Vrije Universiteit Brussel
FACULTY OF ENGINEERING
Department of architectural engineering

UNIVERSITEIT GENT
FACULTY OF ENGINEERING AND ARCHITECTURE
Department of structural engineering

Contribution of the glass cladding to the overall structural behaviour of 19th-century iron and glass roofs

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by
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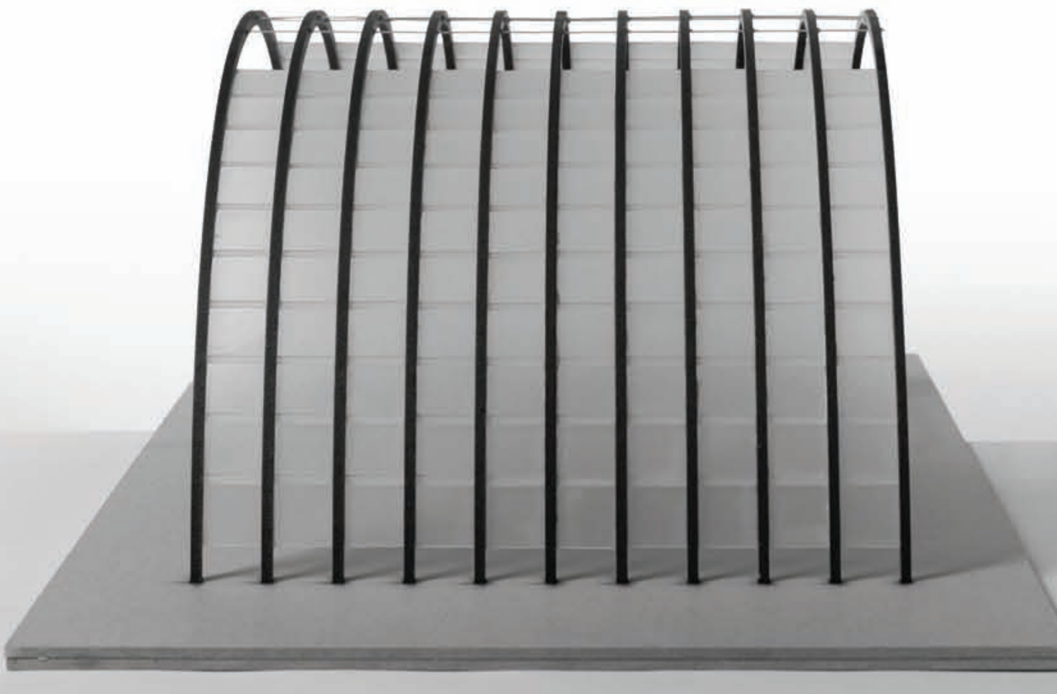

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Members of the jury

Prof. Dr. Ir. Tine Tysmans President
Vrije Universiteit Brussel – Department of Mechanics of Materials and Constructions

Prof. Dr. Ir. Luc Taerwe President
Ghent University – Department of Structural Engineering

Prof. Dr. Ir. Rik Pintelon Vice-President
Vrije Universiteit Brussel – Department of Fundamental Electricity and Instrumentation

Prof. Dr. Ir.-Arch. Ine Wouters Advisor
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Ghent University – Department of Structural Engineering

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Prof. Dr. Ir. Luc Schueremans External member of the jury
University of Leuven – Department of Civil Engineering
Building industry – Frisomat

Dr. Ir. Edwin Huveners External member of the jury
Building industry – Volantis

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Abstract (English)

To determine the historical value of an iron and glass roof structure, insights into the global structure and the way it was built, the applied materials and their fabrication, and the applied connections have to be gained. By means of an extensive literature study of 19th-century course books and manuals, and 20th- and 21st-century overview books, the development of the iron and glass architecture, the evolution in the construction details and the alteration of the glass production process during the 19th-century was investigated.

The production processes of cast iron, wrought iron, and steel underwent major changes during the 19th century. The application of iron and steel in building construction expanded and an architectural vocabulary specific for iron construction was formed. Glass manufacture however stayed a traditional process until the beginning of the 20th century. The traditional manufacturing methods were nevertheless continuously improved, thus providing the possibility to increase the production volume and the quality of the produced glass. The great innovation of the 19th century however was the application of glass in architecture in combination with iron. The slender iron frames cladded with glass allowed light to penetrate to the core of the buildings. New building types, originating from the Industrial Revolution like railway stations, exhibition buildings, shopping galleries, and glasshouses, used this possibility at full extent.

Chemical analysis of glass can be used to date historic glass, if this chemical analysis can be linked to the evolution of the glass production process. The specific Belgian situation in 19th-century glass industry (the economic conditions, the available raw material resources, etc.) is studied. A timeline gives an overview of the evolution from 1790 until 1915 of the economic situation, the maximum glass plate dimensions, the raw materials, the melting furnaces and the processing technology. This timeline can be the start of the development of a methodology to date 19th-century Belgian window glass.

The construction techniques of 19th-century iron and glass constructions contribute to the specific historic value of these buildings. The slender iron glazing bars covered with a high number of small overlapping glass plates define its distinguishable look. The construction techniques are investigated based on manuals and course books from 1847 until 1919 and confirmed by all investigated Belgian case studies. The most often used connection detail comprises a T-shaped iron glazing bars where glass plates of 3 to 4 mm thickness are placed on and sealed with putty. Systems for puttyless glazing were developed to avoid the intensive maintenance of putty connections, however were only rarely used in Belgium.

When renovating 19th-century iron and glass roofs, the question arises how we can preserve this built heritage but make the construction fulfil the modern standards on safety and structural integrity. The heritage value of the whole construction and of the separate components establishes the boundary conditions in which the possible interventions are defined. Possible renovation strategies were illustrated by three case studies, discussing how modern standards and heritage value can be combined.

In 19th-century iron and glass roofs, glass plates were placed on the iron glazing bars using traditional linseed oil putty. Experimental research shows that linseed oil based putty can have significant compression stiffness, so forces can be transmitted between iron glazing bar and glass plate. However, tensile forces cannot be transferred. The influence of parameters specific for the on-site execution of adhesive bonding on historic iron is investigated on modern adhesives. Grit blasting the substrates, resulting in different surface roughnesses, show to have only minor influence on the strength of the adhesive bond. The same conclusion can be drawn for the application of a paint layer. However, the time between cleaning and bonding the surface has a significant negative influence on the adhesive bond strength. In a renovation context, it is barely impossible to apply a paint layer or an adhesive bond quickly after cleaning the surface by grit blasting.

The overall structural behaviour of a 19th-century iron and glass roof is simulated and a parameter study is performed in a finite element software package. The glass plate composition (laminated versus single glass plates) and the stiffness of the connection between iron glazing bar and glass plates were selected for the parameter study. The structural behaviour was studied under seven different load cases, each of them combining self-weight with a variation of wind, snow, or maintenance load.
The influence of including the glass plates in the calculation model is very clear: the deflections and stresses are lower, while the eigenvalues are higher. The exact influence depends on the load case and the specific component.
In the original structure, the glass plates are often sealed to the 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 of a modern adhesive with a higher stiffness (within the range of adhesives with enough filling capacity and flexibility) on the structural behaviour is only limited for most quantities.
Replacing monolithic glass by laminated glass reduces the effective thickness (the thickness simulating the composite action of laminated glass in a monolithic section) of the glass plate. The thinner this effective thickness is, the higher the buckling sensitivity to local glass plate buckling. Increasing the weight of the glass plates has an influence on most of the quantities and has to be avoided as much as possible.

Finally, the whole process of the assessment of 19th-century iron and glass roofs is discussed. For the structural assessment, including the glass plates into the structural model reduces the stresses and deflections, therefore making structural interventions unnecessary or reducing them. Replacing the putty by a modern adhesive with higher stiffness can help reduce some specific local overloading problems (e.g. stresses in the longitudinal iron components). Local overloading of the iron frame might experience a higher positive influence from including the glass plates in the calculation model.

Different approaches are possible for the renovation strategies of separate components. This research gives examples on all levels: sandblasting and painting the iron components, the influence on both safety level and structural performance of the installation of single or laminated glass, the choice of replacing linseed oil putty by a modern sealant due to maintenance issues, etc. The interventions have to be chosen based on their influence on the heritage value of the building as a whole and its components, on the connection details of the construction, on the safety level of the glass roof, and on the structural integrity of the whole construction.

Abstract (Dutch)

Om de monumentenwaarde van een ijzer- en glasdak te bepalen, moet men inzicht hebben in de structuur en hoe deze werd gebouwd, de materialen die daarvoor werden gebruikt en hoe deze werden vervaardigd, en de toegepaste constructietechnieken. De ontwikkeling van de ijzer- en glasarchitectuur, de evolutie van de constructietechnieken en de vooruitgang in de productie van glas worden onderzocht aan de hand van een uitgebreide literatuurstudie van 19de-eeuwse technische handboeken en cursussen, en 20ste- en 21ste-eeuwse overzichtswerken.

De productie processen van gietijzer, smeedijzer en staal veranderden zeer sterk tijdens de 19de eeuw. Het gebruik van ijzer en staal voor constructieve toepassingen breidde uit en een vormentaal voor ijzerconstructies werd ontwikkeld. De productie van glas bleef echter een ambachtelijk proces tot aan het begin van de 20ste eeuw. Die ambachtelijke productie werd tijdens de 19de eeuw wel op punt gesteld, waardoor de productiehoeveelheid en de kwaliteit van het glas omhoog gingen. De grootste vernieuwing in de 19de eeuw was echter de toepassing van glas in combinatie met ijzer. Slanke ijzeren skeletstructuren bedekt met glas verhoogden de lichtinval tot in de kern van de gebouwen. Nieuwe gebouwtypologieën die ontstonden in de context van de Industriële Revolutie, zoals stationsgebouwen, exhibitiehallen, winkelgalerijen en serres, gebruikten deze lichtinval in hun architectuur.

Chemische analyse van glas kan uitsluitsel geven over de datering van de glasplaten, wanneer de chemische samenstelling kan afgeleid worden uit de evolutie van de glasproductie. De glasproductie in België wordt in detail bestudeerd, met onder andere de specifieke economische situatie en de specifieke grondstofvoorraden in België. De evolutie van de Belgische vensterglasproductie wordt weergegeven in een tijdlijn van 1790 tot 1915 waarin de economische situatie, de maximale glasafmetingen, de ruwe materialen, de oventechnologie en de verwerkingsprocessen worden besproken. Deze tijdlijn is een aanzet voor de ontwikkeling van een dateringsmethode voor 19de-eeuws Belgische vensterglas.

De constructietechnieken van 19de-eeuwse ijzer- en glasconstructies zijn bepalend voor de historische waarde van deze gebouwen. De slanke ijzeren glasroedes ondersteunen de vele kleine glasplaatjes die als schubben over elkaar liggen. Dit bepaalt het typische uitzicht van deze constructies. De constructietechnieken worden onderzocht op basis van technische handboeken en cursussen gedrukt tussen 1847 en 1919 en worden bevestigd door alle bestudeerde Belgische case studies. Het meest gebruikte verbindingsdetail bestaat uit een T-profiel als glasroede waarop glasplaten van 3 of 4 mm dik worden gelegd in een bed van lijnolie stopverf. Specifiek ontworpen glasroedes waarbij het gebruik van stopverf vermeden wordt, worden amper toegepast in België.

Bij de renovatie van 19de-eeuwse ijzer- en glasdaken moet een evenwicht worden gezocht tussen het respecteren van de monumentenwaarde en het aanpassen van de constructie aan de huidig geldende veiligheidsnormen en structurele normen. De monumentenwaarde van de gehele constructie en van de aparte onderdelen bepaalt in welke mate er ingrepen mogelijk zijn. Mogelijke renovatiestrategieën worden in het onderzoek geïllustreerd aan de hand van drie case studies waarin de evenwichtsoefening tussen historische waarde en moderne standaarden gemaakt wordt.

De verbinding tussen de glasplaten en de ijzeren glasroedes wordt in de 19de-eeuwse ijzer- en glasdaken verzekerd door lijnolie stopverf. Experimenteel onderzoek wijst uit dat deze stopverf een hoge drukstijfheid kan bereiken, zodat er krachten kunnen worden overgedragen tussen ijzer en glas. Er kan echter geen overdracht plaatsvinden van trekkrachten. De invloed van enkele parameters die eigen zijn aan de uitvoering van de lijmvoegen op historisch ijzer op de werf, wordt onderzocht bij moderne lijmen. De oppervlakteruwheid gegenereerd door het gritstralen van de ijzeren substraten heeft slechts een beperkte invloed op de sterkte van de lijmverbinding. Hetzelfde geldt voor het aanbrengen van een verflaag op het substraat alvorens te verlijmen. De tijd waarin de substraten worden blootgesteld aan lucht (tussen het gritstralen en het verven of verlijmen) blijkt echter wel zeer bepalend. Het beperken van deze tijd bij de renovatie van een ijzer- en glasdak is echter amper mogelijk.

Het structureel gedrag van een 19de-eeuws ijzer- en glasdak wordt gemodelleerd en er wordt een parameterstudie uitgevoerd met een eindige elementen berekening. De opbouw van de glasplaten (enkel of gelaagd glas) en de stijfheid van de verbinding tussen de ijzeren glasroede en de glasplaten zijn de bestudeerde parameters. De structuur wordt berekend onder verschillende belastingen waarbij het eigengewicht gecombineerd wordt met een windbelasting, een sneeuwbelasting of een onderhoudsbelasting.

Het verschil tussen de modellen met en zonder glasplaten is zeer duidelijk: zowel de vervormingen als de spanningen zijn lager in het model met glasplaten, terwijl de eigenwaardes hoger liggen. De numerieke invloed van het modelleren van de glasplaten verschilt per belastingsgeval en per component van de structuur.

Het originele verbindingsdetail maakt gebruikt van traditionele lijnolie stopverf. Bij de renovatie kan deze stopverf vervangen worden door een moderne lijm met een hogere stijfheid (die echter nog wel voldoende flexibel is en in voldoende dikte kan worden toegepast). De invloed van een dergelijke ingreep op het structurele gedrag is echter beperkt voor de meeste grootheden.

Wanneer het originele enkele glas wordt vervangen door gelaagd glas, beïnvloedt dit de effectieve dikte van de glasplaat (de dikte waarbij het samengesteld gedrag van gelaagd glas wordt vereenvoudigd tot een monolithische glasplaat). Hoe dunner de effectieve dikte, hoe gevoeliger de glasplaat zal zijn voor lokaal uitknikken. Wanneer de totale dikte van de opbouw van de glasplaat verhoogt en dus ook het totale gewicht van de glasplaten, heeft dit een negatieve invloed op het structureel gedrag en dit moet indien mogelijk dus vermeden worden.

Tenslotte wordt een overzicht gegeven van het gehele proces van de analyse van ijzer- en glasdaken. In het kader van de structurele evaluatie, worden de spanningen en vervormingen lager wanneer de glasplaten in het model worden opgenomen. Dat kan de benodigde interventies beperken of zelfs helemaal overbodig maken. Het vervangen van de stopverf door een moderne lijm kan vooral een verschil maken bij lokale overbelasting (bijvoorbeeld een overschrijding van de toelaatbare spanningen lokaal in enkele longitudinale componenten). De invloed van het opnemen van de glasplaten in het model op deze lokale fenomenen, kan ook groter zijn dan de gemiddelde invloed op de hele structuur.
Bij de renovatie van de verschillende onderdelen van een ijzer- en glasdak, zijn verschillende benaderingen mogelijk. In dit onderzoek worden de implicaties van enkele interventies behandeld: het reinigen en verven van de ijzeren glasroedes, de implicaties van het al dan niet plaatsen van gelaagd glas op zowel de veiligheidsvoorschriften als het structurele gedrag, het vervangen van de traditionele stopverf door een moderne lijm omwille van de voordelen op het vlak van onderhoud, enz. De interventies moeten geëvalueerd worden op basis van hun invloed op de monumentenwaarde zowel van het gehele dak als van de componenten, op de verbindingsdetails, op de veiligheid van het glasdak en op het structurele gedrag van de constructie.

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Abbreviations and symbols

1. Abbreviations

AIHV	Association International de l'Histoire du Verre
AKP	Archief Koninklijk Paleis (Archives of the Royal Palace)
BBRI	Belgian Building Research Institute
BC	Before Christ
CIP	cast-in-place resin
EN	European standard
ETAG	European Technical Approval report
EVA	ethylene vinylacetate
ICOMOS	International Council on Monuments and Sites
ISO	International Organisation for Standardization
KCML	Koninklijke Commissie voor Monumenten en Landschappen (Royal Commission for Monuments and Sites)
MS polymer	Modified Silane polymer
NBN	Norme Belge / Belgische Norm
prEN	European pre-standard
PVB	polyvinylbutyral
pXRF	portable X-ray fluorescence
SEM	scanning electron microscope
SG	Sentry Glas ®
SLS	Serviceability Limited State
TNO	Nederlandse Organisatie voor toegepast-natuurwetenschappelijk onderzoek
ULS	Ultimate Limate State
UV	Ultra Violet

2. Symbols

A	loaded surface	mm^2
A'	shear surface	mm^2
d	depth	mm
E	Young's modulus	N/mm^2
E_{exp}	Young's modulus of experiments	N/mm^2
E_{strip}	Young's modulus of adhesive strip	N/mm^2
F	force	N
F_{hor}	axial in-plane force	N
F_{vert}	axial out-of-plane force	N

G	shear modulus	N/mm²
G_{exp}	shear modulus of experiments	N/mm²
G_{strip}	shear modulus of adhesive strip	N/mm²
$h_{ef, w}$	effective thickness for deformation of laminated glass plates	mm
$h_{ef, w, \omega=0}$	effective thickness for deformation of laminated glass plates for no composition action	mm
$h_{ef, \sigma, j}$	effective thickness for stresses of laminated glass plates	mm
$h_{ef, \sigma, j, \omega=0}$	effective thickness for stresses of laminated glass plates for no composition action	mm
h_i	thickness of the glass plate i	mm
$h_{m, j}$	distance of the mid-plane of glass plate j to the mid-plane of the laminated glass composition (ignoring the thickness of the interlayers)	mm
k_{compr}	(spring) stiffness under compressive force	N/mm²
k_{hor}	(spring) stiffness under F_{hor}	N/mm²
k_{shear}	(spring) stiffness under shear force	N/mm²
$k_{tensile}$	(spring) stiffness under tensile force	N/mm²
k_{vert}	(spring) stiffness under F_{vert}	N/mm²
L	length	mm
M_{hor}	rotation of the glass plate	Nmm
Ra	surface roughness: arithmetic average of the absolute values of the profile height deviations recorded within the evaluation length and measured from the mean line	µm
t	thickness	mm
t_i	thickness of component i	mm
w_i	width of component i	mm
γ	shear strain	
ΔL	difference in length	mm
Δx	shear strain displacement	mm
$\Delta\theta$	shear strain angle	rad
ε	strain	mm
σ	(axial) stress	N/mm²
τ	shear stress	N/mm²
ω	coefficient representing the shear transfer by the interlayer	

Chapter 0

Introduction

Figure 0-1: Royal Glasshouses of Laeken, Brussels,
Alphonse Balat, Henri Maquet and Charles Girault, 1874-1905 (2010-02-12)

> *"(...) the delicate iron-and-glass palm houses of the 19th century. Their slender metal primary structures are braced by countless small glass panes. This produces a strong, complex structure with a high degree of static indeterminacy, which is impossible to model on a computer. The individual panes of these constructions are embedded in a hard yet slightly resilient putty and so form a myriad of interconnected load paths. Structures like these glasshouses appear rather foolhardy to us today because a metal rib is stabilized by glass and putty in a way that cannot be appraised in engineering terms. Nevertheless, this certainly purposeful and, so far, successful form of construction can be justified if we consider the terms redundancy and residual stability. Redundancy is ensured because failure of a single pane only produces a structurally insignificant defect in the shell. The residual stability of such a structural system is immense because, if broken, the small panes adhere to the putty fillet and pose very little risk to persons underneath. However, trying to verify the effects of a chain reaction of failures, whether by simulation or by experiment, would involve a huge amount of work."*[1]

Although Schittich describes the calculation of an integrated model in 2007 as a huge amount of work, the last decades several engineers and engineering offices specialized in structural glass applications in general[2] and in using glass for the stabilization of building envelopes[3]. The structural role glass plays in modern constructions, poses the question how this could be applied in 19th-century iron and glass roofs.
The contribution glass plates can have on the structural behaviour of an iron and glass roof is illustrated in Figure 0-2 and Figure 0-3. Two scale models consisting of a series of arcs without and with thin plastic plates, respectively, are each subjected to a

[1] Schittich et al. 2007, p.104.
[2] O'Callaghan 2008; webpage Malishev Wilson Engineers.
[3] Weller, Reich, and Ebert 2008; Huveners 2009; Kassnel-Henneberg 2011.

horizontal and vertical concentrated load. The deformation behaviour of both models shows the differences in horizontal stability (Figure 0-2) and vertical deformation and dissipation of a concentrated force (Figure 0-3).

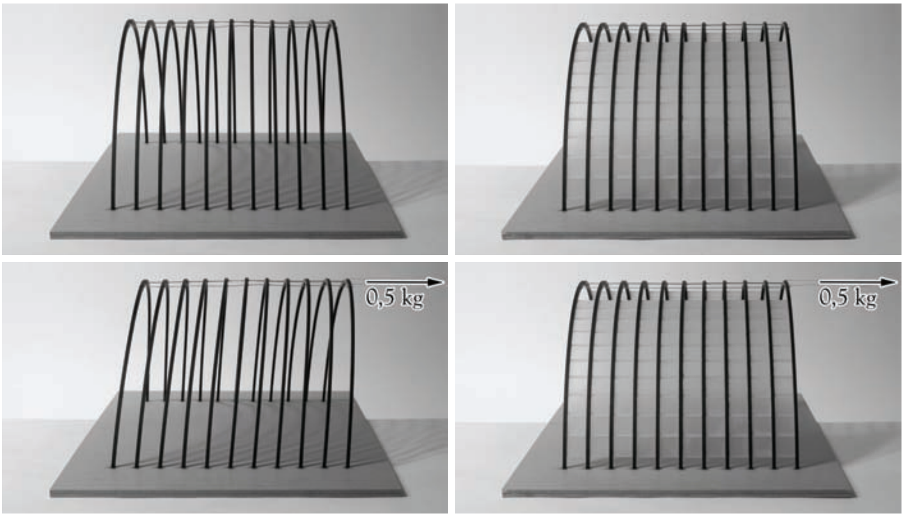

Figure 0-2: Stability of models loaded with horizontal weight:
(left) models without glass plates; (right) models with glass plates;
(top) unloaded models; (bottom) loaded models

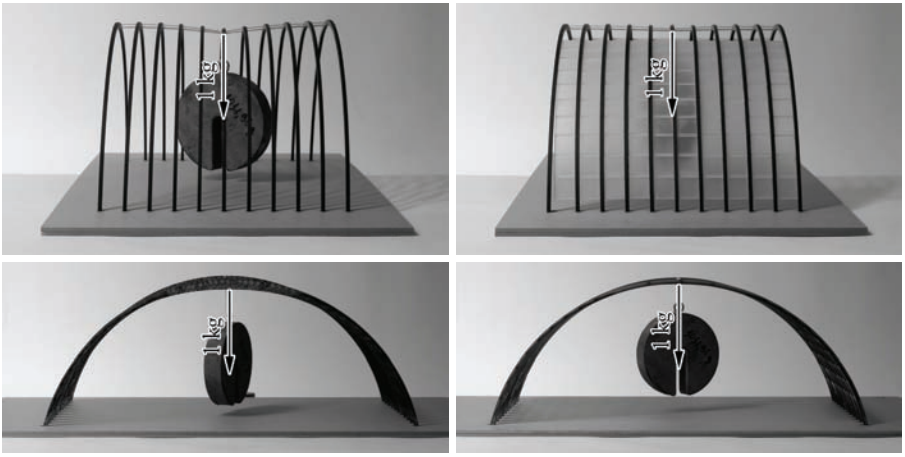

Figure 0-3: Dissipation of concentrated vertical weight in centre of the models:
(left) models without glass plates; (right) models with glass plates

1. Research motivation

During the renovation of 19th-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 the historic and modern aspects are considered.

Modern standards require the application of laminated glass to limit the risk of falling glass fragments for the people walking underneath glazed roofs. As laminated glass is composed of two bonded glass panels, this implies an increased weight on the iron structure. A structural assessment of the roof is in this case necessary. Other factors might also introduce the need for a structural calculation: 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, elaborate corrosion damage can reduce the structural sections of the iron components, etc.

The goal of the structural recalculation is to assess the safety level of the structure. At the same time, the heritage value of the roof and its components define the boundary conditions in which a restoration proposal has to be made. Incorporating the glass cladding into the structural model might limit the necessary interventions to fulfil the modern requirements for structural integrity. This research will focus on non-decorative colourless glazing to study if including the glass in the structural model is useful and which parameters define the contribution of the glass cladding to the strength, stiffness and stability of the structure.

2. Literature review

Van de Vijver gives an overview of the evolution of the construction history research field in Belgium in 2004[4]. He puts forward four research themes: "the building context", "building materials and techniques", "the building site", and "technical equipment". The topic proposed in this PhD research fits within the "building materials and techniques" aspect of construction history research defined by Van de Vijver. This theme is subdivided in general technical culture studies (e.g. on education and research institutions) or research on specific materials and techniques (e.g. on iron constructions). Van de Vijver mentions both iron and glass and the researchers that work on these materials. All mentioned glass research focuses on stained[5] or

[4] Van de Vijver 2004.

[5] Method of making windows with small flat (coloured) glass panes mounted in lead strips.

other decorative glass. The research on iron covers different periods and different regions in Belgium. Within "the building context" theme, studies on different building typologies like warehouses, libraries, glasshouses, etc. are listed, mainly summing up restoration studies on particular case studies that were published in journals or magazines. In iron and glass architecture, two materials are combined, and also the historic context of the Industrial Revolution plays a role with the birth of new building typologies (e.g. railway stations, exhibition buildings, etc.). This study on iron and glass architecture is therefore an interdisciplinary research that is positioned in between all the themes defined by Van de Vijver.
In the next paragraph, an overview is given of the available international and national literature on iron and glass architecture, on historic glass and on the restoration strategies for 19th-century iron and glass architecture.

2.1. Iron and glass architecture

The most elaborate reference dealing with iron and glass architecture is Kohlmaier and Von Sartory's book *Houses of Glass* published in 1991[6]. This book comprises a full overview of aspects that have an influence on glasshouse building: the (social, economic, architectural) context and birth of iron and glass construction, the evolution of glasshouse typologies, the development of heating techniques and iron production, and the development of the iron skeleton frame. Aspects regarding the glass material are less dealt with: the evolution of glass production, the construction techniques involving the glass plates, etc.
Iron construction in general (also outside the 19th century) is already described by many authors. Some authors give a detailed description about the evolution in a country (Bussell, Swailes, Gordon, Oosterhoff, Lemoine[7]). Others like Addis[8] consider the evolution of iron construction in the context of engineering history on an international level. Belgian iron construction is described by Baele and De Herdt[9].
All these authors leave the material glass more or less out of their perspective. But McGrath focuses specifically on the use of glass for architectural and decorative applications in his book *Glass in architecture and decoration*[10]. He gives an overview of the evolution of glass production with a focus on the English situation. He describes

[6] Kohlmaier and Von Sartory 1991.
[7] Lemoine 1986; Oosterhoff et al. 1988; Gordon 1996; Bussell 1997; Swailes and Marsh 2005; Swailes, Watson, and Dakin 2006.
[8] Addis 2008.
[9] Baele and Herdt 1983.
[10] McGrath 1961.

the applications in architecture from an architectural history point of view. Wigginton[11], Schittich et al[12] and Wurm[13] repeat this overview in short.

Next to literature that deals with the historical evolution, books with catalogues of cases give a good overview of which iron and glass constructions were built. Koppelkamm, Woods and Warren, and Ullrich[14] made a selection of glasshouses from around the world. The more extensive catalogue in the book of Kohlmaier and Von Sartory[15] of 19th-century glasshouses provides examples from whole Europe and USA. Together with the elaborate theoretical part, it makes this book the most complete work specifically on 19th-century iron and glass architecture. All catalogues are however mainly focused on historical and architectural aspects, with only rare records of the used materials, construction techniques, etc. Hix[16] gives an updated inventory of glasshouses, without adding new cases to the existing catalogues. Geist[17] on the other hand gives both a historical introduction and an inventory, but on the gallery building type instead of on glasshouses.
All these examples give an international overview while mentioning at the most five Belgian examples. No equivalent exists for the Belgian national situation. Baele and De Herdt give many examples of iron construction throughout their text including some iron and glass constructions. And the vast amount of publications on Art Nouveau touches the topic in an indirect way. However, a structured overview does not exist. The Flemish, Brussels and Walloon Regions have a descriptive inventory of their built patrimony[18], but without mentioning the small skylights that can be found in many 19th-century interiors. Notwithstanding the limited number of publications, already an extended amount of impressive skylights, glasshouses, galleries is known and will be used throughout the text.

[11] Wigginton 1996.
[12] Schittich et al. 2007.
[13] Wurm 2007.
[14] Koppelkamm 1981; Woods and Swartz Warren 1988; Ullrich 1989.
[15] Kohlmaier and Von Sartory 1991.
[16] Hix 2005.
[17] Geist 1983.
[18] *Bouwen door de eeuwen heen in Vlaanderen. Inventaris van het cultuurbezit in België. Architectuur.* 1971; *Le patrimoine monumental de la Belgique. Wallonie* 1971; *Bouwen door de eeuwen heen in Brussel. inventaris van het cultuurbezit in België. Deel Brussel.* 1989; *Le patrimoine monumental de la Belgique. Région de Bruxelles-Capitale.* 1997.

2.2. Historical glass research

Research on the applications of glass mainly focuses on decorative glazing and more particular on stained glass. Corpus Vitrearum, an international society with national committees[19], publishes inventories on stained glass windows from different time periods. The society gathers art historians, scientists, conservators, curators and architects. Technical aspects like the material, the construction technology and the conservation techniques of the stained glass windows are all part of the research carried out by members of the Corpus Vitrearum. The *Association Internationale de l'Histoire du Verre* (AIHV) is "an international organisation devoted to advancing knowledge about glass: its use, history and aesthetic qualities from antiquity to present times"[20]. Their three-yearly conferences and corresponding proceedings are mainly dealing with glass objects (e.g. tableware) and stained glass. In Belgium, the Flemish government institution on heritage conservation (*Vlaamse Overheid, dienst Onroerend Erfgoed*) publishes thematic booklets, of which the first one in 1992 was on stained glass and its conservation[21]. The same governmental institution also publishes a journal every two months. Articles mainly on case studies are included both on stained glass and iron and glass architecture[22]. The information on stained glass is therefore covered very broad, also for its technical aspects: the materials that were used, the colouring techniques for the glass panes, the sections of the lead cames that connect the glass panes, etc.

A similar overview of materials and techniques lacks for flat colourless glass. The evolution of the production processes of glass is described in several books. Douglas[23] wrote about the international evolution of production technology. The Belgian glass industry is described by Pesch, Lefèbvre, Douxchamps, Chambon, Mille and Engen[24]. Specific books are available about the company history of glass manufacturers such as Pilkington and Saint-Gobain[25].

Some aspects on the implications of the evolution of production processes have already been studied in detail. The Practical Building Conservation series published by English Heritage in 2012 give, apart from a broad historical perspective and an overview of all conservation techniques, a methodology to determine the production

[19] webpage Corpus Vitrearum 2008.
[20] webpage AIHV 2008.
[21] Vanden Bemden et al. 1992.
[22] De Maegd 1984; Breydel 1989.
[23] Douglas and Frank 1972.
[24] Lefèbvre 1938; Pesch 1949; Douxchamps 1951; Chambon 1955; Mille 1982; Engen 1989.
[25] Barker 1977; Daviet 1988.

technique of a colourless plane glass (hand-blown or mechanically drawn, cylinder or crown glass, etc.) based on its interior and exterior aesthetics[26]. Applied historic and experimental research on historic glass is carried out at English Heritage. Employee David Dungworth published several research reports on the chemical composition of plain glass from different case studies, from whom a methodology to date English glass based on its chemical composition was extracted[27]. The chemical composition of glass from other periods and countries is published in several other articles[28]. However, a broad overview on the consequences of the history of production specifically for the Belgian situation is missing: which glass types were available during the 19th century (e.g. wired glass, cast glass, etc.), what were the maximum available dimensions of these glass plates, how expensive was glass and how does this relate to the evolution of the use of glass in architecture, etc.

The limited attention for 19th-century plain glass in the art and/or architectural history research field was demonstrated in the previous paragraph. However, in the construction history research field almost no research is available on 19th-century glass, except for the research by Schoenefeldt. His findings were published among others in the Journal of Construction History Society[29]. He describes the scientific experiments that were carried out in Great-Britain in the 19th century for the use of tinted glazing as solar shading to protect the plants in glasshouses.

2.3. The restoration strategies

The evolution and conservation of iron structures are addresses by -among others-Friedman, Bussell, Nieuwmeijer, Tilly, Wouters, de Bouw, and Godfraind et al[30]. However, an overview of possible restoration strategies for 19th-century iron and glass architecture does not exist. The restoration of cases is sometimes published: for instance about the Palm House in Kew Gardens[31], the Saint-Hubertus Galleries in Brussels[32], and the Glasgow Kibble Palace[33]. Technical information and details about which interventions were carried out on the glass cladding and the connections, is rarely provided.

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[26] Pender, Godfraind, and English Heritage 2011, p.180–181.
[27] Dungworth 2009; Dungworth 2011; Dungworth 2012.
[28] Muspratt 1860; Kurkjian and Prindle 1998; Schalm et al. 2007.
[29] Schoenefeldt 2011.
[30] Friedman 1995; Bussell 1997; Nieuwmeijer 2002; Tilly, Frost, and Wallsgrove 2002; Wouters 2002; de Bouw 2010; Godfraind et al. 2012.
[31] Minter 1990.
[32] Reis et al. 1998.
[33] Glasgow City Council 1998; Curtis 1999.

The maintenance of separate iron and glass components is covered in specialist publication. Scotland Heritage for example publishes "Inform Guides" on the maintenance of plain glass and sash windows[34]. Concerning the connections, specialized books about sealants sometimes give a small description and sometimes contain recipes of traditional linseed oil putty, e.g. Bieneman or Scheffler and Connolly[35].

2.4. Observations

Most of the available knowledge on 19th-century iron and glass architecture focuses on the architectural aspects. The publication of Kohlmaier and Von Sartory illustrates the vast amount of information an inventory could hold and the possibilities of sketching a broad context of the evolution of iron and glass architecture. However, it would be interesting to complement this international publication, by focussing on the Belgian situation and adding more specific information on the level of the building details.

Iron is the most widely studied material from a construction history point of view. The material glass, for the production of plates, is given less attention. And the historic adhesive and sealants are still to be discovered. Although lot of publications deal with the specific Belgian context and the works of Art Nouveau architects, the Belgian materials and construction techniques are not covered by the existing literature and research.

Next to the individual materials, the conservation of these heritage buildings has to be addressed. An integrated approach where both the heritage value and modern standards are considered, is still missing. The Practical Building Conservation series of English Heritage are a fine example of how this interdisciplinary methodology could be applied. However, the series do concentrate on separate materials, and are therefore not applicable to iron and glass architecture in general.

3. Research goal

The goal of this research is to propose an integrated approach for the restoration of 19th-century iron and glass constructions in which the heritage value of the roof and its components is taken into account when assessing the structural integrity of the roof. The structural assessment of the roof will be carried out by modelling the iron structure together with the glass cladding, to study if and under which conditions the

[34] Historic Scotland 2007a; Historic Scotland 2007b.
[35] Bieneman 1967; Scheffler and Connolly 1996.

contribution of the glass cladding is useful to limit the necessary interventions on the roof.
As described by the literature review above, the integrated approach of both the heritage value and the modern standards was not carried out before. To limit the amount of intervening factors, the focus will be on non-decorative glazing of single roofs.

4. Research approach

During the renovation of 19th-century iron and glass roofs, both the heritage value and modern standards on safety and structural performance have to be taken into account. The following paragraph will describe how both aspects will be incorporated in this research and which methodologies were used for the investigations.

To determine the historical value of an iron and glass structure, insights into the context of the building, the global structure and the way it was built, the applied materials and their fabrication, and the applied connections have to be gained. The international context of the evolution of iron and glass architecture will be described based on literature research and will be illustrated with Belgian examples.
During the 19th century, cast iron, wrought iron and mild steel were used in the building industry. The material properties and production processes of these three metals are well documented: the evolution of the production processes, the jargon that was used to name them, their mechanical properties, etc[36]. In contrast, the production process of glass and the connection details of a glass-iron connection are less known. Consequently, a literature study is carried out, focusing on 19th-century manuals and course books and 20th-century overview works. An overview of the evolution of glass production techniques in Belgium in the 19th century is drawn up, in such a way that this knowledge can be applied to determine the heritage value of the glass plates and the connection details in a 19th-century iron and glass roof. The focus of this study on manuals and course books from engineering and architectural education (complemented by limited archival research on case studies), limits the conclusions to the knowledge that was available and transferred to the students. To broaden the conclusions, patents and archives of glass manufacturers, educational manuals for contractors and builders, etc. would have to be studied.

To make the glass cladding and the iron frame structurally work together, both materials have to be connected. The historical connection details were investigated as

[36] Bussell 1997; de Bouw 2010.

described in the previous paragraph. On the other hand, there is a constant development of modern glass connections and polymeric adhesives. Petrie gives and overview of the state-of-the-art of modern adhesive connections, while Blandini, Bos et al and Belis et al have published findings on adhesive bonding that will be used later in this research[37]. Nevertheless, some additional aspects have to be investigated before one is able to calculate the structure of a 19th-century iron and glass roof in its current state. Firstly, information is lacking about adhesive materials that fell into disuse, but were widely applied in the 19th century. Linseed oil putty is such a material. Secondly, the available information has to be complemented with aspects related to the renovation context: the application of the adhesive on a paint layer instead of bare metal, the corroded and rough surface of an iron section, etc. Consequently, in the course of this research, lab tests will be carried out to determine the mechanical properties of putty and the influence of surface roughness and surface finishing on the adhesive bonding of historic iron.

The structural assessment of both new and existing structures implies a numerical simulation of the structural behaviour. This research will use finite element models to evaluate the structure and possible interventions for the restoration of the glass roof of the Saint-Hubertus Galleries in Brussels. Abstract simplified loads will be applied, to simulate the structural behaviour under generalized conditions. The mechanical properties of the connections that were determined experimentally will be implemented in these simulations. The various glass plate compositions necessary to fulfil modern standards on safety, will also be evaluated for their influence on the structural behaviour.

International charters define a framework for the restoration of protected monuments, in general and also specifically for a structural assessment[38]. Via the analysis of national and international case studies, insight is given in interventions and their implications. Attention is also given to different typologies and environments, which can require different interventions:

- the structural behaviour of flat, inclined, single curved or double curved roofs is each time different, which makes the contribution of the glass cladding specific for the geometry of the roof;
- to separate the function of weather-proofing and interior decoration, double-walled roofs were sometimes applied instead of single-walled roofs, which has its

[37] Blandini 2005; Petrie 2007; Bos et al. 2010; Belis, Van Hulle, et al. 2011; Belis, Callewaert, and Van Hulle 2011.
[38] Venice Charter: ICOMOS 1964; The charter of Victoria Falls: ICOMOS 2003a; Ministerie van het Brussels Hoofdstedelijk Gewest and KCML 2005.

implication on the loading and on the performance requirements of the different layers of the roof;
- different environments define the aesthetic consequences of possible interventions: a small glasshouse is visible from both interior and exterior, while a high gallery roof is only visible for the public from several meters underneath.

5. Outline of thesis

The structural assessment of 19th-century iron and glass roofs should be carried out taking its historic and modern context into account. Seven chapters give an overview of the different stages in the whole process of the assessment.

Chapter 1 gives an overview of building in iron and glass in the 19th-century. The evolution of iron and glass architecture is described in relation to the social and technical context of the 19th century. The specific material developments during this century are addressed. A brief overview of the evolution in the production processes of wrought iron, cast iron and mild steel is given. The Belgian situation on the production of glass is outlined elaborately. This chapter permits to position iron and glass constructions in time, both based on its architectural qualities, its structure and its materials.

The construction techniques in 19th-century iron and glass constructions are investigated in Chapter 2. These construction techniques are studied by going through 19th-century Belgian manuals and course books. Specific interest went to the connections, the sections of the iron glazing bars, the maximum available dimensions of the glass plates, etc. Built examples were added to illustrate the results found in literature. The overview of all the construction techniques in this chapter can be used as a reference to evaluate the heritage value of the connections in a 19th-century iron and glass roof.

Chapter 3 considers the opportunities and conditions for giving glass a structural role. Modern structural glass applications and research show the possibilities of stabilizing building envelopes by giving the glass cladding a structural function. Modern standards assign requirements for applying glass in roofs and for structural applications. The boundary conditions set by modern standards for applying glass as a structural element in 19th-century iron and glass roofs, are outlined in this chapter.

The boundary conditions for the renovation of 19th-century iron and glass architecture are also defined by the heritage value of these constructions. The theoretical framework defined by international heritage charters is in Chapter 4 applied to 19th-century iron and glass roofs. In three case studies, the global renovation strategy and the interventions on the iron and glass components are evaluated against the theoretical framework of heritage conservation as well as the boundary conditions imposed by modern standards. This chapter gives insight in which parameters define the heritage value of an iron and glass roof and illustrates possible renovation strategies in three case studies.

Chapter 5 focuses on the experimental research performed on adhesive bonding. Experiments were carried out to determine the influence of specific conditions for 19th-century iron and glass construction. Originally, traditional linseed oil putty was used to seal the iron-glass connection, so this putty was studied for its mechanical characteristics. The influence of applying modern adhesives on historic mild steel substrates, the influence of applying a paint layer before applying the adhesive bond, and the influence of cleaning the substrate by grit blasting on adhesive bonding were investigated. This detailed information on the mechanical performance of connection details, is required in order to model the global behaviour of an iron and glass structure.

The structural behaviour of iron and glass roofs is studied in detail in Chapter 6. The geometry of the Saint-Hubertus Galleries in the centre of Brussels is used for the parameter study. The influence of including glass plates in the calculation models, the influence of installing laminated glass to fulfil the standards on safety and the influence of replacing traditional linseed oil putty by a modern adhesive or sealant are investigated under different loading situations. The parameter study leads to an insight in which parameters specific for the renovation of 19th-century iron and glass roofs, are important for simulating the overall structural behaviour.

The results of all chapters are brought together and generalized in Chapter 7. During the renovation of 19th-century iron and glass roofs, a set of interventions need to be decided on. An overview of possible interventions is given within the boundary conditions set by the historic context, the construction techniques, the heritage value, and the modern requirements for safety and structural integrity.

Final conclusions are given in Chapter 8. The research is evaluated against the set research goals and ideas for further research are outlined.

Chapter 1

Building in iron and glass in the 19th century

1. 19th-century Belgium

During the 19th century, Belgium evolved from an agricultural to an industrialised society. Scientific knowledge was applied to improve and develop techniques in all branches of manufacturing and construction. During this century, important innovations were made in iron and glass production and construction. This chapter will give an overview of the context of this time period and the consequences for the building industry.

The international evolution of iron and glass architecture will be outlined and illustrated with Belgian examples. The evolution of the production techniques of iron and glass throughout the 19th and begin 20th century will complement this historical overview.

1.1. The Industrial Revolution

The Industrial Revolution was a period in which crafts were reshaped into industries. The use of steam power, use of coal as an energy source, wide-spread mechanisation, and development of the iron industry were symbols of the Industrial Revolution. The expansion of transportation infrastructure (roads, canals, and railroads), development of the sciences (e.g. chemistry), and beginnings of engineering education were some of the other important, but sometimes less known, consequences.

The impact on the building industry was enormous. An increased knowledge about materials and their structural properties, with iron as a completely new construction material, broadened the possibilities for their use. The production volume of glass factories expanded due to the gradual mechanisation of the manufacturing processes. The renewed interest in statics and structural design in the 19th century gave birth to the profession of the engineer. Stimulating these developments were needs for buildings with new architectural features, adapted to new functions that arose from the Industrial Revolution.

The evolution of the production processes of iron and glass made it technically and economically possible to build with iron and glass. In the context of the industrialisation and the dense cities, iron and glass architecture becomes a symbol for 19th-century architecture.

1.2. Iron and glass architecture

The first confirmed use of glass for windows dates from Ancient Rome. Archaeological evidence from the excavations of Pompeii showed glass embedded in bronze and wooden frames. The merely translucent cast glass plates measured ca.

300x500 mm and were 30 to 600 mm thick[1]. However, nearly all traditional houses from that period had only a minimal amount of openings, which were usually closed with local materials like scraped hides or canvas. It was only during the Middle-Ages with the development of Gothic style that glazed windows were used more often. The typical Gothic stone skeleton made it possible to fill in the walls with stained glass windows, which brought coloured light into the interior. The colder climate in middle and northern Europe made them of particular interest. However, due to the high price of glass, the application was limited to religious buildings.
During the 18th century, the idea of a window's function changed drastically. Instead of protecting the inside from the exterior, the window became a look-out on the environment. The evolution in the production processes (paragraph 2.2) made it possible to manufacture larger glass panes, which eliminated the need for intermediate glazing bars (Figure 1-1). The iron skeleton frame was developed (at first for fireproof construction) at the end of the 18th century. The use of glass as an infill material was, analogous to the gothic period, a logical step. During the 19th and the 20th centuries, the window evolved from an outlined surface in the façade to completely glazed enclosures (Figure 1-2).

Figure 1-1: 17th –century leaded window (right) next to 18th-century sash-window (left)[2]

Figure 1-2: Fagus works, Alfeld an der Leine, Germany, Walter Gropius and Adolf Meyer, 1911[3]

[1] Schittich et al. 2007, p.10.
[2] McGrath 1961, p.192.
[3] Schittich et al. 2007, p.22.

Although glass was used since the Roman age in the building envelope, it only began to be applied together with iron in the 19th century. In 1837-38, the large demand for iron from the Belgian rail network was decreasing[4], which provoked a sharp decrease in the price. This price drop made iron attractive for use in the building industry and iron construction in Belgium around 1840[5].

The introduction of the new material in the building industry generated many discussions. The use of glass in façades was possible without breaking with the traditional architectural styles. However, new functions arose from the Industrial Revolution, which resulted in new types of buildings, e.g. factories, railway stations, department stores, exhibition buildings, and glasshouses[6]. The new public buildings provided opportunities for architects and engineers to develop and express a new architectural language using iron and glass.

Iron and glass architecture was pushed to its limits with the construction of palm houses. The typology originated mainly for two reasons. First, effort was done in the 17th century to prolong the growing period in northern Europe agriculture. Secondly, the import of exotic plants from colonies and voyages of discovery, which started in the 15th century and reached a climax in the 18th century, demanded a controlled climatic environment. The first structures for cultivating plants were temporary wooden orangeries, succeeded by orangeries with three masonry walls and a glazed wall (and sometimes a glazed roof)[7]. They finally evolved to a completely glazed glasshouse. The idea spread to other building types. At the second half of the 19th century, even utopias housed in glass buildings were conceived as a reaction to the greyness of the fast growing industrial cities.
From the 1830s, the heating and ventilation systems reached a point where they could supply larger glasshouses[8]. Joseph Paxton, in addition to his structural engineering improvements and the introduction of a modular building system, helped improve the heating and ventilation systems used in glasshouses.

[4] Linters 1987, p.34.
[5] Vierendeel 1890, p.12.
[6] Glasshouse is in this text used for all buildings with glass enclosing walls where plants are stored. This includes culture houses (for cultivating plants), conservatories (built to grow plants and display them to visitors), and winter gardens (where plants are used as decoration for social events).
[7] Kohlmaier and Von Sartory 1991; Hix 2005.
[8] Addis 2008.

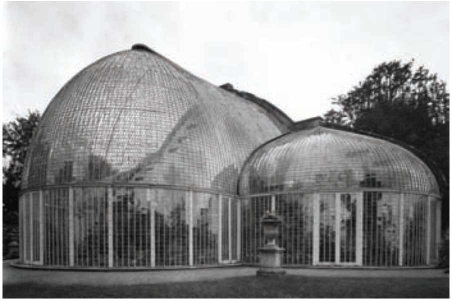

Figure 1-3: Palm House, Bicton Gardens close to Budleigh Salterton, John Claudius Loudon, 1820-1840[9]

Figure 1-4: Palm House, Royal Botanic Gardens, Kew, London, Richard Turner and Decimus Burton, 1844-48 (2006-10-25)

Figure 1-5: Royal Glasshouses of Laeken, Brussels, Alphonse Balat, Henri Maquet and Charles Girault, 1874-1905 (2009-11-17)

John Claudius Loudon, a British landscape gardener, developed the wrought-iron glazing bar and published his findings in 1817 and 1818[10]. The first iron glazing bars were composed of a flat wrought-iron bar to which angles were riveted or screwed to hold the glass. Later, a cruciform section could be manufactured by rolling it in one piece. From that time on, the iron glazing bar could compete with the previous used wooden sash bars. The main advantages were the high strength compared to the width of the section and the possibility to bend the glazing bar without reducing its strength. As a consequence, the slender glazing bars allowed for more light to pass to the interior.

Loudon also studied the geometry of the ideal glasshouses, in relation to the angle of entering sun rays. He improved the curvilinear roof (introduced by Sir G. Mackenzie in 1815[11]) where the idea was to follow the path of the sun during the day. The curved iron glazing bars were covered with small plain glass plates to minimize the

[9] Koppelkamm 1981, p.57.
[10] Loudon 1817; Loudon 1818.
[11] McGrath 1961, p.116.

facetted view. Loudon also developed a second geometry, the ridge-and-furrow roof. The saw-toothed roof collects the low sun rays, but prevents the sun at its highest point from burning the plants. This geometry became famous due to its extensive use after Joseph Paxton included it in the Crystal Palace of 1851.

Loudon, as a landscape gardener, focused on the application of his glazing bar in glasshouses. He developed the idea of the iron glazing bar, but left the commercial exploitation to his partners, W. & D. Bailey. Since 1818, this cooperation was very successful. A very elegant and still existing example of Loudon's framework system is the *Palm House* in Bicton Gardens at Budleigh Salterton in Devon, UK[12] (Figure 1-3). The year of construction is uncertain, but between 1820 and 1840[13], and D. & E. Bailey, the successors of W. & D. Bailey, were the contractors for the work. The cast-iron glazing bars have a cruciform section, are stiffened by the glass plates and are supported by cast-iron columns[14].

One of the earliest Belgian examples of glasshouses was built in 1827-1829 at the Botanique (the former botanical garden of Brussels) by P.-F. Gineste. A neoclassicistic building with a glazed rotunda was flanked by glazed annexes. Unfortunately, the glass cladding and the glazing bars were replaced during the 1979-1983 renovation of the building to a cultural centre.

There are many examples of glasshouses in England. The 1844-48 *Palm House* in the Royal Botanic Gardens at Kew (London), designed by Richard Turner and Decimus Burton[15] (Figure 1-4), was an inspiration for the Belgian architect Alphonse Balat when designing the *Wintergarden* in Laeken, Brussels[16]. The *Wintergarden* is part of the *Royal Glasshouses of Laeken* that were built between 1874 and 1905 by architect Balat and finished by architects Henri Maquet and Charles Girault[17] (Figure 1-5).

Glasshouses are the oldest witnesses of the use of rolled wrought-iron sections in buildings[18]. Around 1817, W. & D. Bailey developed a rolling process in collaboration with Loudon. The first rolled sections had a cruciform shape with rebates for placing the glass. In the 1820s, these rolled sections were used widely in

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[12] Koppelkamm 1981; Baele and Herdt 1983; Wigginton 1996; Addis 2008.
[13] Koppelkamm 1981; Wigginton 1996.
[14] Kohlmaier and Von Sartory 1991, p.132; Pender, Godfraind, and English Heritage 2011, p.318.
[15] Koppelkamm 1981; Minter 1990; Wigginton 1996; Schittich et al. 2007.
[16] Lauriks et al. 2008; Lauriks, de Bouw, and Wouters 2009.
[17] Goedleven, Fornari, and Vandenbreeden 1989; Lauriks 2007.
[18] Kohlmaier and Von Sartory 1991, p.119–120.

glasshouses[19]. In the same decade, the first patent for rolled wrought-iron sections for rail tracks was submitted by J. Birkinshaw in Great-Britain[20]. New rolling mills were built that could roll large, non-rectangular wrought-iron sections, which were used for railroad tracks. These mills also started to produce also smaller L- and T-shaped sections in the 1830s that were used in the building industry[21]. The rolled I-section followed at the end of the 1840s, with probably its first application in the building industry in the *Palm House* at Kew Gardens[22].

In the first half of the 19th century, cast iron was still preferred for structures with a clear hierarchy of primary and secondary load-bearing components (with a higher percentage of cast iron than wrought iron used for the large construction works[23]). Cast iron was cheaper than wrought iron, it had a better corrosion resistance, and it was considered a fireproof material[24]. Until around 1820, wrought-iron sections were limited to rectangular shapes, while cast iron could be moulded in nearly any shape (which was used among others to improve the stiffness of the sections around their weak axis). The length of cast-iron girders, however, was limited due to internally generated stresses, and a bolted connection was not stiff enough to realize a continuous component[25].

Glasshouse building, with its early use of wrought iron, contrasts with the traditional introduction of new materials and techniques in the bridge, shipping, military, and railway before the building industry[26]. Loudon himself already described the two different design principles: closely spaced wrought-iron glazing bars stiffened with puttied glass plates compared to glazing bars at a greater distance supported by cast-iron ribs[27]. Cast iron was also used for vertical window frames[28].

The *Palm House* in Kew is an example of the junction of these two structural design principles. Originally, Richard Turner planned to use cast-iron ribs to support the glazing bars, but eventually changed this in favour of the wrought-iron I-shaped deck beams[29]. The slenderness of the glass shell is ensured by the use of the wrought-iron glazing bars, while the larger span is only possible because of the hierarchical structure with primary wrought-iron arches and purlins.

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[19] Ibid., p.80.
[20] Lemoine 1986, p.48; Kohlmaier and von Sartory 1991, p.79; Addis 2012, p.519.
[21] Addis 2012, p.519.
[22] Kohlmaier and Von Sartory 1991, p.120.
[23] Ibid., p.119.
[24] Ibid., p.77–78.
[25] Ibid., p.119.
[26] De Bouw 2010, p.15.
[27] Loudon 1817, p.36–37.
[28] Kohlmaier and Von Sartory 1991, p.80.
[29] Lemoine 1986, p.48.

