading was examined. For structures that exceed the allowable stresses and deflections to a greater extent (due to an increase in the imposed loads, a change in support conditions, reduced sections due to corrosion, etc.), including glass plates in the model will not eliminate the overall overloading but could limit the necessary interventions. Replacing putty with a modern adhesive with higher stiffness could help reduce some specific local overloading problems (e.g. stresses in the longitudinal iron components). Replacing single with laminated glass was shown to have only a minor influence, but the glass weight is a parameter that needs careful consideration. When calculation of an iron frame highlights some local overloading, including the glass plates in the model could make a major difference (e.g. local buckling of the iron frame or stress peaks in the longitudinal iron components). The contribution of the glass plates was also found to be higher for loading in the plane of the glass plates (under the horizontal load case).

In other renovation studies, the results of this research might be used in two directions. When new glass plates are to be installed, the contribution of the glass to the structural behaviour can be calculated depending on the glass plate composition and the connection between the iron glazing bars and the glass plates. When an existing structure is studied, the structural behaviour of the whole iron and glass roof can be estimated based on the present conditions of both the glass plates and the connections.

This paper has shown that consideration of the contribution of glass cladding to overall structural behaviour can reduce the necessary interventions on nineteenth-century iron and glass roofs. Subsequently, other parameters that define the restoration strategies of nineteenth-century iron and glass roofs should be looked in at more detail. Financial considerations, the residual life of the structure, local corrosion damage, the durability of sealant materials, risk assessment and so on are interesting factors to address in the future.

Acknowledgements

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