The Industrial Revolution owed part of its success to the efficient exchange of knowledge. The world exhibitions contributed to this exchange by displaying many advanced products of industrial manufacturing processes (transport, mining, metallurgy, textile, building physics, printing, photography, etc.). The buildings of these exhibitions provided a perfect context for innovative architecture, which was admired by all visitors.

The first world exhibition was *The Great Exhibition of the Works of all Nations* in 1851 in Hyde Park in London. The *Crystal Palace*, designed by Joseph Paxton with the collaboration of the contractors Fox and Henderson, was the impressive housing for this event (Figure 1-6). Due to the prefabrication of the standardised components and the process of the assembly, the whole building of 563 by 124 meters was built in only 7 months[30]. The glazed ridge-and-furrow roof consumed a third of the annual production of glass in England[31]. The length of the cast glass plates, the longest that could be produced at that time, defined the modular dimensions of the whole building[32]. The structural frame of the building was built up of cast-iron columns, combined cast- and wrought-iron girders and compound timber and wrought-iron arches[33]. In 1852, the whole structure was disassembled and rebuilt in Sydenham as a recreational centre. It was completely destroyed by fire the 30th November 1936[34]. The Crystal Palace was an inspiration for building in iron and glass, followed by many others.

World exhibitions were held all over the globe and glass exhibition buildings were common features of them. The influence of the Paris exhibitions (in 1855, 1867, 1878, 1889 and 1900[35]) was indispensable. The *Galérie des Machines* of the exhibition of 1889 (Figure 1-7), designed by architect Dutert and engineer Contamin, was the result of the growing knowledge of stability of structures. The main frame consisted of a three-hinged arch spanning circa 115 meters[36]. The hall was covered at its centre with large cast glass plates, sealed with putty to the iron, T-shaped glazing bars[37] (Figure 2-12 p.64). Unfortunately, the hall was demolished in 1910.

[30] Baele and Herdt 1983; McKean, Durant, and Parissien 1999.
[31] Baele and Herdt 1983; McKean, Durant, and Parissien 1999.
[32] Vierendeel 1902, p.66.
[33] Addis 2006, p.6–7.
[34] McGrath 1961; Baele and Herdt 1983; Wigginton 1996; McKean, Durant, and Parissien 1999.
[35] Baele and Herdt 1983.
[36] Ibid.; Wigginton 1996; McKean, Durant, and Parissien 1999.
[37] Vierendeel 1902, p.226, 352.

Figure 1-6: Crystal Palace, London, Joseph Paxton, 1851[38]

Figure 1-7: Galérie des Machines, Paris, Contamin and Dutert, 1889[39]

Figure 1-8: Royal Museum of the Armed Forces and of Military History, Brussels, Fr. Heyninx and Gédéon Bordiau, 1888 (2006-09-01)

World fairs were held in Belgium in 1880 (Brussels), 1885 (Antwerp), 1888 (Brussels), 1894 (Antwerp), 1897 (Brussels), 1905 (Liège), 1910 (Brussels), and 1913 (Ghent)[40]. The Brussels *Cinquantenaire's Halls* were built in 1888 by engineer Fr. Heyninx and architect Gédéon Bordiau for the international exhibition *Grand Concours International des Sciences et de l'Industrie*. They were built in the Cinquantenaire Park, which served as the site for nearly all 19th-century Brussels exhibitions, and they are one of the few remaining buildings of the Belgian world exhibitions. The halls were expanded, cut in

[38] McKean, Durant, and Parissien 1999.
[39] McGrath 1961, p.223.
[40] Baele and Herdt 1983.

two and partly demolished in the next 20 years[41]. The two separate halls are now used for museums for automobiles (south hall) and aviation (north hall). The halls consist of a series of three-bay portal frames with a central bay of 230 by 48 meters spanned with wrought-iron trussed arches[42] (Figure 1-8).

The search for large iron spans was also important for the railway stations. These buildings were symbols of the newly improved transport system and were considered ideal to impress the passengers when entering a city for the first time. In many 19th century stations, the rail platforms were covered with a hall usually designed by an engineer, while the passenger lobby and offices were housed in a more traditional building, designed by an architect. In the beginning of the 19th century, the platforms were mostly covered by a series of spans. In England shed roofs were often used for covering the platforms, while in France the Wiegmann-Polonceau trusses were popular[43]. During the nineteenth century, there was an evolution towards covering the platforms with one large arched structure. The *Paddington Station* in London, designed by Isambard Kingdom Brunel and Matthew Digby Wyatt in 1852-54[44], is one of the first examples of the use of wrought-iron arched ribs with only one span for the whole platform hall. In 1900, iron tension rods had to be added to resist the enormous thrust forces[45]. The station was built by the same contractors that built the Crystal Palace in 1851.
The *Saint Pancras Station* in London, designed by engineer William Henry Barlow and bridge engineer R.M. Ordish, was built in 1866-68[46] (Figure 1-9). The wrought-iron rigid arches span 74.8 meters; their ends are tied with iron tie rods in the floor[47]. Roughly half of the roof is covered with glass with the ridge-and-furrow geometry. At the centre of the roof arch, the slope is too low to ensure good water-tightness, but with the ridge-and-furrow roof the arch can still be glazed without interruption[48]. The station now serves as the international station for Eurostar trains.

[41] Collette et al. 2012.
[42] Ibid.
[43] Schittich et al. 2007, p.17.
[44] McGrath 1961; Stoop 1981; Baele and Herdt 1983; Schittich et al. 2007.
[45] Baele and Herdt 1983.
[46] McGrath 1961; Stoop 1981; Baele and Herdt 1983; Wigginton 1996.
[47] Barlow 1870, p.84; Wigginton 1996.
[48] Vierendeel 1902, p.113.

Figure 1-9: Saint Pancras Station, London, William Henry Barlow and R.M. Ordish, 1866-68 (2011-08-03)

Figure 1-10: Antwerp Central Station, Antwerp, C. Van Bogaert, 1895-98 (2012-03-20)

Many Belgian train platform halls built in the 19th century have been demolished (Brussels-South 1869-1949, Brussels-North 1869-1949, Mechelen 1888-1957[49]). Most of these examples were partly covered with glass. At the ridge of the hall, the slope of the arch was too low to continue the glazing, thus a small pitched roof called "lanterneau" was added. The platform hall of the *Antwerp Central Station*, designed by C. Van Bogaert and built between 1895 and 1898[50], is one of the rare remaining 19th-century Belgian platform halls (Figure 1-10). The entrance building was built later (1900-1905) and designed by architect Louis Delanceserie. The three-hinged arches of the platform hall, built by *la Cie central à Haine Saint Pierre*[51], spanned 64 meters. The centre of the roof was glazed with wired glass[52], with a "lanterneau" at the apex. The railway station was drastically enlarged and renovated between 2000 and 2009.

[49] webpage Demeulder 2012.
[50] Plomteux and Steyaert 1989, p.255–260.
[51] webpage Demeulder 2012.
[52] Wired glass was a glass plate with a wire net embedded in it during manufacture so that the risk of falling glass pieces was reduced.

The Industrial Revolution was the start of the consumer economy. New types of buildings originated from the need to present products in an attractive setting. The shopping arcade was a covered street where the bourgeoisie could buy luxury articles in a steady environment protected from cold and rain. The decorated stone façades contrasted with the lightweight iron and glass covering. Nearly every European city had its own covered arcade with shops.
A famous example is the *Galérie d'Orléans* in Paris, built by Pierre François Fontaine in 1828-30[53] (Figure 1-11). It was the first arcade that was completely covered with glass[54] and was part of the *Galéries du Palais Royal*, a large complex of galleries and shops. It had a large influence in Europe, but was unfortunately demolished in 1935.

In Belgium, the most renowned shopping arcade is a complex of three covered streets in the centre of the capital Brussels: the *Saint-Hubertus Galleries* (Figure 1-13). They were built in 1846-47 from designs by the architect Jean-Pierre Cluysenaar, who found inspiration in the *Galérie d'Orléans*. The *Saint-Hubertus Galleries* fit into the existing urban structure: the depth of the shops was adjusted to the surrounding buildings[55]. Deeper plots were filled up with public functions such as a theatre. The galleries were one of the first in Europe where the glass roof was put higher so that it seemed one was walking in a street rather than an interior space (Chapter 4 paragraph 4). The glazing bars served a load-bearing function and held the glass plates in place.
Another international example worth mentioning is the *Galleria Vittorio Emanuele II* in Milan, Italy built in 1865-67[56] (Figure 1-12). Its architect, Giuseppe Mengoni, had been in contact with J.-P. Cluysenaar[57]. The similarities are stunning. The span of the structure is larger compared to the *Saint-Hubertus Galleries* and a separate steel primary load-bearing structure again appeared[58]. The Milanese gallery was built by an English contractor and published in English journals. It became widely known and was used as an inspiration for many other shopping galleries.

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[53] McGrath 1961; Stoop 1981; Baele and Herdt 1983; Wigginton 1996; Schittich et al. 2007.
[54] Schittich et al. 2007.
[55] Baele and Herdt 1983; Reis et al. 1998, p.41.
[56] Stoop 1981; Wigginton 1996.
[57] Baele and Herdt 1983; Geist 1983, p.199.
[58] Gianni 1989, p.197.

Figure 1-11: Galérie d'Orléans, Paris, Pierre François Fontaine, 1828-30[59]

Figure 1-12: Galleria Vittorio Emanuele, Milan, Giuseppe Mengoni, 1865-67[60]

Figure 1-13: Saint-Hubertus Galleries, Brussels, Jean-Pierre Cluysenaar, 1846-47 (2010-04-22)

[59] McGrath 1961, p.240.
[60] Hitchcock et al. 1963, p.305.

The *Palm House* in Kew demonstrated the junction of the two design principles already described by Loudon: one, a shell-like structure with slender wrought-iron glazing bars and glass plates and the other, a hierarchical structure with a wide-span primary component that supports the secondary glazing bars. The typology of the arcade made a comparable evolution. The *Saint-Hubertus Galleries* are clearly influenced by the slender glasshouse building: the wrought-iron glazing bars are at the same time the primary load-carrying structure. The *Galérie d'Orléans,* however, has a clear hierarchical structure. Both galleries span a comparable gallery width, but were covered using a different structural design principle. The span of the *Galleria Vittorio Emanuele II* is larger (almost the double) and built in steel. The characteristic supporting arches illustrate two structural developments: towards a combination of primary structure with slender glazing bars and towards the use of wrought iron and steel instead of cast iron for load-bearing structures.

Apart from the typologies described, iron frames with a glass filling were also used in other buildings where bringing light to the core of the building was advantageous. These include market halls (e.g. *Sint-Goriksmarkt*, Brussels, 1881[61]), stock exchanges (e.g. *Antwerp Exchange*, Antwerp, Joseph Schadde, 1872[62]), warehouses, and department stores (e.g. *Old England*, Brussels, Paul Saintenoy, 1899[63]). On top of that, iron and glass was also applied in many small structures like porches, sky lights in domestic buildings, etc.

The development of the production techniques of the iron and glass materials was very important for the evolution of the 19th-century iron and glass architecture. The following paragraphs will investigate the developments throughout the 19th and begin 20th century.

61 Stoop 1981; Baele and Herdt 1983.
62 Manderyck, Plomteux, and Steyaert 1979; webpage Origin 2009.
63 Stoop 1981; Baele and Herdt 1983.

2. Material developments during the industrial revolution

2.1. Iron and steel

Iron ore was heated in a blast furnace to obtain pig iron. The first blast furnaces in Belgium for which there is documentation date from the 16th century[64]. The invention of Abraham Darby I in 1709 to use cokes instead of coal as a fuel for these blast furnaces made it possible to reach higher temperatures (above the fusion point of iron > 1535 °C) and thus produce better quality pig iron.

When the pig iron was afterwards melted again and cast into moulds, the resulting product was called cast iron: it was an alloy of iron, carbon and silicon. The carbon is present in separate lamina, causing a high compressive but low tensile strength of the material (Table 1-1).

Wrought iron was produced out of pig iron in a puddling furnace, invented by Henry Cort in 1784. By bringing the crude iron in contact with hot air in the puddle furnace, the carbon content in the iron was decreased. The resulting product had to be hammered and worked to reach the final product that could be rolled into sections. Wrought iron contains a carbon content lower than 1% and has a fibrous and layered structure. The mechanical characteristics are thus orthotropic: they are higher in the rolled direction than in the direction perpendicular on the layered structure (Table 1-1).

Table 1-1: Properties and application of wrought iron, cast iron and steel[65]

		Wrought iron	Grey Cast Iron	Mild steel
Proportion of carbon	%	0.02 - 0.05	2.5 - 4.0	0.2 - 1.0
Temperature to manufacture	Deg C	1000	1130-1200	1500
First use in Europe		c.1500 BC	14th C	mid-19th C
First use in European buildings		c.500 BC	1770s	1870s
Fracture behaviour		Ductile	Brittle	Ductile
Tensile strength	N/mm²	280-370	120	350-450
Compressive strength	N/mm²	240-310	600-800	350-450
Stiffness (Young's modulus)	kN/mm²	155-220	85-90	210
Castability		–	Very good	Good
Corrosion resistance		Good	Very good	Poor
Internal structure		Fibrous	Crystalline	Amorphous
Weldability		Possible	Difficult	Very good

[64] De Bouw 2010, p.70.
[65] Addis 2010, p.35.

Wrought and cast iron were used for tools, weapons and other applications. As a building material, wrought iron was first used for iron ties and cramps. The railroad expansion and the development of the fireproof construction gave rise to the spread of the use of both cast and wrought iron as a main building material in the late 18th and the 19th centuries.

The production of steel for structural application was developed in the second half of the 19th century[66]. Steel contains less than 2% carbon, which situates it in between cast and wrought iron. Following the invention of the Bessemer converter in 1855 and the Siemens-Martin converter in 1864-1865, which were adopted in many countries, steel became more widely available. The invention of the Thomas-Gilchrist converter by Sidney Gilchrist Thomas and his cousin Percy Carlyle Gilchrist in 1877-1879 was decisive for the mild steel production in Belgium. This converter was better suited for Belgian iron ores, which had a high phosphorous content. The chemical composition and the mechanical characteristics of these different mild steels can vary (Table 1-1).
With these new converters, the quality of the steel produced was higher and more constant. As a consequence, steel was spreading as a structural building material. With the invention of the electric furnace in 1900, the production of steel comparable to today’s steel got a start.

In the 19th century, the developments of iron production varied considerably among countries. De Bouw[67] gives an overview of the developments worldwide and more specifically in Belgium (Figure 1-14). Belgium was an important manufacturer of iron, the nation with the highest output per blast furnace between 1870 and 1890[68] in the world.

The development of new methods for manufacturing cast iron, wrought iron and mild steel were improving the quality of the resulting metal as well as its quantity. The 19th century developments were decisive for the use of iron and steel in the building industry. The development of the iron and glass architecture in the 19th century is particularly due to this broadened use.
In the next section, the evolution of the production of glass during the 19th century will be examined.

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[66] De Bouw 2010, p.65–115.
[67] De Bouw 2010, Chapter 3.
[68] Ibid., p.68.

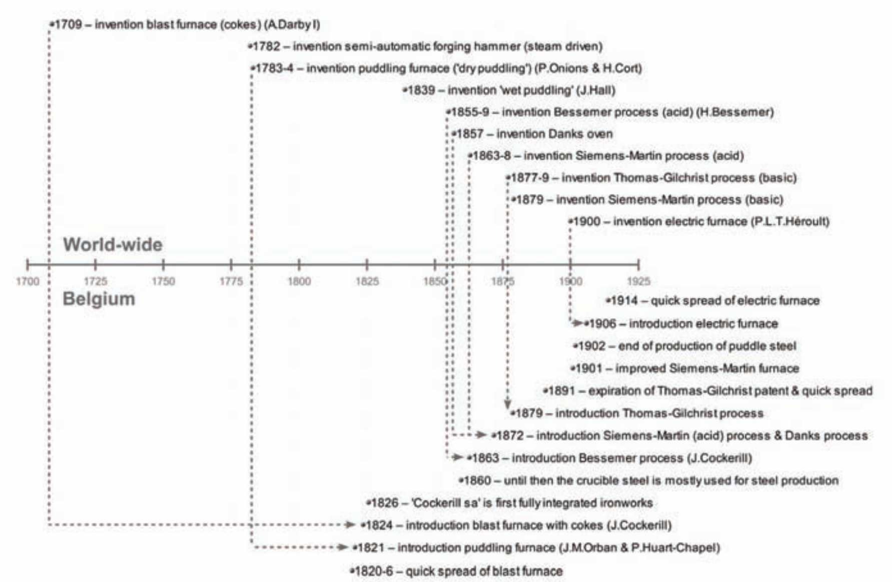

Fig.III - 1: World-wide developments and their adoption in Belgium

Figure 1-14: Timeline of the production and application of iron during the 19th century[69]

2.2. Glass production in the 19th century

Glass is an amorphous material made from silica (sand), alkali (sodium and potassium) and a stabilizer (calcium, magnesium, etc.). These three constituents melt at 1400-1500°C (just below the melting temperature of steel)[70] and have to settle (cool down) to make the glass workable. Once the geometry of the glass product has been formed, the material has to be annealed. Annealing means that the glass has to be cooled down to solidify again at 500°C[71], not too fast to limit the creation of internal stresses due to temperature differences, but also not too slow because then devitrification can occur (it gets a crystalline instead of amorphous structure)[72].

The origin of glassmaking is uncertain. It was probably discovered by chance, in Western Asia, from glazing earthenware in an open fire. The first evidence of glassmaking dates from the 5th century BC in Mesopotamia[73]. The blowing iron was discovered by Syrian craftsmen, between 200 BC[74] and 50 BC[75]. Knowledge of

[69] Ibid., p.69.
[70] Douglas and Frank 1972.
[71] Ibid.
[72] Muspratt 1860.
[73] Schittich et al. 2007.
[74] Ibid.

glassmaking was exported from Syria to Egypt and then further to the Romans. Evidence of the use of glass in the building envelope during the Roman Age was found at the excavations of Pompeii. The glass from the 1st century AD was produced using a primitive method of casting, and it was not transparent[76]. Glassmaking was transferred by the Romans to Western Europe, but exactly when is unknown.

Two methods for making sheet glass were developed by Syrian craftsmen and applied in Western Europe. One was blown cylinder glass, invented in the 1st century AD, while the other, crown glass, was known since the 4th century AD[77]. Cylinder glass was made by blowing a bulb, elongating it by swinging and turning and finally cutting and flattening it to a large sheet (Figure 1-15). Crown glass started the same way, but the sheet was achieved by spinning the bulb and then cutting small panes out of the circular sheet (Figure 1-16).

Crown glass was dominating in England and Normandy (north-west France), while cylinder glass was the major kind in Lorraine (north-east France), along the river Rhine and in Belgium.

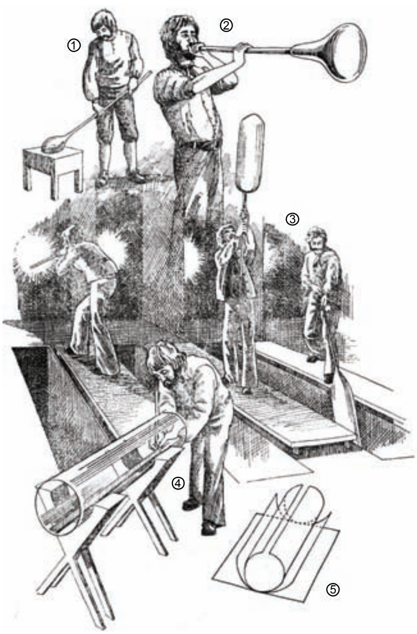

Figure 1-15: Production of cylinder glass[78]

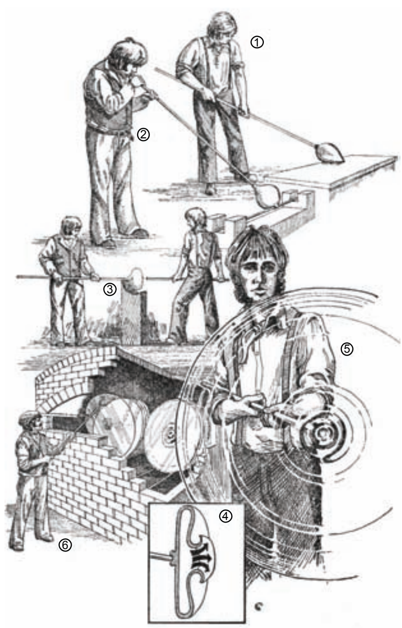

Figure 1-16: Production of crown glass[79]

[75] Douglas and Frank 1972.
[76] McGrath 1961.
[77] Schittich et al. 2007.
[78] Image © Pilkington Group Limited. Reproduced with permission.
[79] Image © Pilkington Group Limited. Reproduced with permission.

During the Middle Ages, glass was very expensive and thus merely used in stained-glass windows in churches, cathedrals, and monasteries. From the 15th century, Venice became a centre for the production of decorative glassware and mirrors. The Venetian craftsmen were able to produce very pure glass without any colour. Encouraged by the immigration of Italian craftsmen during the 15th century, the Belgian region saw a revival of glass production. Glassworks were founded in Belgium in the region of the river Sambre (Figure 1-22), where forests were close-by[80]. In the 16th century, Antwerp, a seaport, and Belgium in general experienced economic prosperity, and more people could afford glass windows. The use of glass spread to more applications than only religious buildings. In the 17th and 18th centuries, glass became common in palaces and houses in the cities. Due to the fast disappearance of forests, experiments for using coal instead of wood as a fuel started in the 17th century. As a consequence, glassworks were concentrating around the coal supplies in Europe, of which Belgium was one of the most important[81].
A third production process, cast glass (later called plate glass) was developed mainly in the second half of the 17th century in France[82]. The production process involved casting the liquid glass on a table and grinding and polishing the surfaces after annealing. The resulting product was of excellent quality but more expensive and thus mainly used for luxury purposes and mirrors.
During the 19th century, glass production became more mechanized, which impacted the operation of furnaces, the quality of raw materials as well as the energy sources. France was the largest industrial glass manufacturer in the world in the beginning of the 19th century, but from around 1830s until 1880 Belgium became the main producer (in m^2 of glass production)[83]. The majority of the country's glass (mainly cylinder) was exported, and Belgian glass was used all over the world in the building envelope (with Great-Britain, the United States, the Netherlands, Hamburg and the Turkish Empire being the main customers[84]). The developments of the glass production in Belgium during this period of enormous export are therefore internationally relevant. In the 1880s, the dominant position of Belgium got a first setback with the introduction of the new tank furnace, which saved on specialized craftsmen. With the invention of the Fourcault process, Belgium contributed to the development of mechanised production techniques in the beginning of the 20th century. Nevertheless, this introduction of production processes that saved on labour reduced the need for skilled craftsmen. It made it easier for non-producing countries

80 Pesch 1949; Douxchamps 1951.
81 Mille 1982.
82 McGrath 1961.
83 Mille 1982.
84 Barker 1977.

to start up their own glass production and as a consequence reduced the international role of Belgium. The invention of the float glass process by Alastair Pilkington in the 1950s blew a new wind of change through the international industry of glass production.

The history of window glass production can be divided in several periods (Figure 1-17). Several researchers have made use of this chronological development to set up a dating technique based on the chemical composition of flat glass. English Heritage carried out a broad investigation on English window glass and summarized the results in a flow chart to date the glass panes based on their chemical composition (Figure 1-18). By using a portable X-ray fluorescence (pXRF) spectrometer for non-destructive chemical analysis, this dating technique can be used on-site (but the quality of the analysis depends strongly on the surface condition of the glass panes)[85].
Belgian researchers have carried out similar research on Belgian window glass dating from between the 12th to the 18th centuries[86]. They provide evidence of when new raw materials and new glassmaking recipes were adopted in plain and stained-glass windows, providing a methodology to date glass based on its chemical composition (Figure 1-19). The following paragraphs provide a first step in extending the approach to dating glass to the 19th century Belgian window glass.

Technical improvements were guided by the need to reduce fuel consumption and desire to improve the aesthetics of glass: to better control the colour (mainly defined by the raw materials) and the opacity, defects, thickness variation and surface finish (defined by melting and working the glass). The development of the glass production techniques will be discussed chronologically while focussing on both technical (raw materials, melting and processing) and economic issues (Figure 1-25).

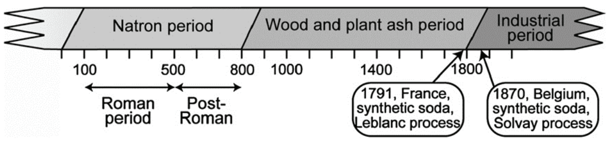

Figure 1-17: Overview of the history of window glass production[87]

[85] Dungworth 2012.
[86] Schalm et al. 2007.
[87] Ibid., p.664.

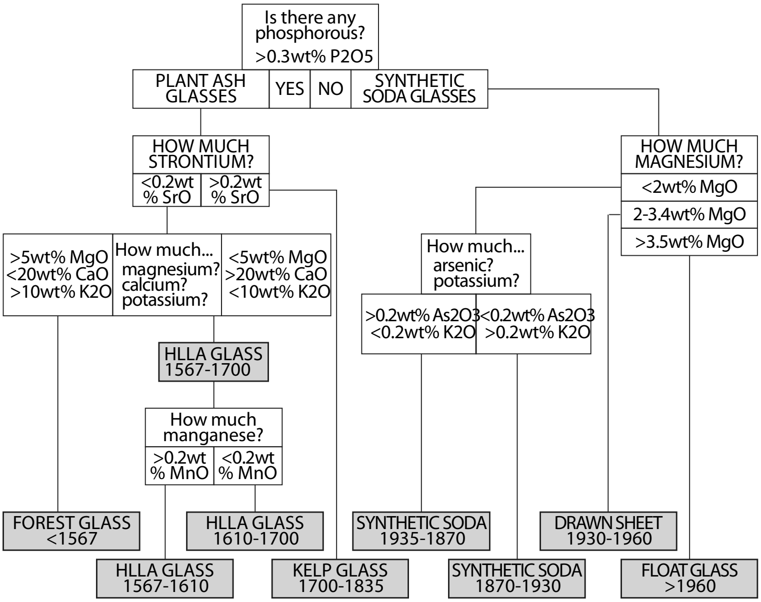

Figure 1-18: Dating English window glass up to 20th century[88]

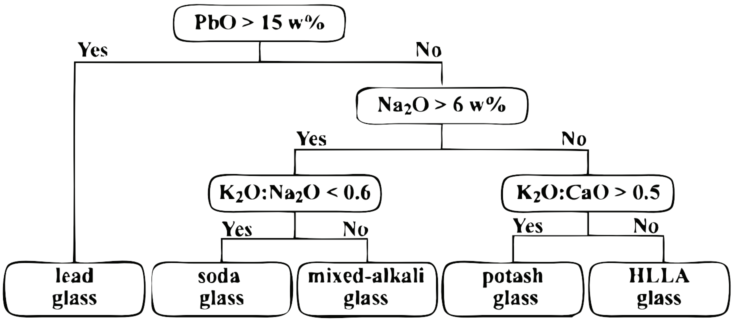

Figure 1-19: Classify Belgian window glass from 12th to 18th century[89]

[88] Dungworth 2012, p.17.
[89] Schalm et al. 2007.

Up to the 18th century

Up to the 17th century, glass had a greenish-yellowish tint due to impure and varying raw materials and uncertain melting circumstances. Kelp (English marine plant ashes), barilla (Spanish plant ashes) and potash (burnt wood) were used as alkali sources. All plant ashes gave colour to the glass due to iron traces, and can be detected by the phosphorous content in the chemical composition of the glass[90].

Plant ashes were used for their alkali content, but also contained lime. Since this was enough for producing glass, lime was not deliberately added to the batch until the 16th century[91]. However, the lime content was not as high as necessary for a durable glass, and as a consequence not much glass from before that time survives today.

The furnaces used until the 18th century could probably not reach temperatures above 1200°C, which resulted in glass with more bubbles, defects, etc[92]. Incomplete melting could lead to inclusions of unmelted material in the glass[93]. The glass was coloured by the raw materials as described above, but also by interaction with the fumes of burning wood to heat the furnace[94]. Glassworks were small firms that used a lot of wood for fuel and potash, so moved en mass when the wood in an area ran out[95].

In the first half of the 17th century, a major change took place in the furnace technology. Due to the fast disappearance of the forests, the use of wood for glass production was prohibited in England in 1615[96]. The price of wood in Belgium also reached high levels after 1625[97]. Coal then replaced wood. It had a higher heat capacity than wood and so larger crucibles and larger furnaces were possible[98]. The first use of coal for glass production was in 1619 in Rouen in France (obliged by the French authorities) and in 1635 in England[99]. Thiry Lambotte, the master glassworker of the glassworks in Namur, was the first to apply coal as a fuel in Belgium in 1643[100]. The only coal with adequate quality came from the region of Charleroi [101]. Glassworks were concentrating around this southern Belgian city (Figure 1-22). Coal furnaces were closed structures designed to prevent sulphurous fumes from acting on the

[90] Dungworth 2011.
[91] Lefèbvre 1938; Pesch 1949.
[92] Douglas and Frank 1972.
[93] Dungworth 2012.
[94] Pesch 1949.
[95] Mille 1982.
[96] McGrath 1961; Douglas and Frank 1972.
[97] Lefèbvre 1938.
[98] Ibid.
[99] Mille 1982.
[100] Lefèbvre 1938; Pesch 1949.
[101] Lefèbvre 1938; Pesch 1949; Douxchamps 1951.

glass[102]. In a vaulted furnace, coal was placed on a grid in between the crucibles for melting glass. It was not until 1708 that coal was used for iron production[103].
The cylinder glass production process was introduced in Belgium in the first half of the 18th century by immigrating German craftsmen. The date 1727 is mentioned in several sources as the start of cylinder glass production by Nicolas de Moreau in Belgium[104]. By 1770, all houses in Belgian cities had glazed windows[105]. The Charleroi region had almost a monopoly on Belgian window glass production.

The 19th century in general
The cost of producing window glass was defined by three major parameters: labour costs, fuel and raw materials (Figure 1-20). The total cost contribution of the raw materials overall decreased from 1840 to 1910, mostly due to the price decrease of alkali[106]. The price of fuel increased, but coal consumption per square meter of glass decreased[107]. The total cost contribution of fuel was very sensitive to changes in both the consumption efficiency and the price of coal, but was almost the same in 1910 as in 1840. In the same period, there was an increase in the salaries but also in worker efficiency. The contribution of labour in the total production cost was slightly higher in 1910 compared to 1840[108]. Labour was cheaper in Belgium than in England and France, but coal was less expensive in England[109]. Probably not the production cost but the assertive Belgian export policy was decisive for the success of the nation's glass industry[110].

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[102] Douglas and Frank 1972.
[103] Ibid.
[104] Lefèbvre 1938; Pesch 1949.
[105] Lefèbvre 1938.
[106] Ibid.; Douxchamps 1951.
[107] Lefèbvre 1938; Douxchamps 1951.
[108] Lefèbvre 1938; Douxchamps 1951.
[109] Bontemps 1868.
[110] Daviet 1988.

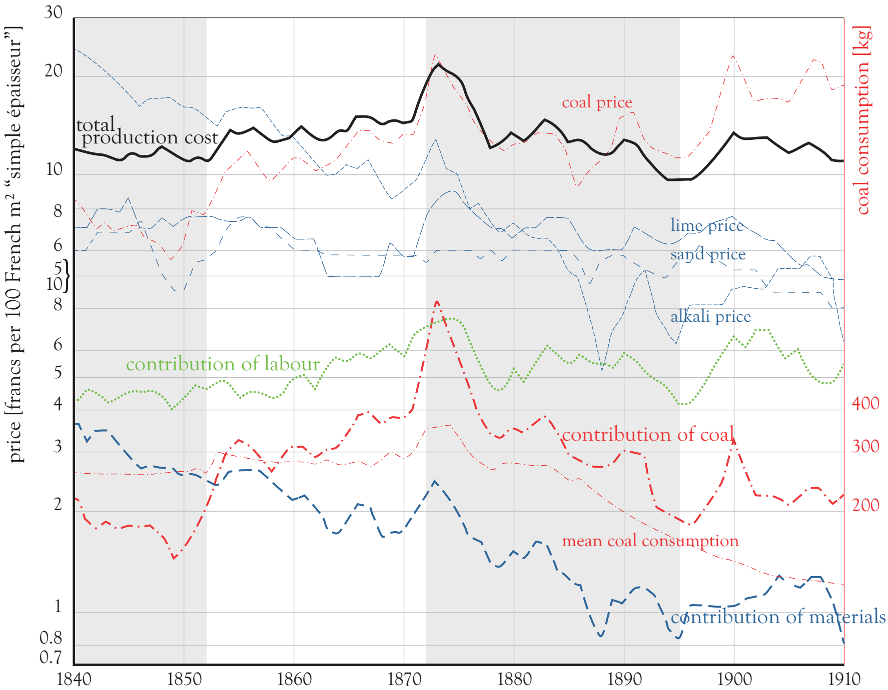

Figure 1-20: Production cost of Belgian window glass[111]

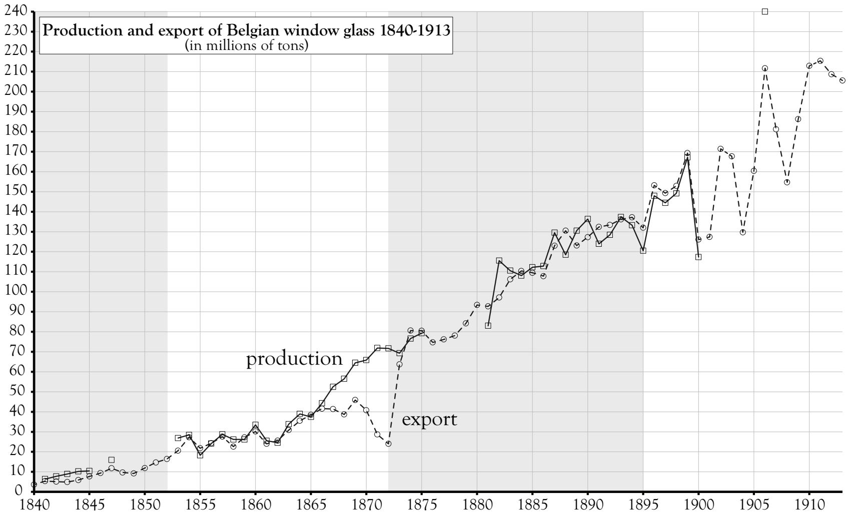

Figure 1-21: Production and export amount of Belgian window glass 1840-1910 (export numbers[112], production numbers[113] converted from m² to tons[114])

[111] Douxchamps 1951, p.479. with layout modifications by the author.
[112] Lefèbvre 1938.
[113] Douxchamps 1951.

Belgian export of window glass was nearly sixty times larger in 1910 than in 1840 (Figure 1-21). This growth was directly linked to changes in production costs, from which four periods can be defined (see the graph in Figure 1-20):
- the production cost from 1840 to 1852 decreased very little while the export increased at an annual growth rate of 10%;
- from 1852 to 1873 the production cost rose but the Belgian manufacturers found new export markets[115] so the export amount was stable;
- a third period, 1873-1895, again saw a decrease of production cost and so an increase of exports;
- the export numbers from on 1895 became very variable, because of the distribution of the tank furnace.

During the 19th century, 90 to 95% of Belgium's glass was exported worldwide, except to France and Russia, with which Belgium had non-import agreements[116]. Belgian manufacturers mainly exported to countries that did not have any glass production, to avoid competition[117]. In 1906, the glass production in Belgium represented 1/5th of the European and 1/6th of the worldwide production[118], of which 88% got exported.

The Belgians had a privileged position in the international market due to its domestic sources of raw materials and coal, improving transport (railroad and steam navigation), the large seaport of Antwerp, cheap labour, and plentiful capital[119]. Before 1909, the amount of production was not linked to what was sold, because the master glassworkers who were organizing the production had limited knowledge of economics and demand[120].

Belgium had supplies for most of the raw materials used in the 19th century to produce glass. Sand was found in two Belgian regions (Figure 1-22). The sand from Waterloo contained a small proportion of iron oxide[121], while the sand from Mol was almost free of iron oxide and was exported to glass factories in Western Europe[122]. The colour of the sand was not representative for its purity. For window glass production, the iron oxide content had to be lower than 0,5%[123].

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[114] Henry 1957.
[115] Douxchamps 1951.
[116] Banque nationale de Belgique 1936; Lefèbvre 1938.
[117] Lefèbvre 1938.
[118] Chambon 1955.
[119] *Nos exportations de verres à vitres jusque 1913* 1930; Douxchamps 1951.
[120] Lefèbvre 1938.
[121] Chance 1883.
[122] Linters 1987.
[123] Chance 1883.

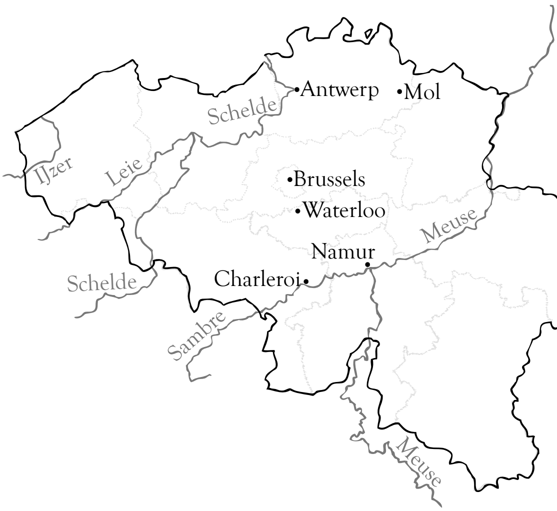

Figure 1-22: Map of Belgium with places and rivers mentioned in glass production history

Calcium could be obtained from limestone or chalk. The valleys of the rivers Meuse and Sambre supplied limestone (Figure 1-22). Limestone could contain iron traces[124], while chalk was often more pure[125]. Lime increased the hardness and the lustre of the glass[126], but increased the tendency of the glass to devitrify[127]. Sources of natural alkali were mainly found in the forests of the Sambre and the Meuse valleys.

Scientist knew that sodium chloride (normal salt) could not be used for glass production. Other insights were that alumina entered the glass composition as an impurity due to disintegration of the crucible or furnace clay[128] and that adding cullet (broken glass) eased the melting of the other constituents[129]. Manganese gave the glass a pink colour and was thus used to balance out the greenish tint originating from the iron oxide, with a slight greyish glass as a result[130].

[124] Muspratt 1860.
[125] Dungworth 2009.
[126] Muspratt 1860.
[127] McGrath 1961.
[128] Chance 1883.
[129] Muspratt 1860; Chance 1883.
[130] Dungworth 2012.

1790-1823: Introduction of Leblanc soda

From the 1790s to the 1810s, Belgium was part of the French Empire. Due to the high prices of natural alkali (potash and barilla), the French academy offered a prize in 1775 to whoever developed a process to make soda out of common salt[131]. In 1787-93, the French chemist Nicolas Leblanc invented a process[132], consisting of two stages[133]:

- common salt (sodium chloride NaCl) was heated with sulphuric acid, forming saltcake (sodium sulphate Na_2SO_4);
- saltcake was heated with lime and charcoal or coal, to obtain industrial soda ash (sodium carbonate Na_2CO_3).

The last reaction could be less effective, resulting in a proportion of saltcake in a batch of soda ash[134].

In the 1810s and 1820s, the production of Leblanc soda was industrialized[135]. Glass produced with Leblanc sodium carbonate as the alkali source was of better quality and colour compared to natural alkali glass[136]. Due to the application of industrial soda ash, lime had to be added separately to improve the chemical durability of the glass[137].

In 1815, the Netherlands annexed Belgium. This was followed by an economic slowdown in glass manufacturing, but the industry recovered, gradually, after 1821 with export to the Netherlands and its colonies[138].

1823-1840: Switch to Leblanc sodium sulphate

After Belgium won its independence in 1830, the country developed from an agricultural to an industrial state[139]. In 1836, the association *Manufactures de Glaces, Verre à Vitres, Cristaux et Gobeleteries* was found, which united several glass manufacturers in the financial institution *Société Générale*[140]. While this brought some capital into the industry, overall investment remained relatively low.

In 1825, the prohibition in France of the use of sodium sulphate for glass production was abandoned[141]. Manufacturers began to experiment with saltcake and learned that saltcake reacted with sand when some coal was added to the batch, after which the

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[131] Douglas and Frank 1972, p.23.

[132] Muspratt 1860; Chance 1883; Douglas and Frank 1972; Mille 1982; Linters 1987.

[133] Dungworth 2009.

[134] Chance 1883; Dungworth 2009.

[135] Chambon 1955; Mille 1982.

[136] Chance 1883.

[137] Kurkjian and Prindle 1998.

[138] Lefèbvre 1938; Chambon 1955.

[139] Lefèbvre 1938.

[140] Ibid.; Chambon 1955.

[141] Chance 1883.

French gradually started to use saltcake[142]. This intermediate product from the Leblanc process gave more colour to the glass than did sodium carbonate. M. Pelouze is said to have solved this by removing iron from the original salt by using lime to obtain a refined sodium sulphate[143]. Glass made with sodium sulphate instead of carbonate was less liable to devitrification, thus there could be more lime added, so that the glass was harder and more durable[144]. Another effect of saltcake was the higher melting temperature of the glass; this encouraged improvements to the furnaces[145].

There was an active exchange of knowledge between Belgium and France (e.g. by exchanging craftsmen), but it is unclear when the use of sodium sulphate started in Belgium. It was only in 1832 that the Chance Brothers, a firm of glassmakers in England, introduced cylinder glass together with the use of Leblanc sodium carbonate in England[146].

The processing of the melted glass into solid glass plates underwent improvements in the 1820s and 1830s. In 1822-25, the *lanceman* was invented, which was a support for the blowing pipe handled by a helper[147]. It allowed the glassblower to handle large pieces of glass, so he could produce larger cylinders. Around 1825, plates of glass could be produced up to 60x36cm. Larger plates also encouraged improvements in the annealing process. Until 1824, annealing was carried out in a *reverberatory furnace* that was heated and cooled down depending on the stage of the flattening and annealing process[148]. The flattening of the cylinder was carried out on a *flattening stone*. The glass plate is afterwards lifted and moved to the *annealing stone* for the start of the annealing process. In 1825-26, the *rotating stones furnace* made moving glass from the flattening to the annealing stone unnecessary, by introducing a system with movable stones[149]. In 1830, the dimensions that could be produced again increased, to 72x60cm or 84x54cm[150].

Cylinder glass had several surface damages due to flattening and moving the plate. Grinding and polishing the plate, as was done with cast glass, could remove these defects and thus improve the transparency but the thinner plate could easily break. In 1839, James Chance of the Chance Brothers invented the *patent plate process*. This

[142] Muspratt 1860; Dungworth 2009.
[143] Chance 1883.
[144] Ibid.
[145] Douglas and Frank 1972.
[146] Muspratt 1860; McGrath 1961; Dungworth 2011.
[147] Lefèbvre 1938; Douxchamps 1951; Engen 1989.
[148] Lefèbvre 1938.
[149] Ibid.; Douxchamps 1951.
[150] Lefèbvre 1938.

involved laying a cylinder glass plate on a soaked piece of leather and then grinding and polishing it to remove surface damage (which originated from flattening)[151].

1840-1852: Improving the efficiency

All adjustments made by the Belgian manufacturers in the period 1840-1852 were aimed at increasing production volume[152]. The repeal of excise duties on the glass production in England was accompanied by a gradual decrease of their import duties. Rising imports of cheap Belgian glass in Great-Britain from 1853 onwards, motivated English glassmakers to come up with technical improvements in order to compete with the Belgian glassmakers[153].

In 1845, a new device, *le crochet d'ouvreau*, replaced the *lanceman*. It was a hook attached to the furnace for supporting the blowing pipe during reheating the cylinder[154]. These constant adjustments to aid the glassblower were helpful, but the reorganisation of the work probably had more influence than these tools on improving the efficiency[155]. From 1845, the largest possible glass plate measured 120x72cm[156].

Another improvement in annealing glass was made to handle the increased glass production, and to reduce fuel and labour costs. In 1844-46, the Chance Brothers developed an annealing tunnel of 12 to 15 meters long where the glass plates were manually moved in stages further from the heat source. This was exported to Belgium, but it is unclear when exactly[157].

1852-1872: Larger crucibles and Solvay soda

In 1863, the Belgian chemist Ernest Solvay invented a new production process for making soda ash (sodium carbonate Na_2CO_3) out of common salt (sodium chloride NaCl) and limestone (calcium carbonate CaCO3), by using ammonia (NH_3)[158]. This ammonia-soda process was more efficient and therefore cheaper and less polluting than the Leblanc process. In 1863-65, the first Solvay factory in Couillet near Charleroi was founded[159]. A struggle between Solvay and Leblanc manufacturers

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151 McGrath 1961; Schittich et al. 2007.
152 Lefèbvre 1938; Douxchamps 1951.
153 Barker 1977.
154 Lefèbvre 1938; Douxchamps 1951; Engen 1989.
155 Douxchamps 1951.
156 Lefèbvre 1938.
157 Ibid.; Chambon 1955.
158 Douglas and Frank 1972; Linters 1987.
159 Linters 1987; Daviet 1988.

followed, and by 1890, half of the production of sodium carbonate was carried out by Solvay factories[160].

After 1845, the production of window glass constantly increased (Figure 1-21). To fulfil the growing demand, the glass manufacturers enlarged their crucibles so they contained more molten glass. As a consequence, the fuel consumption per unit of glass increased[161]. The glass manufacturers looked for ways to improve the furnaces to reduce the fuel cost[162]. With the experiments of Joule in the 1840s, insights into heat phenomena grew, and these were applied to the development of furnaces[163].
In 1856, a naturalised Englishman of German origin, Frederick Siemens, patented a regenerative furnace in Great-Britain and Germany[164]. His brother Charles William Siemens explained the regenerative principle in a paper in 1857 and mentioned experiments using it to produce iron (but not glass)[165]. The furnace was heated by gas flames produced from burning coal, from which the residual heat was used to preheat the fuel that would be used afterwards. In 1861, the brothers C.W. and F. Siemens jointly took a British patent on "improved furnaces"; this patent involved making combustible gas outside the furnace[166]. A report of tests on the application of the regenerative furnace in glass production to England was written by C.W. Siemens in 1860-61[167]. The advantages for glassmakers were a clean production, more constant and higher temperature in the furnace, and fuel savings (30% by gasification, increased to 45-50% by the regenerative principle)[168].

With the crucible method of glass production, the molten glass stayed in one place, namely the crucible. The crucible was heated and cooled down in several cycles depending on the stage of the glass manufacturing, consuming a lot of fuel. Several experiments were carried out to invent a continuous process, where the temperature was constant on defined locations and the glass was transported between those locations. The Siemens brothers were finally the ones who succeeded in realizing this. On the 1st October 1867, Frederick Siemens applied this continuous principle in the tank furnace[169]. Charles William Siemens gave an extensive description of this tank

[160] Daviet 1988.
[161] Lefèbvre 1938; Douxchamps 1951.
[162] Lefèbvre 1938.
[163] Douglas and Frank 1972.
[164] Chambon 1955; Douglas and Frank 1972; Mille 1982; Schittich et al. 2007.
[165] Douglas and Frank 1972.
[166] Ibid.
[167] Ibid.
[168] Chance 1883; Douglas and Frank 1972; Mille 1982.
[169] Douglas and Frank 1972.

furnace in 1870[170]. In the US patent no. 127806, granted to the Siemens brothers[171], the principle is explained (Figure 1-23).

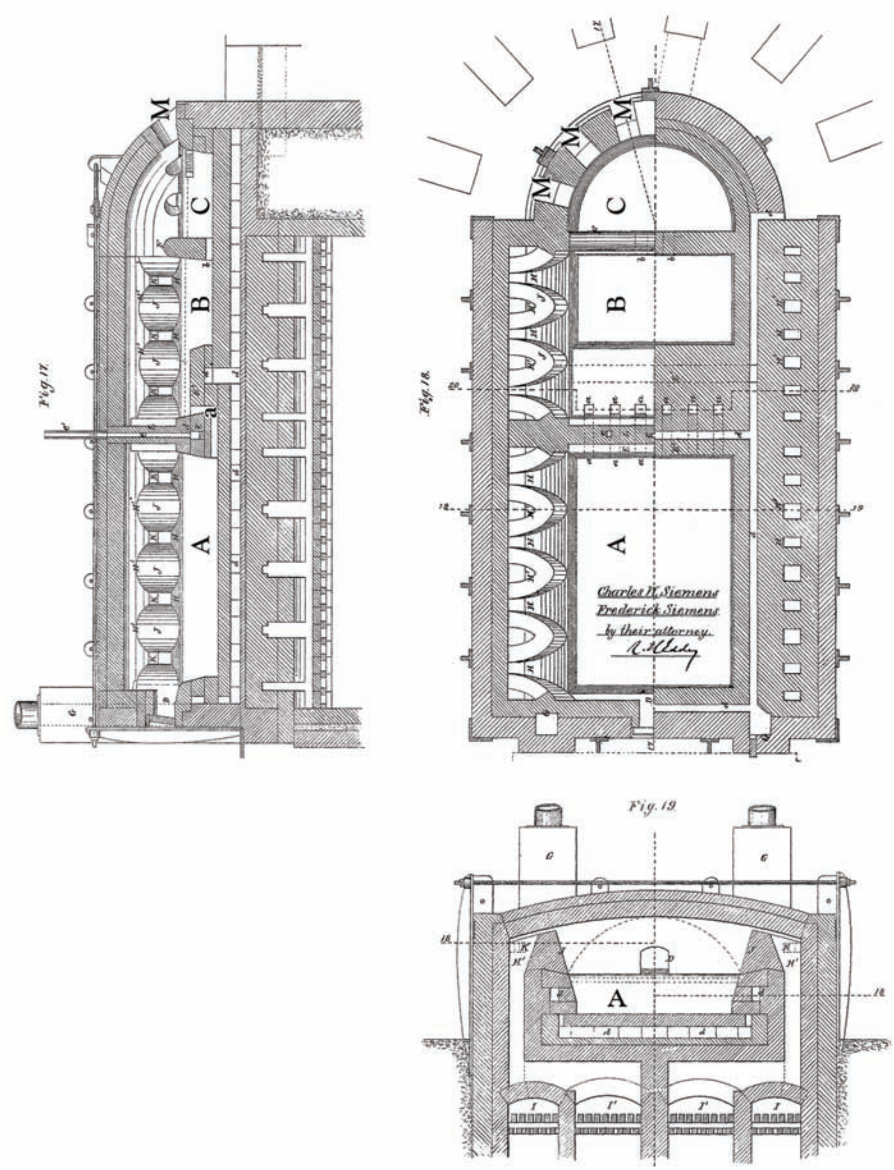

Figure 1-23: Drawing of tank furnace[172]

[170] Chance 1883.
[171] Siemens and Siemens 1872.
[172] Ibid.

"The tank of the furnace is divided, similarly to the before-described glass-pots, into three separate compartments, A B C, of which A serves to receive the raw materials, while B is the clarifying-compartment, and C the working-compartment. (...) By arranging the gas and air ports along the sides of the tank we are enabled to regulate the temperature in the different parts of the furnace according to the various stages of preparation of the glass in the several compartments, this regulation being effected by constructing the gas and air ports of larger dimensions at the compartment A, where greater heat is required than in the compartment B; (...)

As before stated, the raw materials introduced into the compartment A (...) gradually sink down to the bottom of the tank and pass thence up through the passages a (...) into the compartment B. (The glass passes) *(...) into the working-compartment C, where it is worked out through the openings M formed in the semicircular* (sic) *front end of the furnace. The compartment C is only heated by a portion of the flames in the compartment B (...), whereby this compartment is maintained at the requisite cooler temperature for working out the glass.*

(...) To produce this circulation of the melted glass through the three compartments advantage is taken of the gradually-increasing specific gravity of the glass as the melting down thereof proceeds."[173]

The pursuit of larger glass panes led to a final tool used for cylinder glass production. In 1867, the *manique* or *iron man* was introduced, which was a mobile support to help swinging and turning the blowing pipe[174]. One source says it was invented in Belgium, but that its use there stopped by 1883[175]. The largest plate that could be produced in 1870 was 144x96cm, which did not enlarge any more until the introduction of mechanised blowing at the beginning of the 20th century[176].

The annealing furnace also got an important improvement. In 1857, the *stracou Frison-Biévez* was invented by Frison[177]. The process was improved by Désiré Biévez of the Mariemont glassworks in the Charleroi region[178]. The annealing tunnel with mobile rods, where glass cooled down in 20 to 25 minutes, was finally applied in 1862-70[179].

Since 1845, the production of window glass constantly increased (Figure 1-21). More skilled craftsmen were needed to fulfil the demand, which lead to increasing salaries.

173 Ibid., p.4.
174 Lefèbvre 1938; Engen 1989.
175 Chance 1883.
176 Lefèbvre 1938.
177 Chambon 1955.
178 Ibid.; Engen 1989.
179 Lefèbvre 1938; Chambon 1955.

The glasswork employers tried to limit wage growth, which provoked strikes in reaction[180]. In 1872, the Belgian glass manufacturers founded *Le Comité des Verriers* as an association for the master glassworkers, to try to reduce labour costs, but not much agreement was found among the companies[181]. In 1870-72, export to Great-Britain was lost to France due to the high labour and fuel costs, while export to the US was lost due to import duties[182]. The sharp decrease in exports (Figure 1-21 1870-72) encouraged the glass manufacturers to introduce technical improvements[183].

1872-1895: Application of the tank furnace

The need for technical improvements to reduce fuel consumption and labour costs led to old crucible furnaces being replaced by the new regenerative tank furnaces[184]. It was a large investment for master glassworkers, which resulted in the disappearance of a lot of glassworks. Although the coal of the region of Charleroi was not suitable for gasification due to its low volatile content, the window glass production was concentrating around Charleroi for its specialized craftsmen[185].

In the 1870s, glassworks began to adapt tank furnace for glass production and finally the new furnaces were put into operation between 1878 and 1881[186]. The roles of the Siemens brothers and the engineer Martine A. Oppermann in this adaptation process are not clear. Chambon and Lefebvre[187] mention experiments carried out by Oppermann, while Engen[188] says that he installed the first regenerative tank furnace in a glassworks in Faubourg near Charleroi. Different sources disagree on the nationality of the engineer Oppermann: he has been described as German[189] or Belgian[190]. Douglas[191] attributes the development of the regenerative tank furnace completely to the Siemens brothers. By 1894, there were no crucible furnaces left in Belgium for window glass production[192].

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180 Lefèbvre 1938; Engen 1989.
181 Lefèbvre 1938.
182 Ibid.; Douxchamps 1951; Chambon 1955.
183 Douxchamps 1951.
184 Lefèbvre 1938; Douxchamps 1951.
185 Lefèbvre 1938; Chambon 1955.
186 Lefèbvre 1938; Chambon 1955; Engen 1989.
187 Lefèbvre 1938; Chambon 1955.
188 Engen 1989.
189 Ibid.
190 Oppermann 1894; Oppermann 1910; Pesch 1949.
191 Douglas and Frank 1972.
192 Lefèbvre 1938; Douxchamps 1951; Mille 1982.

The regenerative principle spread together with the tank furnace[193]. The gas furnace was also applied to the annealing tunnel beginning in 1872[194]. In 1883, in Great Britain, blown cylinders began to be cut with a diamond, while an iron rod was still used in Europe[195].

In the 1880s, social conflicts were regular in the Belgian glassworks. Periods of high export volume alternated with overproduction, resulting in periods of expansion or high unemployment. The time shifts between production and export (alternating peaks and valleys per year in Figure 1-21 1880-1895) explain why glass manufacturers built up stocks[196]. In 1882, *L'Union Verrière* was founded which united all "warm" glassworkers (blowers, gatherers, flatteners and gasmen[197]. One year later in 1883 *Le Comité des Verriers* tried to reduce salaries, which provokes a major strike in 1884[198]. Strikes continued to return during the 1880s and 1890s. In 1894, the *Nouvelle Union Verrière* was founded by Edmond Gilles, which initially united all workmen but later only the "warm" craftsmen[199]. In 1895, the "cold" craftsmen (cutters, grinders, packers, cashiers) were united in *Syndicat des Magasiniers verriers* founded by Henri Bastin[200]. Because of the large difference in salaries between these two groups of workers, there was a lot of concurrence and discussion during the successive social conflicts.

1895-1914: Improving the efficiency
After 1896, no relevant technical improvements were made to the production processes[201]. There was constant overproduction, so master glassworkers continued to focus on labour cost containment, with strikes in 1900-01 and 1904 as a result[202]. After the *Nouvelle Union Verrière* demanded a partner to negotiate with, the *Mutualité des Maîtres de Verreries* was founded in 1909-12[203]. Agreements were signed about the regulation of the work, which improved efficiency. The production of all glassworks was regulated, avoiding overproduction and so massive unemployment[204]. At the start

[193] Douxchamps 1951.
[194] Lefèbvre 1938.
[195] Chance 1883.
[196] Lefèbvre 1938.
[197] Ibid.; Engen 1989.
[198] Lefèbvre 1938; Engen 1989.
[199] Lefèbvre 1938; Engen 1989.
[200] Lefèbvre 1938.
[201] Douxchamps 1951.
[202] Lefèbvre 1938; Douxchamps 1951; Chambon 1955; Engen 1989.
[203] Lefèbvre 1938; Engen 1989.
[204] Engen 1989.

of the First World War in 1914, the *Nouvelle Union Verrière* closed down all window glass production[205].

At the beginning of the 20th century, hand-blown glass production processes were gradually abandoned (Figure 1-24). A first adjustment was the replacement of blown glass with the Lubbers process. This had been applied in the US between 1900 and 1905[206], but was not used at the time in Europe[207].

In 1901, Emile Fourcault took a Belgian patent for the idea of glass drawn vertically out of the molten material[208]. The idea of drawing glass was already patented in 1857 by William Clark of Pittsburgh, but he was not successful with preventing the glass plate from losing width[209]. At the same time as Fourcault, Irving Colburn was developing a method in the US for drawing glass vertically but turning it horizontally. This Libbey-Owens process was patented in 1903-05[210]. The First World War slowed down the development of the Fourcault process, although the first factory using the process was founded in Dampremy (Belgium) in 1912-14[211]. 1916-17 was the start of the production with the Libbey-Owens process[212]. A third drawing process, the Pittsburgh drawing process, combined the advantages of both processes and started production in Belgian glassworks in 1921[213].

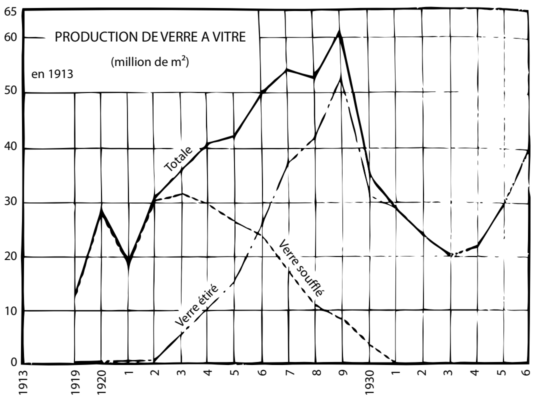

Figure 1-24: Gradual switch from blown to drawn glass[214]

[205] Lefèbvre 1938.
[206] Engen 1989; Schittich et al. 2007.
[207] Mille 1982.
[208] Chambon 1955; Mille 1982.
[209] Chambon 1955; Douglas and Frank 1972.
[210] McGrath 1961; Mille 1982; Schittich et al. 2007.
[211] Chambon 1955; McGrath 1961; Mille 1982; Engen 1989.
[212] Mille 1982; Engen 1989.
[213] Mille 1982; Engen 1989.
[214] Lefèbvre 1938, p.114.

3. Conclusions

In the 19th century, iron production underwent large changes. The processes for producing cast and wrought iron improved and the production of steel was developed. The furnaces for all three metals were improved by applying scientific discoveries, made in the fields of heat and chemistry. Iron was at first mainly used for infrastructure (bridges and railways) and for fireproof buildings. With these applications, the material became well-known and its advantages were demonstrated. In the second half of the 19th century, iron and steel spread to various structural applications in the building industry.

Glass manufacturers in the 19th century only introduced major improvements from other industries. Furnace technology was adopted from the steel industry, driven by the increasing fuel prices. The implementation of the regenerative oven and tank furnace improved the energy-efficiency. The chemical industry offered insight into glass composition and the quality of the raw materials. The invention of Leblanc and later Solvay soda improved the quality and aesthetics of glass plates. Although the production process was industrialized, glassblowing was still the work of an individual. By improving his work tools, the glassmakers were able to create larger glass planes. But the industry had to wait until the invention of the drawn glass in the beginning of the 20th century to fully industrialise the window glass manufacturing process.

The major change in applying glass to buildings in the 19th century was combining it with iron. The slender sections, high strength and form freedom of iron and steel frames filled with glass dramatically expanded the architectural possibilities for enclosing space and increasing light.

The Industrial Revolution created new building typologies like railway stations, shopping arcades and palm houses. A new architectural language was developed, applying the new materials iron and glass and the new insights into statics. The iron and glass architecture became a symbol of the 19th-century architecture.

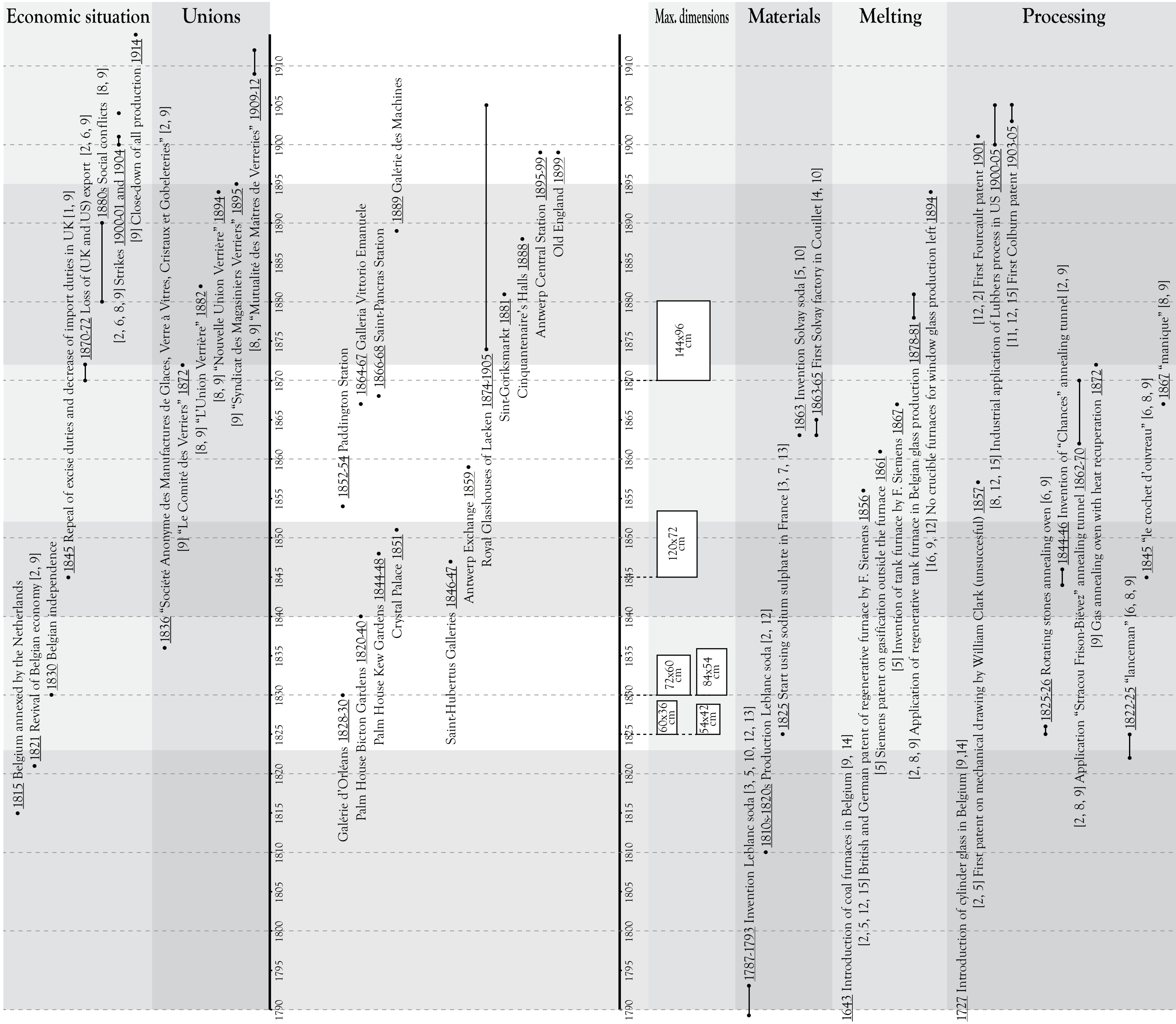

Figure 1-25: Timeline of developments of glass production in 19th century

Sources: [1] Barker 1977; [2] Chambon 1955; [3] Chance 1883; [4] Daviet 1988; [5] Douglas and Frank 1972; [6] Douxchamps 1951; [7] Dungworth 2009; [8] Engen 1989; [9] Lefebvre 1938; [10] Linters 1987; [11] McGrath 1961; [12] Mille 1982; [13] Muspratt 1860; [14] Pesch 1949; [15] Schittich 2007

Chapter 2

Construction techniques of 19th century glass roofs

The heritage value of a 19th-century iron and glass roof includes also the heritage value of its components. The previous chapter dealt with the heritage value of the glass plates. The iron glazing bars and the construction techniques that were used to connect glass and iron, will be the subject of this chapter. By means of an extensive literature study, the evolution of connection details of glass plates and iron glazing bars throughout the 19th and 20th century is outlined.

1. Origin of the iron glazing bar

The first glasshouse was built around 1700. In the beginning of glasshouse building, only part of the building envelope was covered with glass. The glass plates were laid on wooden glazing bars that transferred the loads to a primary load-bearing structure. The English gardener John Claudius Loudon invented the wrought iron glazing bar for the use in glasshouse building (paragraph 1.2 p.19). His findings were first published in 1817[1]. The wrought iron glazing bar could be more slender than the previously used wooden ones (comparison in Figure 2-1). Loudon's solid iron glazing bar could be bent without losing its strength, an advantage to the compound iron sections, cast iron sections or wooden glazing bars. They were applied in curved glasshouses, which improved the penetration of the sun's rays into the interior. In search for more light penetration, Loudon also developed the ridge-and-furrow roof (Figure 2-2).

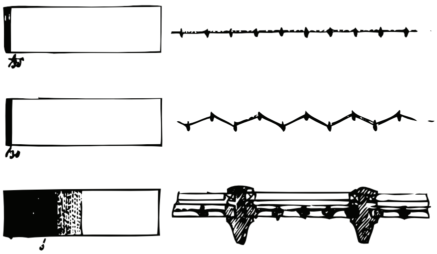

Figure 2-1: Study of different glazing bar systems and the light they admit: (left) ground surface of glasshouse with shaded area, (right) compared glazing systems (iron curvilinear, iron ridge-and-furrow, and wooden glazing bars)[2]

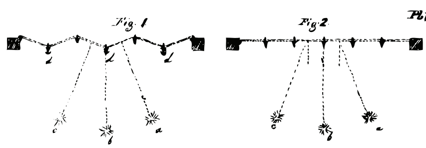

Figure 2-2: Study of roof geometry (ridge-and-furrow and flat roof) to improve the admission of light[3]

[1] Loudon 1817; Loudon 1818.
[2] Loudon 1817, Pl. VIII.
[3] Ibid., Pl. VI.

2. Sources: in search for course books and manuals

The construction techniques of the 19th century can be studied by analysing the knowledge transfer between professors and students at universities and technical schools. Van de Vijver lists the structural engineering and construction courses which were given in Belgium from 1780 until 1930[4]. His article served as a base for tracking down the most important course books from the 19th century.

Another important source consists of the manuals written by architects, engineers, chemists etc. The course books cross-reference to these manuals.

The analysed books can be divided in three categories: course books, directly based on given courses[5]; manuals[6]; and manuals written by professors and teachers[7]. Most of the consulted books belong to the last category and their content lies in between educational and professional books.

Various parameters concerning glass and connection details were recorded (Table 2-1) from literature published between 1847 and 1919. Course books from before 1847 were consulted, but did not contain any information about building with glass.

The course books and manuals sometimes give an overview of iron and glass architecture: Loudon writes separate books about glasshouse constructions[8], while the manuals focus on a brief overview of possible typologies from glass porches to small culture houses to winter gardens (Figure 2-3). These descriptive overviews do not mention technical information about connection details, construction technologies or materials.

The decorative aspects of the glass plates will not be discussed in this chapter. The manuals and course books often contain information about colouring glass plates. The use of texturized cast glass plates as floor tiles is sometimes described, mainly focusing on the decorative patterns that can be chosen from (Figure 2-4).

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[4] Van de Vijver 2003.

[5] Demanet 1847; Demanet 1850; Demanet 1861; Demanet 1862; De Vos 1879; Dechamps 1908.

[6] Moerman 1874; Barberot 1888.

[7] Oslet 1888; Combaz 1895, Vol.1-2, ; Cloquet 1898; Vierendeel 1902; Combaz 1905, Vol.3-6, ; Combaz 1905, Vol.3-7, ; Francken 1910; Nachtergal 1912; Nachtergal 1919.

[8] Loudon 1818; Loudon 1839.

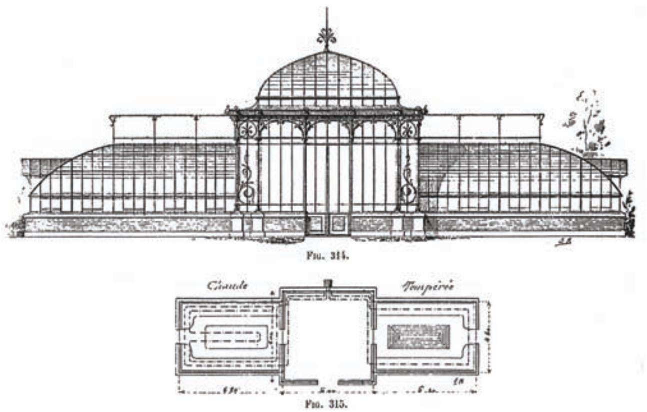

Figure 2-3: Description of iron and glass construction:
example of a wintergarden construction (façade and plan)[9]

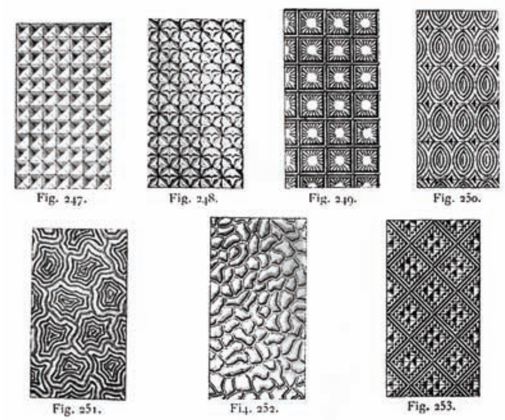

Figure 2-4: Cast glass plates with decorative textures
that are described in some manuals and course books[10]

3. The knowledge of the construction technology in Belgium

The information given by the three classes of books differed. The course books gave general information and were not specific. The manuals written by teachers gave the widest range of information, including very specific drawings, descriptions, etc. The other manuals were somewhere in between the previous two classes. The difference between the course books and the two classes of manuals was most explicit in comparing the definitions of the thickness of glass plates and the descriptions of the sections used as glazing bars (Table 2-1).

[9] Barberot 1888, p.212.
[10] Combaz 1895, Vol.1-2, p.411.

3.1. Glass plates

The composition of glass, the defects, the colour and the production processes were dealt with in these course books and manuals. The description was mostly only a summary of the information described in the literature cited in Chapter 1.
The maximum available dimensions were listed in both literature groups, but to a lesser extent in the course books and manuals. The maximum dimensions commercially available for crown and cylinder glass, described in the manuals and course books, were similar to the dimensions from Chapter 1. They increased during the investigated period (Table 2-1), but were in the beginning of the 20th century still limited by the force of the glassblower.

The third dimension of a glass plate, the thickness, was mentioned in almost every course book and manual. In the 19th century, glass was sold by its weight per square meter, which could roughly be translated to the thickness. The glass was divided into three classes: *verre simple*, *verre demi-double* and *verre double*. According to Vierendeel[11], these terms were the same throughout Europe, but could vary slightly according to the region. Definitions of these terms also varied during the investigated period. The *verre simple* decreased from 2.25 mm thickness in 1847[12] to 1 - 1.25 mm in 1919[12]. The *verre demi-double* decreased from 3 - 4 mm in 1888[13] to 2 – 2.5 mm in 1919[14]. However, the thickness of the *verre double* stayed constant over time, namely 3 - 4 mm (Table 2-1). The technical possibility of producing thinner sheets was visible in the change in meanings of *verre simple* and *verre demi-double*. However, *verre double* was recommended for use in glass roofs. A sufficient thickness ensured the resistance of these plates against snow loads, hail impacts, etc.

Wired glass, today used as safety glass, was already known in the investigated period for its better post-breakage behaviour. It was manufactured by adding a wire net into the mould in which molten glass was cast. It is mentioned in the investigated course books starting from 1905, but described as a non-aesthetic glass plates of which the use was limited to industrial buildings[15]. It was commercially produced by Pilkington from 1898[16].

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[11] Vierendeel 1902.
[12] Demanet 1847.
[13] Barberot 1888.
[14] Nachtergal 1919.
[15] Combaz 1905, Vol.3-7, p.175.
[16] Pender, Godfraind, and English Heritage 2011, p.468.

Table 2-1: Glass plates and glazing bars mentioned in Belgian literature 1847-1919: course books (dark grey); manuals (white); manuals written by professors and teachers (light grey).

author year	position	glass plates				iron glazing bar		
		thickness [mm]			max available dimensions [cm]	T-section	T-section with gutters	special section systems
		"verre simple"	"verre demi-double"	"verre double"				
Demanet 1847	Ecole militaire de Bruxelles	2.25 (1)		x (2)	117 x 71			
Demanet 1850	Ecole militaire de Bruxelles			x (1)				
Demanet 1861	Ecole militaire de Bruxelles	2.25 (1)		x (2)	117 x 71			
Demanet 1862	Ecole militaire de Bruxelles							
Moerman 1874	architecte, Bruxelles	+2.25		3 - 4				
De Vos 1879	Ecole d'application de Bruxelles							
Barberot 1888	architecte, Liège & Paris		3 - 4			x		
Oslet 1888	Ecole Centrale Paris	x	x	3 - 4		x	x	
Combaz 1895	Académie royale des beaux-arts de Bruxelles	2	2,5	3 - 4				
Cloquet 1898	Université de Gand	2.5	3 (2)	4		x	x (3)	x
Vierendeel 1902	Université de Louvain	1.5 - 2	2,4 - 3	3,2 - 4	126 & 66	x	x	x (4)
Combaz 1905	Académie royale des beaux-arts de Bruxelles	1.2 - 2.2 (2)	2 - 3	3 - 4 (2)		x	x	x (5)
Dechamps 1908	Université de Liège					x		
Francken 1910	Ecole de Dessin et d'Industrie d'Anderlecht	1.2 - 2.2	2 - 3	3.5 - 4 (2)		x	x	x
Nachtergal 1912	Ecole Industrielle d'Houdeng-Aimeries	1 - 1.25	2 - 2.5	3 - 4		x		
Nachtergal 1919	Ecole Industrielle d'Houdeng-Aimeries	1 - 1.25	2 - 2.5	3 - 4	126 & 63			x (6)

(1) for windows (3) most often used (5) especially Morglia system
(2) usable for roof covering (4) not common in Belgium (6) mention without explanation

3.2. Placing glass on iron glazing bars

The connection details in iron and glass constructions were extensively described in the course books and manuals. In the following paragraphs, the transverse connection (the connection between an iron glazing bar and a glass plate) and the longitudinal connection (the connection between two glass plates) will be considered successively (Figure 2-5).

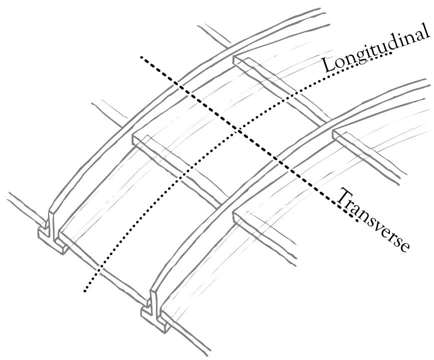

Figure 2-5: Transverse and longitudinal connection in a glass roof

The construction of the transverse connection could be divided in two main categories: connections without and with putty.

For the connections without putty, special section systems were developed (Figure 2-6), called *patent glazing*. A covering section was screwed on an iron base section (scheme in Figure 2-9 left). The contact between the glass and the iron was avoided by using rubber or cotton strips. These strips ensured the water-tightness of the connection. Course books and manuals mention the application of these *patent glazing* systems as being little used in Belgium, however no possible explanation is given. Investigation of the built structures is important to verify this statement (paragraph 4 p.66).

Secondly, the transverse connection can be made watertight using putty. These connections can be subdivided based on the section of the iron glazing bar. The glass plates could be put on a simple T-section or on a more elaborate version with gutters (Figure 2-7 and Figure 2-8). The gutter was added to the cross-section to drain away water that condensed on it, which originated on the interior of the iron and glass skin. To be sure the glass was not lifted from the glazing bar by wind forces, a locking pin could be put through the web of the iron glazing bar and hidden by the putty (schemes in Figure 2-9 middle and right).

The simple T-section and elaborate T-section (called moulded T-section in the subsequent text) both were mentioned in the course books and manuals, starting from 1888 onwards (Table 2-1). The course books and manuals preceding 1888 do not mention any cross-section for the glazing bars. Nevertheless, Loudon already described cruciform rolled sections in his books of 1817 and 1818[17].

[17] Loudon 1817; Loudon 1818.

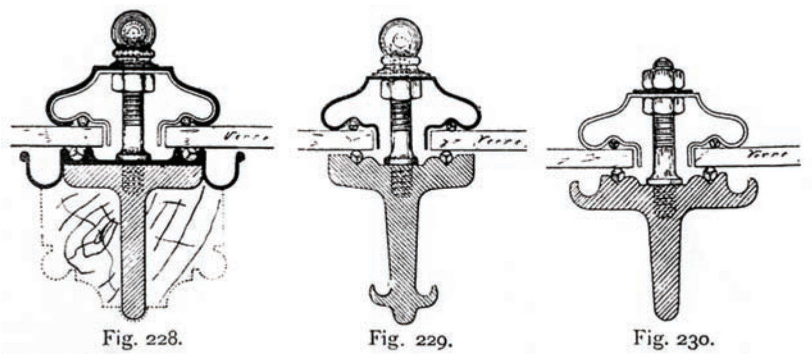

Figure 2-6: Sections of iron glazing bars mentioned in manuals and course books: example of patent glazing systems[18]

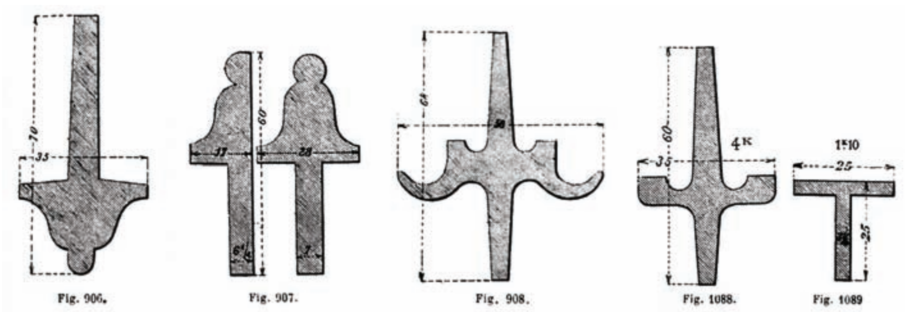

Figure 2-7: Sections of iron glazing bars mentioned in manuals and course books: example of elaborate drawings[19]

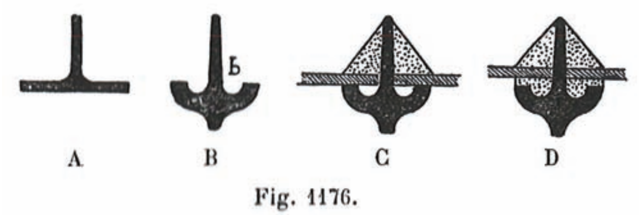

Figure 2-8: Sections of iron glazing bars mentioned in manuals and course books: example of limited detail in drawings[20]

[18] Combaz 1905, Vol.3-7, p.180.
[19] Oslet 1888, p.389, 507.
[20] Cloquet 1898, p.505.

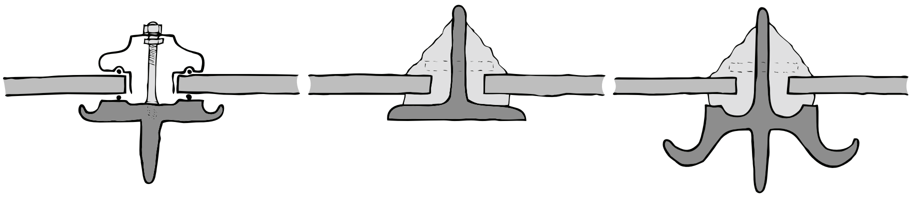

Figure 2-9: Transverse connection in glass roofs: section system with rubber strips (left); simple T-section with putty (middle); and moulded T-section with gutters sealed with putty (right)

The water-tightness of the transverse connection could be ensured by the putty. The composition of the putty was described in some of the manuals and course books (Table 2-2). The functions of the different raw materials in the putty were only rarely and briefly mentioned.

Linseed oil was always mentioned as the basis for the putty throughout the investigated period (Table 2-2) and continued to be used in traditional putty manufacture by glazing craftsmen. The other ingredients vary per recipe. The general principle was to make a paste by mixing linseed oil with a drying agent; in addition, if desired, a pigment was added to it.

The oil was made pasty by adding chalk or ceruse (white lead). Chalk (which was sometimes specified as *Blanc de Meudon* or *Blanc d'Espagne*) and linseed oil mixed together make beige-coloured putty. Ceruse also fulfilled the role of a white pigment (it was also used in the paint industry in the 19th century). Separate pigments could be added: lead monoxide or zinc oxide for a white colour, or the mineral minium as a red pigment.

The course books and manuals mostly recommended placing the glass into a bed of putty. First, a large amount of putty was laid on the glazing bar. The glass plate was pressed into this bed and finally the whole was covered with a second layer of putty. Avoiding contact between the glass and the iron was necessary so that the glass plate would not break.

The necessary depth of the rebate of the iron glazing bars was not described in the course books. Nevertheless, a minimum depth is necessary to prevent the glass from sliding off. Kohlmaier and Von Sartory[21] give a possible guideline: at least twice the thickness of the glass plate plus the thickness of the putty bed.

[21] Kohlmaier and Von Sartory 1991, p.134.

Table 2-2: Putty recipes mentioned in Belgian literature 1847-1919:
course books (dark grey); manuals (white);
manuals written by professors and teachers (light grey).

author year	position	**linseed oil**	**chalk**	**ceruse**	**minium**	**zinc oxide**	**lead monoxide**
Demanet 1847	Ecole militaire de Bruxelles	x	(x)	x	(x)		
Demanet 1850	Ecole militaire de Bruxelles	x		x	(x)		
Demanet 1861	Ecole militaire de Bruxelles	x	(x)	x	(x)		
Demanet 1862	Ecole militaire de Bruxelles	x		x	(x)		
Moerman 1874	architecte, Bruxelles	x	x	(x)			x
De Vos 1879	Ecole d'application de Bruxelles						
Barberot 1888	architecte, Liège & Paris						
Oslet 1888	Ecole Centrale Paris						
Combaz 1895	Académie royale des beaux-arts de Bruxelles	x		x			
Cloquet 1898	Université de Gand	x	x				x
Vierendeel 1902	Université de Louvain	x	x	x			
Combaz 1905	Académie royale des beaux-arts de Bruxelles						
Dechamps 1908	Université de Liège						
Francken 1910	Ecole de Dessin et d'Industrie d'Anderlecht	x	x	x	x		
Nachtergal 1912	Ecole Industrielle d'Houdeng-Aimeries						
Nachtergal 1919	Ecole Industrielle d'Houdeng-Aimeries	x		x		x	

x: mentioned as ingredient (x): mentioned as possible ingredient

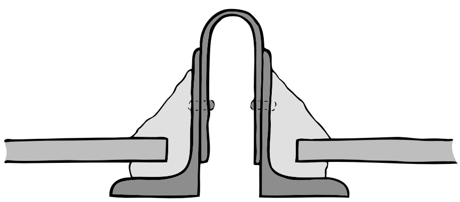

Figure 2-10: Transverse connection: dilatation joint with copper strips

The transverse connections had to accommodate the different thermal movements of iron and glass. This could be achieved by using thermoplastic putty, which becomes soft when heated (e.g. by sun rays) and hardens again after cooling down. Vierendeel[22] cites a composition for this from a German magazine: a mix of resin and tallow with a little bit of lead minium. A second solution was to provide an expansion joint, for example by means of copper strips (Figure 2-10). Vierendeel[23] ascribed the invention of this section to M. Barrault, designer of the *Palais de l'Industrie* in Paris in 1853-54.
The thermoplastic putty would have been used in *King's Cross Station* in London[24]. It is unclear whether any of these dilatation techniques were applied in practice in Belgium.

3.3. Overlapping glass plates

There were several construction methods to realize the longitudinal connection (Figure 2-5).
Three methods do not need extra iron sections in the longitudinal direction. In some books, a system -similar to a slate covering- with a zinc hook was described. The upper glass plate was connected to the lower glass plate via a zinc hook, to keep it in position (Figure 2-11 left). The zinc hooks were placed at the sides of the glass plates and hidden by the putty. The second system prevented the upper glass plate from sliding down by positioning a pin at the end of the glass plate through the web of the glazing bar (Figure 2-11 right). Another possibility was to only count on the transverse connection to hold the glass panes in position, but the literature does not give more details about this alternative.
The gap between two overlapping glass plates could be filled with putty or stay open. These two systems were mentioned occasionally in the course books and manuals, without any pattern or development over time. When the gap between the two glass plates was puttied, there was less ventilation and thus greater chance for condensation. When the gap was open, the overlap between the two glass plates had to be long enough to ensure a water-tight connection. Water-tightness also depended on the inclination of the roof, which ideally had a minimum slope of 18-20°[25]. A zinc strip over the whole width of the glass plate, folded like the slate-like hook (Figure 2-11 left), combined the two conditions. Ventilation and drainage of condensation could take place through a small hole in the middle fold of the strip.

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[22] Vierendeel 1902, p.354.
[23] Ibid.
[24] Walmisley 1888, p.34.
[25] Moerman 1874; Combaz 1895, Vol.1-2, ; Vierendeel 1902; Nachtergal 1912.

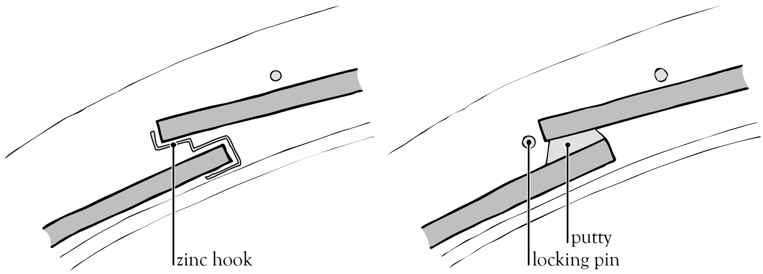

Figure 2-11: Zinc hook (left) or putty and a locking pin (right) for the longitudinal connection

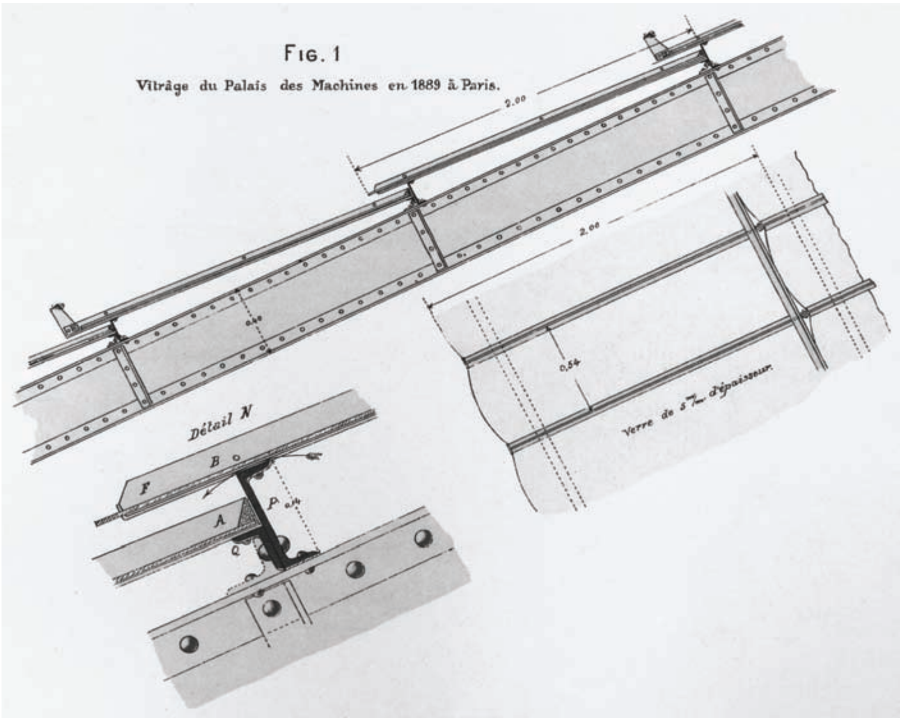

Figure 2-12: Glazing of *Galéries des Machines*, Paris, 1889[26]

Examples in the manuals and course books often show another possibility to define the longitudinal connection of the glass roof by adding iron sections. The roof of the *Galéries des Machines* in Paris from 1889 is a famous example. A combination of U- and L-sections fastened the iron glazing bars, maintaining the gap between the glass plates (Figure 2-12, Detail N). The *Galérie des Machines* was covered with cast glass plates, an expensive production process (mainly used for mirrors), with larger glass plates and glass thicknesses as a result compared to blown glass (Table 2-1).

[26] Vierendeel 1902, Pl. 112.

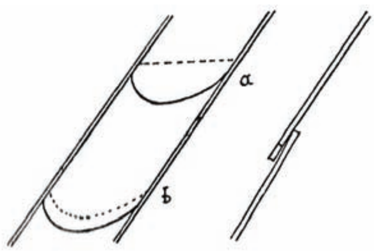

Figure 2-13: Cutting patterns for bottom edge of glass plates to keep rain away from putty connection[27]

3.4. Maintenance of iron and glass roofs

The need to paint the iron glazing bars to avoid oxidation was acknowledged in nearly every course book and manual. Also the deterioration of the traditional linseed oil putty when exposed to the weather was described often. It was given as the main reason for developing a *patent glazing*, to avoid the use of putty and thus the expensive maintenance it requires. The putty should be painted with at least 2[28] or 3 layers[29] after maximum 8 days[30] and repainted regularly to maintain the water-tightness of the connection. According to Pender et al[31], the typical service life of liquid-applied sealants, of which linseed oil putty is an example, is 10 to 15 years.

To keep the rain away from the putty, special cutting patterns for the bottom edge of the glass plate were proposed (Figure 2-13). These cutting patterns were described theoretically, but were very labour intensive to apply. No Belgian examples were found so far.

The cleaning of the glass plates, the dust between two glass plates in the longitudinal connection (Figure 2-5), and the easy replacement of broken glass plates are examples of other maintenance parameters that were mentioned occasionally in the course books and manuals.

[27] Cloquet 1898, p.505.
[28] Vierendeel 1902, p.351.
[29] Cloquet 1898, p.506.
[30] Francken 1910, p.441.
[31] Pender, Godfraind, and English Heritage 2011, p.358.

4. The link to the built structures

The course books and manuals give insight into the knowledge that was communicated to students of architecture and engineering in the 19th century. To check whether the building techniques recommended in the literature were actually used, Belgian iron and glass roofs were analysed. Examples of a compound section, a simple T-section, a moulded T-section and puttyless glazing will be discussed in this part. These examples give an overview of all construction details described in the manuals and their evolution during the 19th century.

Compound iron glazing bar

Simple iron sections (e.g. flat bars and L-sections) could be put together to form a support for glass plates.

In the Saint-Hubertus Galleries such compound section is applied to support the glass. The galleries were built in the centre of Brussels in 1847 by architect J.-P. Cluysenaar (Figure 2-14 and Figure 2-15). A renovation campaign was carried out by A.2R.C. architects from 1993 until 1997 for the 150th anniversary of the gallery[32]. The original glazing bar was composed of a flat iron bar and two angle sections (Figure 2-16). According to the restoration report[33], the connection between the components of the compound section would have been accomplished with small angle sections, however this could not be verified since the L-sections were replaced during the renovation. The glass was placed on the L-sections and sealed with putty. In the restoration report, the glass plates in the transverse connection are drawn in contact with the iron L-section. This drawing could not be verified on-site because the connection is altered nowadays. The connection however was probably not installed as drawn, but with a zone of putty underneath the glass plates.

In the longitudinal direction (Figure 2-17), no additional measures were taken to hold the glass plate in position. The overlap between two glass plates stayed open.

[32] website webpage A.2R.C 2010.

[33] A.2R.C, Forum, and TCA 1996b.

Figure 2-14: The Saint-Hubertus Galleries (1847, Brussel, Cluysenaar) after 1993-96 renovation campaign (2010-04-22)

Figure 2-15: Exterior view of the Saint-Hubertus Galleries before 1993-96 renovation campaign[34]

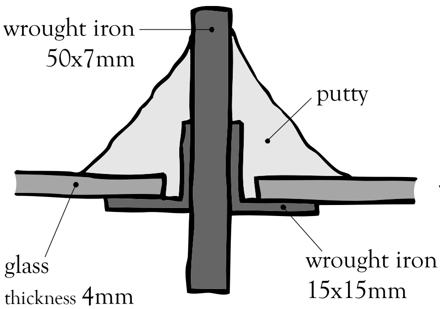

Figure 2-16: The transverse connection at the Saint-Hubertus Galleries: original detail before 1993-96 renovation campaign[35]

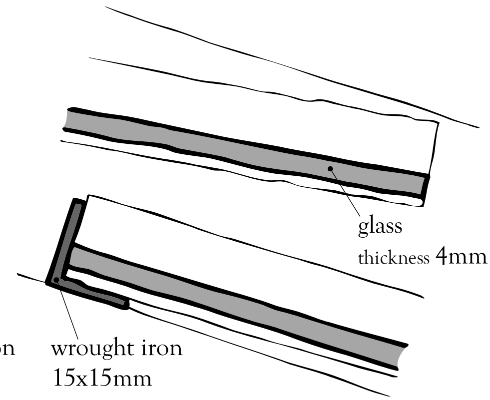

Figure 2-17: The longitudinal connection at the Saint-Hubertus Galleries: original detail before 1993-96 renovation campaign[36]

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[34] Ibid., p.13.
[35] A.2R.C, Forum, and TCA 1996b.
[36] Ibid.

Figure 2-18: State of connections at Saint-Hubertus Galleries before 1993-96 renovation campaign[37]

Figure 2-19: State of connections at the Saint-Hubertus Galleries before 1993-96 renovation campaign[38]

Simple T-section

The Victoria Regia Glasshouse was designed by Alphonse Balat, one of the tutors of the famous Art Nouveau architect Victor Horta (Figure 2-20 and Figure 2-21). It was built in 1854 at the Leopold Park in Brussels, but was moved to the botanical garden in Brussels in 1910 and to the botanical garden in Meise in 1941[39]. Originally, it had very small glass panels to be able to follow the curvature of the structure, but they were replaced by larger ones during restorations (conclusion based on historic photographs, Figure 2-25 - Figure 2-28). The glazing bars seem to be original (but this could not be confirmed due to the lack of original drawings): these are a simple T-section with putty to seal the glass (Figure 2-24). It is unclear how these T-sections were originally connected to the primary load-carrying arches, which are welded at present.

Next to the Victoria Regia Glasshouse, the majority of glass roofs examined in the timeframe of this research used a simple T-section as glazing bar: the shopping mall *Palais du Vin et Merchi-Pède* in Brussels (1892-09, F. Timmermans and F. Symons[40]), the *Palais des Beaux-Arts* (1928, V. Horta), etc.

[37] *TR 772 1/2 2043-0116*: *Dossier renovation-verrières* 1988.
[38] Ibid.
[39] Bulinckx 2010.
[40] Verschueren and Marchi 2006, p.34–35.

Figure 2-20: The exterior of
the Victoria Regia Glasshouse
(1854, Meise, Balat) (2010-05-20)

Figure 2-21: The interior of
the Victoria Regia Glasshouse
(2010-05-20)

Figure 2-22: The exterior truss, the column
and the triangular glass plates at the
Victoria Regia Glasshouse
(2010-05-20)

Figure 2-23: The column and its
connections at the interior of
the Victoria Regia Glasshouse
(2010-05-20)

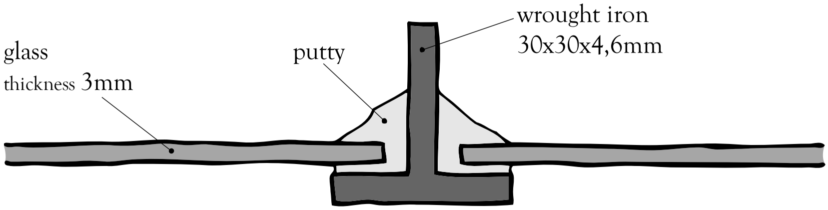

Figure 2-24: The transverse connection at the Victoria Regia Glasshouse: detail as surveyed in present state

Figure 2-25: Victoria Regia Glasshouse in 1910[41]

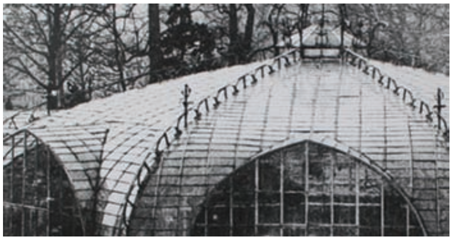

Figure 2-26: Detail of Victoria Regia Glasshouse in 1910: small glass plates[42]

Figure 2-27: Victoria Regia Glasshouse in present state (2008-10-28)

Figure 2-28: Detail of Victoria Regia House in present state: large glass plates

[41] AAM. s.d.

[42] Ibid.. s.d.

The Winter Garden was the first large glasshouse of the complex of the Royal Glasshouses of Laeken (Figure 2-29 and Figure 2-30). It was built in 1874-1876 by architect Alphonse Balat. The structure is built up by trusses that are rotated around a central point and thus forming a cupola of around 57m diameter[43]. T-shaped iron glazing bars with 4 mm thick glass plates fill in the space between the arcs (Figure 2-31). The connection details of this case are not rare, however it was the only case where detailed drawings on the glass cladding were found in the archives. The plans for glazing this masterpiece are preserved in the Archives of the Royal Palace (Figure 2-32 and Figure 2-33).

Figure 2-29: Exterior view of Winter Garden of the Royal Glasshouses of Laeken (1874-76, Laeken, Balat) (2007-03-29)

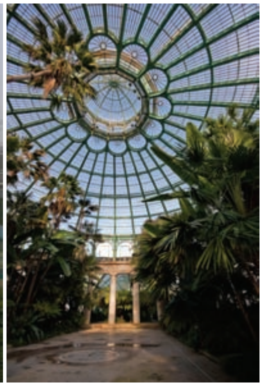

Figure 2-30: Interior view of the Winter Garden (2010-02-12)

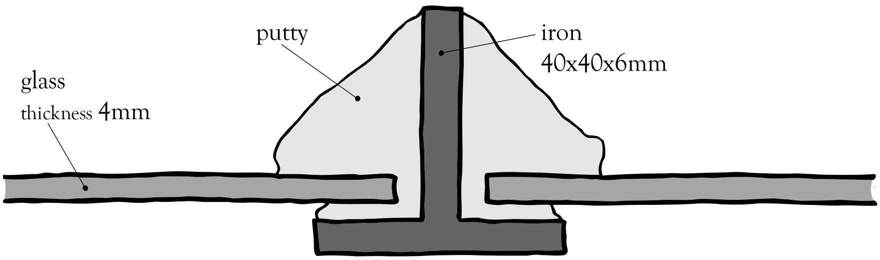

Figure 2-31: The transverse connection at the Winter Garden of the Royal Glasshouses of Laeken: detail as surveyed in present state

[43] Lauriks, De Bouw, and Wouters 2009.

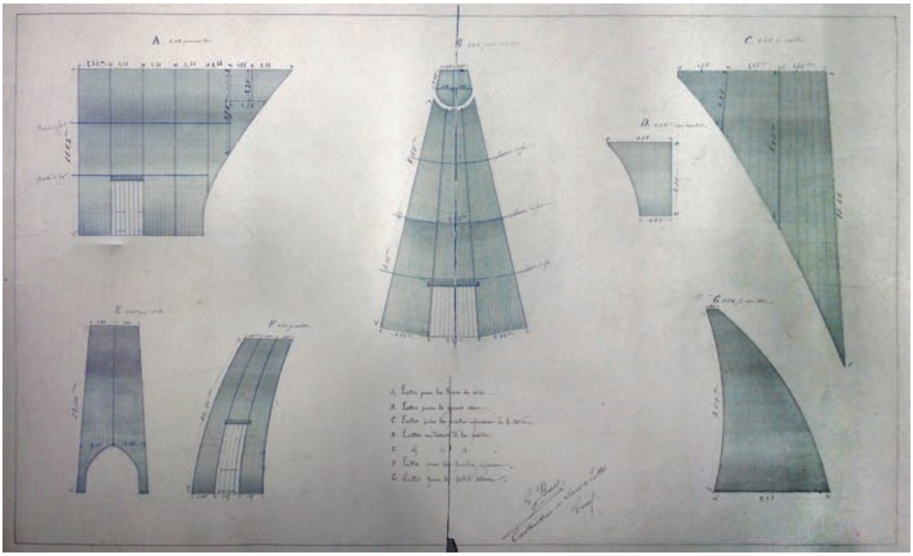

Figure 2-32: The Winter Garden of the Royal Glasshouses of Laeken: glazing plans[44]

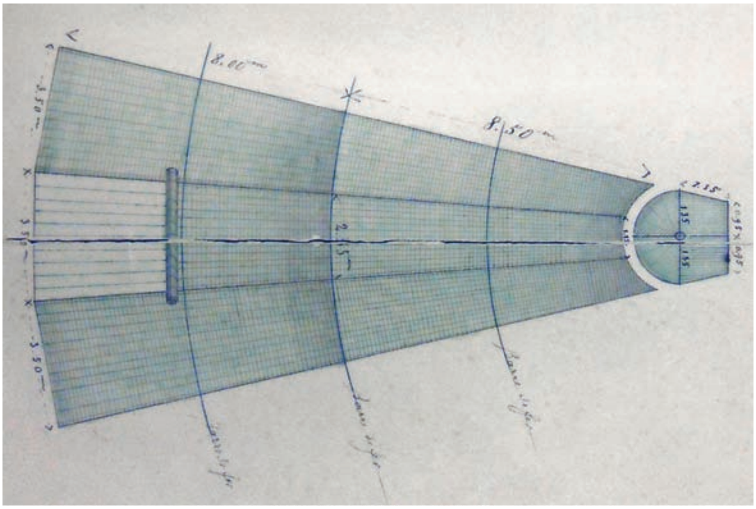

Figure 2-33: The Winter Garden of the Royal Glasshouses of Laeken: detail from glazing plans[45]

[44] Archives of the Royal Palace. *Glazing plans of Winter Garden of Royal Glasshouses of Laeken*.
[45] Archives of the Royal Palace. Ibid.

Moulded T-section

Moulded T-sections also found application in 19th century Belgian glass roofs. The UCB library of the National Bank in Brussels (the former *Union du Crédit*) was designed by architect Désiré De Keyser in 1872 (Figure 2-34 and Figure 2-35). The restoration of the grand hall and smaller roof light was completed in 2010. The glazed roofs are double-layered roofs: the outer layer consists of Polonceau trusses covered with glass, while the inner roof holds decorated glass plates. The iron glazing bars of the inner roof have a moulded section (Figure 2-36). In the rebates of the glazing bars, originally lead hooks were placed at the overlap of two glass plates (system as in Figure 2-11). The historic glass was laminated to improve the safety and the connections were adjusted during the 2003-2010 renovation campaign (Figure 2-37 and Figure 2-38).

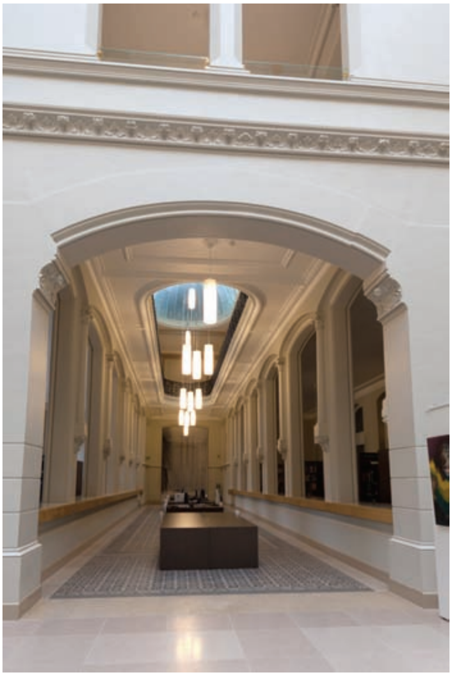

Figure 2-34: Ground floor view of UCB library after 2003-10 renovation campaign (1872, Brussels, De Keyser) (2011-09-17)

Figure 2-35: First floor view of UCB library after 2003-10 renovation campaign (2011-09-17)

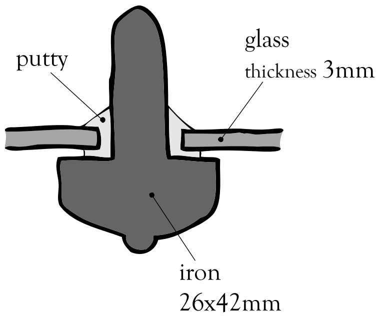

Figure 2-36: The transverse connection at the UCB library: detail as surveyed by the restoration architects before 2003-10 renovation campaign

Figure 2-37: Longitudinal connection at UCB library: adjusted connection after 2003-10 renovation campaign (2010-03-02)

Figure 2-38: Laminated glass panels at UCB library: glass edge of cast-in-place resin laminated panel after 2003-10 renovation campaign (2010-03-02)

Puttyless glazing

Patent glazing was described in literature as not or rarely, found in Belgium[46]. Only a few examples where no putty was used for sealing the connection could be tracked down, despite an intensive search.

Combaz[47] mentioned the municipal school no. 10 "Karel Buls" as one example where the *Morglia patent glazing* was used. The school was built in 1902-06 in the Rollebeekstraat in Brussels (Figure 2-39 and Figure 2-40). The central covered courtyard, typical for the *Ecole Modèle* in the 19th century, was partly covered with wood and partly with glass[48]. The *patent glazing* is clear in the drawings (dated 1904) of Albert Morglia that were found in the municipal archives[49] (Figure 2-43). Due to limited access to the school building, the connection details could only be verified on pictures. It appears the current glazing bar does not correspond to the drawings (Figure 2-41 and Figure 2-42). The glass plates mentioned on the drawings were 6 to 7 mm thick. This was thicker than the standard *verre double*, but could still be produced both by blowing or casting the glass.

Figure 2-39: Central covered courtyard at the Ecole communale no. 10 "Karel Buls" (1902-06, Brussels, A. Samyn) (picture Michael de Bouw, 2007-07-31)

Figure 2-40: Detail of glass part of courtyard at the Ecole communale no. 10 "Karel Buls") (picture Michael de Bouw, 2008-04-16)

[46] Vierendeel 1902, p.362.
[47] Combaz 1905, Vol.3-7, p., 181.
[48] De Bouw 2010.
[49] Municipal archives of Brussels 2008b (Travaux Publics n°4038).

Figure 2-41: Detail of glass part of courtyard at the Ecole communale no. 10 "Karel Buls") (picture Michael de Bouw, 2008-04-16)

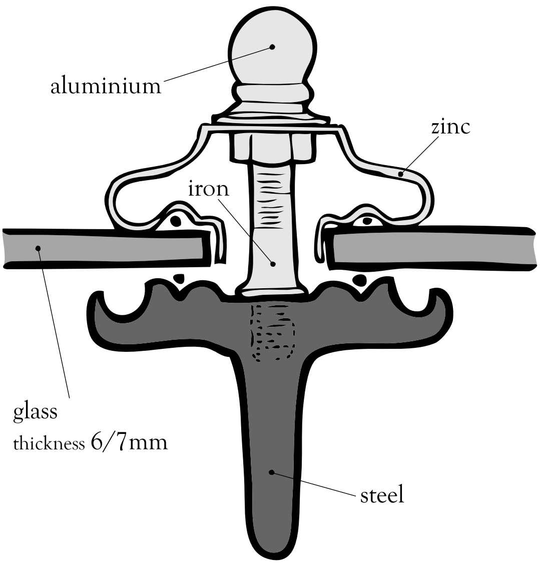

Figure 2-42: The transverse connection at the Ecole communale no. 10 "Karel Buls": detail as found on plan

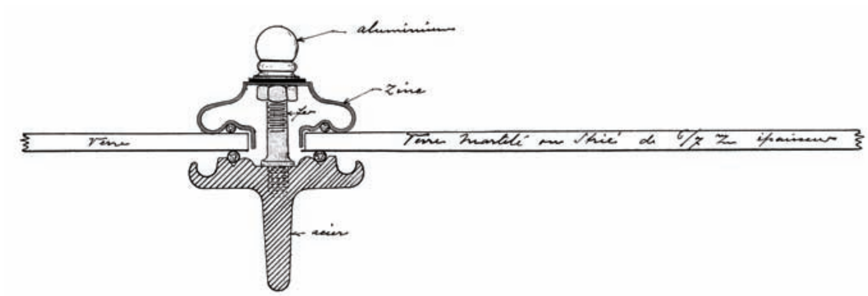

Figure 2-43: The patent glazing systems of the Ecole communale no. 10 "Karel Buls" (1902-06, Brussels, A. Samyn): detail as found on original plans[50]

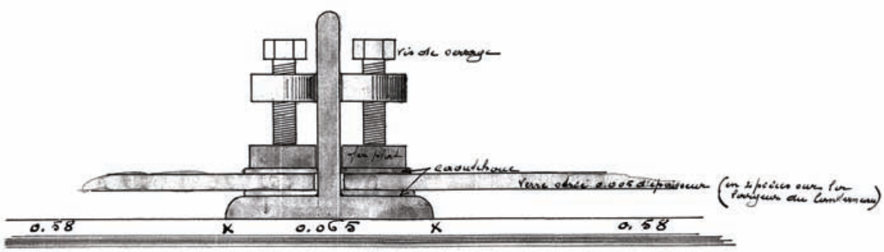

Figure 2-44: The patent glazing systems of the Ecole communale no. 7 "Baron Steens" (1896-97, Brussels, A. Samyn): detail as found on original plans[51]

Adolphe Samyn built another Brussels school in the Hoogstraat in 1896-97, the primary municipal school no. 7 "Baron Steens". The trusses of the central covered courtyard of both schools, "Karel Buls" and "Baron Steens", seem to be identical (but this could not be verified due to limited access to the school building). The original plans show another glazing bar: less elaborate but already without putty (Figure 2-44).

The drawings for both schools mention puttyless glazing, but no evidence can be found that it was also executed. The *Institut des Arts et Métiers*, an arts and crafts school in Brussels, was built in 1933 and designed by the engineer architect Eugène François (Figure 2-45). The renovation study of the glazed roofs (Figure 2-46, Figure 2-47 and Figure 2-48) is finished and renovation works will normally start next year (2013). The lead cover around the iron glazing bar was used for placing the glass (Figure 2-49 and Figure 2-50). This late example is the only proof so far of puttyless glazing used in Belgium.

[50] Ibid. (Travaux Publics n°4038).
[51] Municipal archives of Brussels 2008a (Travaux Publics n°3418).

Figure 2-45: Façade of the Institut des Arts et Métiers
(1933, Brussels, E. Francois) (picture Origin, 2005-10-04)

Figure 2-46: Exterior view of glazed roof
(picture Origin, 2005-05-28)

Figure 2-47: Interior view of glazed roof[52]

[52] Bouafif-Hoebanx 1986.

Figure 2-48: Inside space in double-walled roof[53]

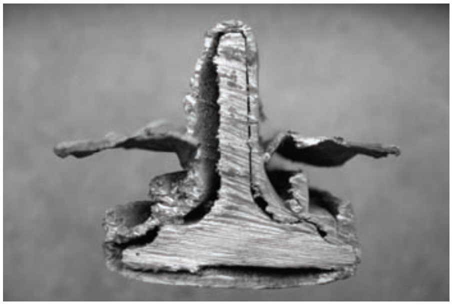

Figure 2-49: Picture of cross-section of glazing bar of outer roof of the double-walled roof (picture Origin, 2006-02-02)

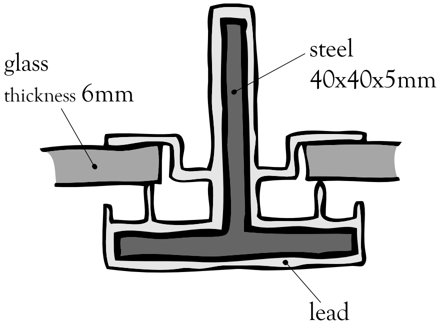

Figure 2-50: The construction techniques at the Institut des Arts et Métiers: detail as found in restoration report[54]

[53] Origin report, 20/03/2007 in *D 2043-0627.0: Institut des Arts et Métiers* 2007.
[54] Origin report, 20/03/2007 in ibid.

5. Conclusions

In the 19th century, glass plates were classified not by their thickness but by their weight per square meters or their equivalent thickness. The different classes were named *verre simple*, *verre demi-double* and *verre double*. The *verre double* was mainly used for glass roofs. These glass plates had a thickness of 3 to 4 mm, and this definition of *verre double* stayed the same during the investigated period (1847-1919). *Verre double* was used for glass coverings and therefore needed to be strong enough to resist snow and hail, which means in practice a sufficient thickness.

Different sections for iron glazing bars were described in the course books and manuals from the 19th century. The systems without putty, where a covering section is screwed on an iron base section, found only limited use in Belgium. The systems that make use of putty to seal the connection between the glass plates and the glazing bars were the usual ones. The iron glazing bars of this system have a T-section or an elaborated T-section with gutters. The gutters drain away the condensed water that originated at the inside of the iron glazing bars. The putty was based on linseed oil and made pasty by adding a drying agent (chalk or ceruse) and if desired a pigment (ceruse, zinc oxide, minium, etc.).

When examining executed iron and glass roofs, it is clear that the more elementary building techniques were used most in practice. Most structures were built with iron glazing bars designed as simple T-sections carrying glass plates with a thickness of 3 to 4 mm.

Chapter 3

Glass envelopes from the 19th century to modern times

Glass is traditionally used as a material to separate two spaces, often between interior and exterior. Glass can protect the interior from moisture, reduces thermal and sound transmissions, but it allows light to pass. The last two decades, this enclosing function is extended with a load-bearing function. When glass panels of a building envelope contribute to the structural integrity of the façade, the supporting structure can be minimized thus optimizing the light transmittance. The same principle was already used in 19th-century iron and glass architecture, however by intuition and without calculations. This chapter will give a brief overview of the possibilities of glass as a structural material and of the standards that are valid for application in the building industry.

1. Structural glass envelopes

1.1. The role of the glass in 19th-century iron and glass roofs

Already in the 19th century, architects were aware of the possibilities of glass as a structural material. Based on their experience and intuition, builders understood the phenomenon of glass plates stiffening a slender iron frame. John Claudius Loudon described one glasshouse in particular, that was built in 1827 at Bretton Hall in Yorkshire for Mrs Beaumont.

> *"It was constructed entirely of cast and wrought iron; all the perpendicular supports being of the former, and all the sash-bar composing the ribs of the roof of the latter, material. (...) It is worthy of remark, that there were no rafters or principal ribs for strengthening the roof besides the common wrought-iron sash-bar, which is only two inches deep, and half an inch thick in the thickest part, and weighs only about one pound per lineal foot. The upper dome had an independent support from cast-iron pillars. When the ironwork was put up, before it was glazed, the slightest wind put the whole of it in motion from the base to the summit; (...) As soon as the glass was put in, however, it was found to become perfectly firm and strong, nor did the slightest accident, from any cause, happen to it, from the time it was completed, in 1827, till, on the death of Mrs Beaumont, in 1832, it was sold by auction, and taken down."* [1]

This principle is also illustrated during renovation of 19th century iron and glass roofs. Dismantling the glass plates of the lower roof at the library of the National Bank of Belgium in Brussels during the renovation campaign in 2004 (Chapter 4 paragraph 5 p.116), the iron frame was 'dancing' due to the decreased lateral stiffness. The architects of the renovation campaign of the Saint-Hubertus Galleries in Brussels (Chapter 4 paragraph 4 p.107) included a temporary lateral support in their process design of dismantling the structure.

[1] Loudon 1839, p.980.

Table 3-1: Experiments on the strength of glass mentioned in Combaz' manual of 1895[2]

	mean plate thickness [mm]	weight of lead ball (falling from 18m height) under which the glass plate fails [g]
verre double	3.5	8
ground *verre double*	3	2
verre triple	6	22
ground *verre triple*	6	6

1.2. Building with glass: mechanical properties and design methods

During the study of the course books and manuals investigated in Chapter 2, attention was given for text parts about the mechanical characteristics of glass. However, only Combaz mentioned experiments on the strength of glass carried out in the 19th century in his book from 1895[3] (Table 3-1). Although Combaz provides no discussion of the results, a conclusion regarding the value of the bending strength is given in the few lines that were added on the experiments.

> *"Expériences faites sur le verre de Saint-Gobain. Résultats obtenus par M. Thomasset (1876-1877) et communiqués à la Société des Ingénieurs civils: des essais sur des verres bruts et des verres polis, donnent un coefficient moyen de rupture par centimètre carré de 260 kilog. pour le verre brut et de 340 kilog. pour le verre poli. Au minimum, on peut donc prendre 250 kilog. à la rupture par centimètre carré à la flexion pour un verre convenablement recuit."* [4]

However, the modern calculation principle of traditional building materials based on partial safety factors cannot be applied for the design of glass components. The design strength of glass cannot be simplified by a single accurate value. Research about the strength of glass has given a broader insight in the specific mechanical properties of the material[5].

The theoretical strength of glass is very high due to its strong atomic bonding forces. However, cracks in the body of the material and on the surface decrease the real strength of a glass plate. Stress concentrations at the tips of these flaws cannot be

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[2] Combaz 1895, Vol.1-2, p.413.
[3] Ibid.
[4] Ibid., Vol.1-2, p.414.
[5] Haldimann, Luible, and Overend 2008, p.49–84.

spread over the material due to its inability to exhibit plastic behaviour. The glass strength is therefore determined by the quality of the material and its surfaces: the production process of the glass plates (both for the surface and the edge quality), the manipulation of the plate (drilling holes, handling of the glass plate), the duration of the mechanical actions (longer duration leads to lower allowable stresses), the increasing surface damage of a plate during its lifetime (e.g. due to erosion), etc. The probability for critical flaws increases with the plate size, therefore the strength of glass determined in experiments also depends on the size of the test sample. This is called the scale effect of glass strength.

Prestressing can increase the strength of glass panes. By introducing a compression stress, the implied loads will first counteract the prestressed force before any tensile stress can occur. Thermal and chemical treatment of a glass plate apply a compressive prestress only at the plate surfaces. Thermal treatment can also have an influence on the fracture pattern of glass.

For traditional building materials, each structural component is dimensioned to prevent its individual failure. Due to the large spread on the glass strength and the brittle failure of glass, another design principle is more obvious[6]. A structure where other members can take over the loads for a minimum evacuation time when one member fails is ideal for the design of structural glass components. This is called robustness, both on component and structure level. Different failure scenarios should be incorporated to include enough redundancy in the structure.

In addition, the design of the connections is important to avoid stress concentrations in the glass. Requirements for the post-breakage behaviour of glass components can be included in the performance requirements of a design. At component level, this is often achieved by laminating the glass. At the level of the whole structure, the redundancy can play its role.

1.3. Structural glass for stiffening glass envelopes

Transparent building envelopes are often achieved by cladding a structural steel frame with glass plates. Modern grid-shells are most cost- and time-efficiently built with articulated connections. However, stability then has to be ensured by braces. The glass cladding can take over the stiffening function when it is included in the structural design. This principle was already applied by 19th-century glasshouse builders: a slender iron frame was stabilized by the glass cladding (paragraph 1.1).

[6] Schittich et al. 2007; Wurm 2007.

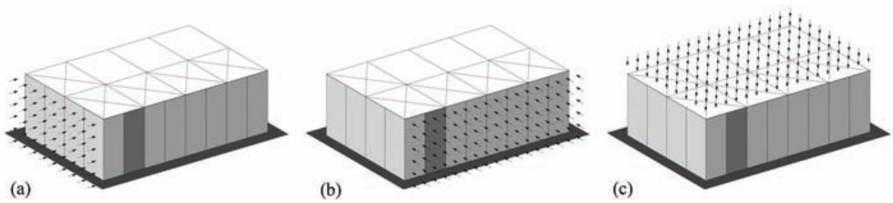

Figure 3-1: Actions on individual glass panel (dark grey panel): (a) in-plane shear force due to wind force in longitudinal direction; (b) out-of-plane bending due to wind force in transversal direction; (c) in-plane compressive force due to gravity loads on roof[7]

The loads acting on structural glass in the envelope are a combination of three simple load cases: in-plane shear forces, out-of-plane distributed loads and in-plane compressive forces (Figure 3-1). Torsion of the glass plates can also occur. This applies to both modern and historic iron and glass roofs. Both in-plane load cases can cause buckling of the glass plates. When the glass cladding of 19th-century iron and glass roofs has to contribute to the overall structural behaviour, the slender glass plates will be subjected to loads that might cause buckling.

Buckling behaviour of glass components (column buckling as well as lateral torsional buckling of beams and plate and/or shear buckling) have been studied fundamentally the last decade[8]. An overview of the most important parameters influencing the buckling behaviour of glass plates will be given in the following paragraph.

Luible[9] investigated column buckling, lateral torsional buckling and plate buckling experimentally, analytically and numerically. For the plate buckling analysis, monolithic and laminated glass plates (simply supported along the four edges) were loaded in-plane by a distributed compression force along two opposite edges. The analytical calculation based on plate bending theory and sandwich theory for laminated glass can predict the critical buckling load, but for laminated glass the prediction is less accurate (up to 8% overestimation compared to the numerical results). The post-buckling behaviour can only be predicted by experimental research or numerical (finite element) calculations. A parametric study by numerical modelling pointed out that the nature and size of the initial out-of-straightness of the glass panels has an influence on the buckling strength. The buckling failure origins at the plate surface under tension and the critical buckling load is therefore influenced by the surface quality. The glass plate thickness, the supports of the plate edges and the shear stiffness of the interlayer of laminated glass (if sufficiently high) also have an

[7] Mocibob 2008, p.27.
[8] Haldimann, Luible, and Overend 2008.
[9] Luible 2004.

important influence on the buckling behaviour. A calculation method using buckling curves (like in the stability analysis of steel components) is proposed where reduction factors take into account the slenderness and the initial deformation of the glass plate.

Wellershoff[10] focused on plate buckling of heat-strengthened single and laminated glass plates under shear and combined shear and out-of-plane loads. The properties of glass were studied and a probability method was developed, interlayer materials for laminated glass were tested and a statistical method was proposed to define load combinations acting on glass envelopes. For the investigation of plate buckling, two load introduction systems were studied analytically, numerically and experimentally: point connections in the corners of the glass plate and circumferentially glued connections (using an acrylate or a polyurethane adhesive in a two-sided adhesive bond). The stiffness of the adhesive affected the stress flow in the glass plate with circumferentially bonded edges. The method of using buckling curves was expanded for both shear and combination shear and out-of-plane loads with their specific reduction factors.

An elaborate research on different boundary conditions was carried out by Englhardt[11]. For the experimental research, three methods for introduction of compressive forces were used: a distributed load directly on the length of two opposite edges, a distributed load on two opposite edges via a soft interlayer, and a concentrated load on two setting blocks at each corner of two opposite edges. The numerical research added one more load case, where the setting blocks at the corners were used to introduce a diagonal compression force. The shape of the initial imperfection showed to define the most critical buckling shape (which is not necessarily the first buckling mode) and therefore also the maximum in-plane stress. The critical buckling load of the two systems with a distributed edge load was equal, but the post-buckling behaviour was different and defined by the stiffness of the interlayer material. The plates subjected to an axial load via setting blocks exhibited a higher critical buckling load. For all load introduction systems, buckling curved were developed with reduction factors extracted from numerical calculations taken into account the dimensions and the slenderness of the glass plates.
The influence of the stiffness of a supporting steel structure was investigated in detail for the plates subjected to axial loading by setting blocks. The higher the stiffness of the supporting structure, the higher the critical buckling load and the stiffer the post-

[10] Wellershoff 2006.
[11] Englhardt 2007.

buckling behaviour. Even the lowest stiffness still has a contribution compared to the buckling behaviour of a glass plate only supported at the corners.

All possible combinations of in-plane compression, in-plane shear, and out-of-plane distributed loads were first studied by Mocibob[12]. The research was carried out on heat-strengthened laminated glass panels. The loads were introduced in the glass plates by bolted connections at the corners or by linear adhesive bonds along two opposite edges. For the load introduction via adhesive bonds, mortar setting blocks were added to be in accordance with the regulations that adhesive joints cannot be applied to carry permanent loads (paragraph 2.2). During a parameter study, the quantities of critical shear buckling force, maximum out-of-plane deflection, in-plane displacement, maximum principle tensile stresses, and support reactions were evaluated. The glass thickness had to largest influence on all quantities, while the glass panel width, the PVB shear modulus and the adhesive stiffness had a small or moderate influence. A first proposal was made for a design method with buckling formulas, graphs and curves with a shear buckling reduction factor depending on the shear plate slenderness.

The possibility to use glass plates for shear panels was investigated more in detail by Huveners[13]. A single annealed glass plate was adhesively connected to a steel frame and subjected to a horizontal in-plane concentrated force. Three adhesive connections were under investigation: a connection along the thickness of the pane with a flexible polyurethane adhesive, a two-sided (on both glass surfaces) stiff epoxy connection, and a one-sided stiff epoxy connection. The flexible polyurethane connection had low in-plane stiffness where contact of the glass plate with the steel frame led to glass failure, but showed to have a good residual capacity after failure. The buckling behaviour was influenced by the width-height ratio of the glass plate and the normal and shear stiffness of the adhesive joint. With this flexible adhesive, a large percentage of façade panels would have to be bonded to attain sufficient stability and redundancy. The epoxy connections showed high in-plane stiffness, but the residual capacity of the one-sided connection was very poor. The buckling behaviour was affected by the width-height ratio and thickness of the glass plate and the shear stiffness of the adhesive. To avoid stress concentrations at the ends of the adhesive joints and to have enough flexibility to take up differential movement, equilibrium has to be found in the stiffness of the adhesive. More research is needed to develop elaborate design rules.

[12] Mocibob 2008.
[13] Huveners 2009.

Bedon[14] examined the results of all the above mentioned studies and proposed an effective thickness principle for glass plate buckling. By introducing reduction factors for in-plane compression and in-plane shear loads to include the specific behaviour of glass plates, the buckling behaviour of laminated glass can be modelled by modelling a monolithic glass plate with an equivalent thickness. The effective thickness principle showed to be accurate for different loads, different glass plate thicknesses, and different boundary conditions to predict the critical buckling load of laminated glass with a very stiff or a very soft interlayer material. The post-buckling behaviour and intermediate interlayer stiffnesses are only accurate in some cases.

From all this research, we can conclude that the important parameters that define the buckling behaviour of slender glass plates are:

- the real glass thickness (inside the tolerances on the nominal glass thickness) and the geometric slenderness of the glass component;
- the initial out-of-straightness[15];
- the tensile bending strength of glass, because failure origins at the glass surface under tension;
- the boundary conditions and supports, which define the buckling length but are also important to avoid eccentricity in the loads;
- the interlayer material behaviour of laminated glass (dependent on term of the loads and on temperature).

The critical buckling load of monolithic and laminated glass plates can be calculated analytically, but this leads to an overestimation of the buckling strength for compact plates and underestimation for slender plates[16]. Numerical analysis is more appropriate for accurate results and is indispensable to predict the post-buckling behaviour.

2. Modern standards on the use of glass

Glass envelopes have to fulfil a wide range of modern standards. Standards on energy performance, fire protection, acoustic performance, lighting, solar shading, and maintenance and cleaning might be applicable to some (parts of) 19th-century iron and glass roofs. These are however not specific for glass and will therefore not be discussed in detail here. Product standards and standards on the application of glass in particular cases will be discussed in the next paragraphs, specifically in the European and Belgian context.

[14] Bedon and Amadio 2012.
[15] Belis, Mocibob, et al. 2011.
[16] Haldimann, Luible, and Overend 2008.

Table 3-2: Tolerance examples for float glass and drawn sheet glass for renovation

	float glass[17]	drawn sheet glass for renovation[18]
thickness tolerance (1)	4 ± 0,2 mm	4 ± 0,3 mm
height and width tolerance	± 5 mm	± 5 mm
gaseous inclusions length	any number of ≤ 0,5 mm limited number of ≤ 3,0 mm	any number of ≤ 5 mm limited number of ≤ 30 mm

(1) the thickness tolerance depends on the nominal thickness

2.1. Product and testing standards

For new glass, product and testing standards are applicable. All glass products have their own standards specifying production tolerances and certification methods (e.g. insulated glass units, laminated glass and laminated safety glass, heat-strengthened soda lime silicate glass, etc). EN 572[19] groups all basic soda lime silicate glass products (float glass, wired glass, etc). An example of specifications on both float glass and drawn sheet glass for renovation is given in Table 3-2. Different test methods for glass are also elaborately described in standards. EN 1288[20] for example defines how the bending strength of glass has to be tested.
Similar product and testing standards are published for sealants and adhesives.

2.2. Standards on the application of overhead glazing

When glass is applied in overhead glazing, the security of the people walking underneath is primordial. Overhead glazing should be able to carry its own weight and a reduced snow weight for a limited number of time after failure[21]. At the same time, overhead glazing should be resistant to some impact loads (falling objects,

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[17] *EN 572-2: Glass in building - Basic soda lime silicate glass products - Part 2: Float glass* 2012.
[18] *EN 572-4: Glass in building - Basic soda lime silicate glass products - Part 4: Drawn sheet glass* 2012.
[19] *EN 572: Glass in building - Basic soda lime silicate glass products* 2012.
[20] *EN 1288: Glass in building - Determination of the bending strength of glass* 2000.
[21] Schittich et al. 2007.

vandalism, etc.) and therefore the experimental methods to determine different impact classes are defined in EN 12600[22].

Since 2007, the standard NBN S 23-002[23] is valid in Belgium[24]. For overhead glazing, surface temperatures between -20°C and 80°C must be considered. Especially, it defines that laminated glass must be used for overhead glazing, with a minimum of two panes of 3 mm thickness connected by two PVB layers. The possible interlayers for laminated glass are PVB foil (polyvinylbutyral), SG foil (SentryGlas ®), EVA foil (ethylene vinylacetate), and cast-in-place resin (CIP). PVB is most often used, but for structural applications also SG is used. CIP is less stiff at room temperature than PVB, but can be applied without lamination in an autoclave (which might be useful in renovation projects, Chapter 4 paragraph 5 p.116).

Specific technical guidelines are published in Belgium by the Belgian Building Research Institute (BBRI). Guideline 176 dating from 1989[25] was specifically dealing with glass applications in roofs. Rules of thumb for the thickness of monolithic and laminated glass, tables of maximum span per thickness and per load case, and execution principles were all covered. The guideline was outdated and therefore replaced by more recent guidelines, however not all topics were covered. Guideline 214[26] still references to the old guidelines for the calculation of the thickness of inclined or horizontal glass plates.

2.3. Standards on the use of glass as a structural component

Structural sealant glazing systems comprise the use of silicon sealants for adhesive bonding of glass façade panels to the supporting structure to carry wind loads. Permanent loads are still taken up by separate mechanical fixings. A European Technical Approval report ETAG 002[27] and a European standard EN 13022[28] are available that specify the products, performance requirements and testing methods for these systems.

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[22] *EN 12600: Glass in building - Pendulum tests - Impact test method and classification for flat glass* 2002.

[23] *NBN S 23-002: Glaswerk* 2007.

[24] A European pre-standard that will replace this Belgian standard is in preparation.

[25] *TV 176: Glas in daken* 1989.

[26] *TV 214: Glas en glasproducten, functies van beglazing* 1999.

[27] *ETAG 002: Guideline for European Technical Approval for Structural Sealant Glazing Systems (SSGS) - Part 1: Supported and unsupported systems* 1999.

[28] *EN 13022: Glass in building - Structural sealant glazing* 2006.

Table 3-3: Allowable stress [N/mm²] depending for annealed float glass depending on the load duration and the limit state[29]

	short (e.g. wind)	medium (e.g. snow)	long (e.g. self-weight)
ULS	17.0	8.5	6.4
SLS	30.7	15.3	11.5

The extensive use of glass for structural components, made a new series of standards for a broader structural application necessary. The technical guideline of the BBRI 242[30] on some specific glass constructions is available and gives design rules for aquaria and stairs. The mechanical properties of glass are presented and a new concept of effective thicknesses for laminated glass in structural applications is reported. Effective thickness is a concept in which laminated glass can be calculated by a monolithic plate with an equivalent thickness for the simulated structural behaviour.

The technical guideline refers to a European pre-standard for more details on the principle of effective thickness. The standards only cope with loads perpendicular to the glass plates, but this already covers an important part of all structural glass applications. The first part of the standard prEN 13474-1[31] deals with the non-structural application of glass in windows. The principle of partial safety factors is used to define the necessary thickness of the glass plate. The second part prEN 13474-2[32] focuses on all applications of simple forms of glass plates are subjected to perpendicular loading. The strength of glass is defined dependent on the type of glass and the load duration (Table 3-3). The third part prEN 13474-3[33] proposes a general method of calculating the strength of glass plates with the use of partial safety factors. The recommended material safety factors for annealed glass are 1.8 (for ULS) and 1.0 (for SLS). Analytical formulae for the calculation of the allowable stress for annealed glass are included in the standard: the bending strength of glass, the material partial factor, a factor to include the glass surface profile as a consequence of the production process, and a factor to include the load duration.

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[29] *prEN 13474-2: Glass in building - Design of glass panes - Part 2: Design for uniformly distributed loads* 1999.

[30] *TV 242: Bijzondere bouwwerken uit glas - Deel 1: Structurele toepassingen* 2011.

[31] *prEN 13474-1: Glass in building - Determination of the strength of glass panes - Part 1: Glass and glass products for fenestration* 2005.

[32] *prEN 13474-2: Glass in building - Design of glass panes - Part 2: Design for uniformly distributed loads* 1999.

[33] *prEN 13474-3: Glass in building - Determination of the strength of glass panes - Part 3: General method of calculation and determination of strength of glass by testing* 2008a.

The European pre-standard gives calculation methods for common application (part 1 and 2) and a general method based on the linear theory for non-common applications (part 3). The linear theory is only applicable for small deformations. When the glass plates deform more than half their thickness, the linear theory would overestimate stresses and deflections. For serviceability considerations, the standard specified the deflection of the glass to be lower than the span/65 or 50 mm, whichever is the lower value.

The effective thickness principle is mentioned in part 2 of the European pre-standard but elaborately explained in part 3. The effective thickness of laminated glass panes subject to perpendicular loading has to be calculated separately for the ultimate (stresses) and serviceability (deflections) limit state. The shear stiffness of the interlayer material influences the composite action of the different panes of the laminated glass. The formulae for the effective thicknesses are given in (Eq. 3-1) and (Eq. 3-2) both in general and in the case of no shear stiffness of the interlayer. Table 3-4 gives an example of an effective thickness calculation for a laminated glass of two panes of 3 mm thickness with two PVB layers in between (notation 33.2). The calculation is performed for zero interlayer stiffness and for a composite coefficient of 0.25. The latter is a simulation of the interaction of a PVB layer under normal conditions, but for long load durations and/or high temperatures the assumption of zero interaction is more accurate.

$$h_{ef,w} = \sqrt[3]{(1-\omega)\sum_i h_i^3 + \omega\left(\sum_i h_i\right)^3}$$
$$h_{ef,w,\omega=0} = \sqrt[3]{\sum_i h_i^3} \qquad \text{(Eq. 3-1)}$$

$$h_{ef,\sigma,j} = \sqrt{\frac{\left(h_{ef,w}\right)^3}{\left(h_j + 2\omega h_{m,j}\right)}}$$
$$h_{ef,\sigma,j,\omega=0} = \sqrt{\frac{\sum_i h_i^3}{\left(h_j\right)}} \qquad \text{(Eq. 3-2)}$$

$h_{ef,w}$ =	effective thickness for deformation of laminated glass plates
$h_{ef,w,\omega=0}$ =	effective thickness for deformation of laminated glass plates for no composition action
$h_{ef,\sigma,j}$ =	effective thickness for stresses of laminated glass plates
$h_{ef,\sigma,j,\omega=0}$ =	effective thickness for stresses of laminated glass plates for no composition action
ω =	coefficient representing the shear transfer by the interlayer (0 = no transfer, 1 = full shear transfer)
h_i , h_j =	thickness of the glass plates
$h_{m,j}$ =	distance of the mid-plane of glass plate j to the mid-plane of the laminated glass composition (ignoring the thickness of the interlayers)

Table 3-4: Effective thicknesses for calculating bending deflection and for calculating the stress of glass plies for glass plate composition 33.2 assuming an interlayer contribution of zero (no shear force transfer) or 0.25 (PVB layer at room temperature for short duration loads) [mm]

	ω=0	ω=0.25
$h_{ef,w}$	3.78	4.55
$h_{ef,\sigma,j}$	4.24	5.02

3. Conclusions

Modern standards make it mandatory to use laminated glass for overhead glazing during the renovation of 19th-century iron and glass roofs. An effective thickness can be calculated according to the European pre-standard prEN 13474, which will simulate an equivalent structural behaviour by a monolithic glass plate. The stiffness of the interlayer material defines the structural contribution of one glass plate to the other.

The large number of small glass plates in 19th-century iron and glass roofs can give the structure the necessary redundancy when one glass plate fails. The glass plates will be subjected to a combination of out-of-plane and in-plane loads. The small and slender plates will thus be liable to buckling failure. The stiffness of the connection between the iron glazing bar and the glass plates, and the structural thickness of the glass plates will be the major parameters influencing the buckling behaviour.

For the design of glass structures, engineers have to reference to recent research. A design standard like for other building materials however is indispensable. The preparation of "Eurocode 10" on the design of structural glass did start very recently.

Chapter 4

Restoration of 19th century glass roofs

1. Updating the Charter of Venice?

In 1830, Belgium became an independent country. The new nation was searching to define its identity, among others by its cultural legacy. The first king of Belgium Leopold I realised that historic buildings could have a contribution to this common identity. In 1835, he founded the *Commission des Monuments* to give advice to the government about protecting and restoring the built heritage[1]. In 1912 this was extended to the *Koninklijke Commissie voor Monumenten en Landschappen* or *KCML* (Royal Commission for Monuments and Sites), a multidisciplinary advisory council.
The first law on monuments in Belgium was only installed on 7th August 1931. The advice of the commission became compulsory for protecting and adjusting protected heritage. The KCML was divided in a Dutch-language and French-language division in 1968. Only in 1989 a third commission specific for the Brussels region was founded next to the two commissions for Flanders and Wallonia.

In the 19th century, various philosophies about the preservation of monuments were developed mostly in a national context. The two world wars in the 20th century made the awareness for an international policy grow. The ICOMOS (International Council on Monuments and Sites) was founded in this context in 1964 at the Second Congress of Architects and Specialists of Historic Buildings in Venice[2].
The present context for the advices of the three Belgian Royal Commissions for Monuments and Sites is defined by international charters. At the Congress in Venice in 1964, the International Charter for the Conservation and Restoration of Monuments and Sites[3] was written. The Venice Charter defined the first internationally acknowledged guidelines for the preservation of monuments and sites. The main principles of the charter were:

- the respect for monuments and sites and urban landscapes;
- the respect for contributions of all historical periods and for all aspects of cultural value;
- the importance of the context of the cultural heritage;
- the principle of minimal intervention;
- the principle of integration and at the same time distinction of adjustments;
- the scientific (instead of artisanal) approach;
- the need for an appropriate function for the heritage;
- the detailed documentation;

[1] Ministerie van het Brussels Hoofdstedelijk Gewest and KCML 2005; Van Santvoort 2011.
[2] webpage ICOMOS 2011.
[3] ICOMOS 1964.

- and the interdisciplinary aspects of all of these aspects.

The success of the Venice Charter is due to its universal and timeless principles, but a proper interpretation of this theoretical framework is indispensable. The charter also has limits, for example that its principle of authenticity is based on a mainly Western context and in its reduced attention for the balance between cultural value and present performance[4].

The Nara document on Authenticity from 1994[5] completed one of the lacunas. The authenticity of monuments and their components was designated as a major qualifying factor for heritage value. Understanding the authenticity is therefore crucial and depends on the availability, credibility and cultural context of information. The Nara document proposes a layered assessment of authenticity where form and design, materials and substance, use and function, tradition and techniques, location and setting, and spirit and feeling are evaluated[6]. Cultural diversity makes it impossible to judge authenticity based on fixed criteria. Van Balen[7] proposed to assess authenticity for continental Europe heritage by *the Nara Grid*, a matrix where all layers of authenticity defined by the Nara document are evaluated by their artistic, historic, social and scientific dimension. The Nara Grid can be used to evaluate restoration strategies of a monument, without being a measuring tool for authenticity values.

Updates of the Venice Charter are primarily focusing on specific sorts of heritage (gardens, underwater heritage, historic towns, etc). The Australia ICOMOS Burra Charter for places of cultural significance was adopted by the Australian National Committee of ICOMOS. Inspired by the Venice Charter, the cultural significance is defined as dependent on the cultural context and the conservation process is outlined more in detail. An attempt for an updated universal charter was made in 2000 with the Cracow Charter on Principles for Conservation and Restoration of Built Heritage[8], but it was never adopted by the ICOMOS General Assembly[9]. The concept of reversibility was more prominent in this charter according to present-day ideas about preserving monuments and sites.

The Principles for the Analysis, Conservation and Structural Restoration of Architectural Heritage[10] in 2003 were adopted by the ICOMOS General Assembly (called the Victoria Falls Charter in the following text). The charter was formulated to counteract the inappropriate application of modern building standards (with their

[4] Robert 2011.
[5] ICOMOS 1994.
[6] Ibid. Article 13.
[7] Van Balen 2008.
[8] ICOMOS 2000.
[9] Robert 2011.
[10] ICOMOS 2003a; ICOMOS 2003b.

stern safety factors) to the bearing structure of heritage buildings. The principles of the Venice Charter were translated to structural principles (e.g. context, multidisciplinarity, reversibility, compatibility but distinction, etc). Some other aspects were added or highlighted by the specific application in structural restoration:

- the safety evaluation defines the need for intervention;
- the safety evaluation is based on a combination of historical and qualitative (observation of damage and decay) and quantitative (tests, analysis, monitoring) approaches;
- dismantling and reassembly can only be applied after alternatives were considered as not applicable;
- repair is preferred over replacement;
- etc.

The reliability of all the sources (both historical, qualitative as quantitative) should be an integral part of the explanatory report of the structural restoration of architectural heritage.

2. The interpretation of the charters for iron and glass roofs

In Belgium, all listed monuments are considered equally protected. The margin in which the restoration architect can make interventions on a monument, the level of "touchability", is in Belgium defined on a case-by-case basis. The building as a whole can be listed, while on the listed building decree special attention can be given to specific components. This is in accordance with the Nara document on Authenticity, where it is considered impossible to consider authenticity based on universal criteria for all heritage buildings and sites.
The application of modern building standards on heritage buildings is often not possible in a strict way, no matter whether they relate to safety, energy performance or structural integrity. Therefore, the possible renovation strategies should be defined by weighing up the heritage value and the present-day requirements. The consequences of renovation strategies on the heritage value of the roof and its components have to be evaluated.

Possible interpretations of the charters for iron and glass roofs will be illustrated in the following pages by three case studies.

3. Case study: A restoration campaign at the Kibble Palace, Glasgow

3.1. Description

The Kibble Palace is the dominant building at the Glasgow Botanic Gardens. It is named after its original owner, John Kibble, who built a small glasshouse at his Coulport House at Loch Long. This glasshouse was dismantled and re-erected with enormous enlargement at the Glasgow Botanic Gardens in 1872-73[11]. It was officially opened on 6 May 1873[12]. The Kibble Palace functioned as a wintergarden, accessible for visitors of the Botanic Gardens during the day and available for concerts and social events in the evenings.

During the summer of 1881, the Kibble Palace was converted into a temperature house for growing plants. Hot water heating was installed, the central pond and the orchestra pit were filled in, the layout of the paths was changed and the main dome was raised to install windows for ventilation in the clerestorey (Figure 4-2)[13]. A collection of tree ferns was built up in the Kibble Palace since September 1881, which became recognised as the "National Collection" in 1990[14].

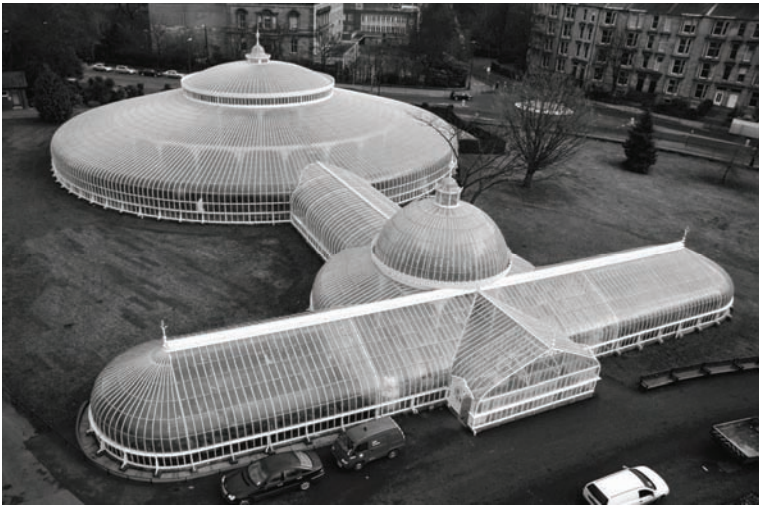

Figure 4-1: Kibble Palace after 2003-06 restoration campaign[15]

[11] Glasgow City Council 1998, p.3; Curtis 1999, p.30; webpage mast architects.
[12] Curtis 1999, p.33.
[13] Glasgow City Council 1998, p.3; Curtis 1999, p.54–56.
[14] Curtis 1999, p.56, 66.
[15] Development and Regeneration Services, Glasgow City Council 2003_2006.

The City of Glasgow Act in 1891 changed the ownership of the Kibble Palace to the city of Glasgow[16]. In 1924 a new road was laid skirting the Kibble Palace, thus harming the setting of the Kibble Palace[17]. The glasshouse suffered serious damage during the Second World War due to a landmine at the nearby river in March 1941, which caused the Kibble Palace to close down until November 1946[18].

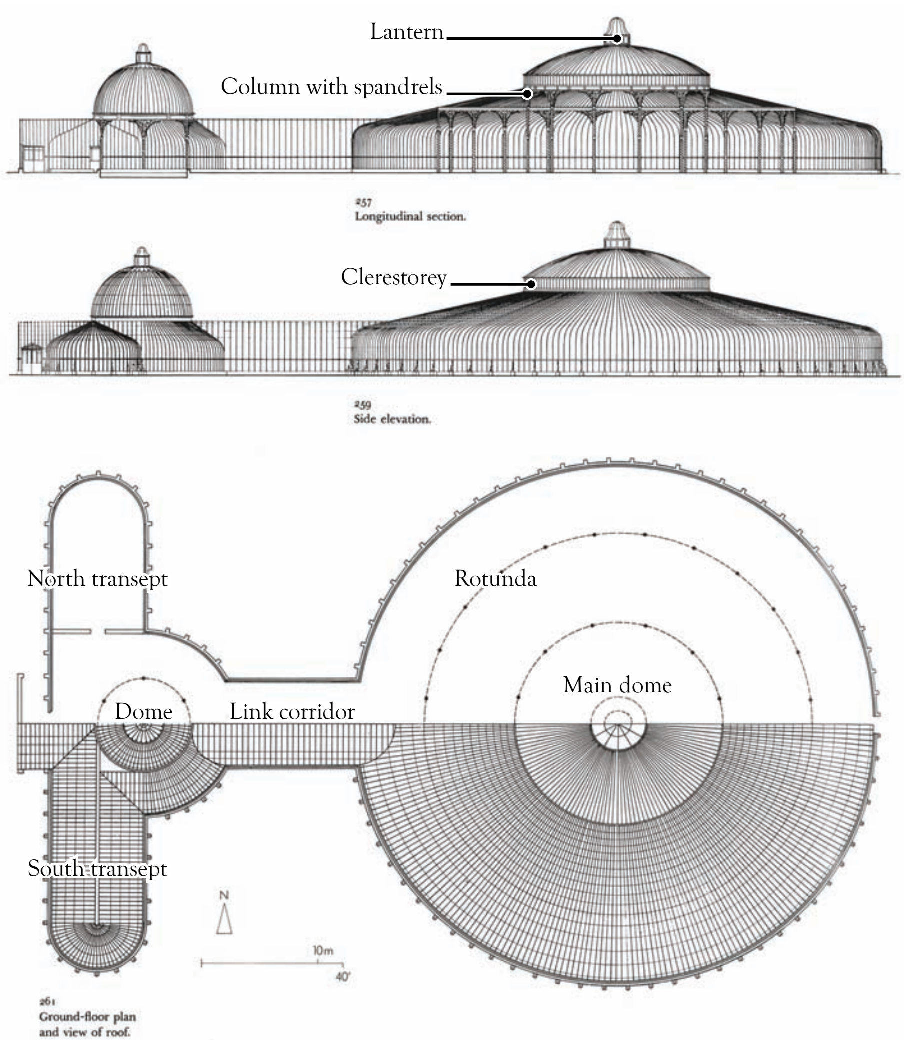

Figure 4-2: Ground plan, section and elevation of Kibble Palace[19]

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[16] Curtis 1999, p.62.
[17] Ibid., p.66.
[18] Ibid., p.64–66.
[19] Kohlmaier and Von Sartory 1991, p.247–248 with scale from Koppelkamm 1981.

The following text will focus on the main dome and the rotunda of the Kibble Palace (Figure 4-2).

The main dome is supported by a circle of 12 cast-iron columns. The columns are tied at their head by a circular cast-iron frame forming the clerestorey. The cast-iron spandrels at both sides of the columns give an extra support to this circular frame. The glazing bars of the main dome span radially from the cast-iron circular frame to a top ring which carries the lantern. Scotland Heritage and Kohlmaier draw a similar cross-section but respectively a moulded or compound section for this glazing bar (Figure 4-3 respectively c1 and c2). It has to be noted that the glazing bar section of the main dome and rotunda differs from the section used in the transepts and the dome. The different span of the glazing bars and the different time period from which they date (the transepts and dome were built originally by John Kibble, while the main dome and rotunda were added at the time of moving the glasshouse to the Glasgow Botanic Gardens) can be an explanation for this difference.

The rotunda consists of an inner and an outer ring, with an intermediate support by a circle of 24 cast-iron columns. The wrought-iron glazing bars start at the perimeter frame with a curved section, evolving to a linear profile and ending at the column ring supporting the main dome. Wrought-iron ties piercing the flange of the glazing bars give the structure resistance against radial forces (Figure 4-4). All iron components are connected using bolted and forged connections.

Sandstone and brick foundations carry the whole structure.

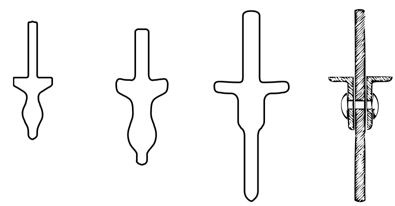

Figure 4-3: Glazing bars used in Kibble Palace: (a) transepts; (b) dome; (c1) main dome and rotunda[20]; (c2) main dome and rotunda[21]

[20] Glasgow City Council 1998, p.7.

[21] Kohlmaier and Von Sartory 1991, p.135.

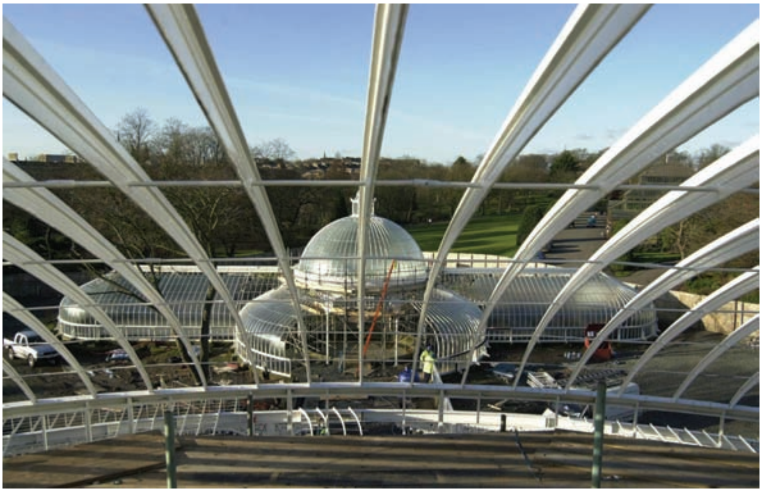

Figure 4-4: Glazing bars and ties in main dome (type c) during 2003-06 restoration campaign[22]

3.2. Historical value

Regular glazing repairs and painting of the iron structure has always been necessary[23]. Major renovation campaigns were carried out in 1932-33, 1953 and 1972. All external decorative ironwork (e.g. a circular grillage on top of the clerestorey) has been removed, probably in the 1940's[24]. In 1972, affected glazing bars in the link corridor and the small dome were strengthened locally using mild steel components, which caused some severe galvanic corrosion at the connected ends of these glazing bars[25]. The iron frame of the main dome and rotunda is generally in a good condition, mostly due to sufficient ventilation (regulated manually by opening the windows at different levels) and the original lead paint protection[26]. Cast-iron components are corroded only superficially. The wrought iron glazing bars are suffering from severe corrosion at their ends supported by the cast iron circular frame[27]. The wrought iron ties are corroded at their connections with the glazing bars[28].

The iron components can be addressed a high heritage value due to the preserved original material, and their specific moulded or compound sections.

[22] Development and Regeneration Services, Glasgow City Council 2003_2006.
[23] Curtis 1999, p.70.
[24] Ibid.
[25] Glasgow City Council 1998, p.3, 9; Curtis 1999, p.71.
[26] Glasgow City Council 1998, p.11.
[27] Ibid., p.10.
[28] Ibid.

The original glass is annealed glass of 3 mm thickness, sealed with putty to the glazing bars (which received regular mastic repairs)[29]. The listed building consent requires the original glass to be reinstalled if possible[30]. Dismantling the glass however takes a lot of time without the guarantee that the glass will not break. Therefore a compromise was made for the Kibble Palace: a strip of original glass was dismantled and reinstalled[31].

Slightly green tinted glass was present in a segment on the south side of the main dome and rotunda[32]. Since the early 1840s, the scorching of plants grown under sheet glass was a known problem, among others at the glasshouses in Kew Gardens[33]. Elaborate experiments were carried out before building the Palm House at Kew Gardens (Chapter 1 paragraph 1.2 p.19) to protect the tree ferns from the burning sun by colouring the glass plates[34]. This practice was abandoned in the late 1950's[35]. However, records about reinstalling the glass plates in the Kibble Palace, do not mention the green tinted glass[36].

3.3. Interventions

A major renovation campaign was carried out in 2003-06 (Figure 4-5 and Figure 4-6). The whole process was followed up by Historic Scotland. The campaign included dismantling and re-erecting the structure. The interventions carried out on the main dome and rotunda are described below.

The wrought-iron glazing bars were taken off-site[37] to apply sandblasting, repairing corroded spots with recycled forged wrought iron[38] and repainting. This is in contradiction with the Victoria Falls Charter in which dismantling is discouraged, however no problems with reassembling were reported. The loss of section of the tie bars was also compensated by using recycled wrought iron sections[39]. A replica of the decorative ironwork on top of the clerestorey was put back on the building (compare Figure 4-5 and Figure 4-6).

[29] Ibid.
[30] Turner and Scottish Ministers 2004.
[31] Wareing and Shepley Engineers Limited 2006.
[32] Glasgow City Council 1998, p.10; Schoenefeldt 2011, p.36.
[33] Schoenefeldt 2011, p.25.
[34] Schoenefeldt 2011.
[35] Glasgow City Council 1998, p.21; Curtis 1999, p.67.
[36] Wareing and Shepley Engineers Limited 2006.
[37] Glasgow City Council 1998, p.21.
[38] Ibid., p.11.
[39] Ibid.

Figure 4-5: Kibble Palace before 2003-06 restoration campaign[40]

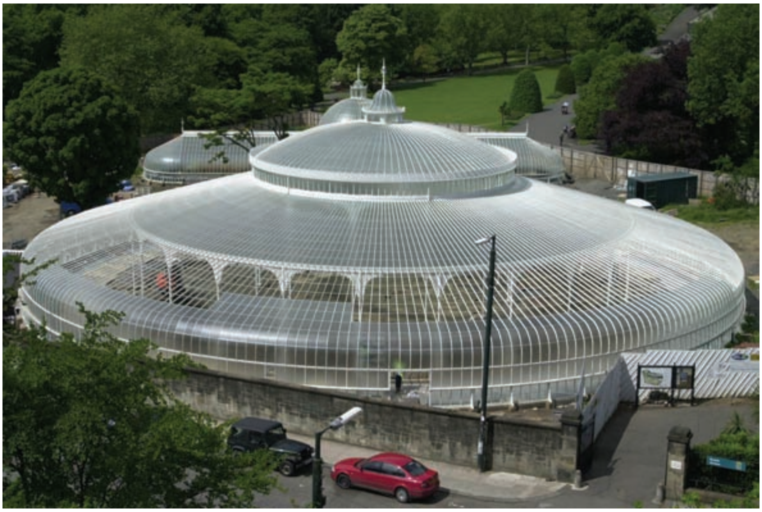

Figure 4-6: Kibble Palace near completion during 2003-06 restoration campaign[41]

An unmistakable part of the glass panels were broken before the restoration campaign started, causing a real danger[42]. All glass panels were thus dismantled, while taking care that a strip of original glass was preserved to be reinstalled later[43]. New glass

[40] Development and Regeneration Services, Glasgow City Council 2003_2006.
[41] Ibid.
[42] Glasgow City Council 1998, p.10.
[43] Wareing and Shepley Engineers Limited 2006.

panels were installed and sealed with a new silicon adhesive[44]. Due to weight restrictions, 4 mm thick toughened and laminated glass was only be installed around the perimeter of the rotunda (above the area where the public walks), while the rest of the structure was covered with 3 mm thick annealed glass[45]. This is a fine example of finding equilibrium between heritage value (reinstalling part of the original single glass) and modern requirements (installing laminated glass for safety reasons).

3.4. Restoration strategy

Two issues concerning the restoration of the structure to an original or later state are interesting to address.

First, there was the question which function to assign to the Kibble Palace after the last restoration campaign, keeping in mind the principle of the Venice Charter to assign an appropriate function to heritage. Originally it was a so-called wintergarden, a venue where the flora is subordinate to the entertainment function. When the Glasgow Botanic Gardens bought the Kibble Palace in 1880, the Palace was converted into a temperate glasshouse where plants were cultivated. Both functions contribute to the history of the building. The Kibble Palace at present is used for a combination of both. It is a tourist attraction for the building itself and for its temperate plant collection (among which the tree fern collection) and marble statues. In the evenings, small scale events and concerts can be organised and the Kibble Palace is available for venue hire. However, the Glasgow City Council warns renters about the consequences of hiring this historic glasshouse: the Palace might have to close during stormy weather, water penetration is possible during heavy rainfall, high temperatures occur during hot summer days, etc[46].

Secondly, the glazing bars of the main dome were seriously distorted during the history of the building (Figure 4-7). By interdisciplinary research of the design team of the 2003-06 restoration campaign, a possible explanation was given based on historical records, structural analysis and survey of the structure. The alterations in 1881 probably caused the distortion due to overloading of the structure: the main dome was raised (introducing the clerestorey) causing a decreased stiffness of the support and extra weight was added by integrating windows in the lantern[47]. Although Historic Scotland preferred to maintain the distorted shape, structural analysis showed that the lantern in that case had to be replaced by an aluminium replica to

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[44] Glasgow City Council 1998, p.11.
[45] Ibid., p.21.
[46] Glasgow City Council s.d.
[47] Glasgow City Council 1998, p.3, 10.

avoid further overloading[48]. The lantern was replaced[49] and pictures (Figure 4-8) seem to reveal a straightening of the glazing bars, however the available reports do not give a clear explanation.

Figure 4-7: Distorted shape of main dome of Kibble Palace before 2003-06 restoration campaign[50]

Figure 4-8: Straightened shape of main dome of Kibble Palace after 2003-06 restoration campaign[51]

[48] Ibid., p.21.
[49] information obtained via email conversation with Historic Scotland
[50] Development and Regeneration Services, Glasgow City Council 2003_2006.
[51] Ibid.

4. Case study: Iron structure of Saint-Hubertus Galleries, Brussels

4.1. Description

The Saint-Hubertus Galleries were built in 1846-47 as a new pedestrian connection and shopping gallery in the centre of Brussels. The complex is made up of three galleries: the Queen's Gallery with a slight angle goes over into the King's Gallery with the Prince Gallery as a side alley (Figure 4-9).
Before designing the Saint-Hubertus Galleries, architect Cluysenaar (1811-1880), already built a small gallery 'Cité' in Antwerp in 1844-45 but this was torn down in the 1860s due to the limited commercial success[52]. In 1846-47, Cluysenaar also designed the small passage Marché de la Madeleine in the centre of Brussels[53].
The Saint-Hubertus Galleries were part of a larger project to upgrade the centre of Brussels. The narrow and twisting alleys from the Middle Ages were not considered healthy and hygienic. Furthermore, the accessibility of the Grand-Place from the north was insufficient for both carriages and pedestrians.

Figure 4-9: Location of the Saint-Hubertus Galleries in the centre of Brussels[54]

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[52] Geist 1983, p.131, 199.
[53] Ibid., p.199.
[54] Reis et al. 1998.

Cluysenaar writes in his notes that he was inspired by the Galérie d'Orléans in Paris from 1828-30 (Figure 1-11 p.27)[55]. In its turn, Cluysenaar inspired architect Mengoni for his design of the Galleria Vittorio Emanuele II in Milan after a visit in the 1860s[56].

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 to the solely private funding before);
- it was unique in combining a commercial space (with retail, culture, leisure and dwelling function) with a public road in a monumental building[57] (Figure 4-12);
- 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[58] (Figure 4-11);
- the height on which the iron and glass roof was installed (which was normally in between the first floor with shops and the above floors with private housing) was almost doubled in comparison with the Galérie d'Orléans which emphasized the impression of the street[59].

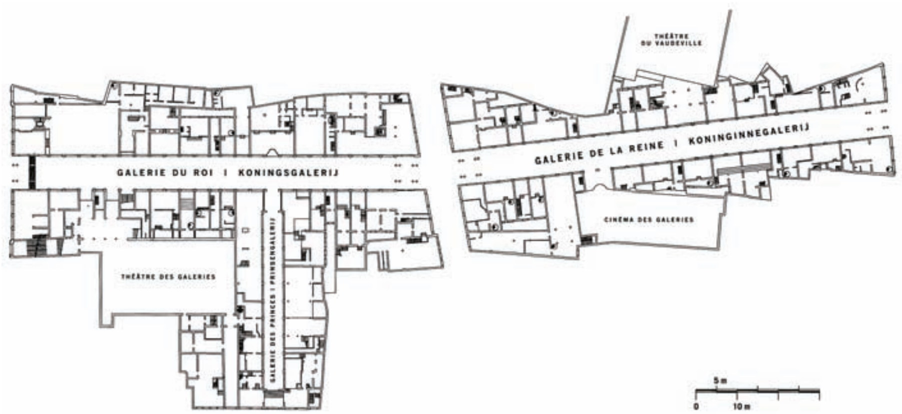

Figure 4-10: Ground plan of Saint-Hubertus Galleries[60]

[55] Ibid., p.51.
[56] Geist 1983, p.199.
[57] Plevoets and Cleempoel 2011, p.141.
[58] Geist 1983, p.113; Reis et al. 1998, p.28.
[59] Geist 1983, p.100, 106; Reis et al. 1998, p.55.
[60] Reis et al. 1998.

Figure 4-11: Queen's Gallery of Saint-Hubertus Galleries (2010-04-22)

Figure 4-12: Exterior view of the King's (in front) and Queens' Gallery (in back)[61]

The King's and Queen's Gallery together are 213m long. The Queen's Gallery consists of 230 iron arches, the King's Gallery of 214[62]. The glazed section is 18m high and 8m wide. The altering depth of the parcels is filled up with the apartments, shops, the theatre and cinema (Figure 4-10).
The King's and Queen's Gallery were built in 15 months from March 1846 until June 1847[63]. 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[64].

The iron and glass roof spans 8 m and consists of a series of wrought iron circular arches. Each arch is, at the central part with a low inclination, topped-off with a small pitched roof called the "lanterneau" (Figure 4-13). The lanterneau is raised from the arches via "columns". All arcs are supported with a hinge at both ends on a cast iron support strip which rests on top of the masonry walls[65]. One arch is constructed every 40cm. The arc and columns have a rectangular cross section of 50x7mm with a radius of 4m23, while the lanterneau has a section of 40x4mm[66].
Perpendicular to the arched main frame, rectangular sections are fixed at the ridge and the 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. L-shaped glazing bars connected to the wrought iron arches carry the glass plates of 40x46 cm[67].

[61] webpage A.2R.C 2010.
[62] A.2R.C, Forum, and TCA 1996b, p.6.
[63] Reis et al. 1998, p.36.
[64] Ibid., p.37, 53.
[65] Ibid., p.55.
[66] A.2R.C, Forum, and TCA 1996b, p.7.
[67] Ibid., p.10.

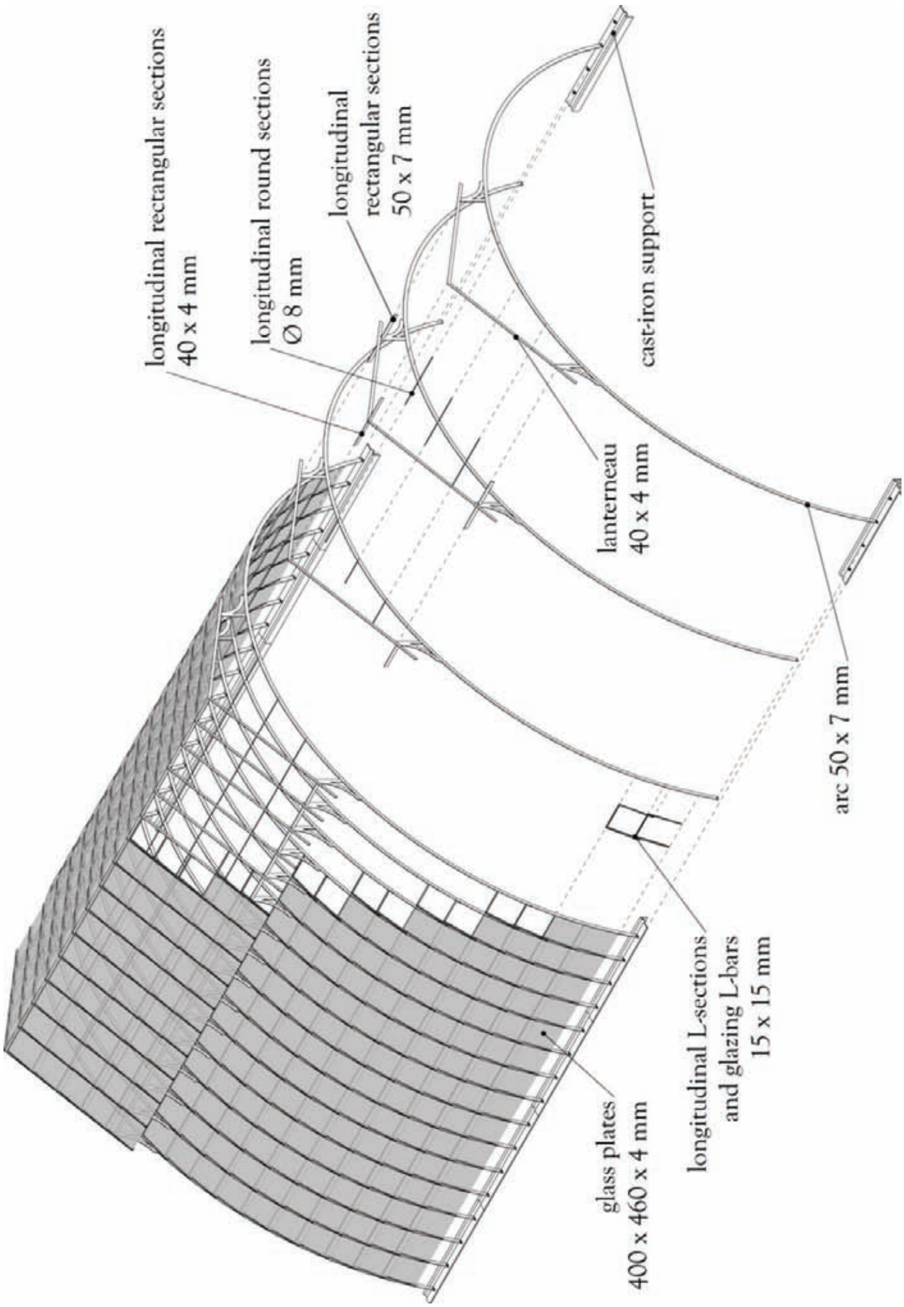

Figure 4-13: Isometric exploded view of iron and glass roof of Saint-Hubertus Galleries[68]

[68] Lefevre & Mechelynck Architectes Associes Scprl 1993, p.6.

4.2. Historical value

The galleries were protected as a whole by the Brussels Hoofdstedelijk Gewest in 1986. The "Société des Galeries" added an extract to the original lease contract of the shops to preserve the architectural homogeneity of the galleries[69]. The facades, sign boards for the shop names, the shop-windows etc. were therefore withheld from large changes. The atmosphere of the gallery is still unique due to this strategy of the Société, which is a major contribution to the heritage value of the Galleries.

During its lifetime, the Queen's Gallery underwent differential settlements up to 20cm during major urban planning works in Brussels. The glass roof was flexible enough to take up these settlements, but the cast iron supporting rails cracked and large deformations occurred in the longitudinal components[70].
On top of that, some specific components were severely corroded[71]. The columns in between the arch and the lanterneau are exposed to the weather and thus vulnerable for corrosion. The L-shaped glazing bars were also severely corroded, and some of them had already been replaced.
Other inconsistencies in the structure were observed due to maintenance[72]. Broken glass plates were replaced case-by-case with different varieties of glass, causing a patchwork of colours. The restoration report does not mention anything on the authenticity of the glass plates and the latter can thus not be evaluated for their heritage value. The putty connection was often covered with an aluminium strip to ensure the water tightness, thus obstructing some light to enter in the gallery[73].
The non-recorded maintenance works on the iron and glass components described above, contribute in a negative way to the heritage value of these roof components. The original material of the iron arches however is still authentic and can be considered of high heritage value. The heritage value of the glass plates cannot be defined. Interventions from different time periods can also contribute to the heritage value, however information is missing to date the separate glass plates.

[69] Reis et al. 1998, p.46.
[70] A.2R.C, Forum, and TCA 1996b, p.12.
[71] Ibid., p.13.
[72] Ibid., p.15.
[73] Ibid.

4.3. Interventions

The first major renovation campaign was carried out from 1993 until 1997 by A.2R.C architects under supervision of the KCML, to celebrate the 150th birthday of the Saint-Hubertus Galleries.

The glass was replaced with new laminated glass to decrease the risk of falling glass pieces (Figure 4-18): 2 panes of 2 mm thickness were laminated with a PVB intermediate layer of 0,38mm[74]. This thin glass plate composition does not fulfil the later introduced standards on safety glass (which prescribe a minimum of 2 panes of 3 mm thickness). The glass is protected against sliding down by a screw at the end of each plate (Figure 4-18).

The original L-shaped glazing bars needed replacement (Figure 4-16). The new "parclose" clamps the glass plates in between neoprene rubbers so that it stays in place (Figure 4-17 and Figure 4-18). The system allows for individual glass plates to be replaced. The top joint between the parclose and the iron arch is sealed with a one-component polyurethane modern sealant[75], although the drawings from the restoration report mention a silicone sealant[76]. The parclose completely covers the top of the arch section. The external view from the top of the galleries is thus changed (Figure 4-14 and Figure 4-15), but this is not visible except for maintenance workers. There is one parclose section per glass plate and they overlap each other when the glass plates do overlap (Figure 4-17 top). Due to the different inclinations, the parclose system is different for the arch and the lanterneau. The clarity of the original system is lost, but the maintenance friendliness is improved.

Figure 4-14: Exterior view of Saint-Hubertus Galleries before 1993-97 restoration campaign[77]

Figure 4-15: Exterior view of Saint-Hubertus Galleries after 1993-97 restoration campaign (2011-09-17)

[74] A.2R.C, Forum, and TCA 1996a.
[75] Ibid.
[76] A.2R.C, Forum, and TCA 1996b.
[77] *TR 772 1/2 2043-0116: Dossier renovation-verrières* 1988.

Technical studies were carried out for the restoration of the iron frame: a calculation of the iron bearing frame, a mechanical test on the iron material and a weldability test[78].

The iron frame of the roof was treated on-site: the paint was removed and the remaining section was evaluated (structural calculation showed that the thickness of the arch could be reduced from 7 mm to 5.5 mm[79]). The iron showed to be forgeable, so the deformations could be repaired by hot treatment[80]. The stability check of the iron frame revealed a critical instability problem at the base of the arches, so it was decided to add an extra longitudinal L-section of 15x15 mm at the bottom of the arcs[81]. The columns between the arch and the lanterneau were replaced with a welded component when too heavily corroded[82] (Figure 4-20).

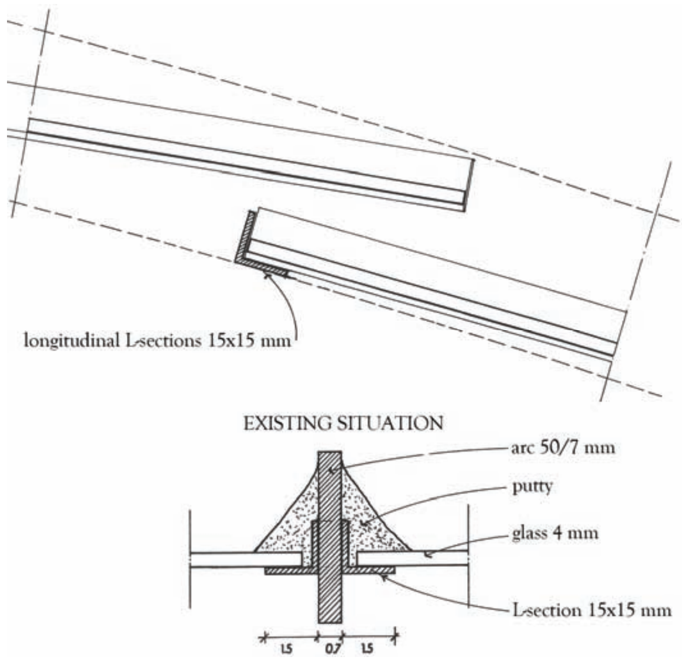

Figure 4-16: Transverse (*) (bottom) and longitudinal (top) connection of Saint-Hubertus Galleries before 1993-97 restoration campaign[83]
(*) with original drawing of putty section, see remark p. 66

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78 T.C.A., CEWAC, and OREX 1996.
79 Ibid.; A.2R.C, Forum, and TCA 1996b, p.13.
80 T.C.A., CEWAC, and OREX 1996; A.2R.C, Forum, and TCA 1996b, p.13.
81 A.2R.C, Forum, and TCA 1996b, p.13.
82 Ibid.
83 Ibid., p.26.

The longitudinal rectangular sections were preserved, but the round bars were replaced with new ones (but it is unclear from the restoration file with which material)[84]. The L-shaped glazing bars needed replacement. At the same time, the architects were looking for a solution low in maintenance compared to a putty connection. A stainless-steel "parclose" to hold the glass plates is connected to the arch with stainless-steel bolts[85] (Figure 4-17).

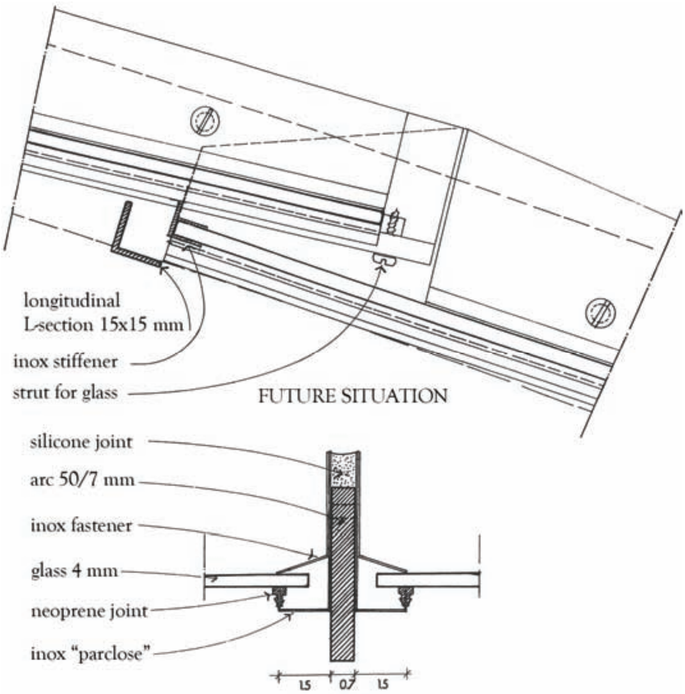

Figure 4-17: Transverse (bottom) and longitudinal (top) connection of Saint-Hubertus Galleries after 1993-97 restoration campaign[86]

[84] Ibid., p.35.
[85] Ibid., p.36.
[86] Ibid., p.26.

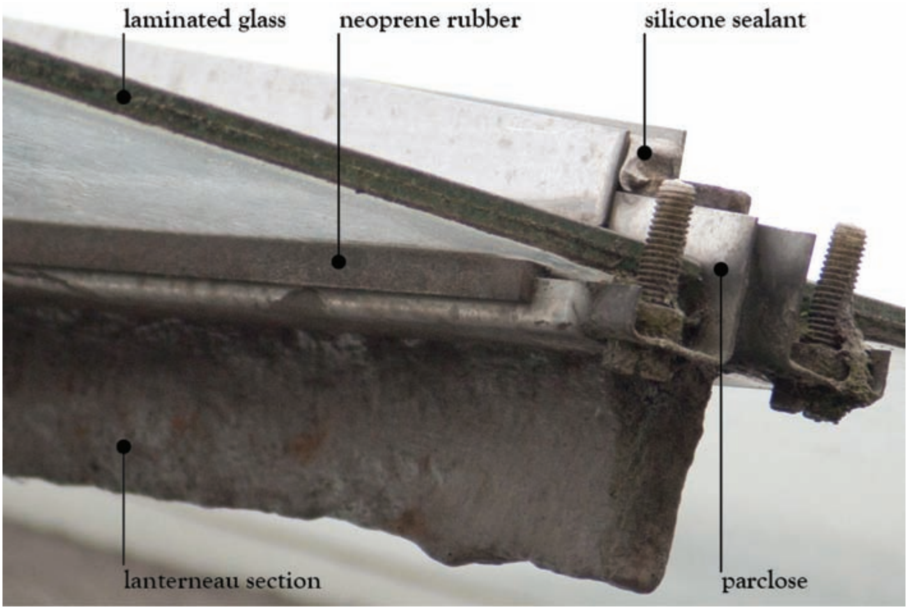

Figure 4-18: Interventions on connections of
the Saint-Hubertus Galleries, Brussels (2011-09-17)

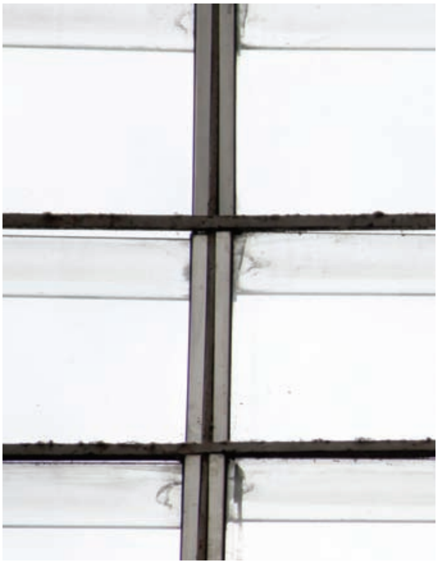

Figure 4-19: Arc with stainless-steel
parclose and longitudinal L-sections
after 1993-97 restoration campaign
(2012-07-27)

Figure 4-20: Columns between
arc and lanterneau
after 1993-97 restoration campaign
(2012-07-27)

4.4. Restoration strategy

Diverse studies were performed for the restoration campaign, covering interior and exterior aspects. The renovation of the glass roof did include interventions on the iron structure, the glass plates and the connection details. Laminated glass plates were installed to ensure the safety in this public street. The connections between the iron frame and the glass plates were replaced by a maintenance friendly solution. And finally, the iron frame was checked for its strength, stiffness and stability according to modern standards.
The interventions on this roof were mainly necessary due to a lack of maintenance in the past. During this renovation campaign, the interventions on the connections caused some loss of heritage value but decreased the necessary maintenance intensity. The height of the glass roof limited the visible impact of these interventions.

5. Case study: 19th century glass in UCB, Brussels

5.1. Description

In 1848, the bank company "Société Anonyme de l'Union du Crédit" was founded. The Brussels architect Désiré De Keyser was asked in 1872 to design a new building located at the Warmoesberg close to the Saint-Hubertus Galleries in Brussels[87]. In 1969, the building was bought by the United California Bank of Los Angeles. The first two owners had the same initials, which is the reason why this monument is still called UCB-building. The National Bank of Belgium bought the complex in 1979. Nevertheless, the building was vacant from 1981 until the start of the renovations in 2003.
Numerous modifications and additions were made to the complex: in 1900 architect Henri Maquet added a conference room which was demolished in 1982, in 1969 the new owner altered the façade in Warmoesberg, originally the large and small 'counter halls' (Figure 4-21) did not contain counters, in 1948 architect Polak made large modifications to the interior, etc.
The building consists of brickwork walls with steel floor beams and brick vaults. The roof lights above the 'counter halls' are double-walled roofs. Polonceau trusses carry both the outer roof which seals of the external climate and the inner roof which is decorated for the view from below (Figure 4-23, Figure 4-24, Figure 4-25 and Figure 4-26). The outer roof was originally covered with single armed glass.

[87] ‘Historische schets’ in *TR 1540/2: UCB - restauratiedossier* 2003.

Figure 4-21: Interior view of UCB library under the large roof light looking towards the small roof light (2012-08-10)

Figure 4-22: Interior view of stairwell of UCB library (2012-08-10)

Figure 4-23: Bottom view of large roof light (2011-09-17)

Figure 4-24: First floor view of small roof light (2011-09-17)

Figure 4-25: Double roof of small roof light (2010-03-02)

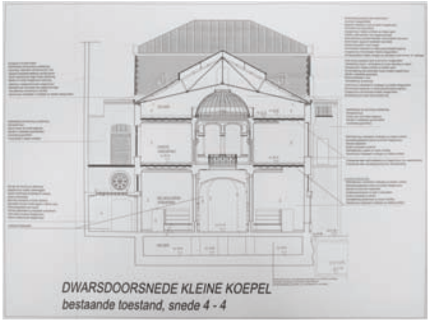

Figure 4-26: Transversal section through small roof light[88]

5.2. Historical value

In 1981, the Sint-Lukasarchief handed in a proposition to protect the building[89]. The main reasons for this proposition were:

- it is the only bank building from that period still remaining in Brussels;
- the building has a unique bank interior with visible structural elements;
- only two interiors designed by architect Désiré De Keyser are preserved until today, one of which is the library of the National Bank and the other one is the synagogue in Regentschapsstraat which he won the competition for in 1868 (it was built 1875-1878)[90].

In 1984, 3 years after the first proposition, part of the complex was protected. The large counter hall, small counter hall, stairwell, the corridor from the street to the large counter hall and the according roofs of the building at Warmoesberg 57 in Brussels were listed by law on 29/02/1984[91] (Figure 4-27, Figure 4-21 and Figure 4-22). The rest of the building underwent too many alterations to be representative for a bank building from the 1870s. The stucco, stone and carton-pierre decorations in the protected zone are of high quality and thus high heritage value[92].

[88] *TR 1540/3.3: UCB - Plannen* 2004. Transversal section, existing situation, 2002-09-20.

[89] 'Aanvraagdossier Sint-Lukasarchief' in *D 2043/0086: UCB - Intérieur d'immeuble* 1984.

[90] *TR 1540/2: UCB - restauratiedossier* 2003.

[91] *D 2043/0086: UCB - Intérieur d'immeuble* 1984.

[92] 'Voorbereidend restauratiedossier' in *D 2043/0086: UCB - beschermingsdossier* 1984.

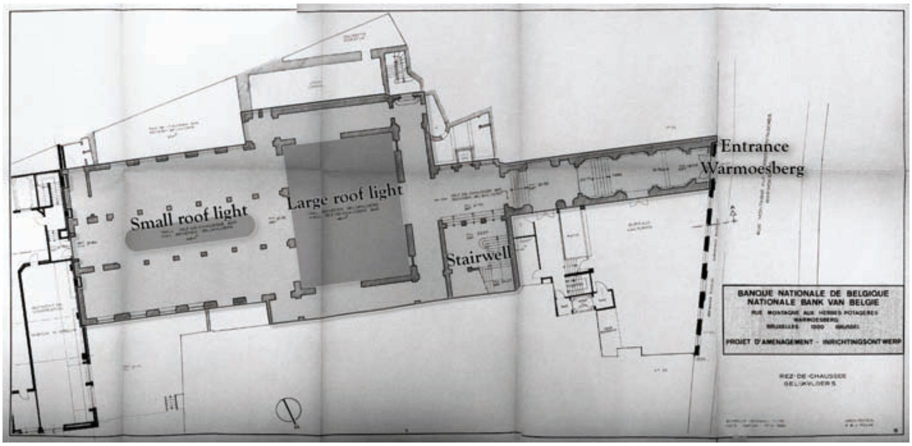

Figure 4-27: Ground plan of protected zone of UCB building[93]

The inner glass structures of both roof lights were recorded in detail in 2004 (Figure 4-28 and Figure 4-29). The original 3 mm thick glass showed to be acid treated on one side[94]. The glass plates of the small roof light were sandblasted after installation[95], which is clear because the acid treatment is still visible in the rebate part[96]. The original iron-glass connections of both roof lights were still present and can thus be considered of high heritage value. The original transversal connection was different for both roof lights: a narrow strip of putty was put on top of a lead strip at the two supported glass edges for the small roof light, while the four supported edges at the large roof light were all sealed off with putty only [97]. Later replacements of glass plates in the small roof light can be recognised by the wider strips of putty. The original longitudinal connection consisted of a lead hook of 5 mm width, which was placed at the end of the overlapping glass plates in the rebates (system described in Chapter 2 paragraph 3.3).

[93] *D 2043/0086: UCB - Intérieur d'immeuble* 1984. Plan Architectes Polak A.&J. 17/05/1982 with notes from the author.

[94] 'Verslag aanvullend onderzoek van de binnenafwerking en restauratieadvies', Studiebureau L. De Clercq in *TR 1540/2: UCB - restauratiedossier* 2003; 'Report Glasmalerei Peters' in *TR 1540/3.1: UCB - koepels en vloeren* 2004.

[95] 'Restauratie glazen binnenkoepels', Glasmalerei Peters in *TR 1540/3.1: UCB - koepels en vloeren* 2004.

[96] The rebate is a continuous groove along the edge of a window frame in which the glass is placed. For T-shaped glazing bars, this means the flange of the section where the glass is placed on.

[97] 'Rapport' by B.I.E. Verres in *TR 1540/3.1: UCB - koepels en vloeren* 2004.

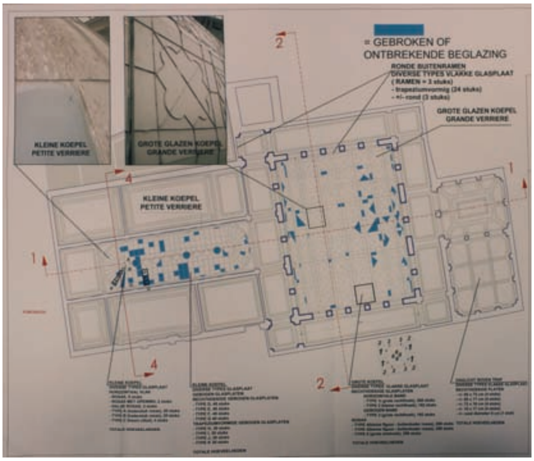

Figure 4-28: Plan of inner roof lights with indication of the broken or missing glass panes[98]

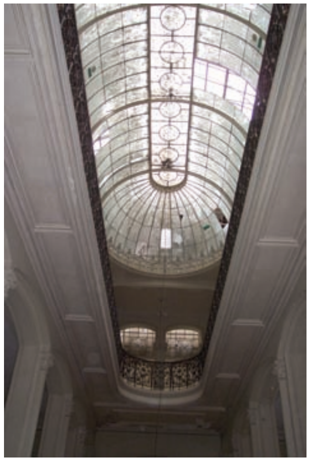

Figure 4-29: View on small roof light[99]

Figure 4-30: Interior view of newly installed small roof light (2010-03-02)

Figure 4-31: Laminated glass plates (2010-03-02)

5.3. Interventions

A first renovation proposition was handed in at the Direction of Monuments and Sites in 1994. It was rejected because the historical study was insufficient and a structural calculation of the roofs was considered indispensable[100]. A new proposition was submitted 9 years later in 2003[101].

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[98] *TR 1540/3.3: UCB - Plannen* 2004. Plan of roof lights: existing situation, 2004-03-31.

[99] picture by Glasmalerei Peters, 2004-03-15

[100] *D 2043/0086: UCB - Intérieur d'immeuble* 1984.

[101] *D 2043/0086: UCB - beschermingsdossier* 1984.

In 1994, 'Verdeyen & Moenaert' made a calculation of the Polonceau trusses of both roof lights[102]. The stresses were acceptable for all classical loads. Nevertheless, the trusses showed to have insufficient resistance against upwind forces. The concluding recommendations were to perform material tests to check whether extra loading was possible and to make the roof completely airtight. Different propositions were made in 2004 to renovate the outer roofs. The Royal Commission of Monuments and Sites (KCML) rejected some of these propositions[103]. They emphasized that the Polonceau trusses were part of the protected zone and had to be renovated in-situ without disassembly, according to the recommendations in the Victoria Falls Charter. Finally, the outer roof was renovated by adding new aluminium frames to hold thermally insulated laminated security glazing[104] on top of the preserved Polonceau trusses which received a fire-resistant paint[105].

Some specific considerations were made during the restoration process about the original glass that was still present in the inner roofs (the decorative side) of both roof lights. The following paragraphs will therefore only focus on the inner roofs of the small and the large roof light.
The restoration of the inner roof was a major concern of the KCML. Many of the original glass plates were still present in the structure, thus increasing the heritage value of this roof component (Figure 4-28). At the same time, the modern standards on overhead glazing for public areas required the installation of laminated glass (Chapter 3 paragraph 2 p.88). In March 2004, the Direction of Monuments and Sites agreed to place a coating on top of the original single glazing to decrease the chance of glass fragments falling down[106]. In November 2004, the National Bank of Belgium made a contra proposition to laminate the original panels to fulfil all security standards for overhead glazing[107]. The following arguments were highlighted:

- technical advice from glass restoration specialists of Glasmalerei Peters on the casting resin lamination technique[108];
- a rumour about a glass shard accident in the Brussels Centre for Fine Arts[109];

[102] ‘Structuur- en Stabiliteitsonderzoek’ in ibid.
[103] *TR 1540/3.2: UCB - restauratie koepel, restauratie, reiniging en inventarisatie vloeren* 2004.
[104] *TR 1540/3.3: UCB - Plannen* 2004.
[105] "Voorbereidend restauratiedossier" in *D 2043/0086: UCB - beschermingsdossier* 1984.
[106] *TR 1540/2: UCB - restauratiedossier* 2003.
[107] ‘Note of 24/11/2004’ in *TR 1540/3.2: UCB - restauratie koepel, restauratie, reiniging en inventarisatie vloeren* 2004.
[108] Ibid.

- a reference to a brief calculation note from March 2004 that the iron frame of the large roof light is able to resist an increased glass weight and a wind pressure and suction load without reinforcements[110];
- uncertainty about the efficiency of the coating during fire and its colour and adherence variability on the long-term.

Two reports from B.I.E. verres and Glasmalerei Peters proved their experience with the casting resin technique but unfortunately it is irreversible[111]. The KCML accepted this proposition in December 2004 under certain conditions. In 2005, several studies and experiments were carried out with the technique[112]:

- reports were delivered on colour invariability and efficiency of lamination effect of the casting resin technique;
- some original glass plates should be kept aside as witnesses of the original material and the evolution of the laminated panes should be followed up;
- tests were carried out with original plates laminated to newly manufactured cylinder glass, to check whether the colour of the laminated pane matched the original colour (the overall results were satisfying, but more experiments needed to be carried out to find a match between new and original laminated panes);
- lamination of the original glass plates enlarged the weight of the panes and therefore the original system of longitudinal connection with lead hook was too flexible which made the design of a new connection necessary.

The renovation works started with removing the glass plates from the iron frame. With a thin and flexible blade, the putty could be removed by carefully scraping and lightly hammering[113]. Covering the putty connection with compresses soaked with dimethylsulfoxide was also suggested, but was a labour intensive alternative[114]. Next, the iron structure was sandblasted and repainted. The dismantled glass plates were brought to a workshop where they could be laminated. While a mould held the plates in position, a resin was poured into a cavity of 2 mm wide[115] (Figure 4-31). The

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109 Ibid.

110 'Grande coupole' in *TR 1540/3.1: UCB - koepels en vloeren* 2004.

111 'Internal Direction of Monuments and Sites email' from 23/11/2004 in ibid.

112 *TR 1540/3.2: UCB - restauratie koepel, restauratie, reiniging en inventarisatie vloeren* 2004.

113 "Rapport", B.I.E. Verres in *TR 1540/3.1: UCB - koepels en vloeren* 2004.

114 "Restauratie glazen binnenkoepels", Glasmalerei Peters GmbH in Ibid.

115 'Report 08/11/2004' in *TR 1540/5.1: UCB - werfvergaderingen*.

technique was only acceptable for the KCML because it was applied to flat and colourless glass plates[116].

The glass plates were put back in place with new connection details for both the transversal and longitudinal connection. The Direction for Monuments and Sites initially did forbid the use of silicones to seal the glass plates to the glazing bars in favour of linseed oil putty[117]. Eventually the latter was not possible because of incompatibility with the lamination resin[118]. The transversal connection was finally executed with a narrow silicone strip (Figure 4-33).

Figure 4-32: Original longitudinal connection of small roof light[119]

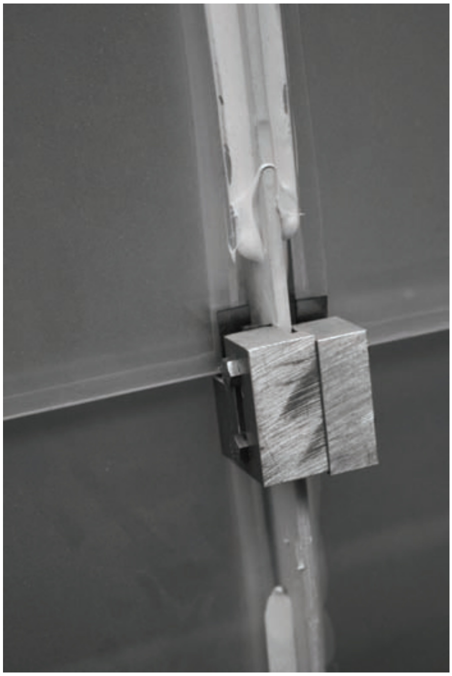

Figure 4-33: New longitudinal connection of small roof light (2010-03-02)

[116] *TR 1540/3.2: UCB - restauratie koepel, restauratie, reiniging en inventarisatie vloeren* 2004.

[117] 'Letter KCML to Direction Monuments and Sites' from 30/6/2005 in *TR 1540/2: UCB - restauratiedossier* 2003.

[118] 'Letter KCML to Direction Monuments and Sites' from 08/03/2005 in *TR 1540/3.2: UCB - restauratie koepel, restauratie, reiniging en inventarisatie vloeren* 2004.

[119] picture Glasmalerie Peters GmbH.

The longitudinal connection needed to be redesigned because of the increased weight of the glass plates. The overlap between two glass plates needed to be kept open[120]. The original lead hooks were soft thus ensuring the movable capacity of the whole roof which was probably the reason why so many original glass plates were preserved[121]. In April 2006, two systems were evaluated: the use of copper hooks, very similar to the original lead hooks; and a system of two linked stainless-steel sections which were clamped onto the iron glazing bar[122]. The copper hooks were still insufficient to carry the weight of all the glass plates above. The stainless-steel sections were finally installed with a copper strip underneath to have a softer contact surface (Figure 4-32 and Figure 4-33). The blocks are visible from beneath the structure, but the translucency of the roof is preserved (Figure 4-30).

5.4. Restoration strategy

The roof lights of the UCB library are double-walled roofs, where the inner roof still contained original decorative glass that had a high heritage value. Although double-walled roofs will not be studied further on in this research, the specific restoration strategy for preserving the original glass is outlined to give a full overview of the possible interventions on iron and glass constructions.

To fulfil the modern standards on safety and preserve the historic glass plates, a resin lamination technique was applied. Because the outer roof carries all the climatic loads (snow, wind, etc), the increase of the weight of the glass cladding could be taken up by the inner roof structure. Major concerns went to the reversibility of the lamination process (in accordance with the Cracow Charter), however the present-day resin techniques are not reversible. Some samples of original glass plates were kept aside and are stored on-site, as a witness of the original texture and material for if something goes wrong with the resin laminates. The connection details were adjusted to the increased weight of the glass plates, thus losing some of the heritage value that these connections were adding. The visible implications of these changes are almost invisible seen from the public area of the building.

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[120] 'Letter KCML to Direction Monuments and Sites' from 30/6/2005 in *TR 1540/3.2: UCB - restauratie koepel, restauratie, reiniging en inventarisatie vloeren* 2004.

[121] 'Letter from KCML to Direction of Monuments and Sites' from 10-12-2004 in ibid.

[122] 'Letter KCML to Direction Monuments and Sites' from 14/04/2006 in ibid.

6. Conclusions

The three case studies illustrate different restoration strategies for finding equilibrium between heritage value and modern requirements. At the UCB library, the main aim was to preserve the authentic glass material, while avoiding any restrictions on the accessibility of the building. The case of the Saint-Hubertus Galleries however was more focused on the iron structure. Decreasing the necessary maintenance intensity was the major concern for the interventions. On the contrary, the Kibble Palace is an example of an integrated approach, where all the components of the roof were taken into account. The interdisciplinary team that dealt with the restoration studies, proposed some restoration options that tried to give an answer to modern requirements so that the heritage value of the roof could be preserved.
However in all three cases, more attention could have been given to the principle of minimal intervention. With a change of the function of the UCB library, or with an elaborate study on possible adhesive materials that could replace the putty in the connections of the Saint-Hubertus Galleries, applied interventions could maybe have been avoided. A more detailed structural recalculation of the Kibble Palace could have explained the structural equilibrium that the distorted main dome had reached. Now both the distorted shape and the original lantern were lost.

The next chapters will go more into detail into the structural modelling possibilities for 19th-century iron and glass roofs. The mechanical properties of on-site executed historic putty connections will be investigated in Chapter 5. These connection characteristics will be implemented in a global structural recalculation of a 19th-century iron and glass roof in Chapter 6.

Chapter 5

Sealing the connection between iron glazing bar and glass plate

If the glass panels and the iron frame must structurally work together, these two parts have to be bonded in such a way that loads can be transferred. In 19th-century joints, the glass plates were sealed to the iron glazing bar by linseed oil putty (Figure 5-1 and Chapter 2 paragraph 3.2). Starting from the 1950's silicones were used[1]. Later in the 20th century, also acrylates, polyurethanes and other polymers were used for sealing and bonding iron-glass connections. To evaluate the force transfer between iron and glass, the mechanical properties of the joining material must be known.

In practice, we face two problems. On the one hand the mechanical properties of the 19th-century putty are not known. On the other hand, renovation asks for circumstances, deviating from the ideal workshop conditions. This chapter will go into the experimental work that was carried out to determine the mechanical properties of joining material, taking into account parameters important during renovation.

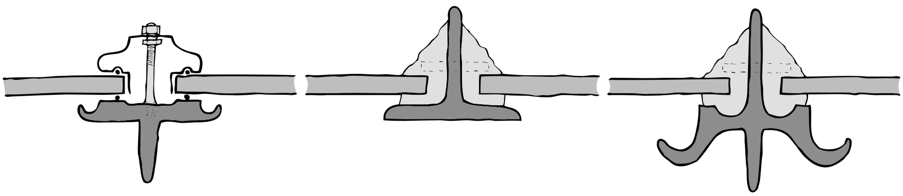

Figure 5-1: Transverse connection in glass roofs: section system with rubber strips (left); simple T-section with putty (middle); and moulded T-section with gutters sealed with putty (right)

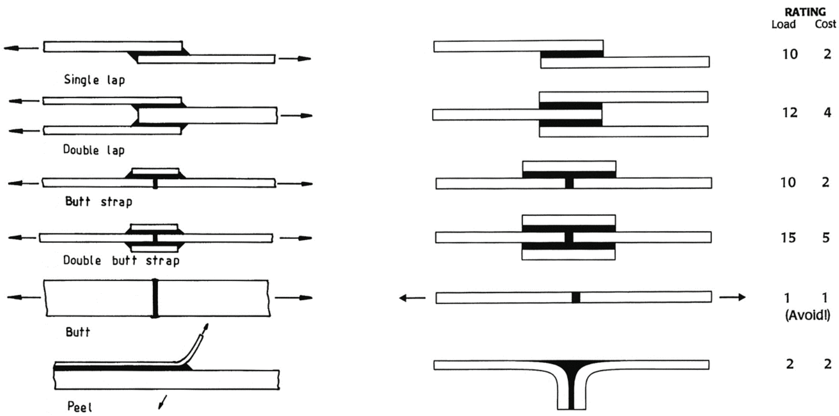

Figure 5-2: Comparison of common engineering adhesive joints and their structural efficiency based on strength and cost (the higher the load rating, the stronger is the joint)[2]

[1] Scheffler and Connolly 1996.

[2] Adams, Comyn, and Wake 1997, p.13, 121.

1. Modern adhesive bonding

Nowadays, adhesive bonding is often applied in the automotive and aerospace industries. Research from these application fields provided an increasing knowledge about adhesion technology and the development of new adhesive products.
The last years, adhesive bonding is more often applied in building industry[3]. The esthetical look and the thermal insulation properties of an adhesive joint are considered advantageous for some applications. Moreover, adhesive bonding can be applied to connect two completely different materials. In the glazing industry, another important benefit is the uniform stress distribution which avoids stress concentrations in the brittle glass plates.
The mechanisms that define the adhesion between different materials are still not completely understood. Different theories are under investigation (adsorption theory, mechanical interlocking theory, weak boundary layer theory, etc.[4]), while a combined effect of different theories seems to be the most plausible explanation for the adhesion phenomenon.

The geometrical design of a structural adhesive joint is an important parameter for its efficiency to transmit loads. In general, adhesive joints are best loaded in shear, whereas peeling forces should be avoided. A selection of possible joint geometries with an indication of their structural efficiency is illustrated in Figure 5-2.

The products used for joining two materials can be classified as fillers, sealants or adhesives depending on their function[5]. Fillers are used to fill crevices and seams. Sealants also fill up a joint but also play a different role: exclude moisture to penetrate through the joint, contain a liquid or gas inside the joint, contribute to a fire barrier, insulate electrical or thermal currents, etc. Linseed oil putty is used as a sealant. Adhesives on the other hand are applied for structural applications. Adhesives can also have the same function as a sealant and some sealants can resist some forces.

2. Experimental research objectives

Taking into account the common joints mentioned in Figure 5-2, the structural behaviour of a typical connection of a glass plate on a T-section (Figure 5-1 left) is similar to a butt strap joint: a combination of a single lap-shear (working in shear) and a butt joint (working in tension or compression). When the glass is loaded in-plane,

[3] Adams, Comyn, and Wake 1997.
[4] Petrie 2007, p.39–58.
[5] Petrie 2007.

the single-lap shear part is located at the flange of the T-section and the butt joint at the web.
When a tension force is applied parallel to the glass surface, the butt joint is loaded in tension. Adams[6] shows that in that case the lap shear part will resist the majority of the forces (Figure 5-2 a and h). Thus, to gather information about the structural behaviour of the connection, a single-lap shear test can be performed on the adhesive.
When a compressive force is applied parallel to the glass plate, the contribution of the butt joint could be important if the ratio between the compressive strength and the shear strength of the sealant is high. The latter is the case for traditional linseed oil putty. Determining the compressive strength of the adhesive via a compression test is then appropriate.

The properties of modern adhesives, provided in technical data sheets or research projects, are often determined in lab conditions. However, in case of historic structures, it will often be necessary to make the adhesive joints on-site. The ideal execution parameters are thus not always easy to implement. The question rises whether the properties from the lab experiments can also be used in the context of on-site bonding in historic structures.
Some specific research projects already took into account some parameters specific for on-site execution, such as a thick adhesive layer[7] and a contamination of the bonded surface[8]. A recent project at Ghent University investigated a broad range of modern adhesives, among other things on some execution parameters (contamination of surface, temperature, humidity and UV light environmental conditions)[9].

The first part of the experimental research consisted of the determination of the mechanical properties of traditional linseed oil putty. Both lap shear and compression tests were performed on linseed oil based products.
The second part of the experimental program focused on parameters important during the execution of an adhesive joint. The influence of three surface conditions (Figure 5-3 and Table 5-1) on the adherence behaviour was investigated on a lap shear joint with a modern polymer adhesive:
- a non-ideal surface due to on-site removal of corrosion and paint layers (with roughness deviations);
- the presence of coatings and paints necessary for corrosion protection;
- replacing modern steel by 19th-century mild steel.

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6 Adams, Comyn, and Wake 1997.
7 Blandini 2005; Callewaert et al. 2011.
8 Bos et al. 2010.
9 Belis, Callewaert, and Van Hulle 2011.

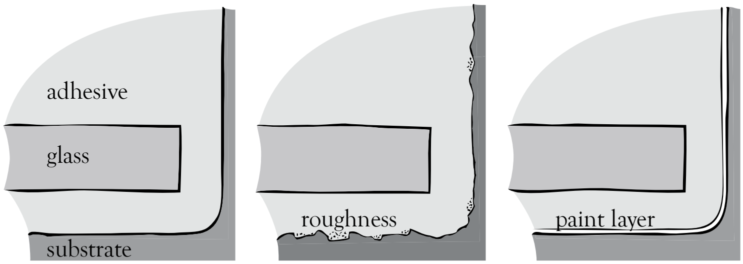

Figure 5-3: Deviations from ideal adhesive bond circumstances under investigation

Roughness is considered to be a parameter that might improve the joint strength of bonding metal surfaces (opposite to some polymeric surfaces) due to the mechanical interlocking principle (one of the adhesion theories)[10]. Many researchers thus focused on the influence of the substrate roughness on adhesion and adhesives (for an overview see Chaudhury[11], for a theoretical approach see Petrie[12]). As the researchers used different substrates, adhesives and joint geometries, one cannot conclude whether the joint strength increases with increasing adherend surface roughness obtained by mechanical treatment. Moreover, literature demonstrates that it is difficult to isolate the influence of one specific parameter on the joint strength, as the surface chemistry of substrates, the surface tension of substrates and the surface treatment of substrates are interacting. Furthermore, the majority of the published research focused on lower values of roughness. Therefore it is important to perform single-lap shear tests on actual materials and apply surface preparation methods used in practice when renovating 19th-century iron or steel structures.

3. Substrate materials

When a 19th-century iron or steel structure is renovated, the metal is (nearly always) grit blasted to remove the paint layer and, if present (depending on the maintenance carried out), also the oxide layers. Soon after being grit blasted, iron oxides start to form at the interface (sources differ on the exact time from 4 hours[13] to 10 hours[14]). Consequently, it is important to protect the iron as quickly as possible with a paint layer (for common renovations) or to apply the adhesive bond (in case the iron is to be adhesively bonded).

[10] Petrie 2007, p.65–66.
[11] Pocius 2002, p.317–349.
[12] Petrie 2007, p.65–66.
[13] Rogers 1966.
[14] Allam, Arlow, and Saricimen 1991.

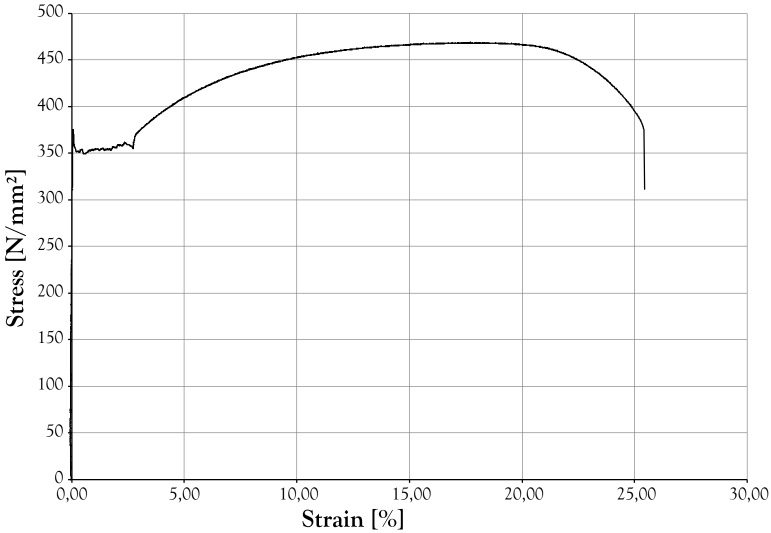

Figure 5-4: Stress-strain curve of the mild steel substrates from the Janlet wing of the Museum of Natural Sciences in Brussels[15]

3.1. Metal substrates

The substrates of the specimens were manufactured out of three sorts of metal (Table 5-1). Three lap shear sample sets were made of modern construction steel, namely S235 (with a yield stress of 235 MPa and a Young's modulus of 210 GPa). Two other lap shear sample sets were made of mild (ingot) steel (with a yield stress of 353 MPa and a Young's modulus of 199 GPa), cut out a recycled I-section dating from 1905 (stress-strain curve in Figure 5-4)[16].

The substrates for the putty compression samples were manufactured out of aluminium (AlMgSi0,5).

3.2. Paint layer on 19th-century mild steel

The paint layer was a two-component epoxy zinc phosphate primer E81 of Aalterpaint NV[17]. The paint has a very good adherence to steel according to tests carried out by Aalterpaint NV. The hardener is polyamide, which improves the wetting[18] and so the

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[15] De Bouw 2010.
[16] Ibid.
[17] Aalterpaint N.V. 2007.
[18] Wetting of a surface is the combined action of the spreading of the adhesive over the surface and the penetration of the adhesive into the surface cavities. Good wetting of the surface ensures a close molecular contact between the adhesive and the substrate, improving the adhesion forces. For more information, see Petrie 2007, p.62-66.

adherence to the historic mild steel. The zinc phosphate makes the paint applicable on surfaces prepared by grit blasting as well as by scraping and brushing with mechanical or hand tools. Epoxy paint layers are widely used in renovation projects. Most of the time, a polyurethane top coat is applied after all other treatments to ensure an excellent durability. In normal conditions, the selected paint layer is dry for overcoating or adhesive bonding after 12 hours to 7 days.

Table 5-1: Matrix of sample series

	number of samples	adhesive			surface roughness			metal substrate			paint	
		MS polymer adhesive	MS polymer sealant	linseed oil putty sealant	low	intermediate	high	modern steel	19th century mild steel	aluminium	without paint	with paint
MST – unpainted (1)	6	x					x		x		x	
MST – painted (1)	6	x					x		x			x
S235 – Sa3 (2)	10	x					x	x			x	
S235 – Sa1 (2)	10	x				x		x			x	
S235 – Sa0 (2)	10	x			x			x			x	
S235 – polymer sealant	9		x					x			x	
S235 – putty sealant	9			x				x			x	
ALU – putty sealant 1	5			x						x	x	
ALU – putty sealant 3	5			x						x	x	

(1) in the nomenclature of the sample series, S235 simulated the modern steel samples and MST stands for "mild steel"

(2) the Sa classification describes the preparation grades of steel surfaces defined by the European standard ISO 8501-1:2007 (see paragraph 4.1)

1. Sealant and adhesive materials

1.1. Traditional sealants

Originally, glass claddings were sealed to the iron glazing bars with linseed oil putty. The traditional putty consisted of linseed oil with chalk or ceruse (Table 2-2 p.62). Both the drying oil and the filler pigment were decisive for the quality of the putty: equilibrium had to be found between flexibility for handling during construction and hardening on the longer term[19]. Glaziers often made their own linseed oil putty on-site or in their workshop, adjusting the composition based on experience. Glaziers today still use these traditional recipes to make their own putty for restoring historic glazing (both plane and stained glass).

Products based on the traditional recipes are also made by adhesive manufacturers. In this experimental research, Soudal putty (*Stopverf* or *Mastic vitrier*) was tested. This glazier's putty is made of synthetic resin modified linseed oil. It can be painted. The technical data sheet states a maximum allowable strain of 5%. It will be referred to as "putty sealant" in the proceeding text.

According to literature, traditional putty has to be repainted regularly after which it has a limited life expectancy of three to five years[20]. Therefore adhesive manufacturers search for replacement products with increased life span. Modern polymers are developed to obtain a product with a similar workability, viscosity, etc. as linseed oil putty. The main goal is to create a product that stays elastic to ensure the water-tightness of a joint over a longer period of time.

Soudaseal Tradition is a MS polymer that was developed as a replacement product for traditional linseed oil putty. The maximum allowable strain is around 20%, illustrating the higher elasticity than the Soudal putty. This product will be referred to as "polymer sealant" in the proceeding text.

1.2. Polymeric adhesive

A modern adhesive, used in the context of a renovation, should meet the following criteria:

1) to have a low stiffness;
   A numerical simulation of a small-scale prototype[21] indicated that a small stiffness already had a significant influence on the structural performance of a set of glass plates connected to load-bearing steel sections. This was confirmed by the detailed

[19] Bieneman 1967.
[20] Scheffler and Connolly 1996, p.87.
[21] carried out by the author in SCIA Engineering software

calculations later in this research (Chapter 6). Moreover, according to Vrenken[22], an adhesive with a low E-modulus also avoids stress concentrations on the glass plates and is therefore less dependent on the surface conditions of the substrates.

2) to be paintable;

   The primer paint layer (if necessary supplemented with an intermediate layer) provides the corrosion protection of the iron structure. Adhesive bonding is done after these first coating layers. Afterwards, the whole construction (the adhesive as well as the historic iron sections) receives a top coat to improve the durability of the whole painting system. This means that the adhesive has to be paintable, which excludes most silicones from the selection. The paint layer has to able to follow the flexibility of the adhesive.

3) to have a high filling capacity.

   At historic iron and steel constructions, the dimensional tolerances are higher than with modern structures and thus the adhesive layers have to be thicker. For example in 19th-century glasshouses, small glass plates are placed on top of each other to follow the curvature of the iron glazing bars, so that the adhesive layer on one side of the glass plate is very thick (Figure 5-5). A second example is that 19th- or 20th-century iron and steel sections are sometimes deteriorated causing a loss of section and a rough surface. Therefore, a thicker and non-uniform adhesive layer will be generated. A good filling capacity of the adhesive is thus desirable.

Other parameters that were taken into account are a good weather resistance, the capability to follow different thermal dilatations of iron and glass, the compatibility with epoxy resin paints and a glass transition temperature as high as possible (because of the rising temperatures in glass roofs).

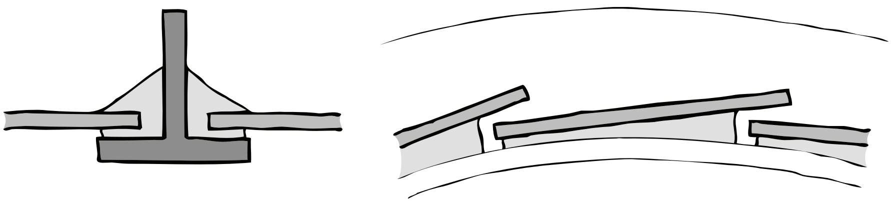

Figure 5-5: Filling capacity of sealant or adhesive: (left) typical 19th-century connection detail; (right) longitudinal connection of the same connection principle showing the difference in adhesive thickness alongside the length of the glass plate

[22] Vrenken 2006.

Property	Silyl terminated polyether MS sealants	Polyurethane sealants	Silicone rubber sealants
Environmental friendliness	10	5	9
Quick cure	10	7	10
Weather resistance	8	6	10
Adhesion to various substrates	10	5	8
Mechanical properties	10	10	10
Paintability with water based paint	10	10	3

Rating: 10 = high; 1 = low.

Figure 5-6: Comparison of selection of performance criteria of construction sealants[23]

Figure 5-6 gives a comparison between three modern polymer sealant types that might fulfil these criteria.

For the selection of the polymeric adhesive, the following adhesive types were excluded: acrylic adhesives have to be applied in thin adhesive bonds, epoxies have a high stiffness and are often not capable of following different thermal dilatations, polyurethanes often have a bad UV resistance, and silicones are not paintable and are often not compatible with PVB interlayers[24]. An MS polymer adhesive (Modified Silane polymer) was selected because of its low E-modulus with relatively good structural performance, good paintability and its good weather resistance and durability (Figure 5-6). MS polymers are also known for their lower sensitivity to surface conditions. The adhesive *HQ Bond 4.5* from HQ Bonding B.V. [25] is an adhesive fulfilling all these criteria and was used for the experiments in this research. This recently developed adhesive was tested in 2010 in shear, tension and creep by TNO[26]. It will be referred to as "polymer adhesive" in the proceeding text.

[23] Hashimoto 1998. Cited from Petrie 2007, p.574.
[24] Belis, Callewaert, and Van Hulle 2011, p.7–15.
[25] HQ Bonding, Technical data sheet.
[26] Botter and Van Straalen 2010a; Botter and van Straalen 2010b; Botter and van Straalen 2010c.

2. Specimen manufacture

2.1. Putty sealant compression test samples

The compression samples were made out of aluminium[27]. The individual moulds were built up of a PVC holder and an aluminium spacer block (Figure 5-8). The samples were constructed as a representation of a traditional 19th-century iron and glass connection: a thin plate (representing the glass pane) is attached to an L-shaped section (Figure 5-7 and Table 5-2). Four samples were tested after one month curing while five samples were given three months of curing time. The excess of putty of all samples was cut off after one month curing.

Table 5-2: Geometry of putty sealant compression specimens

	number of samples	average width b [mm]	average alu thickness l [mm]	average thickness d_a [mm]
ALU – putty sealant 1	5	25.12	4.03	2.09
ALU – putty sealant 3	5	24.97	3.91	2.29

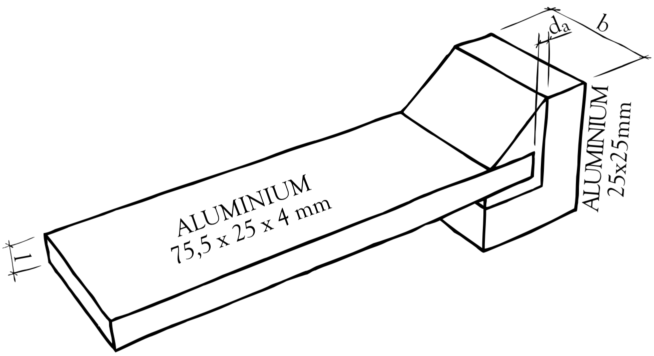

Figure 5-7: Specimen geometry of the putty sealant compression samples in aluminium

[27] The material of the substrates is not critical for these experiments. Firstly, the expected stiffness of the putty sealant is much lower than the stiffness of aluminium, thus the geometry of the samples will be invariable. Secondly, when acting in compression, the influence of the adhesion to the substrate surface is negligible, therefor the compression strength is independent from the substrate material.

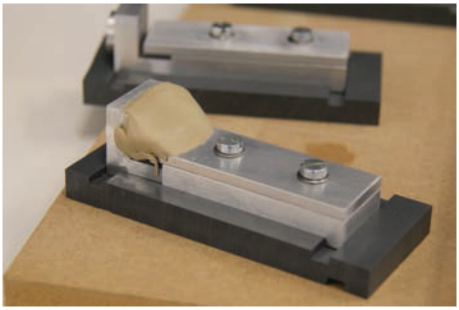

Figure 5-8: Mould for the manufacture of the putty compression samples

2.2. Traditional sealants lap shear test samples

Two modern construction steel sample series were manufactured for the lap shear tests of the putty sealant and the polymer sealant. The whole surface was sandblasted to remove all corrosion and dirt. The samples were cleaned with acetone both before and after the sand blasting. Subsequently, the samples were bonded with the putty sealant or the polymer sealant (Figure 5-9 left and middle and Table 5-3). The excess of sealant was cut off the sides of the test samples one day before testing.
Both series of lap shear samples cured one month before being tested.

Table 5-3: Geometry of traditional sealants lap shear specimens

	number of samples	average width b [mm]	average overlap l_a [mm]	average thickness d_a [mm]
S235 – polymer sealant	9	24.87	11.15	2
S235 – putty sealant	1 (1)	25.44	11 (2)	2

(1) the dimensions are given for the one sample that was included in the results
(2) due to the fragility of the putty sealant samples, the overlap could only be measured with 1mm accuracy

Figure 5-9: Sample series: (left) polymer sealant lap shear samples;
(middle) putty sealant lap shear samples; (right) putty sealant compression samples

2.3. Polymer adhesive lap shear test samples

The aimed geometry of the lap shear samples was a lap shear joint of two plates of 5 mm thick, 80 mm long and 25 mm wide (Figure 5-10 left and middle). Nevertheless, the starting plates were 270 mm wide (and 175 mm wide for the mild steel). Subsequently the modern steel samples were grit blasted to give the surfaces three different roughness values. The 19th-century mild steel strips were grit blasted to obtain a high roughness and one pair of strips was painted. Then all sample pairs were adhesively bonded with the polymer adhesive. Before the grit blasting, before the painting and again before applying the adhesive, the samples were cleaned with a clean cloth dipped in isopropylalcohol. After the adhesive cured for at least 28 days, the samples were cut in individual test specimens with a width of 25 mm (Figure 5-10 b) by water jet cutting. Some of the dimensional tolerances of the water jet cut samples are illustrated in Figure 5-11. The resulting sample sets are depicted in Figure 5-12.

Due to the rarity of the 19th-century mild steel sections, there are only 6 samples of these series per tested parameter (with and without paint) (Table 5-4).

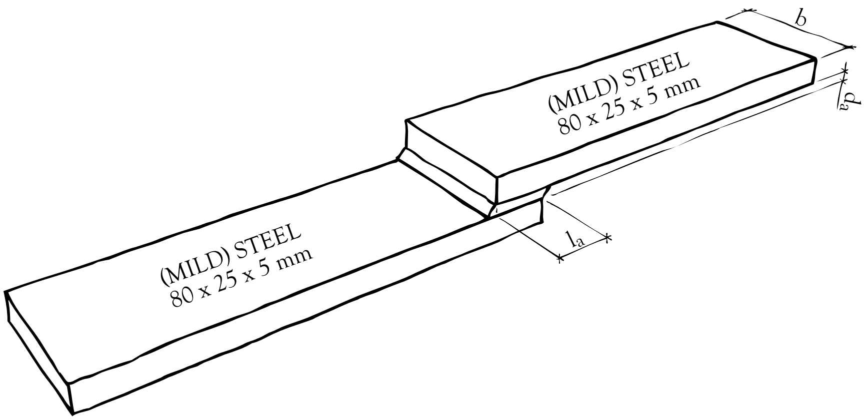

Figure 5-10: Specimen geometry of the lap shear samples in steel or mild steel

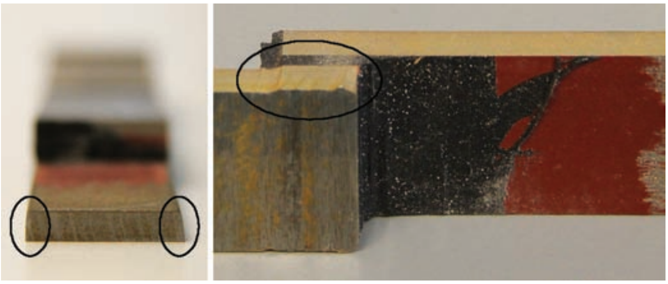

Figure 5-11: Geometrical imperfections due to water jet cutting: (left) tapered cross-section; (right) deformed cut line

Table 5-4: Geometry of polymer adhesive samples

	number of samples	average width b [mm]	average overlap l_a [mm]	average thickness d_a [mm]
S235 – Sa0	5	24.97	7.97	1.765
S235 – Sa1	10	24.59	8.87	1.766
S235 – Sa3	10	24.64	7.92	1.619
MST – unpainted	6	24.88	8.70	1.745
MST – painted	6	24.71	8.78	1.757

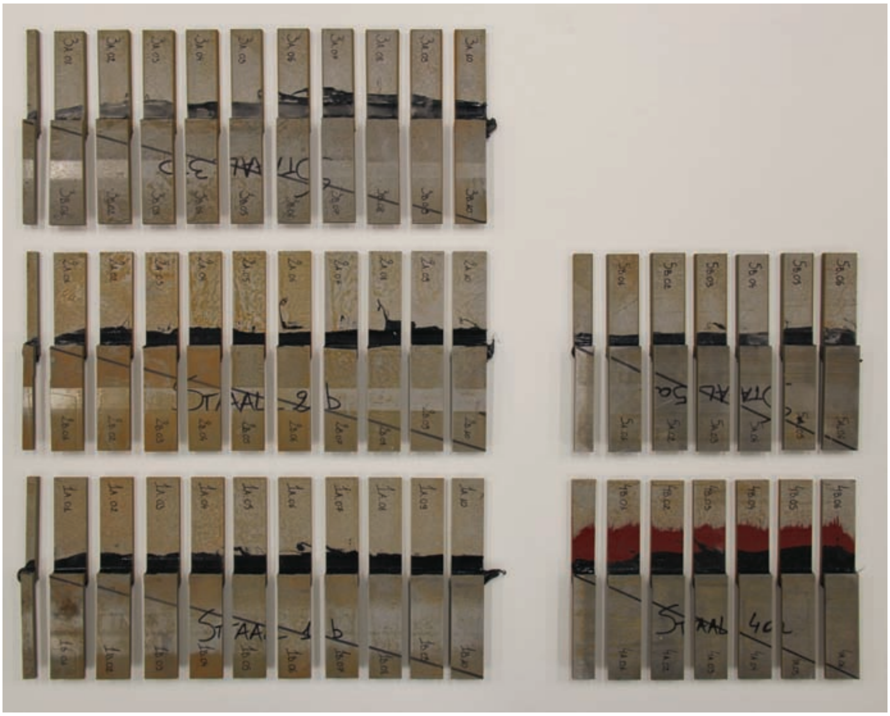

Figure 5-12: Polymer adhesive sample series:
(left) from top to bottom: S235 – Sa1, S235 – Sa0 and S235 – Sa3;
(right) from top to bottom: MST – unpainted and MST – painted

Table 5-5: Roughness measurements on polymer adhesive samples

	average roughness R_a [μm]	standard deviation on R_a [μm]
S235 – Sa0	3.91	0.40
S235 – Sa1	8.45	1.62
S235 – Sa3	12.20	0.74
MST – unpainted	10.28	0.57
MST – painted	7.26	2.79

2.4. Surface roughness

The optimal roughness for the adhesion of the paint layer to the mild steel was taken as the highest roughness of the specimens: a surface roughness Ra of about 10 μm[28], defined as the arithmetic average of the absolute values of the profile height deviations recorded within the evaluation length and measured from the mean line. To be able to investigate the influence of the roughness, two other roughness values were picked for the experiments: a very low (for ground and grit blasted iron) and an intermediate roughness value of 1 μm and 5 μm respectively.

The surface preparation of the substrates was carried out at a grit blasting workshop. The aimed roughness values were communicated to the skilled worker by preparation grades of steel surfaces defined by the European standard ISO 8501-1:2007[29]. Sa3 was used for a roughness value Ra of circa 10 μm (S235 – Sa3) and Sa1 for Ra of circa 5 μm (S235 – Sa1). Both preparation grades were obtained by blasting with a steel grit. The lowest roughness was obtained by blasting with sand grit and was named by a preparation grade "Sa0" (S235 – Sa0).

Subsequently, the adhesive bonding of the modern steel substrates was carried out 4.5 hours after the grit blasting.

Finally, the paint layer was put on the substrates with a brush, as thin as possible. It was applied to one sample set of the 19th-century mild steel (MST - painted), 4 hours after the grit blasting was carried out. The paint cured for 72 hours after which the two mild steel sample series were adhesively bonded. Thus, the unpainted 19th-century mild steel (MST - unpainted) was adhesively bonded 76 hours after being grit blasted.

The average paint layer thickness, measured with a coating thickness gauge after curing of the adhesive, was 43.8 μm with a standard deviation of 4.8 μm.

The surface roughness measurements were done after the adhesive had cured for circa 1 month. The measurements were performed by a contact stylus profiler "Dektak 8" of Veeco. The resulting averages and standard deviations of Ra are illustrated in Figure 5-13 and summarized in Table 5-5. The sample series grit blasted to Sa3 (S235 – Sa3, MST – unpainted and MST –painted) have a roughness value Ra between 10 and 12.5 μm, corresponding to the aimed roughness value. The roughness of the painted mild steel samples was a second time measured on top of the paint layer, resulting in a lower roughness value of 7.26 μm. The standard deviation on this roughness is very high, despite the increased number of measurements.

[28] Berendsen 1989.

[29] International Organization for Standardization. 2007.

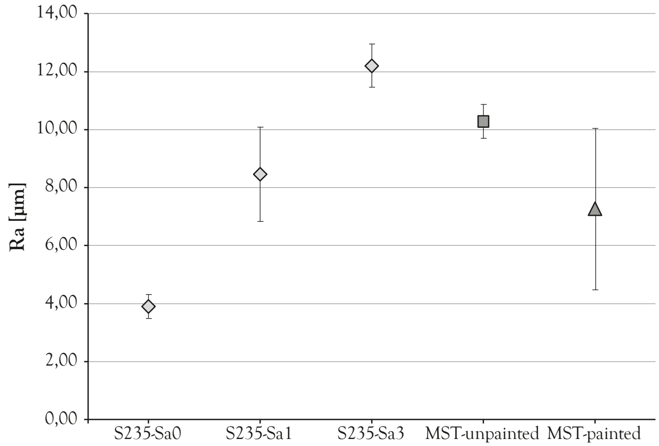

Figure 5-13: Roughness measurements on MS polymer adhesive sample sets

3. Testing equipment

3.1. Lap shear tests

The single-lap shear tests on the traditional sealants (putty sealant and polymer sealant) were performed with a Zwick 10 kN universal electromechanical test machine at the Soudal research lab. The grips were designed to be adjustable so that the loading is put centric on the adhesive bond. The tests were carried out at room temperature. The displacements of the grips were recorded and supposed equal to the displacements of the adhesive bond (which is accurate for flexible adhesives and sealants). The shear strain was calculated out of these displacements. Nine samples of the putty sealant lap shear joints were manufactured. Seven samples failed while positioning them in the testing machine due to their fragility (the putty was still very soft). Two samples were tested. Only one sample resisted force during deformation. This high loss of samples is to be taken into account in the statistical interpretation of the results of the experiments.

The single-lap shear tests on the polymer adhesive samples were performed with a Zwick 250 kN universal electromechanical test machine at the Adhesion Institute of TU Delft in collaboration with UGent (Figure 5-14), also with adjustable grips. All tests were performed at a temperature of 22.2 °C and at 50% relative humidity. The displacements from these experiments were not recorded. From sample series S235 – Sa0, the results of 5 out of 10 samples are excluded because of asymmetric loading due to slipping of the grips. This high loss of samples is to be taken into account in the statistical interpretation of the results of the experiments.

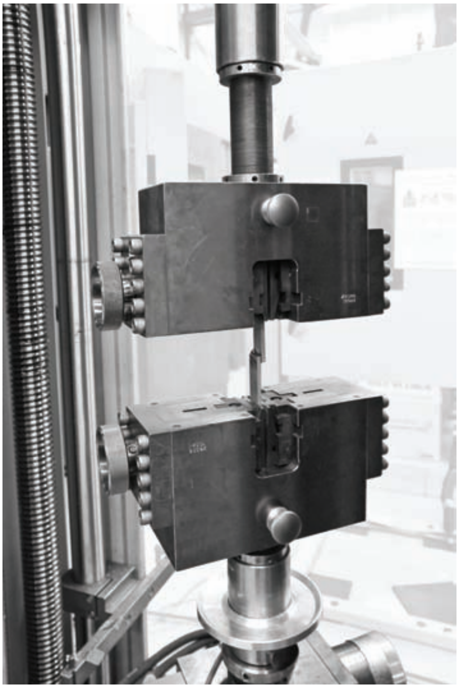

Figure 5-14: Single-lap shear tests at TU Delft with Zwick 250 kN universal electromechanical test machine

3.2. Compressive tests

The compressive tests on the putty sealant samples were performed with an Instron 100 kN universal electromechanical test machine at the VUB (Figure 5-15). The tests were carried out at room temperature. The strains of the experiments were recorded as the displacements of the grips (which is accurate for flexible adhesives and sealants). The samples were placed on a hinge to reduce the eccentricity in the test samples (Figure 5-16). One sample of the series that cured for one month (ALU – putty sealant 1), failed while positioning it in the hinge.

Figure 5-15: Compression tests at VUB with Instron 100 kN test machine

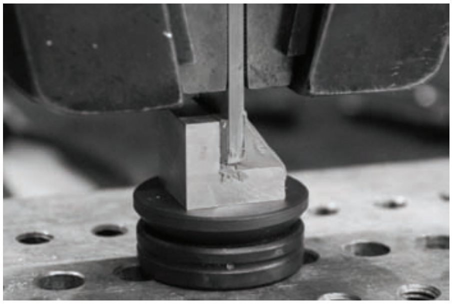

Figure 5-16: Avoiding eccentric forces by using a spherical cap hinge during compression tests

4. Results

4.1. The putty sealant lap shear and compression tests

The results of the putty sealant tests are summarized in Table 5-6 and Table 5-7.

The sole tested putty lap shear sample did not break but underwent such a major elongation that the test was interrupted. The shear strength is the maximum stress reached during testing. The bond at the surfaces was tested manually afterwards and was very weak (Figure 5-19 middle).

The putty sealant compression tests were interrupted at 100N (for the samples after one month curing) or 300N (for the samples after three months curing), assuming contact between the aluminium surfaces above these values. The compression samples were opened up after the tests, showing a diversity of curing stages with often an alternation of soft material with harder grains (Figure 5-19 top).

Table 5-6: Shear strength of single-lap putty sealant sample series (displacement rate 1 mm/min)

	shear strength [N/mm²]	G-modulus [N/mm²]
S235 – putty sealant	0.0028	0.0097

Table 5-7: Compressive strength of putty sealant sample series (displacement rate 1 mm/min)

	compressive strength [N/mm²]	standard deviation on strength [N/mm²]	E-modulus [N/mm²]	standard deviation on E-modulus [N/mm²]
ALU – putty sealant 1	0.11	0.05	1.20	1.11
ALU – putty sealant 3	1.72	0.66	20.65	9.81

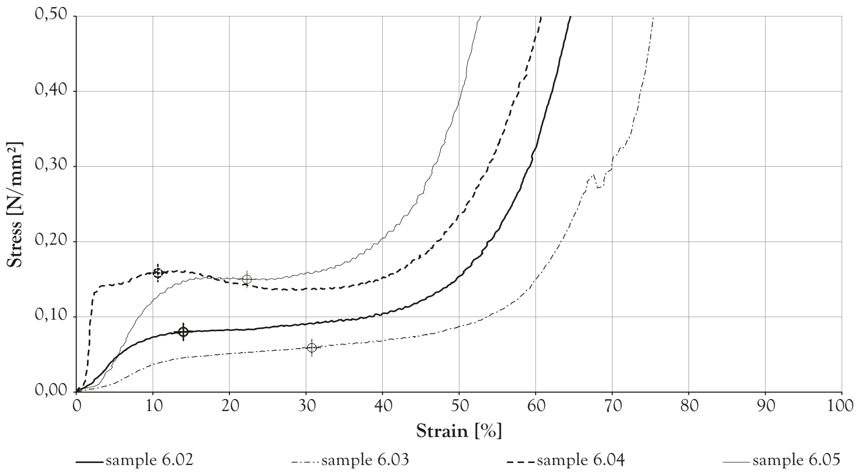

Figure 5-17: Stress-strain curves of putty sealant compression tests after 1 month curing

For the putty compression tests, the E-modulus was taken as the stress value at 5% strain (the maximum allowable strain defined in the technical data sheet).

The compressive strength of the putty compression samples was determined by constructing the linear behaviour (the E-modulus) at 5% strain, so that the intersection with the experimental curve is the maximum allowable compressive strength (indicated points at Figure 5-17 and Figure 5-18).

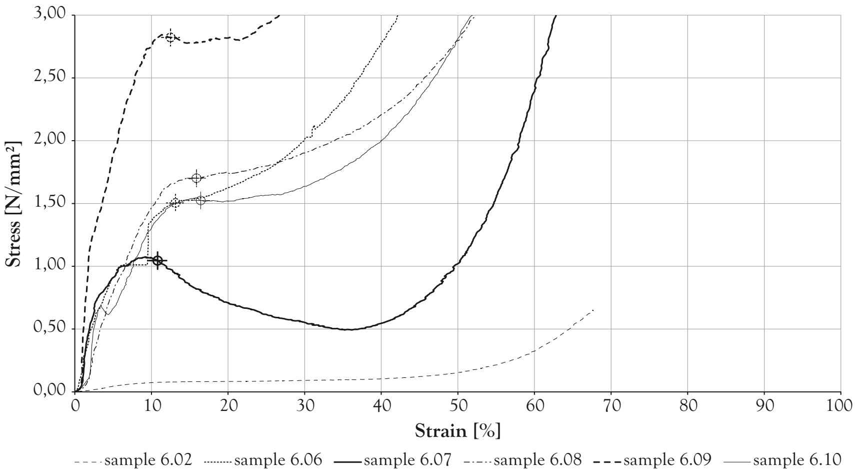

Figure 5-18: Stress-strain curves of putty sealant compression tests after 3 months curing

Table 5-8: Shear strength of single-lap polymer sealant sample series (displacement rate 5 mm/min)

	shear strength [N/mm²]	standard deviation on strength [N/mm²]	G-modulus [N/mm²]	standard deviation on G-modulus [N/mm²]
S235 – polymer sealant	0.77	0.05	1.24	0.24

4.2. The polymer sealant lap shear tests

The results of the polymer sealant tests are summarized in Table 5-8. The mean shear strength of all sample series is an arithmetic mean of all samples in the series of the maximum stress reached during testing. The polymer sealant lap shear samples all failed cohesively (Figure 5-19 bottom).

For silicone sealant connections, linear behaviour of single-lap shear tests is considered to last at least up to 12.5% of the strain at failure[30]. The same limit was used for the polymer sealant lap shear samples: 240% ultimate strain means linear behaviour up to 30% strain (Figure 5-20). The G-modulus was thus calculated at 30% strain.

[30] Haldimann, Luible, and Overend 2008, p.157–158.

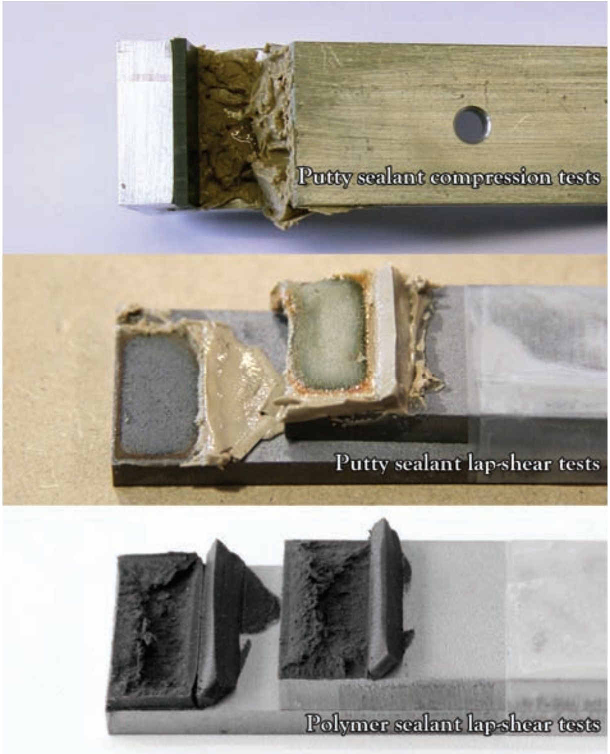

Figure 5-19: Failure surfaces: (top) putty sealant compression tests;
(middle) adhesive failure of putty sealant lap shear test;
(bottom) cohesive failure of polymer sealant lap shear tests

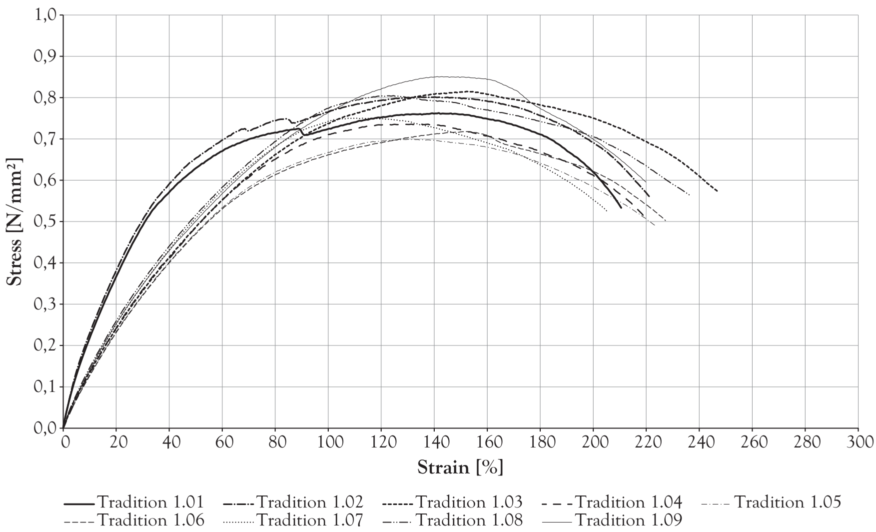

Figure 5-20: Stress-strain curve of polymer sealant single-lap shear tests

4.3. Polymer adhesive lap shear tests

The results of the single-lap shear tests on the polymer adhesive are summarized in Table 5-9. The mean shear strength of all sample series is an arithmetic mean of all samples in the series of the maximum stress reached during testing.

All the samples failed adhesively on both surfaces with a cohesive rupture in between (Figure 5-21). With the naked eye some (black) spots of left adhesive can be observed (most notable at the S235 – Sa1 and S235 – Sa3 sample series), suggesting the adhesive was not pulled out of the valleys of the roughness and waviness profile. Subsequently, SEM (scanning electron microscope) analysis of sample series S235 – Sa3 and MST – unpainted revealed that a very thin layer of adhesive was left at the entire failure surface (compare substrate and failure surface structures in Figure 5-22). The samples thus failed adhesively when visually assessed, however some adhesive is left on the substrate surfaces.

Table 5-9: Shear strength of single-lap polymer adhesive sample series (displacement rate 5 mm/min)

	shear strength [N/mm²]	standard deviation on shear strength [N/mm²]
S235 – Sa0	2.75	0.10
S235 – Sa1	2.82	0.23
S235 – Sa3	2.91	0.17
MST – unpainted	2.46	0.13
MST – painted	2.76	0.31

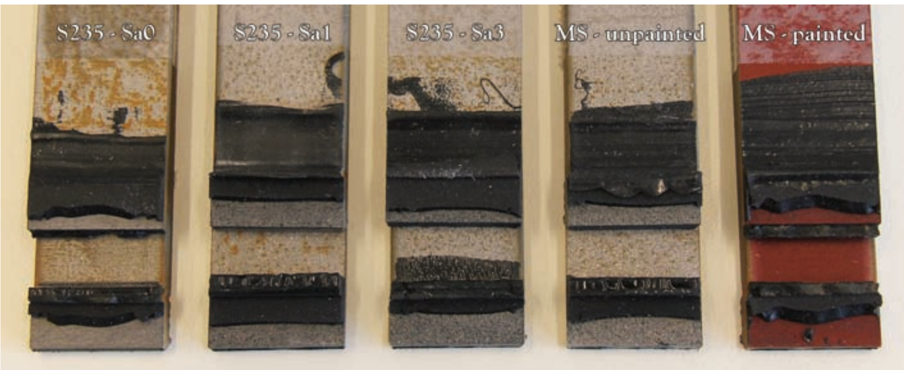

Figure 5-21: Adhesive failure of MS polymer adhesive sample series

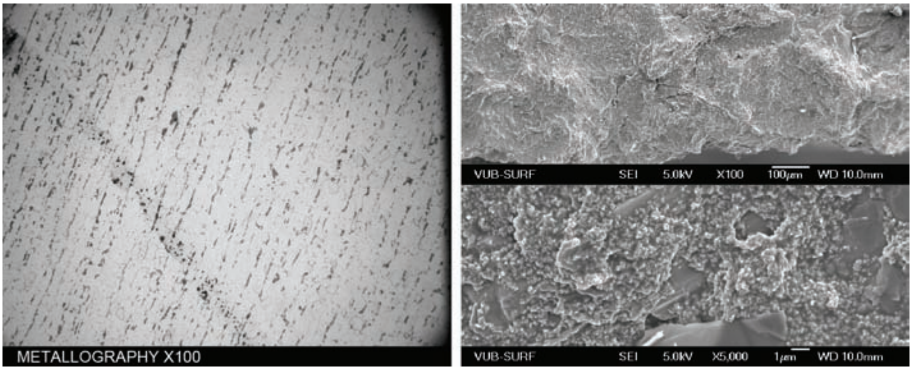

Figure 5-22: (left) Linear structure from metallography of mild steel substrates with magnification x100[31]; (right) more random structure from SEM analysis of failure surface of S235 – Sa3 sample with magnification x100 on top and magnification x5000 on bottom

5. Discussion of the results

5.1. Influence of adhesive material

Three different adhesive and sealant materials were tested in these series. The polymer adhesive (HQ Bond XS4,5) had the highest shear strength of 2,91 N/mm^2 (S235 – Sa3). The polymer sealant (Soudaseal Tradition) reached a mean shear strength of 0,77 N/mm^2, whereas the shear strength of the putty sealant (Soudal Putty) was negligible (Table 5-10).
A main difference between the lap shear tests of these materials was the failure mechanism. Cohesive failure was only present at the polymer sealant samples. All other samples are thus possibly subject to improvement when the adhesion to the substrates could be improved.
The elongation at the maximum stress of the lap shear samples was also different for all materials. The flexibility of the polymer sealant was significantly higher (136% elongation) than of the putty sealant samples (31% elongation) and significantly lower than the polymer adhesive samples (360% elongation) (Table 5-10). The polymer sealant was explicitly developed to improve the flexibility compared to the traditional linseed oil putties.

The E-modulus of the adhesives and sealants can also be compared. The E-moduli from the polymer sealant and polymer adhesive were taken from the technical data sheets of the adhesive manufacturer (at 100% strain)[32]. The putty sealant reaches a comparable stiffness after one month curing, but only under compressive forces. The

[31] De Bouw 2010, p.95–97.
[32] Soudal 2012; HQ Bonding.

stiffness increases significantly after three months curing time. The elongation at the maximum stress also decreases, which confirms the hypotheses that the putty becomes more brittle after a longer curing time.

Finally, it is clear that the standard deviation on the putty sealant sample series is proportionally much higher than the sample series of both the polymer adhesive and sealant (Table 5-10). When high accuracy is needed for the design of structural adhesive joints, this could be taken into account when selecting the appropriate bonding material.

Table 5-10: Comparison of adhesive and sealant materials

	shear strength [N/mm²]	standard deviation on strength [N/mm²]	elongation at max. stress [%]	standard deviation on elongation [%]
S235 – Sa3	2.91 (1)	0.17	360 (3)	
S235 – polymer sealant	0.77 (1)	0.05	135.57 (1)	13.70
S235 – putty sealant	0.0028 (1)		31.46 (1)	
ALU – putty sealant 1	0.11 (1)	0.05	19.47 (1)	9.01
ALU – putty sealant 3	1.72 (1)	0.66	13.78 (1)	2.37

sources:(1) own experimental research
(2) technical data sheets
(3) TNO reports

Table 5-11: Comparison of adhesive and sealant materials

	E-modulus 100% [N/mm²]	standard deviation on E-modulus [N/mm²]
S235 – Sa3	1,0 (2)	
S235 – polymer sealant	1,35 (2)	
S235 – putty sealant		
ALU – putty sealant 1	1,20 (1)	1.11
ALU – putty sealant 3	20,65 (1)	9.81

sources:(1) own experimental research
(2) technical data sheets

5.2. Influence of surface roughness for polymer adhesive

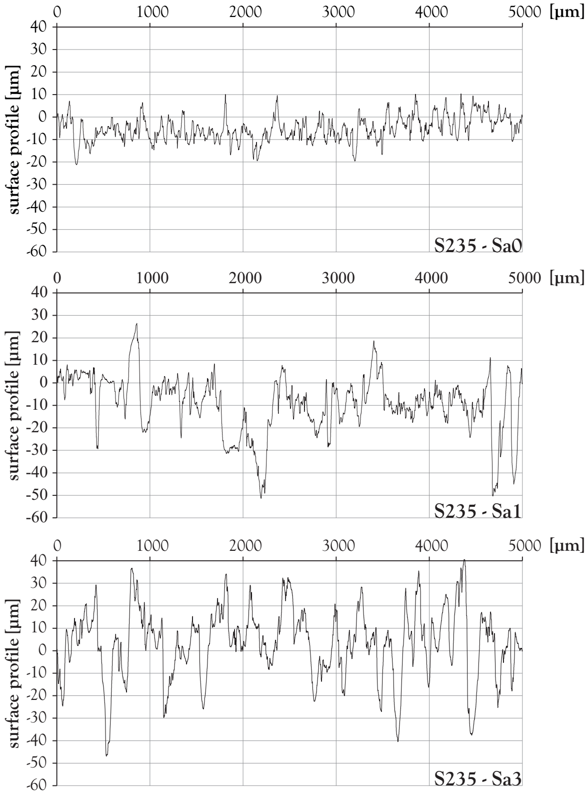

Figure 5-23: Surface profiles of series S235 – Sa0 (top), series S235 – Sa1 (middle) and series S235 – Sa3 (bottom)

Three sample series of modern construction steel S235 were grit blasted to reach different roughness values Ra. The different surface profiles measured with the stylus are depicted in Figure 5-23. The average shear strength (and the standard deviation) of the S235 samples in relation to the roughness Ra is illustrated in Figure 5-24.

The average shear strength increased with increasing roughness. A statistical analysis was carried out to investigate this influence to be relevant or not (regarding the limited number of test samples). A 95% confidence interval on the difference in means was calculated[33]. This analysis showed that the difference in average shear strength between S235 – Sa0 and S235 – Sa3 (lowest and highest roughness) is relevant. The difference of these two series with S235 – Sa1 (intermediate roughness) is not relevant.

[33] Montgomery 2001, p.235.

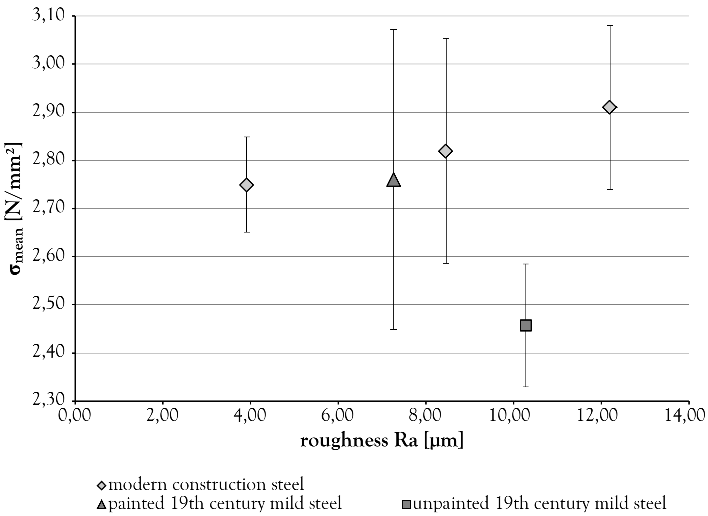

Figure 5-24: Roughness influence graph on modern construction steel and 19th century unpainted mild steel

However, the effect of increasing shear strength with increasing substrate roughness was very small. An increase of the roughness Ra of 300% (from 3.91 ±0.40 µm to 12.20 ±0.74 µm) increased the average lap shear strength with only 5% (from 2.75 ±0.10 N/mm² to 2.91 ±0.17 N/mm²). This result is positive for renovation works, as the accuracy of the obtained roughness is rather poor when grit blasting is applied on-site in difficult execution circumstances.

5.3. Influence of substrate material

The shear strengths for the S235 samples and the unpainted mild steel samples in relation to the roughness are illustrated in Figure 5-24. The average shear strength of the unpainted mild steel was significantly lower than the average shear strength from the S235 sample series.

The unpainted mild steel plates were only adhesively bonded 76 hours after being grit blasted, thus giving enough time for the oxides to develop and form a weak boundary layer. Rogers states that the maximum allowable time between the surface preparation and bonding of the metal substrates for sandblasted steel is 4 hours[34]. This time period is only just exceeded for the adhesive bonding of the modern construction steel (4.5 hours), in accordance to the ideal conditions described by adhesive manufacturers, but the unpainted mild steel has gone well over this period of time. This can be an explanation of the low average shear strength of the unpainted mild

[34] Rogers 1966.

steel samples. The SEM analysis of the failure surfaces could not confirm this, because of the very thin layer of adhesive left on the surface. Further analysis is needed to be able to confirm this theory. However, repainting after a time period of four hours is not realistic in a renovation project.

5.4. Influence of paint layer

The painted mild steel samples (MST – painted) were reaching nearly the same average shear strength as the modern construction steel samples (Table 5-9). The two-component epoxy paint layer did not form a weak boundary layer for the adhesion of the MS polymer adhesive. At the same time, the standard deviation of the shear strength measurements of the painted samples is higher than the modern construction steel and the unpainted mild steel samples. The painted samples also failed adhesively on the surface between paint layer and adhesive, so the paint layer thickness had no influence on the shear strength.
Improvements in paint layers can play an important role in increasing the shear strength of the connection. The research on corrosion protection primers and paints, developed to increase the adhesion for structural adhesive bonding on steel (e.g. 3M Scotch-Weld EC-1945 B/A) will be interesting for future applications in 19th-century iron and glass roofs.

6. Conclusions

The load transfer in a representative 19th-century iron and glass connection is done in shear (single-lap shear joint) and in tension and/or compression (a butt joint). The experimental research extended the knowledge about both the mechanical properties of traditional sealants and the influence of some execution parameters that are relevant when renovating historic structures.

The linseed oil putty samples revealed that the shear strength of this material is very low, especially when compared to new adhesives as well as sealants. However, the compressive tests on the Soudal putty pointed out that the stiffness of this material can be high if enough curing time can be reached. This is not applicable for tensile forces, since the putty becomes brittle when drying out. Replacement products like the tested MS polymer sealant Soudaseal Tradition can give a more flexible alternative.
The experimental results of the stiffness moduli (in shear and in compression) will be used in the next chapter for the determination of the stiffness of the connection detail between the iron frame and the glass plates.

The single-lap shear tests on the MS polymer adhesive HQ Bond XS 4,5 delivered insights which are useful for the renovation practice.
When grit blasting has to be executed on-site, the accuracy of the obtained roughness is relatively poor. After grit blasting, the bare metal has to be protected for corrosion by a paint layer. The single-lap shear tests indicated that these two parameters only had a minor or no influence on the average shear strength. This means that neither the substrate roughness nor the paint layer is a decisive parameter.
Although it was not stated as a parameter in the beginning of the tests, the speed of applying the adhesive bond on the bare grit blasted surface, indicated to be very important. The average shear strength of the modern construction steel samples (adhesively bonded 4.5 hours after grit blasting) and 19th-century mild steel samples (adhesively bonded 76 hours after grit blasting) were significantly different. The oxide layer that has been formed on the surface of the unpainted mild steel probably caused a weak boundary layer and bond failure. More research is needed to examine the influence of the time period between surface preparation and application of the adhesive bond. However, it is already clear that a short time period, difficult to reach in on-site circumstances, can be a decisive factor.

Chapter 6

Parametric study of the contribution of glass to the overall structural behaviour

The goal of simulating the structural behaviour of a 19th-century iron and glass roof during renovation studies is to assess the safety level of the structure. At the same time, the heritage value of the roof and its components define the boundary conditions in which a restoration proposal has to be made.
As described before in Chapter 3, glass plates can play a structural role in modern glass constructions. In this chapter, the contribution of the glass cladding to the strength, stiffness and stability in historic constructions is investigated. Incorporating the glass cladding into the structural model might limit the necessary interventions to fulfil the modern requirements for structural integrity.

A parameter study will be carried out to investigate the circumstances in which the contribution of the glass cladding is useful. Different parameters that are specific for the renovation of 19th-century iron and glass roofs could affect the contribution of the glass cladding:

- the mechanical properties of the historic glass cladding: The course books and manuals investigated in Chapter 2 almost never mention the mechanical characteristics of 19th-century glass (one exception was reported in Chapter 3 paragraph 0), so the mechanical properties are considered to be unknown. The mechanical properties of modern glass are used as a basis for the further research.
- the 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 will be investigated.
- the 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. The experimental results from Chapter 5 are incorporated in this chapter.

The contribution of the glass will be studied by calculations of an iron frame cladded with glass plates. Finite element calculations[1] allow to integrate frame and plate structures and to simulate a connection with a dedicated stiffness. Linear static analysis will give results of deformation, stresses, and reaction forces. A linear buckling analysis will provide insight in the instability phenomena of the whole structure. All these calculations will be performed under gravity, maintenance, snow and wind load, and under varying glass plate thickness and connection stiffness. The following chapter describes the calculations and results in detail/

[1] performed in the software package *Abaqus /CAE* 2011.

The contribution of the glass plates will be evaluated by simulating three possible interventions. The structural behaviour of the original structure (4 mm thick monolithic glass plates included in the model and connected to the iron arches with traditional linseed oil putty) will be compared to the structural behaviour of the following structures:

- the original iron frame (without the glass plates);
- the original iron frame and glass plates, but with an adjusted connection detail to obtain a higher stiffness so larger forces are transmitted between the iron glazing bar and the glass plates;
- the original iron frame, but cladded with laminated glass instead of the original monolithic glass.

The results of this comparison are discussed in paragraph 6.

The calculations will be based on the geometry of the roof light of the Saint-Hubertus Galleries. The restoration of this roof was described in Chapter 4. The galleries are particularly interesting as a case study because:

- The glass roof is an example of a slender non-hierarchical roof construction. The glazing bars consist of a curved slender iron rectangular section with straight iron L-sections on the sides to place the glass on. The glazing bars were placed every 40 cm. They serve as glazing bars and at the same time as the primary loadbearing structure, which leads to a very slender structure.
- The geometry of the roof is single curved. The structural contribution of the glass in both the curved and the longitudinal direction can thus be studied.
- The span of the roof in the curved direction is 8.32 m. This span is comparable to an elaborate number of other 19th-century iron and glass roofs, from glasshouses (e.g. Victoria Regia House, Meise, architect Balat, 1853, p.70) to skylights in public buildings (e.g. UCB library, Brussels, architect D. De Keyser, 1872, p.116) and private dwellings (e.g. skylight in Horta museum, the former house of Victor Horta, Brussels, architect Victor Horta, 1898).
- The restoration campaign (from 1993 until 1997) was very well documented. The differential settlements of the hinged supports of the arches (Chapter 4 paragraph 4.2 p.111) were measured and one segment was defined where this was a major concern for the loadbearing structure.
- Recalculations of the roof structure during the renovation studies, which took only the iron frame in account, concluded that the structure had to be strengthened to carry all external loads. The new connection which replaced the severely corroded L-sections of the glazing bars was used as a strengthening element for the iron frame.

1. The geometry of the Saint-Hubertus Galleries

1.1. The geometry of the iron and glass components

The Saint-Hubertus Galleries are a barrel vault consisting of a series of arcs with a lanterneau on top of each arc (Figure 6-1). The entity of the arc, the lanterneau and the columns in between will be called "arch" in the proceeding text. Longitudinal iron components span between the arches: round bars in the central part of the arc, plates at the ridge and the column supports of the lanterneau and L-sections across the bottom parts of the arcs. The structure is cladded with small glass plates, with an interruption at the transition from arc to lanterneau. Each glass plate lies on a pair of L-shaped glazing bars, which on their turn are supported at their top by the longitudinal L-sections (Figure 6-2).
The nomenclature of all iron components is appointed in Figure 6-3.

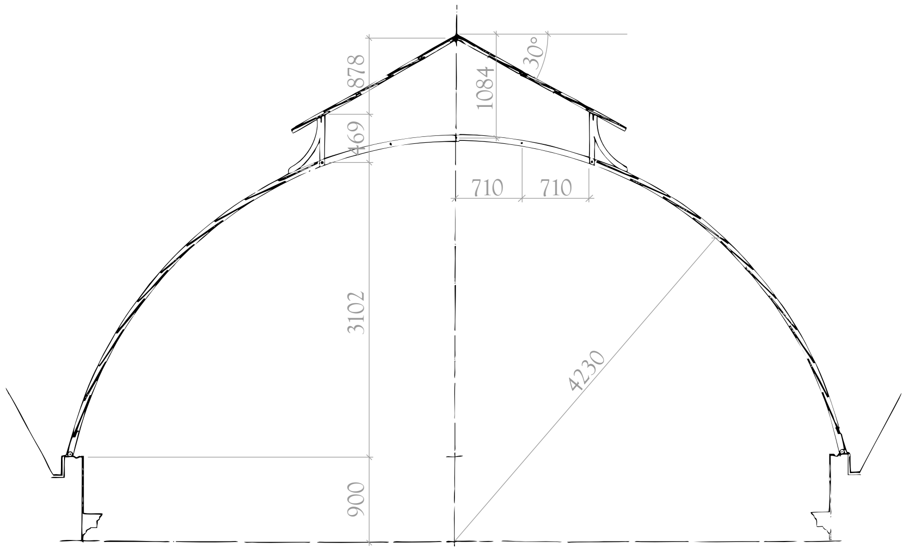

Figure 6-1: Cross section of Saint-Hubertus Galleries arch[2]

[2] A.2R.C, Forum, and TCA 1996b, p.7 with annotations of the author.

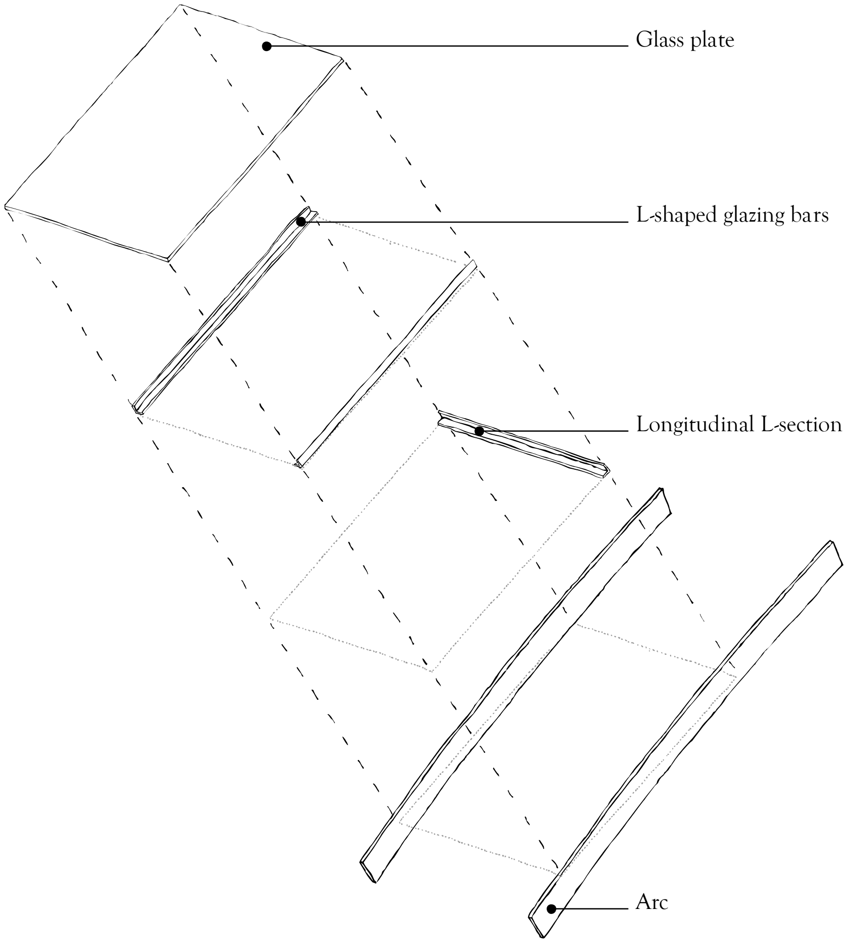

Figure 6-2: Exploded view of placing glass covering of the Saint-Hubertus Galleries

The geometry of the Saint-Hubertus Galleries was reconstructed in AutoCAD[3] based on different sources. The overall geometry (the span, the height, the radius of the circular arc, etc.) are all agreed on in all sources, but the overall geometry of the lanterneau (the height, the inclination of the rafters, etc.) was consequently taken from the plans drawn by A.2R.C for the renovation campaign[4]. The dimensions of all iron sections were taken from the restoration file of A.2R.C[5] while the glass plate dimensions and exact location were extracted from the plans of the existing situation of A.2R.C[6].

An overview of all dimensions is given in Table 6-1.

[3] Autodesk 2011.
[4] A.2R.C 1996a; A.2R.C 1996b.
[5] A.2R.C, Forum, and TCA 1996b.
[6] A.2R.C 1996a.

Table 6-1: Geometry of iron components of the Saint-Hubertus Galleries

component name	**component geometry**
arc	50x7 mm
columns	50x7 mm
lanterneau	40x4 mm
glazing bar L-sections	15x15x2 mm
longitudinal L-sections	15x15x2 mm
longitudinal bars	Ø8 mm
longitudinal column plates	50x7 mm
longitudinal ridge plate	40x4 mm

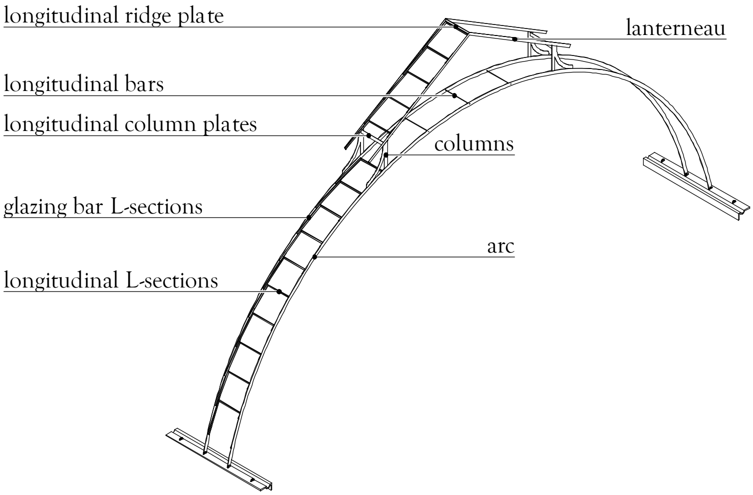

Figure 6-3: One segment of the Saint-Hubertus Galleries without glass plates with the nomenclature of the iron sections for this chapter

Figure 6-4: Hinged support at the bottom of each arch[7]

[7] Lefevre & Mechelynck Architectes Associes Scprl 1993.

1.2. The external supports

The arcs are at their bottom connected with a hinge to a cast-iron strip (Figure 6-4). The external support for the calculation is modelled as a hinge about the Z-axis (Figure 6-7). The resistance of the connection to the calculated moment about the other axes have to be checked as part of the interpretation of the calculation results.
The Saint-Hubertus Galleries consist of two sections, each with more than 200 iron arches. The interval of 40 cm between the arches is cladded with glass plates. The same structure is repeated for every segment of 40 cm. The calculation model of the Saint-Hubertus Galleries will therefore isolate a limited number of segments out of the length of the galleries. At their ends, the masonry entrances of the galleries form a massive support for lateral displacements. The longitudinal iron components are connected to a final arch. The final arch and the longitudinal bars are connected to the end walls, but the execution of the connection cannot be verified (Figure 6-5). It can be assumed that the connection can resist horizontal forces. Consequently, the horizontal displacement of the final arches will be prevented to simulate the buttressing of the end walls.

1.3. The internal connections

The columns and arcs are connected with a fixed connection because of the triangulated base of the columns, while the top of the columns is fixed to the lanterneau section with the help of the longitudinal column plates (Figure 6-6 and Figure 6-8). The connection method of the longitudinal components to the arches is not clearly described in the restoration file and neither can be extracted unmistakably from the pictures. These connections will be modelled as fixed connections, but care must be taken that no unrealistic stress concentrations arise near the internal connections due to moments.

Figure 6-5: Connection of the longitudinal bars to the end walls of the galleries (2012-08-10)

Figure 6-6: Interior view of connections between iron components: columns connected to the arcs, longitudinal column plates to the columns, longitudinal L-sections to the arches (2011-09-17)

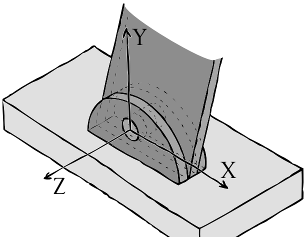

Figure 6-7: Hinged connection at bottom of iron arches

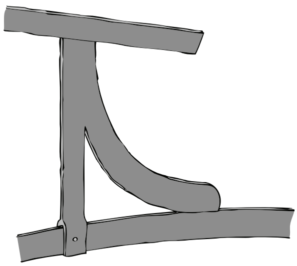

Figure 6-8: Connection of columns to iron arch

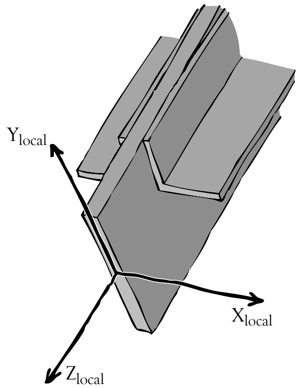

Figure 6-9: Glazing bars connected to iron arch

The renovation report[8] cites that the L-shaped glazing bars would have been connected to the iron arches with small angle sections, however this could not be verified since the L-sections were replaced during the renovation. The L-shaped glazing bars are not modelled as separate parts in the calculation model. Their contribution to the stiffness of the iron arches is assumed to be limited: about the strong axis (Figure 6-9: X_{local} axis) their contribution will be limited because the majority of the section lies close to the neutral axis; about the weak axis (Figure 6-9: Y_{local} axis) their contribution will be limited compared to the contribution of the glass plates because of their limited section. The contribution of the glazing bars to the support of the glass plates will be taken into account in the calculation of the connection stiffness described in paragraph 3.

[8] A.2R.C, Forum, and TCA 1996b.

1.4. A specific model to study the influence on the horizontal displacements

The iron and glass roof of the Saint-Hubertus Galleries is buttressed by the end walls to prevent horizontal displacements. However, in other 19th-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 is 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.

2. The research approach for the calculations

A structural recalculation of the Saint-Hubertus Galleries was carried out during the renovation studies[9]. It covered a two-dimensional calculation of one arch of the Saint-Hubertus Galleries (according to the Belgian buildings standards that were valid before the present-day Eurocodes were introduced). The calculation investigated the influence of corrosion: a change of articulated to fixed supports (reduction of the movement capacity due to rust and dirt layers) and a reduction of the cross-section of the arch. It was concluded that the arch fulfilled the criteria of structural integrity, as long as the arc section width was at least 5.5 mm (compared to the original 7 mm).

In this chapter, a three-dimensional calculation will be carried out integrating the longitudinal iron components and the glass plates into the model. The goal of this calculation is to study the contribution of the glass cladding and its influencing parameters. The geometry and boundary conditions of the Saint-Hubertus Galleries serve as a basis for the structural simulations, however a nominal structural assessment of the Galleries will not be carried out:

- The loading on the structure will be an abstract representation of the climatic loads (paragraph 2.1). For a structural recalculation as part of a renovation study, the loading profiles would have to be more elaborate instead of abstract simplifications. The reduction of the loads in specific conditions might also be considered when calculating the structure in the context of a restoration: the wind loading could be studied by wind tunnel tests instead of via the simplified methods of the Eurocodes, the snow loading might be reduced because of the limited snow that would remain on the glass cladding, etc.

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[9] T.C.A. ‘Etude de stabilité’ in T.C.A., CEWAC, and OREX 1996.

- Apart from the exact determination of the loads, load combinations would also have to be composed and safety factors would have to be applied to the loads (as well as to the materials).
- The supports of the structure would have to be examined for their resistance against rotations. Corrosion could limit the movement of the hinged support about the Z-axis (Figure 6-7) while the assumption of a fixed support about the other axes would have to be verified.
- The mechanical characteristics of the iron material would have to be determined. In the course of the restoration studies, a tension test was carried out (Figure 6-13) on one sample extracted from the Galleries. However, the uncertainty on this one sample is high.
- The condition of the structure would have to be incorporated into the simulations. The reduction of the cross-section of the iron members due to corrosion would have to be implemented, the deterioration of the internal connections would have to be surveyed, the differential settlements of part of the structure would have to be included in the geometry, etc.

2.1. The applied 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 the climatic loads. The self-weight of the structure is considered in combination with one of the considered live loads. These loads simulate four load cases:

1) a concentrated load at the attachment points of the maintenance ladder (at the ridge of the arch or on top of the small columns supporting the lanterneau) simulating maintenance loads of 1kN (both concentrated loads are illustrated in Figure 6-10);
2) a vertical distributed load on the glass plates simulating snow load (both symmetrical over the whole structure and asymmetrical on only one half of the structure) of 0.5 kN/m^2 (asymmetric snow load is illustrated in Figure 6-11);
3) a distributed load perpendicular on the glass plates simulating 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;
4) 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 (illustrated in Figure 6-12) 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).

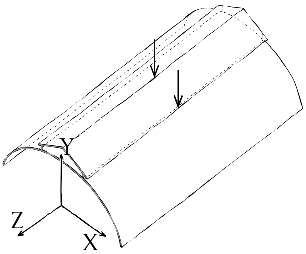

Figure 6-10: Concentrated maintenance load

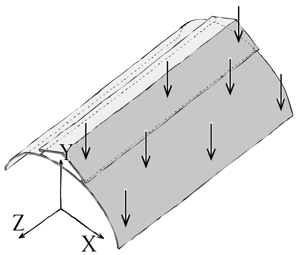

Figure 6-11: Asymmetric snow load

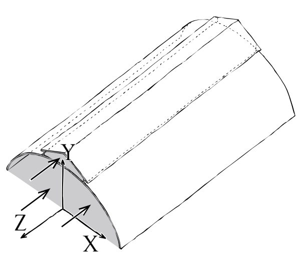

Figure 6-12: Horizontal wind load

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[10]; 0.5 kN/m² is the characteristic value of snow load on the ground in Belgium[11]; 0.5 kN/m² is the maximum external wind pressure on cylindrical roofs in Belgium[12]. The distributed loads are considered to be constant over all segments of the structure.

2.2. The glass plate thickness

The glass plate thickness parameter has an influence on the glass weight and the stiffness of the individual glass plates. The stiffness of the glass plate has an influence on the buckling resistance of a single glass plate, as discussed in Chapter 3. The influence of the glass thickness on the global behaviour of the roof will be analysed by using four glass plate compositions.

- The original glass plate thickness of 4 mm.
- A new glass plate composition of 2 panes of 2 mm glass laminated against each other. This is the glass plate composition that was applied at the Saint-Hubertus Galleries after the 1993-97 renovation campaign (Chapter 4 paragraph 4).
- A very stiff glass plate composition of 2 panes of 4 mm thick glass laminated against each other. This glass plate composition is added to the parameter study to study the influence when the glass weight is doubled.
- A simulation of the application of a new 4 mm thick glass plate onto which the original 4 mm thick glass plate is laminated using the resin technique (Chapter 4

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[10] *NBN EN 1991-1-1: Eurocode 1 - Belastingen op constructies - Deel 1-1: Algemene belastingen - Volumieke gewichten, eigen gewicht en opgelegde belastingen voor gebouwen* 2002.

[11] *EN 1991-1-3: Eurocode 1 - Actions on structures - Part 1-3: General actions - Snow loads* 2003; *NBN EN 1991-1-3 ANB: Eurocode 1 - Belastingen op construction - Deel 1-3: Algemene belastingen - Sneeuwbelasting - Nationale bijlage* 2007.

[12] *EN 1991-1-4: Eurocode 1 - Actions on structures - Part 1-4: General action - Wind actions* 2005; *NBN EN 1991-1-4 ANB: Eurocode 1: Belastingen op constructies - Deel 1-4: Algemene belastingen - Windbelasting - Nationale bijlage* 2010.

paragraph 5). The original glass plate is considered to have no contribution to the stiffness of the glass plate composition as the new glass plate should not transmit stresses to the original plate. The original glass plate thus only contributes to the glass weight.

All glass compositions will be modelled in the finite element model as a monolithic glass plate with an effective thickness calculated with the method proposed in the technical report of the BBRI based on the European Draft Standard[13]. Two possible effective thicknesses can be calculated: one for the calculation of the deformation in SLS and one for the stresses occurring in the glass plates in ULS. The formula for the deformation effective thickness will be used, so the deformation of the glass plates is modelled according to the proposed standard. The nominal values of the stresses in the glass plates can therefore not be used.

$$h_{eff,\omega} = \sqrt[3]{(1-\omega)\sum_i h_i^3 + \omega\left(\sum_i h_i\right)^3} \qquad \text{(Eq. 6-1)}$$

The effective thickness for the deformation of a laminated glass pane depends on the coefficient of participation ω of the interlayer material. This participation coefficient varies between zero (for no transfer of shear forces between the laminated panes) and 1 (for a perfect monolithic behaviour of the two plates). The effectiveness of the interlayer material (thus the amount of shear forces that it can transmit) depends on the load duration, the temperature, the size of the glass plates, etc[14]. The different effects and their mutual dependency are hard to model in one value for ω. Therefore, it is safer to model the material behaviour with a value of zero for the participation coefficient ω.

The same effective thickness can be used for modelling the buckling behaviour of laminated glass panels. Under both in-plane compressive forces and in-plane shear forces, recent research proved that an extra parameter needs to be added to accurately estimate the effective level of shear connection provided by the interlayer material[15]. Since no shear force transfer is assumed for this study (ω =0), the effective thickness in the calculation model is valid for the buckling analysis of the plates.

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[13] *prEN 13474-3: Glass in building - Determination of the strength of glass panes - Part 3: General method of calculation and determination of strength of glass by testing* 2008b, p.20–22; *TV 242: Bijzondere bouwwerken uit glas - Deel 1: structurele toepassingen* 2011, p.77–78.

[14] Callewaert, Belis, and Van Impe 2011.

[15] Bedon and Amadio 2012.

The difference between the effective thickness and the real thickness has to be added as extra self-weight to the loads. An overview of the glass plate compositions and the characteristics that have to be used in the finite element model is given in Table 6-2.

Table 6-2: Overview of effective thickness and weight of glass plate compositions with $\omega=0$

	$h_{eff,\omega}$ [mm]	total weight [N/mm²]	extra self-weight load [N/mm²]
original 4mm	4.00	10.00×10^{-5}	0.00
new laminated 2x2mm	2.52	10.00×10^{-5}	3.70×10^{-5}
new laminated 2x4mm	5.04	20.00×10^{-5}	7.40×10^{-5}
new 4mm + original 4mm	4.00	20.00×10^{-5}	10.00×10^{-5}

Table 6-3: Tensile test results of sample of Saint-Hubertus Galleries

yield stress at 0.2% strain	240.9 N/mm²
tensile strength	336.7 N/mm²
strain at failure	13.46 %

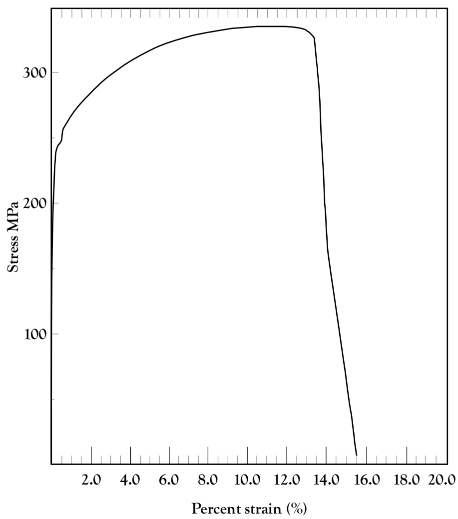

Figure 6-13: Stress-strain curve of iron material of Saint-Hubertus Galleries[16]

[16] T.C.A., CEWAC, and OREX 1996.

2.3. Comparing the models in the parameter study

The results of all the models will be compared to study the influence of the glass plate thickness and the connection stiffness. The reaction forces, the deformation of the iron arches, the deformation of the adhesive connections, the stresses in all iron components and the buckling behaviour will be evaluated, for both parameters and under different loads. The results of all calculation models will be normalized 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 for every result graph but the influence of the parameters can be read directly from the graphs.

To give an idea of the scale to the results, a brief description will be given of the nominal results. Therefore, a limited number of evaluation criteria are defined. These criteria are based on the structural design criteria of the Eurocodes[17]. However, no partial safety factors are applied on the material properties and the loads. The nominal evaluation criteria are:

1) The reaction forces at the supports. Attention is given to the moment reactions at the hinges of the arcs, to check whether these moments about the transversal axis (X-axis) can be taken up by the real connection detail.
2) The vertical (Y-direction) deformation of the iron components. For a span of 8.32 m, a vertical deflection of 28 mm (L/300) is taken as the maximum allowable limit.
3) The relative deformation of the adhesive connections. This deformation in compression has to be lower than the width of the connection to avoid contact of the glass and iron.
4) The maximum Von Mises stress in the iron structure (the arches and the longitudinal components). Tensile tests were carried out in 1995 and reported by OREX (Figure 6-13 and Table 6-3)[18]. The report only mentions the results of one sample. Literature confirms that yield strength of 235 N/mm² is realistic for 19th-century iron materials[19].

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[17] *EN 1990: Eurocode - Basis of structural design* 2002.

[18] Essais de traction, OREX in T.C.A., CEWAC, and OREX 1996.

[19] De Bouw 2010, p.115.

3. Modelling the connection stiffness

3.1. The equivalent beam theory to define the connection stiffnesses

The stress state of the putty connection

Loads are transferred between the glass plates and the iron arches via the putty connection. The stiffness of the sealant or adhesive material (Chapter 5) is one of the parameters that define 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 will be simplified to develop an analytical method to calculate the stiffness of the connection out of the properties of the sealant or adhesive (Figure 6-14 and Figure 6-15).

Three load types can be considered in the cross-section of the joint: an axial in-plane force F_{hor}, an axial out-of-plane force F_{vert} and a bending moment of the glass plate M_{hor} (Figure 6-15). Since the glass plates are very stiff (the E-modulus of the glass is much higher than of the putty and the glass plates have a very small span), the bending of the glass plate and thus the rotation force transmitted to the putty can be neglected. The behaviour of the connection is considered orthotropic: the connection stiffnesses in the cross-section and perpendicular to the cross-section are equal, but are separated for axial and shear behaviour (the E- and G-modulus of the connection are independent).

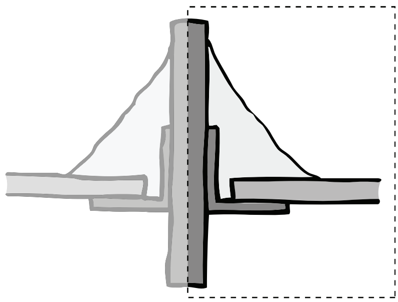

Figure 6-14: Connection detail at Saint-Hubertus Galleries with annotation of half of the connection that will be illustrated in following figures

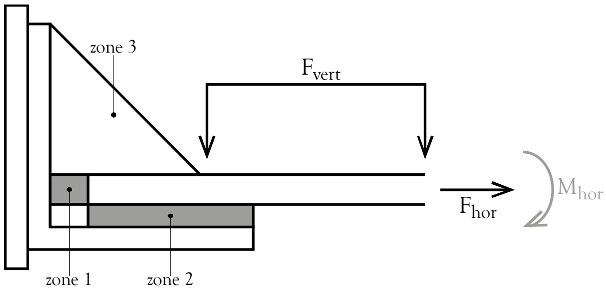

Figure 6-15: Loaded zones of the real connection detail

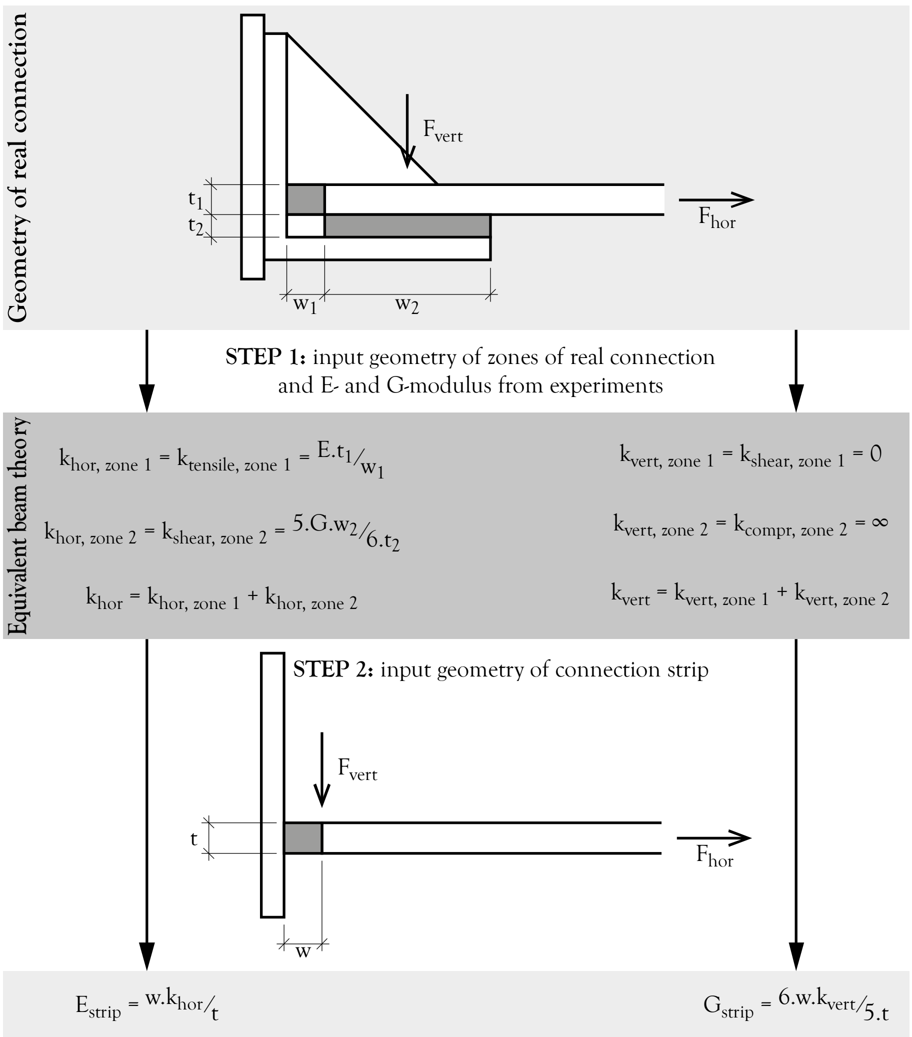

Figure 6-16: Scheme of calculation of fictitious material parameters

The connection detail can be divided in different loaded zones. For the analytical calculation of the joint stiffness, zone 1 and zone 2 are taken into account for the force transfer through the joint (Figure 6-15). The reason for not considering zone 3 will be explained in the following paragraphs.

A positive out-of-plane force F_{vert} generates a shear force in zone 1 and a pressure on zone 2. The compression of zone 2 will be limited, due to the large loaded surface and the limited compressibility, so this stiffness is very high. The shear strain of zone 1 will therefore be very low and the loaded surface is small, so the shear force transfer is negligible. When the connection is sealed with a material that cannot resist tensile forces (e.g. linseed oil putty), zone 3 will have a major contribution for a negative out-of-plane force F_{vert} (occurring with e.g. wind suction).

A positive in-plane force F_{hor} generates a tensile force in zone 1 and a shear force in zone 2. Due to the large difference in dimensions, zone 2 will have the largest contribution to the stiffness of the joint. However, for a negative in-plane force F_{hor} compressive forces will act on zone 1 and the proportion of the contribution of the two zones can be different.

Modelling the connection as a strip of a fictitious material

To model the joint in the calculation model, the force transfer has to be simplified to a simple geometry and material. The joint will be simplified as a strip with a fictitious material which is located at zone 1 but behaves identical to the combined zones 1 and 2 in the real connection (Figure 6-17). The stiffness against the in-plane force F_{hor} is simulated by the E-modulus, while the stiffness against the out-of-plane force F_{vert} is simulated by the G-modulus. To model these two independently, the fictitious material is modelled as an orthotropic material.

The whole process of calculating the connection strip parameters is illustrated in the scheme in Figure 6-16.

Figure 6-17: Loads on strip of fictitious material that simulates the connection stiffness

The E- and G-modulus of a material define the relationship between the stress and the deformation of the material. These characteristics can be defined by experimental research. The experiments described in Chapter 5 are used to determine the characteristics of the adhesive in the real connection detail. Subsequently, the beam theory can be used to calculate the equivalent connection stiffnesses independent from the connection detail geometry. These equivalent stiffnesses are defined analogously to spring stiffness: the force per deformation unit. For each load case (F_{vert} and F_{hor}), the equivalent stiffness of both zones can be superposed via the principle of springs arranged in series. These total equivalent stiffnesses can be translated with the equivalent beam theory to the characteristics of the connection strip: the E- and G-modulus of the orthotropic material (a fictitious material with a Poisson's ratio of zero).

Derivation of the equivalent beam stiffness under axial loading

For a theoretical case with geometry illustrated in Figure 6-18, the equivalent beam stiffness under axial loading (which is the force per displacement) can be expressed using Hooke's law.

$$\begin{aligned} &\sigma = E.\varepsilon \\ &\Leftrightarrow \frac{F}{A} = E.\frac{\Delta L}{L} \\ &\Leftrightarrow \frac{F}{\Delta L} = \frac{E.A}{L} \end{aligned} \qquad \text{(Eq. 6-2)}$$

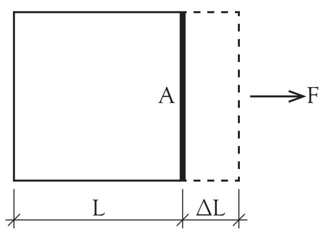

Figure 6-18: Translational stiffness under axial loading

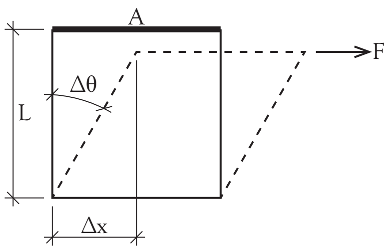

Figure 6-19: Rotational stiffness under shear loading

Derivation of the equivalent beam stiffness under shear loading

For a theoretical case with geometry illustrated in Figure 6-19, the equivalent beam stiffness under shear loading (which is the force per displacement) can be expressed analogously to Hooke's law.

$$\tau = G.\gamma$$

$$\Leftrightarrow \frac{F}{A'} = G.\Delta\theta$$

$$\left| \textit{for small displacements}: \theta \approx \tan\theta = \frac{\Delta x}{L} \right.$$

$$\Leftrightarrow \frac{F}{A'} = G.\frac{\Delta x}{L} \qquad \text{(Eq. 6-3)}$$

$$\Leftrightarrow \frac{F}{\Delta x} = \frac{G.A'}{L}$$

$$\textit{with } A' = \frac{5}{6}.A$$

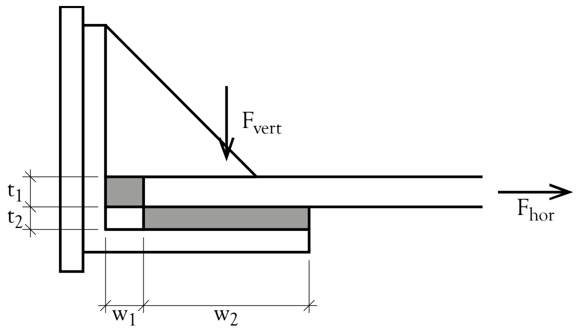

Figure 6-20: Geometric parameters of the two loaded zones on the real connection detail

Table 6-4: Geometric parameters of the real connection detail in relation to the equivalent beam equations

loading condition		F_{hor}		F_{vert}	
zone 1	A	axial loading	$d.t_1$	shear loading	$d.t_1$
	L		w_1		w_1
zone 2	A	shear loading	$d.w_2$	axial loading	$d.w_2$
	L		t_2		t_2

The shear force induces a strain both due to bending and due to shear stress. The proportion of the shear strain to the total strain can be calculated analytically[20] and computed numerically[21]. For a rectangular section of a material with a Poisson's ratio of zero, this proportion is close to 5/6 or 1.2. The shear area is therefore expressed as a proportion of the geometric area.

Step 1: Input of the geometry of the zones of the real connection

The first step illustrated in Figure 6-16 comprises the input of the geometry of the real connection and the input of the experimental results for E- and G-moduli.

The geometric parameters (expressed using the symbols of the equivalent beam equations) of the two loaded zones in the real connection detail are illustrated in Figure 6-20 and summarized in Table 6-4. The depth *d* of the joint perpendicular to the cross-section is equal for all parts of the joint.

The expressions of the stiffness have to be divided by the depth d of the joint to get stiffnesses per length unit that are applicable to linear connections. The following equations per zone are the result:

$$\begin{aligned} k_{hor,zone1} &= \frac{F}{\Delta L}.\frac{1}{d} \\ &= \frac{E.A}{L}.\frac{1}{d} = \frac{E_{\exp}.t_1.d}{w_1}.\frac{1}{d} \\ &= \frac{E_{\exp}.t_1}{w_1} \end{aligned} \qquad \text{(Eq. 6-4)}$$

$$\begin{aligned} k_{hor,zone2} &= \frac{F}{\Delta x}.\frac{1}{d} \\ &= \frac{5}{6}.\frac{G.A}{L}.\frac{1}{d} = \frac{5.G_{\exp}.w_2.d}{6.t_2}.\frac{1}{d} \\ &= \frac{5.G_{\exp}.w_2}{6.t_2} \end{aligned} \qquad \text{(Eq. 6-5)}$$

[20] Renton 1991; Renton 1997.

[21] Schramm et al. 1994.

Step 2: Input of the geometry of the connection strip with fictitious material
The second step of the calculation scheme is the input of the geometric parameters of the connection strip in the equations of the equivalent beam stiffnesses. Figure 6-21 and Table 6-5 show the geometry of the connection strip elements (translated to the symbols of the equivalent beam equations). The depth *d* of the joint perpendicular to the cross-section is equal for all parts of the joint.

The expressions of the stiffness have to be divided by the depth d of the joint to get stiffnesses that are applicable to linear connections. The following equations are the result:

$$\begin{aligned} k_{hor} &= \frac{F}{\Delta L}.\frac{1}{d} = \frac{E.A}{L}.\frac{1}{d} = \frac{E_{strip}.d.t}{w}.\frac{1}{d} \\ &= \frac{E_{strip}.t}{w} \\ &= k_{hor,zone1} + k_{hor,zone2} \end{aligned} \quad \text{(Eq. 6-6)}$$

$$\begin{aligned} k_{vert} &= \frac{F}{\Delta x}.\frac{1}{d} = \frac{5}{6}.\frac{G.A}{L}.\frac{1}{d} = \frac{5}{6}.\frac{G_{strip}.d.t}{w}.\frac{1}{d} \\ &= \frac{5.G_{strip}.t}{6.w} \\ &= k_{vert,zone1} + k_{vert,zone2} \end{aligned} \quad \text{(Eq. 6-7)}$$

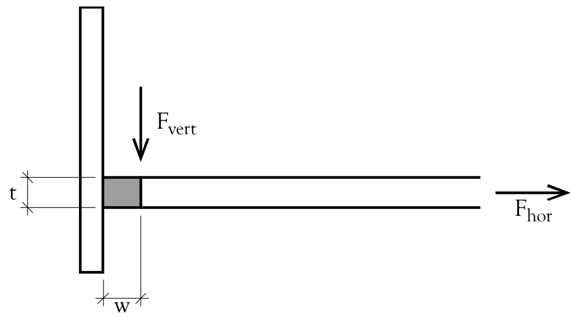

Figure 6-21: Geometric parameters of the connection strip

Table 6-5: Geometric parameters of the connection strip translated to the equivalent beam equations

loading condition	F_{hor}	F_{vert}
A	$d.t$	$d.t$
L	w	w
k	$k_{hor, zone1}+k_{hor, zone2}$	$k_{vert, zone1}+k_{vert, zone2}$

The width w of the joint is defined by the geometry of the model: it is the opening that is left between two plates and is thus known. The thickness of the material strip is chosen as equal to the default glass thickness (4 mm). The two above equations can be rewritten so that the material's Young's modulus E_{strip} and the shear modulus G_{strip} can be calculated out of the horizontal and vertical connection stiffnesses. These equations are valid for orthotropic materials and when the Poisson's ratio is set to zero[22].

$$E_{strip} = \frac{k_{hor} \,.\, w}{t}$$
$$G_{strip} = \frac{6}{5} . \frac{k_{vert} \,.\, w}{t} \qquad \text{(Eq. 6-8)}$$

3.2. Input of the parameters from the experimental research

The E- and G-moduli from the experimental research are summarized in Table 6-6. Soudal Putty is a traditional linseed oil putty which was used in the original connection detail. Soudal Soudaseal Tradition is a MS polymer sealant that is developed as a replacement product for traditional putty. Both sealants were tested and reported in Chapter 5. The research project at Ghent University on the application of adhesives for glass connections demonstrated that the HQ Bond XS 4,5 did perform weak when tested for environmental conditions. However the same research revealed that the Soudaseal 2k from manufacturer Soudal showed to be a modern MS polymer adhesive that is durable and behaves well under the tested surface contaminations (oil, water and sand). The latter adhesive is therefore included in the overview[23].

[22] The use of the equivalent beam theory to calculate the connection strip characteristics out of a set of chosen connection stiffnesses was validated numerically in Abaqus. A model with a series of point springs (of which the sum was the total connection stiffness) was compared to a model with the equivalent material strip. This was done both for an in-plane force F_{hor} as for an out-of-plane force F_{vert}. The connected substrates were modelled as *rigid bodies* (undeformable elements), so a possible influence of the stiffness of the substrates is eliminated. The displacements were compared to the forces acting in the connection. For the in-plane force, the spring and connection strip model exactly matched. For the out-of-plane force, a deviation of 0.07% was recorded. This deviation is due to the approximate value of the proportion of the shear area 5/6. The method of using equivalent stiffnesses in a fictitious material strip was also used and validated by A. Bagger in her doctoral thesis.

[23] Experimental results of this adhesive were provided by Soudal. From these experiments, a E-modulus (at 100% strain) of 1.20 N/mm² and a G-modulus (at 1% strain, which is 12.5% of the ultimate strain of 8%) of 31.65 N/mm² were extracted.

Table 6-6: Overview of the E- and G-moduli from the experimental research

	E_{exp} [N/mm²]	G_{exp} [N/mm²]
Soudal Putty ("Mastic vitrier")	20.65 (no tension)	0.01
Soudal Soudaseal Tradition	1.35 (1)	1.24
Soudal Soudaseal 2k	1.20 (2)	31.65 (2)

(1) result from technical data sheet provided by the sealant manufacturer
(2) result from experimental data provided by the adhesive manufacturer

Table 6-7: Equivalent stiffnesses under in-plane force F_{hor}

	$k_{hor, zone 1}$ [N/mm²]	$k_{hor, zone 2}$ [N/mm²]	k_{hor} [N/mm²]
Soudal Putty ("Mastic vitrier")	41.30 (no tension)	0.05	41.35
Soudal Soudaseal Tradition	2.70	5.68	8.38
Soudal Soudaseal 2k	2.40	145.06	147.46

From the stiffness moduli, the equivalent stiffnesses of the two loaded zones under the in-plane force F_{hor} can be calculated ((Eq. 6-4), (Eq. 6-5) and Table 6-4). The resulting stiffnesses are summarized in Table 6-7.

3.3. Parameters for the global model

The equivalent stiffnesses are an approximation for the connection stiffnesses that can be expected. The global model will be built with three classes of stiffnesses (Table 6-9). The resistance against the out-of-plane force F_{vert} is considered to be very high, thus the G-modulus as if it were a solid steel strip was taken as a guideline.

- The first stiffness class is a simulation of the glass plates sealed with traditional linseed oil putty. From the experimental research (Chapter 5), it was clear that this putty could transmit a relevant amount of compressive forces but could not resist any shear or tensile forces.

  The resistance against the out-of-plane force F_{vert} is considered equal to the connections with the other adhesives, although extra measures might be necessary to prevent the glass plates from lifting up from the glazing bars (e.g. the use of pins, see Chapter 2).
- The second stiffness class simulates the behaviour of a connection sealed with a modern adhesive with low stiffness characteristics. The values will be based on the characteristics of the Soudal Soudaseal Tradition sealant.

- The third stiffness class is a variation of the third class, with the application of a modern adhesive with a relatively high stiffness (in the range of a flexible adhesive that fulfils the criteria listed in Chapter 5). The values are based on the characteristics of the Soudal Soudaseal 2k adhesive.

4. Finite element analysis

4.1. Overview of parameter study

The parameter study will be carried out by the finite element calculation software Abaqus (version 6.11)[24]. Sixteen models will be analysed per load case. Four glass thicknesses are combined with three connection stiffnesses and a model without glass plates. The model without glass plates is equivalent to the standard calculation method used often by engineers, when only the iron frame is taken into account. For these models, the actual loading is transferred to line loads onto the iron arches instead of surface loads on the glass plates.

Table 6-8: Overview of effective thickness and weight of glass plate compositions with ω=0

		$h_{eff,\omega}$ [mm]	total weight [N/mm²]	extra self-weight load [N/mm²]
4a	original 4mm	4.00	10.00×10^{-5}	0.00
4b	new laminated 2x2mm	2.52	10.00×10^{-5}	3.70×10^{-5}
8a	new laminated 2x4mm	5.04	20.00×10^{-5}	7.40×10^{-5}
8b	new 4mm + original 4mm	4.00	20.00×10^{-5}	10.00×10^{-5}

Table 6-9: Overview of simulations of adhesive strip material characteristics

	k_{hor} [N/mm²]	E_{strip} [N/mm²]	G_{strip} [N/mm²]
model without glass plates	/	/	/
simulation of putty	50	125 (no tension)	80 000
simulation of new adhesive with low stiffness	5	12.5	80 000
simulation of new adhesive with high stiffness	100	250	80 000

[24] *Abaqus* /CAE 2011.

Table 6-10: Overview of parameter matrix per load case with the definition of the filename for every parameter combination

	no glass	putty	low stiffness	high stiffness
original 4mm	4a_none	4a_putty	4a_low	4a_high
new laminated 2x2mm	4b_none	4b_putty	4b_low	4b_high
new laminated 2x4mm	8a_none	8a_putty	8a_low	8a_high
new 4mm + original 4mm	8b_none	8b_putty	8b_low	8b_high

4.2. Analysis steps

The structure will be analysed by linear calculations only, which limits the results to small deformations. All steps are executed for each external load case (paragraph 2.1).
A linear static analysis of the original structure loaded only with its own weight will give the results of deformation, section forces, stresses, reaction forces, etc. Subsequently, a linear buckling analysis on the structure deformed by the gravity load (called Buckle 1) will give insight in the instability phenomena of the whole roof.
A second linear static analysis is performed. This analysis includes both the gravity load (propagated from the first linear static analysis) and the selected external loading. A second buckling analysis is performed after this linear static analysis (called Buckle 2), which starts from the deformed structure by both the gravity and the external loading.
The linear buckling analysis steps are set to calculate the first 5 eigenvalues (reduced to 3 when more than 1000 iterations were necessary to calculate the first 5).

4.3. Convergence and mesh density

The iron arches are modelled in Abaqus as shells (section of arc, columns and lanterneau as a surface with shell thickness defined on this surface). These are meshed with general purpose shell elements S4 (4-node doubly curved general-purpose shell element) and S3 (equivalent 3-node triangular shell element). The glass plates are meshed with the same elements. The longitudinal iron components are modelled as linear elements and meshed with B33 elements (2-node cubic beam elements). Different stages of convergence studies were performed for different connection stiffnesses and under different load cases.

1) The convergence of the displacements, stresses and eigenvalues in a model limited to one circular arc supported by hinges depending on the mesh size of the arc.
2) The convergence of the displacements, stresses and eigenvalues in a model of 2 segments (3 arcs) depending on the mesh size of the glass plates and the connection strip.

3) The convergence of the displacements, stresses and eigenvalues in a model of 2 segments (3 arcs) depending on the mesh size of the arcs and the longitudinal iron components.

With the resulting mesh parameters, a study was performed for the number of arcs that had to be modelled. The distributed loads are constant over all segments, however enough segments have to be modelled to isolate the impact of a concentrated load. This was done with different connection stiffnesses and evaluated based on the dissipation of a central concentrated load.

The resulting geometry and mesh size parameters are summarized in Table 6-11.

Table 6-11: Geometry and mesh size parameters of different components in global model

arc and lanterneau seed size	36 mm
longitudinal iron components seed size	36 mm
glass plates seed size	20 mm
connection strip mesh size	20 x 2 mm
number of arcs	33

4.4. Python script

The parametric study is performed by generating the geometry and the analysis in a python script. This script is built up in several steps, which will be described as an illustration of the calculation methodology. Steps that are left out in the models without glass plates are placed in italic font. An example script is added in the Appendices.

1) Import the geometry of the arc, columns and lanterneau via dxf files.
2) Create materials and cross-sections. The materials are summarized in Table 6-12.

Table 6-12: Materials definition in Abaqus python script

	density 10^{-9} kg/mm^3	E [N/mm^2]	ν
Iron	0.000 007 8	210 000	0.30
Glass	0.000 002 5	70 000	0.22
Adhesive	0.000 001 4	variable	0.00

3) Create arc, columns and lanterneau in one geometry part. Geometrically define lines and nodes where longitudinal components will have to be positioned. Assign different cross-sections to the arc and columns (50x7mm) than to the lanterneau

(40x4mm). Mesh the geometry with seed size 36mm and element types S4 and S3 (general shell elements).

4) *Create a plate in one geometry part with partitions to form 10mm wide connection strips along the left and right edges. Assign different cross-sections to the glass segment and the connection strip segments. Create surfaces at left and right plate end edges where connection with the arch will be defined. Partition the connection strips further in their width to converged mesh width of the strips (paragraph 4.3). Mesh the whole geometry part with seed size 20mm and element types S4 and S3 (general shell elements).*
5) Create longitudinal geometry parts (all iron components interconnecting the arches). Mesh them with seed size 36mm.
6) Create an assembly of one arch together with one segment of glass plates:
    a) Add a first arch (= arc + columns + lanterneau) to the assembly.
    b) *Add the 30 glass plates to the assembly. Align each edge of the glass plate to the corresponding line on the arch.*
    c) Add all the longitudinal iron components to the assembly. Align each component to the corresponding node on the arch.
7) Make a linear pattern of this arch and of this single segment with a spacing of 400mm.
8) *Use "tie constraints" to create a fixed connection between the arches and the plates. The arches are acting as master surfaces (they are considered stiffer and have a coarser mesh). These tie constraints prevent all translations and rotations.*
9) Use "coupling constraints" to create fixed connections between the arches and the longitudinal iron components. All translations and rotations of the longitudinal component are fixed.
10) Create the analysis steps and their corresponding output requests. A first linear static analysis under gravity loading for which the stresses ('S', 'MISESMAX'), the displacements ('U'), the strains ('E') and the reaction forces and moments ('RF') are calculated. A first buckling step is carried out for which all the displacements ('U') are calculated. The third step is the linear static analysis of the structure under gravity and external loading, for which again the stresses ('S', 'MISESMAX'), the displacements ('U'), the strains ('E') and the reaction forces and moments ('RF') are calculated. Finally, a buckling analysis is performed on the deformed structure of step 2, for which again the displacements ('U') are calculated.
11) Create the boundary conditions and apply them to all the analysis steps.
    a) In-plane hinges (UR_{XX}=UR_{YY}=0 and UR_{ZZ}=free) at the end edges of each arch.
    b) Vertical rollers (U_Z=0) along the whole surface of the end arches.
12) Create the external loads, both concentrated and distributed loads, both symmetrical and asymmetrical versions.

a) The gravity load, automatically calculated by the Abaqus software, based on the density factors defined with the material characteristics and a component in global Y-direction equal to -9.81.
b) A vertical (in the negative global Y-direction) distributed load on the glass plates, simulating the self-weight difference between the actual and the modelled thickness of the glass plate (paragraph 0).
c) A vertical distributed load on the glass plates, defined separately on both halves of the arches so that both symmetrical and asymmetrical loads can be applied.
d) A distributed load perpendicular on the glass plates, defined separately on both halves of the arches so that both symmetrical and asymmetrical loads can be applied.
e) A concentrated load in the negative global Y-direction at the ridge (symmetrical) or on top of the columns (asymmetrical) of the central arch.
f) A horizontal (in the global Z-direction) distributed load on the side of the first arch.

For all distributed loads, an equivalent line load on the arches is defined. These loads replace the distributed loads in the models without glass plates. The line loads are defined separately on both halves of the arches so that both symmetrical and asymmetrical loads can be applied.

13) Create and submit the analysis job.

The results of the analysis (only the quantities that were specified as output requests of the analysis steps) are written to an output database. This output databases of the whole parameter study are again read out with a python script.

5. Nominal results

The most onerous load case for both deformations and stresses is the asymmetric perpendicular load roughly simulating wind load. The asymmetric load cases are in general more severe for the structure to resists than their symmetric equivalents. In the following paragraph, the nominal results of the models which include glass plates will be discussed. For the static calculations, the results of the models loaded with self-weight combined with a live load case will be discussed. An overview of all calculation results can be consulted in Appendices.
The nominal results are compared to the evaluation criteria defined in paragraph 0. This comparison is carried out to give an idea of the scale of the quantities. It is however no strict evaluation of the structural performance of the iron and glass roof (paragraph 2).

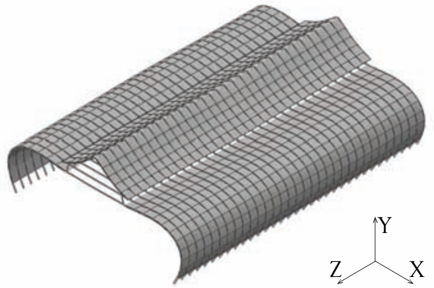

Figure 6-22: Deformed structure under symmetric concentrated load (scale 150)

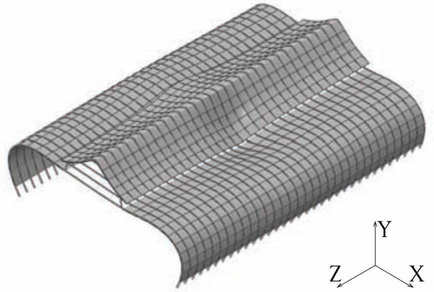

Figure 6-23: Deformed structure under asymmetric concentrated load (scale 150)

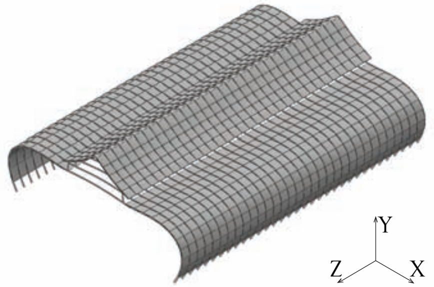

Figure 6-24: Deformed structure under symmetric vertical load (scale 50)

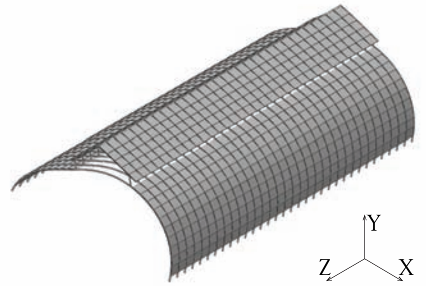

Figure 6-25: Deformed structure under asymmetric vertical load (scale 10)

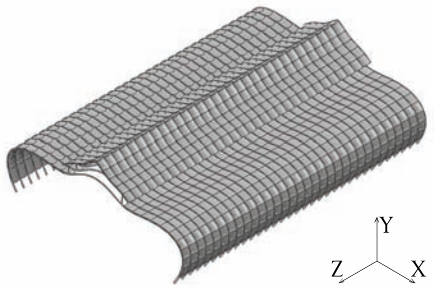

Figure 6-26: Deformed structure under symmetric perpendicular load (scale 150)

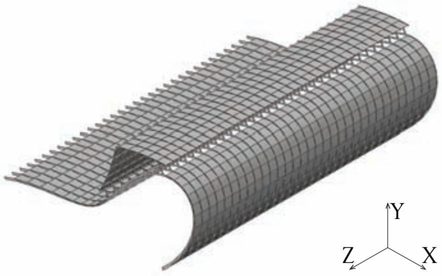

Figure 6-27: Deformed structure under asymmetric perpendicular load (scale 10)

Deformations

The shape and the values of the deformation of the structure are different for every load case. The deformed structures of the 4a_putty models (the original geometry of 4 mm thick monolithic glass and a putty connection) for all loads are illustrated in Figure 6-22 until Figure 6-27. The deformations are plotted on different scales for clarity. It is clear that the asymmetric perpendicular and vertical load cases are the most onerous.

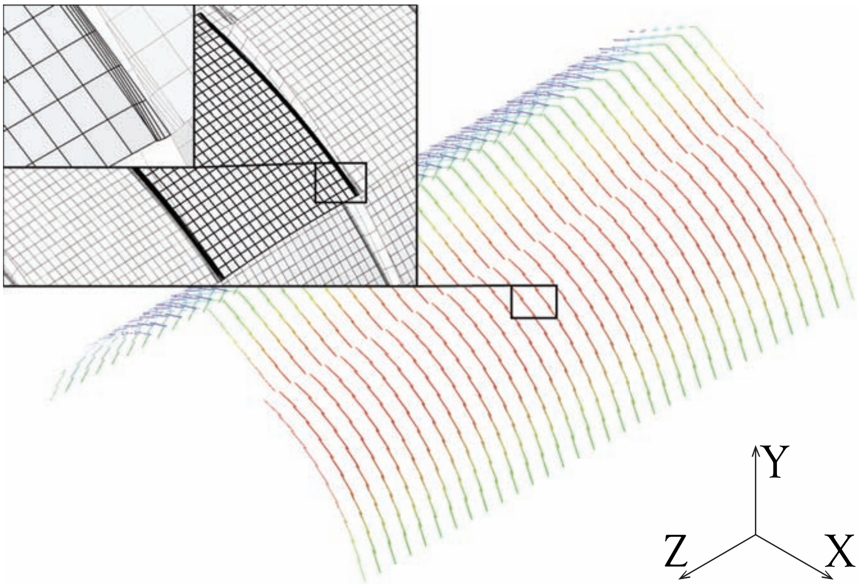

Figure 6-28: Deformation of adhesive material in Z-direction in the 4b_low model under asymmetric perpendicular loading

The maximum allowable vertical deformation in negative Y-direction (28 mm) is exceeded under the asymmetric perpendicular and vertical load cases (respectively to 631% and 210% of the maximum allowable deflection both at the 4b_low models). These deformations are thus unacceptable.

The relative deformations of the adhesive strips never exceed the maximum allowable value of 2 mm. The maximum deformation is 0.26 mm which occurs in the 4b_low model under asymmetric perpendicular loading (Figure 6-28).

Support reactions

The supports have to resist the reaction forces and reaction moments. The maximum and minimum reaction moments in the bottom hinges about the X-axis are the highest for the symmetric vertical load case. The maximum reaction moment of 296 Nmm occurs at the model 8b_low. This means that a force of 29.6 N has to be taken up over a distance of 10 mm (supposing this is the width of the hinge), which is a small moment to resist. The reaction moment is the highest at the end arches and diminished towards the central arch (Figure 6-29).

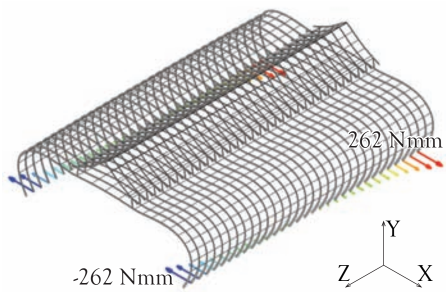

Figure 6-29: Reaction moments at the bottom hinges (about X-axis) for the 4b_low
model under symmetric vertical loading

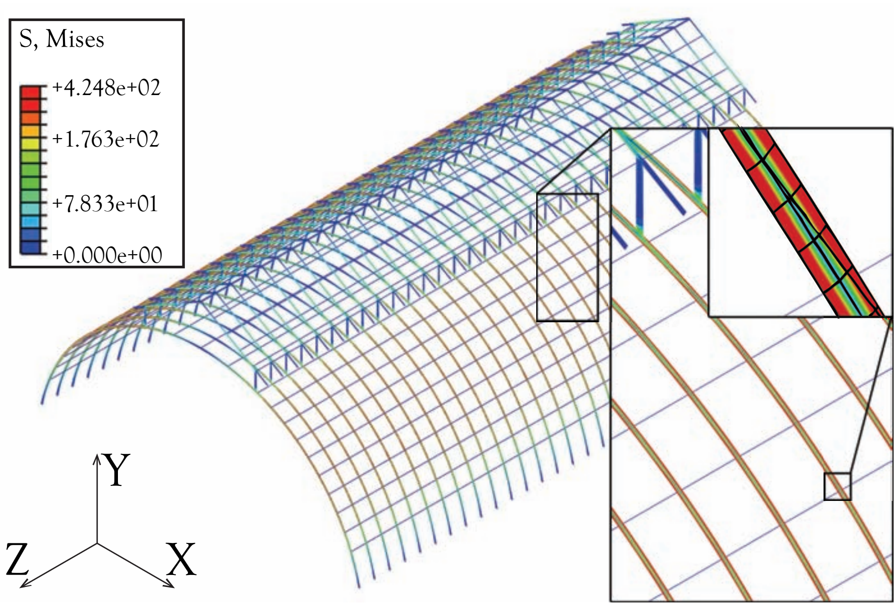

Figure 6-30: Von Mises stresses in iron components for the 4b_low model under asymmetric perpendicular loading

Stresses in iron components

The maximum allowable stress (235 N/mm²) is exceeded in the arches in all models under asymmetric perpendicular load. It reaches a maximum of 170% of the maximum allowable stress in the 4b_low model. The maximum stress is located at the bottom and top nerves of the arcs. The exceeding of the stresses is thus not due to stress concentrations and thus unacceptable. The distribution of the Von Mises stresses in the model and across the arch is illustrated in Figure 6-30.

The stresses in the longitudinal components are the highest in the longitudinal L-sections, except for the concentrated loads where the components the closest to the load introduction resist the highest stresses. The nominal values of the Von Mises stresses in the longitudinal components however, do not exceed the maximum allowable stress. A maximum of 23% of the capacity is used in the longitudinal columns plates in the 4b_low model under the asymmetric concentrated load.

Stability of the structure

The stability of the structure is defined by both global and local buckling modes. Two buckling calculations are performed with an incremental live load: firstly on the structure deformed under self-weight and secondly on the structure deformed under self-weight and live load. The eigenmodes of both buckling steps are equal. For all load cases, the eigenvalues of the second buckling step are exactly minus one of the eigenvalues of the first buckling step.

The buckling modes differ per load case and per studied parameter. Both global and local buckling modes (of the glass or of the iron components) are present. The models under symmetric and asymmetric concentrated load, only experience local buckling (for the first five eigenmodes). The eigenvalues of local buckling modes are often very close to each other.

The models with the lowest glass plate thickness (4b) under the asymmetric perpendicular load, experience local buckling of the glass plates even before the external load is reached. The critical buckling load is 0.47 N/mm² (compared to 0.50 N/mm² assumed wind load).

6. Influence of glass thickness and connection stiffness

The results in the following paragraphs will not be discussed for their nominal values, but the influence of the parameters will be illustrated. The results are normalized to the value of that quantity in the 4a_putty model (model simulating the original structure with 4 mm thick monolithic glass and a connection sealed with putty).

Graphs in the following paragraphs are drawn in function of both parameters: the connection stiffness and the glass thickness. For the latter, the effective thickness is taken. The two models with an effective thickness of 4 mm are plotted slightly to the left or right, to make a distinction between the two based on the thickness of the whole glass composition: 4a models are monolithic 4 mm thick glass and are plotted slightly left of value 4; 8b models comprise of two glass plates of 4 mm thickness and are plotted slightly right of value 4.

6.1. Influence of the presence of the glass plates

The models without glass plates simulate the results when only the iron frame is taken into account for the structural performance, but the glass weight is included as line loads on the iron frame. For all graphs in the following paragraphs, the markings of the models without glass plates are shaded grey and assigned zero connection stiffness.

Deformations

For all load cases, the presence of the glass plates has a significant positive influence on the maximum vertical deflection of the arches. This is illustrated for two load cases in Figure 6-31 and Figure 6-32. Proportionally, the influence is lower for the more onerous load cases (the deflection in the 4a_none model is 111% of the deflection in the 4a_putty model for the asymmetric perpendicular load case, while it is 122% for the symmetric concentrated load case).

Stresses in iron components

The maximum Von Mises stresses in the arches are overestimated in the models without glass plates. The Von Mises stresses for the asymmetric perpendicular and symmetrical vertical load case are plotted in Figure 6-35 and Figure 6-36. Proportionally, the most onerous loads exhibit the less influence of the presence of the glass plates (the maximum Von Mises stress in the arches in the 4a_none model is 117% of the stress in the 4a_putty model for the asymmetric perpendicular load case, while it is 128% for the symmetrical vertical load case).

The maximum Von Mises stresses in the longitudinal iron components are again overestimated in the models without glass plates. The highest stresses longitudinal components are the longitudinal L-sections. The stresses for the 4a models under symmetric vertical loading are plotted in Figure 6-37. The influence of the presence of the glass plates differs per longitudinal component (the maximum Von Mises stress in the longitudinal L-sections in the 4a_none model lies between 141 and 516% of the stress in the 4a_putty model over all load cases, while it is 183% for the symmetric vertical load case illustrated in Figure 6-37). The stresses in the longitudinal bars are the only ones that increase when glass plates are added to the model (the maximum Von Mises stress in the longitudinal bars in the 4a_none model is only 61% of the stress in the 4a_putty model).

Stability of the structure

The presence of the glass plates has an influence on the buckling modes that occur in the structure. In the models without glass plates, only global eigenmodes occur for the first five eigenvectors with all eigenvalues close to each other. The eigenvalues of the models including glass plates are only higher when the eigenmode changes from global to local buckling mode. This is illustrated in Figure 6-39 for the symmetric vertical load case: the first eigenvalue equivalents a global buckling mode and is barely influenced by the presence of the glass plates, but the second and third eigenvalue are much higher for the models including glass plates.

6.2. Influence of the connection stiffness

Deformations

The influence of the connection stiffness on the vertical deflection of the arches is not negligible, however depends on the load case. The influence can be seen in the different series in Figure 6-31 and Figure 6-32. For the most onerous load cases, the influence is proportionally smaller than for the less onerous load cases (the maximum vertical deflection in the 4a_low model is 104% of the deflection in the 4a_putty model for the asymmetric perpendicular load case, while it is 107% for the symmetric concentrated load case).

The relative deformation of the adhesive strips is to a higher degree affected by the connection stiffness. Figure 6-33 and Figure 6-34 illustrate that the relative deformation decreases with increasing connection stiffness (the maximum relative deformation of the adhesive strip for the 4a_low model is 343% and for the 4a_high model is 84% of the relative deformation for the 4a_putty model under the symmetric concentred load). The influence is the highest for the symmetric perpendicular load case (between 80 and 409% of the 4a_putty model).

Stresses in iron components

The maximum Von Mises stresses in the arches slightly influenced by the connection stiffness. The differences are shown for the asymmetric perpendicular and symmetric vertical load case in respectively Figure 6-35 and Figure 6-36. The Von Mises stresses of the putty and high connection stiffness models lie close to each other for all load cases (the maximum Von Mises stress in the arches in the 4a_high model is 98% of the stress in the 4a_putty model for both the asymmetric perpendicular and symmetric vertical load case). The difference with the models with the low connection stiffness is slightly higher (the maximum Von Mises stress in the arches in the 4a_low model is 105% of the stress in the 4a_putty model for both the asymmetric perpendicular and symmetric vertical load case).

The maximum Von Mises stresses in the longitudinal components are to a higher degree influenced by the connection stiffness. The higher the connection stiffness, the lower the stresses in the longitudinal components, except for the longitudinal bars (the maximum Von Mises stress in the longitudinal L-sections in the 4a_low model is 117% and for the 4a_high model is 92% of the stress in the 4a_putty model for the symmetric vertical load case).

Stability of the structure

Both the eigenvalues and the eigenmodes of the models with high and putty connection stiffness are very similar for all load cases (Figure 6-39). The models with the low connection stiffness exhibit less chance for a local glass buckling mode with a lower eigenvalue as a consequence, but the impact is different per load case.

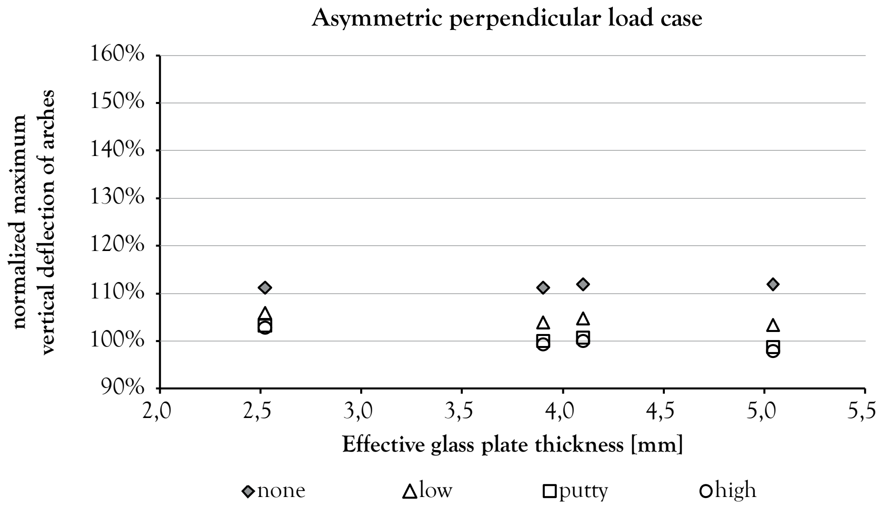

Figure 6-31: Maximum deflection of the arches for the asymmetric perpendicular load case (normalized for 4a_putty model)

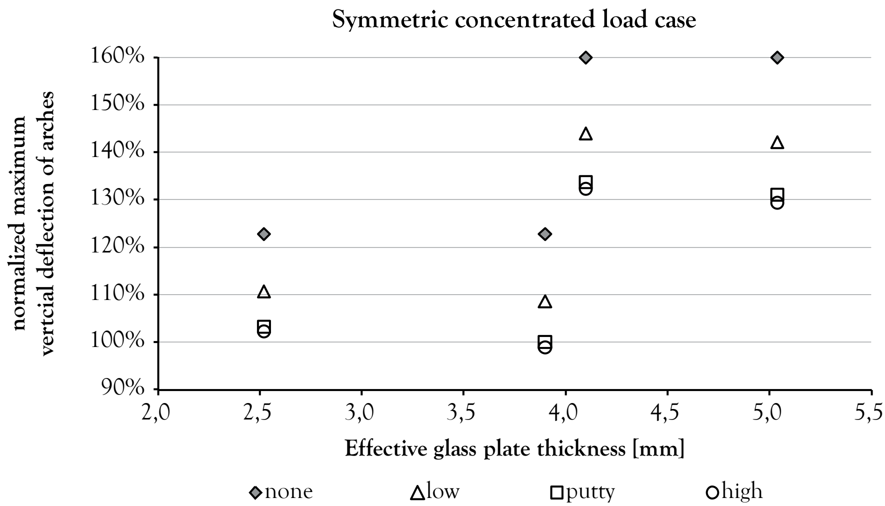

Figure 6-32: Maximum deflection of the arches for the symmetric concentrated load case (normalized for 4a_putty model)

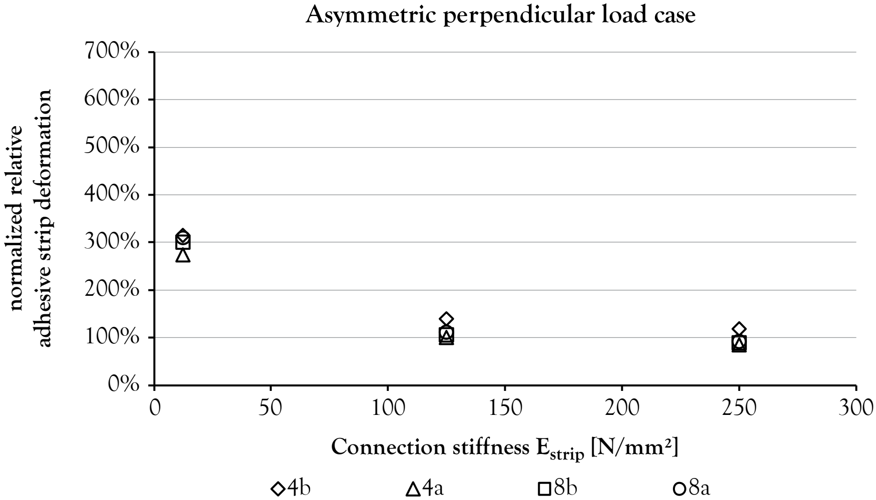

Figure 6-33: Relative deformation of the adhesive strips for the asymmetric perpendicular load case (normalized for 4a_putty model)

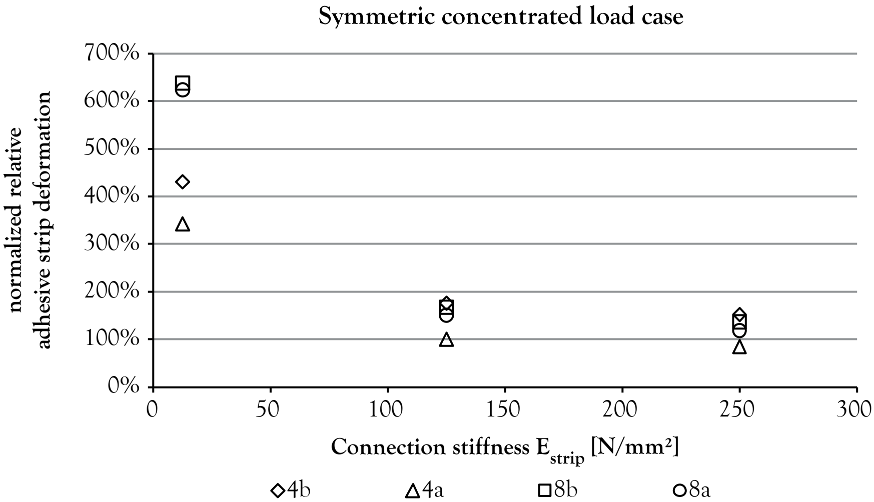

Figure 6-34: Relative deformation of the adhesive strips for the symmetric concentrated load case (normalized for 4a_putty model)

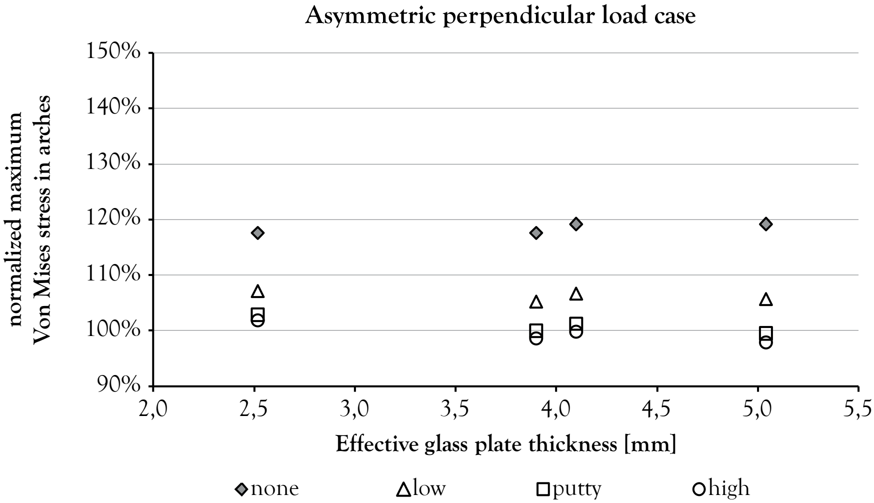

Figure 6-35: Maximum Von Mises stress in the arches for the asymmetric perpendicular load case (normalized for 4a_putty model)

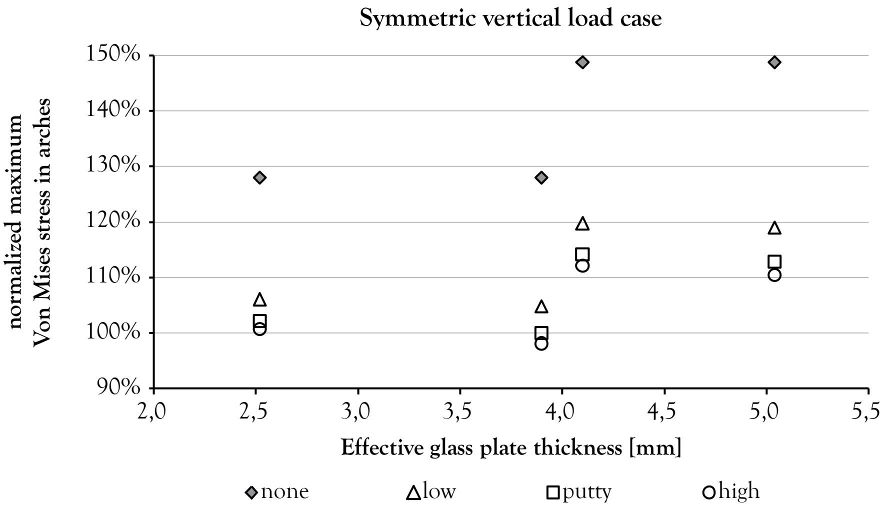

Figure 6-36: Maximum Von Mises stress in the arches for the symmetric vertical load case (normalized for 4a_putty model)

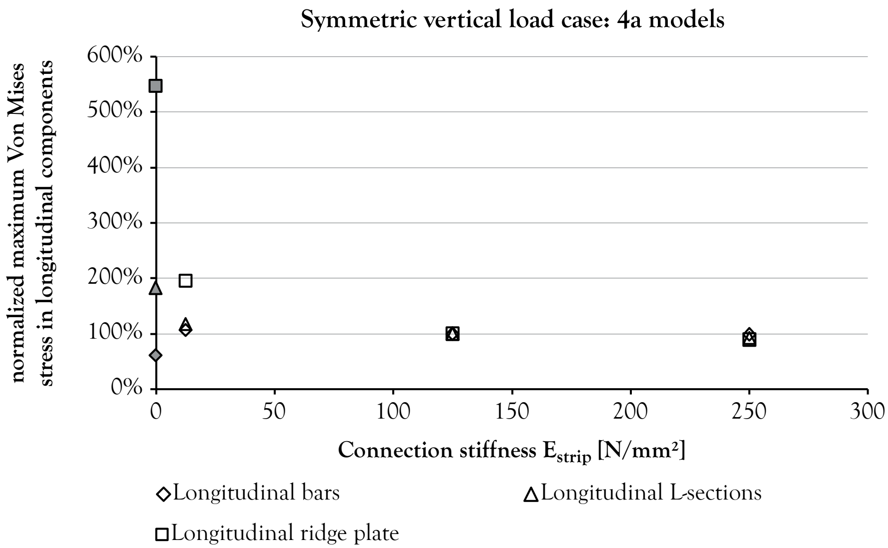

Figure 6-37: Influence of connection stiffness on maximum Von Mises stress in the longitudinal iron components for 4a models for the symmetric vertical load case (normalized for 4a_putty model)

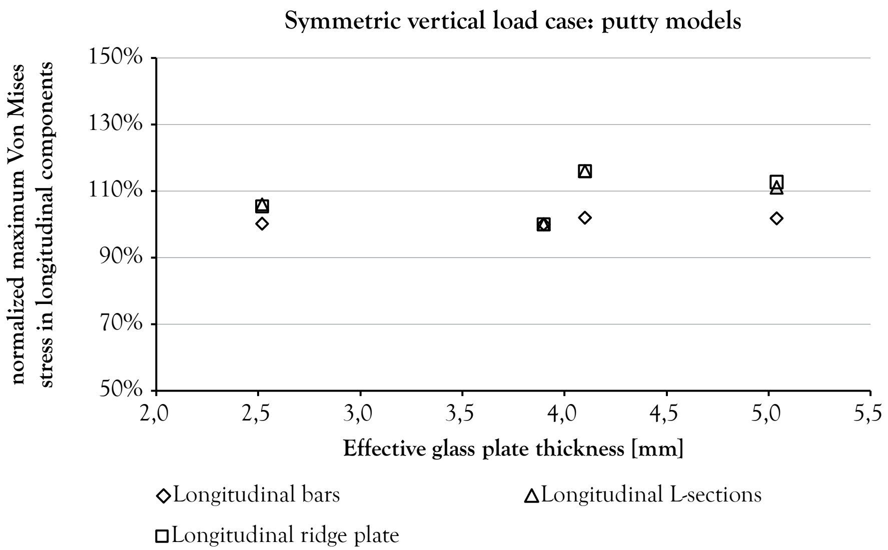

Figure 6-38: Influence of glass thickness on maximum Von Mises stress in the longitudinal iron components for putty models for the symmetric vertical load case (normalized for 4a_putty model)

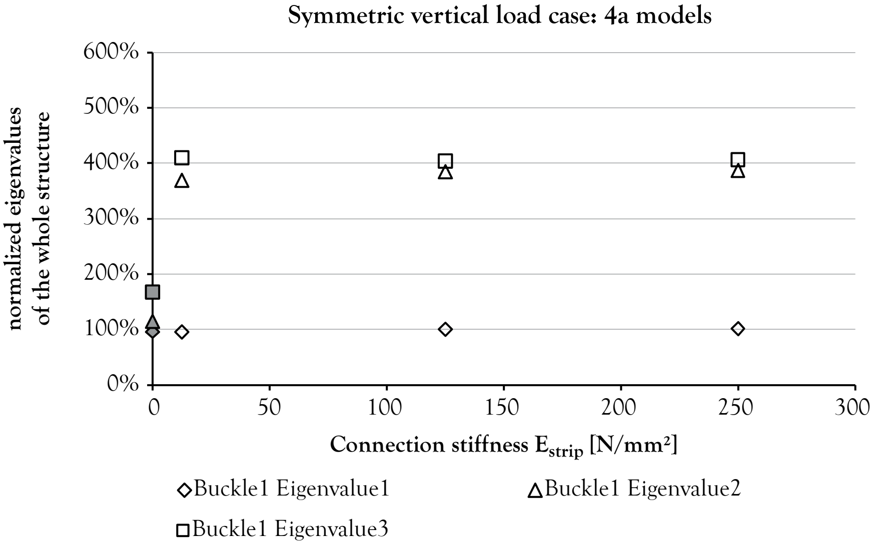

Figure 6-39: Influence of connection stiffness on absolute value of eigenvalues of the first buckle step for 4a models for the symmetric vertical load case (normalized for first eigenvalue in 4a_putty model)

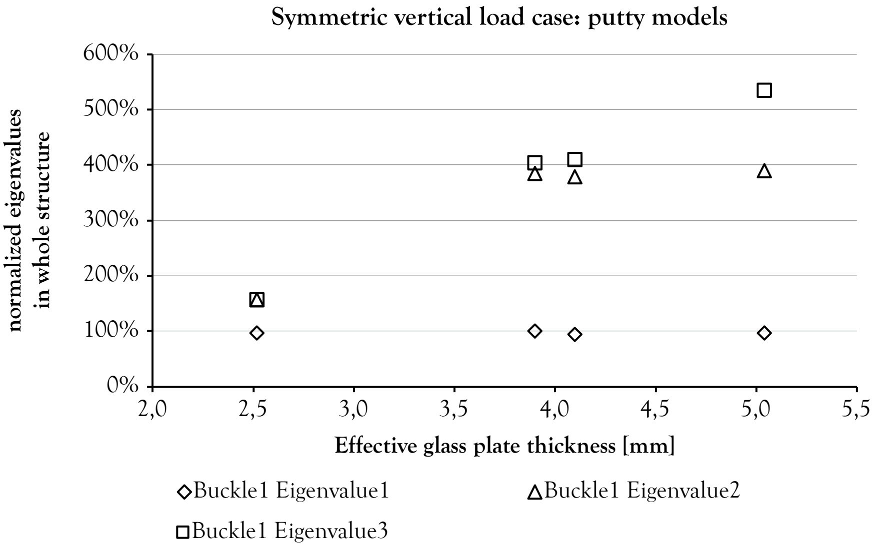

Figure 6-40: Influence of glass thickness on absolute value of eigenvalues of the first buckle step for putty models for the symmetric vertical load case (normalized for first eigenvalue in 4a_putty model)

6.3. Influence of glass plate thickness: stiffness and weight

The glass plate thickness involves two separate parameters: the effective thickness of the glass plate (defining the structural stiffness of the plate) and the real thickness of the glass plate composition (defining its weight). By considering four glass plate thicknesses, these parameters can be studied separately. In the graphs, the effective thickness is plotted on the X-axis. Two model series have the same effective thickness of 4 mm (4a and 8b model) while two pairs of model series have the same glass weight (models 4a and 4b carry 4 mm thick glass plate composition and models 8a and 8b carry 8 mm thick glass plate compositions).

Deformations

The maximum vertical deflection of the arches is only slightly influenced by the glass weight and stiffness for the most onerous load cases. However, for the less onerous load cases, the glass weight has an increased influence. This is illustrated for the asymmetric perpendicular and symmetric concentrated load case in Figure 6-31 and Figure 6-32 (the maximum vertical deflection in the 8b_putty model is 101% of the deflection in the 4a_putty model for the asymmetric load case, while it is 133% for the symmetric concentrated load case).

The relative deformation of the adhesive strips is affected by the glass weight and stiffness dependent on the load case. The influence for two load cases is illustrated in the different series in Figure 6-33 and Figure 6-34. For the most onerous load cases, the influence is lower than for the less onerous load cases (the maximum relative deformation of the adhesive strips for the 8b_putty model is 106% of the relative deformation for the 4a_putty model for the asymmetric perpendicular load case, while it is 167% for the symmetric concentrated load case).

Stresses in iron components

The maximum Von Mises stresses in the arches are illustrated for two load cases in Figure 6-35 and Figure 6-36. The influence of the glass stiffness on the maximum Von Mises stresses in the arches is small (the maximum Von Mises stress in the arches in the 4b_putty model is maximum 103% of the stress in the 4a_putty model over all load cases). The weight of the glass has an impact, but only for the less onerous load cases (the maximum Von Mises stress in the arches in the 8b_putty model is 101% of the stress in the 4a_putty model for the asymmetric perpendicular load case, while it is 114% for the symmetric vertical load case).

A similar influence can be observed for the maximum Von Mises stresses in the longitudinal components. The glass weight has an impact for the less onerous load cases, illustrated in Figure 6-38 (the maximum Von Mises stress in the longitudinal L-sections in the 8b_putty model is 116% of the stress in the 4a_putty model).

Stability of the structure

The buckling analysis pointed out that the glass plate thickness might affect the buckling modes. The models with the thinnest effective glass thickness (4b models) have more chance to experience local glass buckling, but only when the connection stiffness is above a certain level (putty and high stiffness connection models) to transfer enough loads to the glass plates (Figure 6-40).

7. Model of galleries under horizontal loading case

The parameter study based on the Saint-Hubertus Galleries was extended with one series of models where the horizontal buttressing of the end walls of the gallery was neglected. A horizontal surface load was applied that was a simulation of a wind load on the side surface enclosed by the final arc and lanterneau.

7.1. Nominal results

Deformations

The deformation of the 4a_putty model under horizontal loading is illustrated in Figure 6-43. The horizontal displacement (in negative Z-direction) takes place mainly by a deformation of the central part of the arches where no glass stiffens the structure. The maximum horizontal displacement of the arches is 30.3 mm in the 4b_low model. This is a deformation which is unacceptable when compared to the height of the roof: the height of the roof is 4.41 m which means that the horizontal deformation of 30.3 mm is more than 1/150 of the height of the roof.

The relative deformations of the adhesive strips reach a maximum of 0.06 mm in the 8b_low model, which is only 2.9% of the maximum allowable deformation of 2 mm.

Support reactions

The reaction moments in the bottom hinges around the X-axis (horizontal axis across the gallery) are much higher in the models under horizontal loading than under the other load cases. A maximum moment of 42254 Nmm occurs in the 4b_low model (Figure 6-41), which means a force of 4225 N over a width of 10 mm. This is a large force for the small hinge as executed (Figure 6-4). The resistance of the hinges against this reaction moment has to be verified.

Stresses in iron components

The maximum Von Mises stresses in the arches and in the longitudinal L-sections exceed the maximum allowable stress of 235 N/mm² in the models without glass plates. The stresses in the longitudinal components are the highest in the longitudinal L-sections for all models.

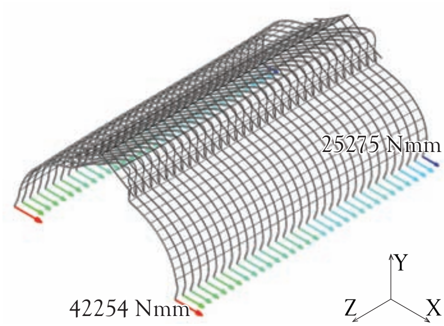

Figure 6-41: Reaction moments at the bottom hinges for the 4b_low model under horizontal loading

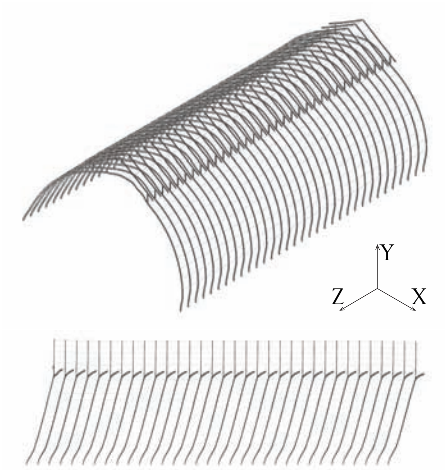

Figure 6-42: Deformed structure of 4a_none model under horizontal load (scale 25)

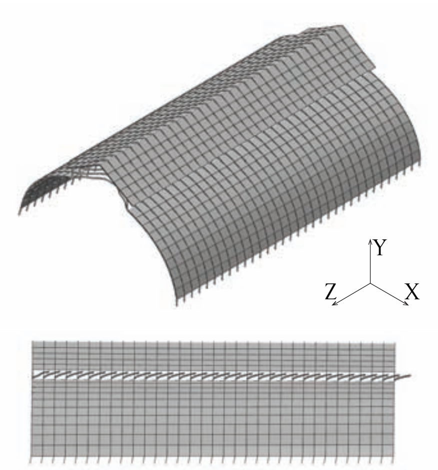

Figure 6-43: Deformed structure of 4a_putty model under horizontal load (scale 25)

7.2. Influence of the presence of the glass plates

Deformations

The influence of the glass plates on the structural behaviour can be illustrated by comparing the deformed structure of the 4a_none and 4a_putty models (Figure 6-42 and Figure 6-43). The maximum horizontal deformation of the 4a_none model measures 110 mm and is a result of a continuous inclination of the whole structure. The glass plates prove to have a major stiffening contribution for the horizontal load case, which is also illustrated in Figure 6-46 (the maximum horizontal deformation of the arches in the 4a_none model is 401% of the deformation in the 4a_putty model).

Stresses in iron components

The presence of the glass plates has a major influence on the maximum Von Mises stresses in the arches. The stresses are more than halved when the glass plates are included, which is illustrated in Figure 6-47 (the maximum Von Mises stress in the arches in the 4a_none model is 252% of the stress in the 4a_putty model).
The influence of the presence of the glass plates is even higher for the stresses in the longitudinal L-sections, illustrated in Figure 6-49 (the maximum Von Mises stress in the longitudinal L-sections in the 4a_none model is 455% of the stress in the 4a_putty model).

Stability of the structure

The eigenvalues of the models barely differ between the models with and without glass plates. All buckling modes are equal (local buckling of the iron arcs in their central part where no glass is present to stiffen the arcs) and the eigenvalues are almost equal (the eigenvalues of all none models are 99% of the eigenvalues of their equivalent model with glass plates).

7.3. Influence of the connection stiffness

Deformations

The maximum horizontal deformation of the arches is affected by the connection stiffness, as illustrated in the different series in Figure 6-46. The influence of the connection stiffness is nearly equal for all glass plate thicknesses (the maximum horizontal deformation of the arches in the 4a_low model is 111% of the deformation in the 4a_putty model). The deformations of the models with putty and high connection stiffness are nearly equal.
The relative deformation of the adhesive strips is to a higher degree influenced by the connection stiffness, illustrated in Figure 6-48 (the relative deformation of the adhesive strips in the 4a_low model is 185% of the relative deformation in the 4a_putty model, while it is 94% for the 4a_high model).

Support reactions

For some models, a vertical reaction force acting in the negative Y-direction occurs. The higher the connection stiffness, the less negative the vertical reaction forces are (Figure 6-44 and Figure 6-45). The negative vertical reaction force only occurs in the final arch on which the horizontal load is introduced.

Stresses in iron components

The influence of the connection stiffness on the maximum Von Mises stresses in the arches in small (Figure 6-47). The stresses in the models with putty and high connection stiffness are nearly equal. However, the stresses in the models with the low connection stiffness are slightly lower (the maximum Von Mises stress in the arches in the 4a_low model is 97% of the stress in the 4a_putty model).

The maximum Von Mises stresses in the longitudinal iron components are affected by the connection stiffness, illustrated in Figure 6-49. The impact on the longitudinal ridge plate is the highest however less relevant because of the low stress levels in these components. The highest stress levels are reached in the longitudinal L-sections on which the connection stiffness has still a significant influence (the maximum Von Mises stress in the longitudinal L-sections in the 4a_low model is 128% of the stress in the 4a_putty model, while it is 91% is the 4a_high model).

Stability of the structure

The first five eigenmodes of the structure are equal for all connection stiffnesses, namely a local buckling mode of the arc in the central part where no glass is present. All eigenvalues of all models lie within a deviation range of 0.1%.

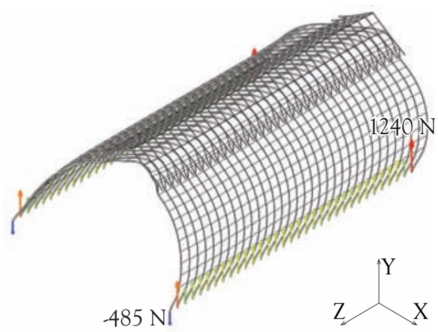

Figure 6-44: Vertical reaction force at the bottom hinges for the 8a_none model under horizontal loading

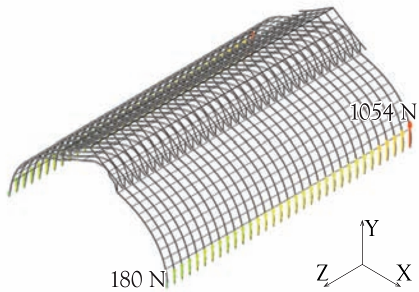

Figure 6-45: Vertical reaction forces at the bottom hinges for the 8a_putty model under horizontal loading

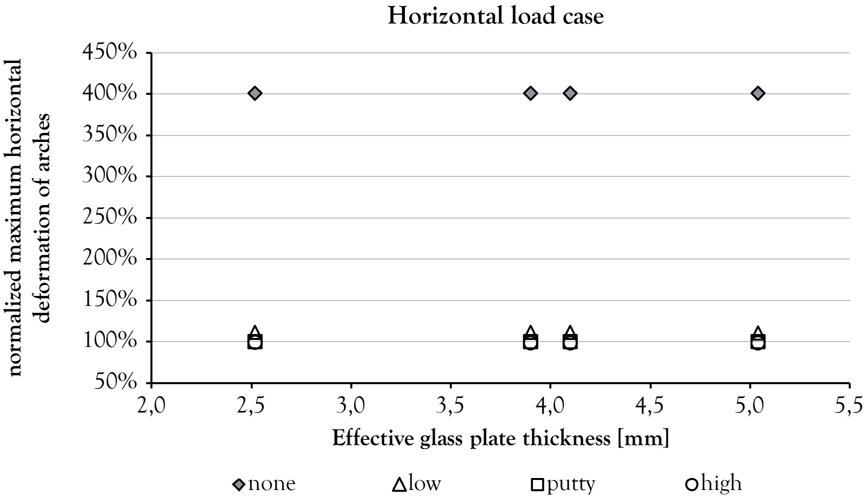

Figure 6-46: Maximum horizontal deformation of the arches
for the horizontal load case (normalized for 4a_putty model)

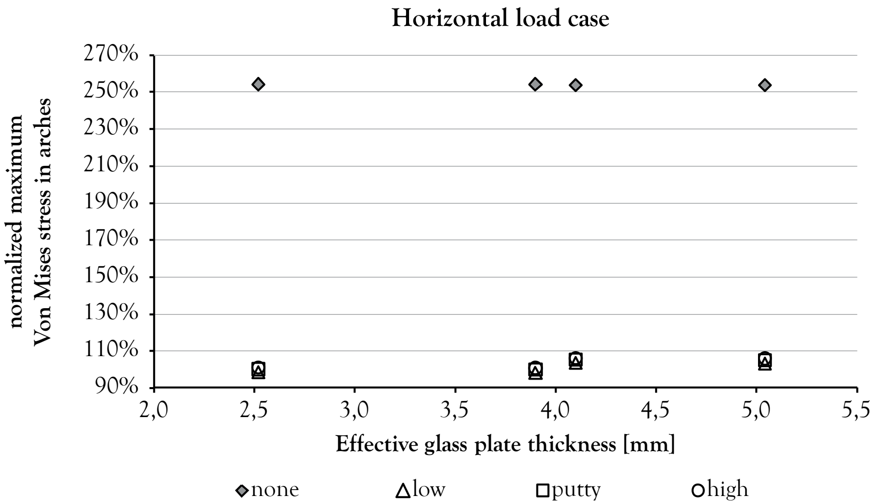

Figure 6-47: Maximum Von Mises stress in the arches
for the horizontal load case (normalized for 4a_putty model)

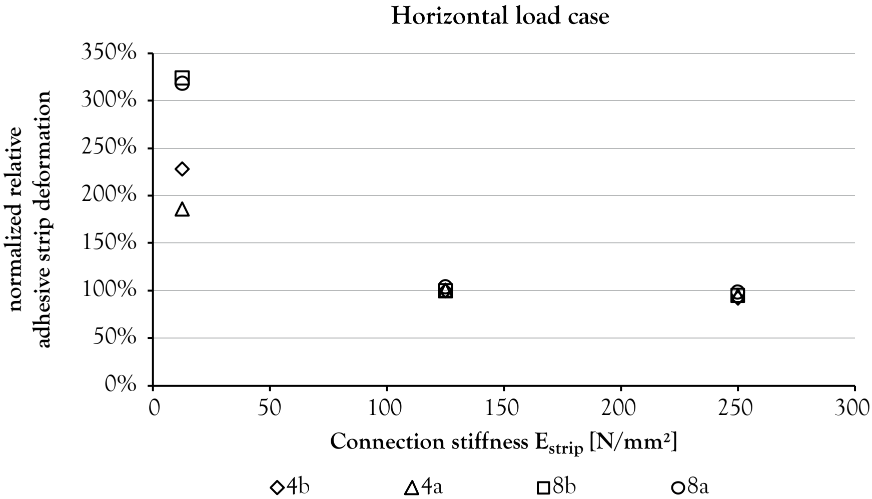

Figure 6-48: Relative deformation of the adhesive strips for the horizontal load case (normalized for 4a_putty model)

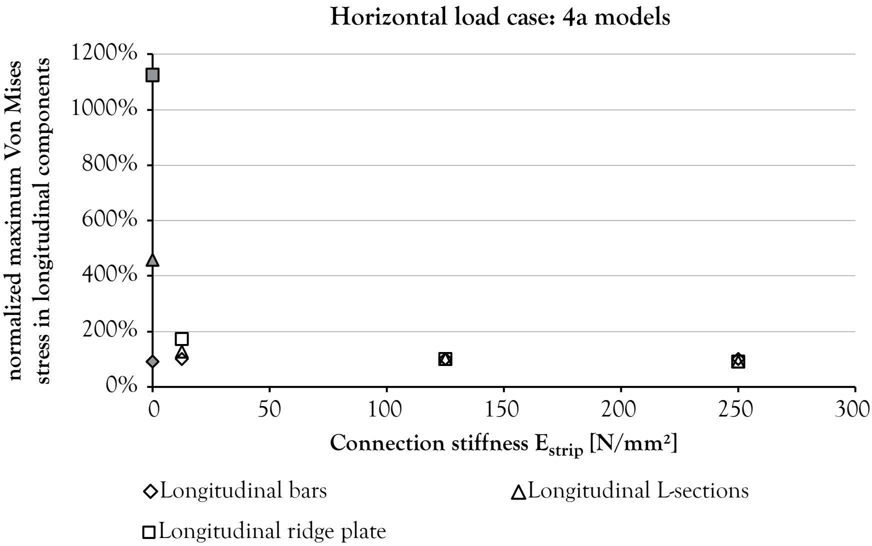

Figure 6-49: Influence of connection stiffness on maximum Von Mises stress in the longitudinal iron components for 4a models for the horizontal load case (normalized for 4a_putty model)

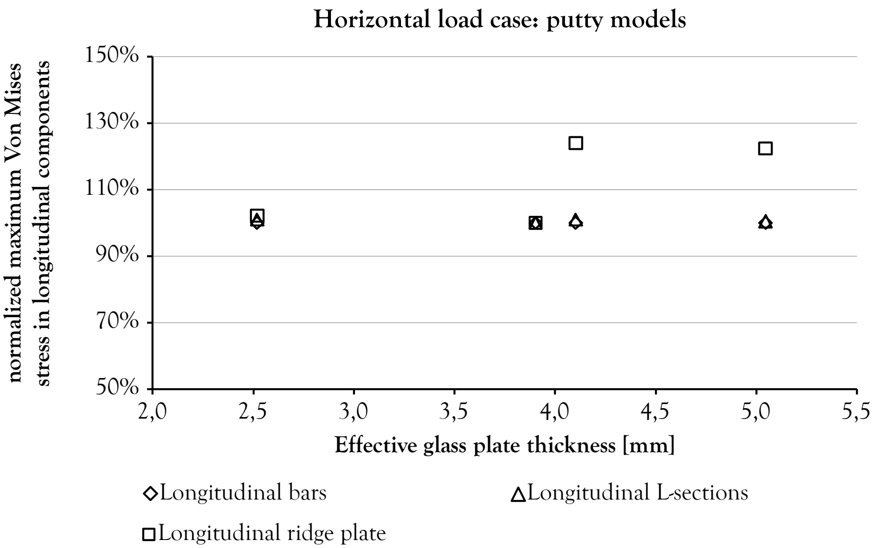

Figure 6-50: Influence of glass thickness on maximum Von Mises stress in the longitudinal iron components for putty models for the horizontal load case (normalized for 4a_putty model)

7.4. Influence of glass plate thickness: stiffness and weight

Deformations

The glass plate thickness has nearly no influence on the maximum horizontal deformation of the arches.

The relative deformations of the adhesive strips are however influenced by the glass thickness for some connection stiffnesses. Figure 6-48 shows that the influence is only significant for the models with low connection stiffness (the relative deformation of the adhesive strips in the 8b_low model is 175% of the relative deformation in the 4a_low model).

Support reactions

The positive vertical reaction forces are obviously higher with a higher glass weight. An increasing glass weight therefore reduces the chance for negative vertical reaction force.

Stresses in iron components

The glass plate stiffness has no impact on the maximum Von Mises stresses in the arches. The glass weight however has a slight influence, for all connection stiffnesses, illustrated in Figure 6-47 (the maximum Von Mises stress in the arches for the 8b_putty model is 105% of the stress in the 4a_putty model).

The maximum Von Mises stresses in the longitudinal iron components are only influenced by the glass plate thickness for the components with low stress rates. For the highest stresses components, the longitudinal L-sections, the influence is negligible (Figure 6-50).

Stability of the structure

The first five eigenmodes of the structure are equal for all glass plate thicknesses, namely a local buckling mode of the arc in the central part where no glass is present. All eigenvalues of all models lie within a deviation range of 0.1%.

8. Conclusions

The Saint-Hubertus Galleries were used as a basis for a parameter study of the structural behaviour of a 19th-century iron and glass roof. The selected parameters were chosen based on the specific historic context: the glass plate composition (with its according stiffness and weight) and the stiffness of the connection between the iron glazing bar and the glass plates. The study was performed under a combination of the self-weight with one of seven live loads, simulating different set-ups of maintenance, snow and wind loads. The quantities of reaction forces and moments, deformations, stresses, and eigenvalues were evaluated.

The influences of three possible interventions on the Saint-Hubertus galleries are listed in Table 6-13 to Table 6-15:

- the influence of including the glass plates into the calculation model;
- the influence of changing the adhesive/sealant of the connection from traditional linseed oil putty to a modern adhesive with a high stiffness;
- the influence of replacing monolithic glass plates by a laminated glass composition that has a total thickness equal to the monolithic glass.

The influences are expressed in comparison with the model simulating the original structure of the Saint-Hubertus galleries with 4 mm thick monolithic glass plates sealed to the iron glazing bars with traditional linseed oil putty (4a_putty model).

Table 6-13: The influence of the presence of the glass plates: the quantity listed in the left column in the 4a_none model expressed as a percentage of the quantity in the 4a_putty model

	asymmetric wind load	other snow, maintenance and wind 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 6-14: The influence of the changing the connection from a putty connection to a connection with an adhesive with high stiffness: the quantity listed in the left column in the 4a_high model expressed as a percentage of the quantity in the 4a_putty model

	asymmetric wind load	other snow, maintenance and wind 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 %
relative deformation of the adhesive strips	84 %	80 – 87 %	94 %
stability of the structure	no difference in buckling behaviour	no difference in buckling behaviour	no difference in buckling behaviour

Table 6-15: The influence of the changing from a monolithic to a laminated glass composition that have the same total thickness: the quantity listed in the left column in the 4b_putty model expressed as a percentage of the quantity in the 4a_putty model

	asymmetric wind load	other snow, maintenance and wind 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 %
relative deformation of the adhesive strips	139 %	127 – 218 %	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

Changing the glass plate composition from monolithic to laminated glass has a small influence on most of the structural behaviour of the Saint-Hubertus galleries. However, it is important to note that the laminated glass composition is more vulnerable for buckling of the glass plate. For example under 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 (paragraph 0 p.167: $\omega = 0$). This is conservative especially for short-term loads like the wind load. The relative deformation of the adhesive strips also increases for all snow, maintenance, and wind loads, however it does not reach the maximum allowable deformation of 2 mm.

In the original structure, the glass plates are often sealed to the 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 of a modern adhesive with a high stiffness (within the range of adhesives that were found appropriate in Chapter 5 paragraph 1.2 p.134) on the structural behaviour is only limited for most quantities. The quantities out-of-plane of the cross-section of the Saint-Hubertus Galleries (the stresses in the longitudinal iron components and the relative deformation of the adhesive strips) can be influenced in all load cases. The critical quantities, the deflection of the arches and the stresses in the arches, are however barely influenced.

The influence of including the glass plates in the calculation model is very clear. The influence on the most critical quantities (the deflection of the arches and the stresses in the arches) under the most onerous load case (the asymmetric wind load) is not enough to lower these quantities under their maximum allowable values, but the influence is more than significant. The impact on the other quantities and for the other load cases might be explicitly higher. The influences under the horizontal wind load are clearly higher.

During renovation studies, the results of this research might be used in two directions. When new glass plates will 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. An assessment methodology for the structural behaviour of 19th-century iron and glass roofs will be proposed in the next chapter.

Chapter 7

Interventions on 19th-century iron and glass roofs

The previous chapter gave an insight in the structural contribution of the glass plates to the structural behaviour of the Saint-Hubertus Galleries. The conclusions that can be drawn for other 19th-century iron and glass roofs will be discussed in this chapter. Structural interventions can however not be seen apart from the heritage value of all components of the roof. An overview of possible interventions will be discussed regarding their historic context, their contribution to fulfilling modern standards on comfort and safety, and their heritage value.

1. Including glass plates in the model of a structural calculation: time consuming or worth the effort?

Only taking the iron frame into account is often the most time efficient way to recalculate a 19th-century iron and glass roof. However, when this 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 influence of the presence of the glass plates on the structural behaviour of the Saint-Hubertus Galleries was discussed in Chapter 6. For the most onerous load case (self-weight combined with asymmetric wind load), the positive influence of including the glass plates in the structural model was low. Nevertheless, the overall deformations and maximum stresses were clearly reduced. Thus, for other 19th-century iron and glass roof structures that show to be slightly overloaded when only modelling the iron frame, including the glass plates in the calculation model might lead to the conclusion that no (or reduced) structural strengthening is necessary. This overloading can be due to an increase of the imposed loads on the structure (e.g. due to increasing glass thickness and therefore weight), due to a severe deterioration of the iron frame (e.g. reduction of iron section due to corrosion), or due to a change in the boundary conditions (e.g. differential settlements). A calculation with the glass plates included can in that case give a more accurate estimation of the real structural behaviour so that the impact of the structural interventions on the historic fabric can be minimised.
Adjusting the glass plate composition or the connection stiffness can have a minor contribution when the overloading is only small or if it is located in some specific components (the positive influence on e.g. the stresses in the longitudinal iron components is much higher than for the arches).

When the overloading of the structure is located at some specific spots, the introduction of glass plates in the calculation model or adjusting the glass plate composition or connection stiffness can positively contribute to solving this overloading. Corrosion at local spots in the iron structure (Figure 7-1), local buckling of iron arches around a quarter of the span, or stress peaks in longitudinal iron components at their connection with the transversal structure, are examples of local phenomena where the glass plates can have a major contribution to the structural performance of these components.
The effect of the glass plates under horizontal wind loading was for the Saint-Hubertus Galleries larger than under the other loads. Research on modern structural glass applications proved the efficiency of glass loaded in-plane (Chapter 3). We can conclude that the contribution of glass plates in the direction of their plane to the overall structural behaviour of 19th-century iron and glass roofs is very valuable.

The effect glass plates can have on the structural behaviour depends also on the geometry of the glass roof. The Saint-Hubertus Galleries are an example of a barrel vault. The contribution of the glass cladding proved to be large for loading acting horizontally, in the longitudinal direction of the barrel shape (in the plane of the glass plates). The same conclusion will probably be valuable for pitched roofs. The contribution of the glass cladding in flat roofs however will presumably be limited to local phenomena. The three-dimensional stiffening effect for double-curved roofs was already proven for a three-dimensional iron frame[1], which enforces the expectation that the impact of the glass plates will be large for cupolas.

Figure 7-1: Section loss at the connection due to corrosion at the Winter Garden in Laeken (2007-03-29)

Figure 7-2: Distorted dome of Glasgow Kibble Palace[2]

[1] Lauriks, De Bouw, and Wouters 2009.
[2] Development and Regeneration Services, Glasgow City Council 2003_2006.

An example of the possible contribution of glass plates on the horizontal load bearing behaviour can be found in the Kibble Palace in the Glasgow Botanical Gardens (case description p.99). The large dome of this glasshouse comprises of a rotunda and a main dome, with a cast iron frame in between which forms a height difference between the two. Both the rotunda and the main dome are supported by cast-iron columns, cast-iron circular frames, wrought-iron glazing bars spanning radially and wrought iron tie rods connecting the glazing bars in concentric direction. The main dome glazing bars got twisted horizontally to S-shaped glazing bars, probably during previous alterations of the glasshouse (Figure 7-2).
For the restoration campaign in 2003-2006, a structural analysis was carried out on the original (non-twisted) geometry. The cast iron lantern on top of the main dome showed to be too heavy for the glazing bars. The lantern was replaced by an aluminium replica to save on weight. The glazing bars were straightened. A master thesis on the structural analysis of the Kibble Palace in 2000 concluded that the wrought-iron tie rods between the glazing bars were overstressed and the main dome got twisted in a calculation of only the iron frame[3]. The tie rods were heavily corroded in the structure before restoration, so the elevated stresses were considered critical in this thesis. The same thesis also carried out a study of including the glass plates in the calculation model, but due to non-realistic assumptions on boundary conditions and modelling method, these results cannot be used.
For the example of the Kibble Palace, including the glass plates in the structural model could have led to a major contribution of the glass plates to the overall structural behaviour of the main dome. The stresses in the iron glazing bars due to the weight of the cast iron lantern could have been lowered, but detailed calculations would be necessary to conclude if this would be sufficient. The effect of the glass plates in the concentric direction however would probably make a major positive difference, both for the stresses in the wrought-iron tie rods and for the twisting effect of the main dome. The structure had found equilibrium in the twisted state, proving that the glass plates in practice did stabilize the main dome[4].

[3] Godinho and Swailes 2000, p.123–125.
[4] Glasgow City Council 1998, p.10.

2. Maintenance of iron and glass roofs

The principle of minimal intervention is important in present-day heritage care and the international charters (Chapter 4). Maintenance of the heritage is a first step to reach this principle. The following paragraphs will give a brief overview of which maintenance interventions can be considered during the restoration of an iron and glass roof.

For the iron components (both the main structure and connection components), maintenance includes removing oxide and dirt layers from the iron sections and afterwards protecting the iron against corrosion (Figure 7-3). Cleaning the iron sections can be carried out in many ways, of which grit blasting is a popular one. However, grit blasting requires a closed space to protect the environment from the resulting dust, and is thus complicated (however not impossible) to apply on-site. Chemical cleaning, water-base cleaning, mechanical cleaning and flame-cleaning can all be applied in varieties of different levels of aggression. A full overview of all cleaning techniques is given by Godfraind et al[5].
The experimental research (Chapter 5) showed that neither the sandblasting neither the painting has a negative influence on the adhesion of the tested adhesive to the substrate. The time between cleaning and treating the iron however can have a negative effect.

Cleaning the iron sections often implies that the glass plates have to be dismantled (Figure 7-4). Moreover, cleaning the glass plates sometimes requires dismantling the connections. The slope of the glass plates determines the effectiveness of automatic cleaning by rainwater. Even with a sufficient slope, the overlap between glass plates is a gathering place for dirt (compare the same glass plate geometries but difference in restoration stages between Figure 7-5 and Figure 7-6).

Maintenance ladders were often incorporated in the original design of the constructions, for example at the Winter Garden of the Royal Glasshouses of Laeken. Nevertheless, the accessibility often poses problems for maintenance and restoration works: a scaffold was built to keep the public street free at the Saint-Hubertus Galleries; the Kibble Palace in Glasgow was emptied during the restoration campaign (Figure 7-7); a temporary closed space was created with textiles to protect the plants at the Winter Garden of the Royal Glasshouses in Laeken (Figure 7-8); etc.

[5] Godfraind et al. 2012, p.196–207.

Figure 7-3: Removing paint layers at Winter Garden in Laeken (2006-11-16)

Figure 7-4: Dismantling glass plates at Winter Garden in Laeken (2006-11-16)

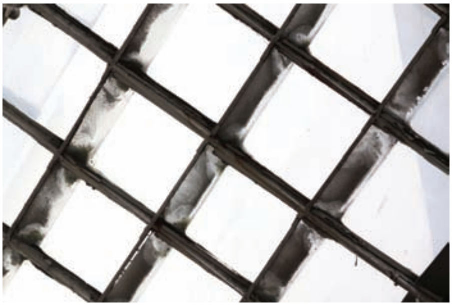

Figure 7-5: Interior view of Prince Gallery of Saint-Hubertus Galleries: original connection detailing (2012-08-10)

Figure 7-6: Interior view of King's Gallery of Saint-Hubertus Galleries: adjusted connection detailing after the 1993-97 restoration campaign (2010-04-22)

Figure 7-7: Emptied Kibble Palace in Glasgow during 2003-06 restoration campaign of Kibble Palace in Glasgow[6]

Figure 7-8: Maintenance works at the Winter Garden in Laeken (2006-11-16)

[6] Development and Regeneration Services, Glasgow City Council 2003_2006.

3. Minimal restoration interventions on iron and glass roofs

Some interventions might be necessary to ensure the conservation and continued use of the construction. Reporting on all conservation works is important to keep track of the authenticity aspects of materials and components.

Due to corrosion, iron cross-sections are sometimes reduced to a level that is no longer sufficient to carry the loads. Repair of decayed iron components is difficult but possible in some specific cases. Cast iron can be metal stitched[7], while existing wrought-iron sections can be repaired by forging with recycled wrought iron (Figure 7-9, this was also applied in the Glasgow Kibble Palace). Distorted wrought-iron components can be straightened by heat treatment[8].

Original glass being present in a 19th-century iron and glass roof is rare, due to glass breakage during the lifespan of the building. The replacement of broken glass panels can be considered an indispensable intervention, to ensure the water tightness of the roof. For double-walled roofs, where the inner glass shell is protected by an outer glazed roof, the chance for original glass is higher (case study Chapter 4 paragraph 5 p.116). Glass that was installed in a later period, although not original, can also hold historic significance.

When the glass plates were taken into account in the structural calculation model, broken glass panels have to be replaced as quickly as possible after their failure. The high number of small glass plates in 19th-century iron and glass roofs however, will give the structure enough redundancy to withstand all loads when one glass plate is broken. Failure scenarios could be included in the structural study if the allowable number of broken glass plates has to be determined.

The repair and/or replacement of connection components can be part of the minimal interventions on an iron and glass roof. For example, the replacement of lost rivets and bolts can be indispensable for the structural safety of the roof.

The experimental research on linseed oil putty (Chapter 5 p.145) revealed that the stiffness of the putty is high enough to have a positive effect on the structural behaviour. The experiments showed however that this stiffness is only reached after a time period of more than a month. Thus, the lower stiffness in the first months after installing the glass plates should be followed-up during supervision of the works.

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[7] Tilly, Frost, and Wallsgrove 2002, p.185–187; Godfraind et al. 2012, p.212–213.

[8] Tilly, Frost, and Wallsgrove 2002, p.189.

Figure 7-9: Repair of wrought-iron sections by using recycled material[9]

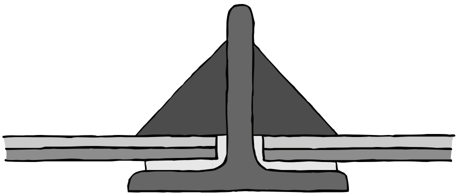

Figure 7-10: Interventions on standard T-shaped glazing bar: laminating historic glass (darker grey at bottom side) with new glass plates; replacing putty (partly or completely) by modern sealant or adhesive

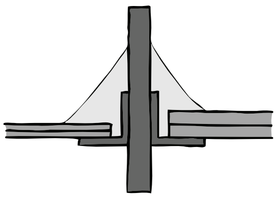

Figure 7-11: Different proportions of glass plate thicknesses compared to the iron glazing bars when applying laminated glass in historic connection detail: (left) 2 panes of 2 mm thickness; (right) 2 panes of 4 mm thickness

4. Interventions to fulfil modern standards on safety

Original single glass in roofs poses safety problems in publicly accessible buildings. When single glass stays in place, the accessibility of the building is limited and thus the function needs to be adjusted to the limited safety level. For example, the venue hire contract of the Kibble Palace in Glasgow mentions that the Glasgow City Council can cancel an event when high winds or heavy rainfall occurs. The Victoria Falls heritage charter (Chapter 4) mentions the possibility of reducing the safety level for historic constructions:

[9] Swailes, Watson, and Dakin 2006, p.150.

> *"Often the application of the same safety levels as in the design of new buildings requires excessive, if not impossible, measures. In these cases specific analyses and appropriate considerations may justify different approaches to safety."* [10]

Modern standards oblige to apply laminated glass for overhead glazing (Chapter 3 paragraph 2.2 p.89). An original glass plate can be laminated to a new glass plate with a resin in between the panes (case study paragraph Chapter 45 p.116). However, this process is irreversible. In that case, the historic glass will be installed at the side which is most visible for the public (Figure 7-10).
The installation of new laminated glass plates is a plausible intervention when no historic glass is present. Restoration glass, which is new glass produced according to the traditional production processes, is available for application in restoration works[11]. The compatibility of the laminate material (PVB, SG, ...) of laminated glass with the connection sealant or adhesive has to be guaranteed. When the thickness of the whole glass plate composition increases, the proportion of the glass plate thickness compared to the dimensions of the glazing bar has to be checked. It gets out of proportion, when for example at the Saint-Hubertus Galleries a laminated glass plate composition of 2 panes of 4 mm thickness is placed onto the L-shaped iron glazing bar of 15 mm high (Figure 7-11). During the 1993-97 renovation campaign of the Galleries, laminated glass of two panes of 2 mm thickness was installed.

5. Interventions to improve the maintenance requirements

Regular maintenance is important to conserve a structure. Some interventions during a restoration campaign might decrease the required maintenance intensity. The necessary interventions depend also on the typology of the iron and glass roof: a double-walled roof for example might be easier to access in between the two glass layers. For all roofs however, the implications on the heritage value have to be considered in combination with the maintenance gain.

[10] ICOMOS 2003a, article 2.8.
[11] webpage Van Ruysdael 2011; webpage Schott 2012.

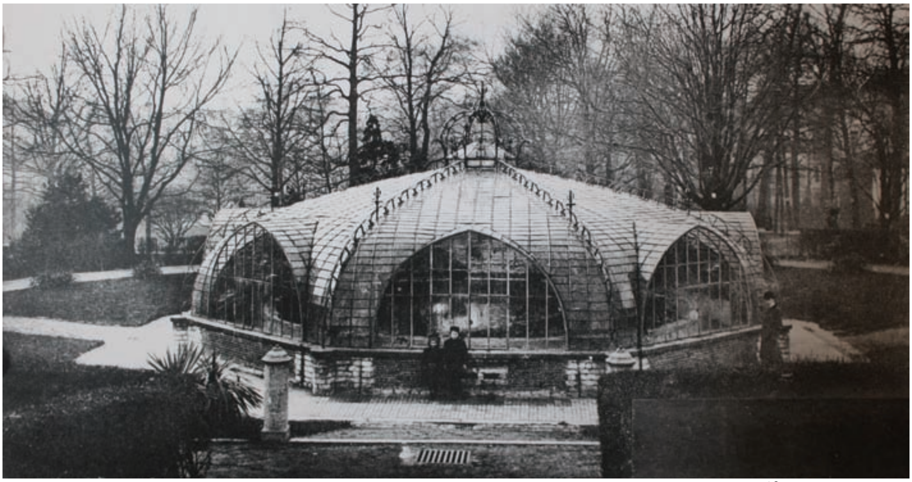

Figure 7-12: Victoria Regia House in 1910: small glass plates[12]
(1854, Meise, Balat)

Figure 7-13: Victoria Regia House in 2008: larger glass plates
and two very long glass plates near the centre
(2008-10-28)

Installing larger glass plates decreases the number of longitudinal joints and thus decreases the chance for leakages and heat loss. However, the appearance of a 19th-century iron and glass roof is among others defined by the scaly cladding (compare the glass plates in Figure 7-12 and Figure 7-13). Changing the length and width of the glass plates is therefore unacceptable for buildings with cultural value.

[12] AAM. s.d.

The transverse connection of an iron and glass roof can be accomplished by putty or by an extra section clamping the glass (Figure 2-9 p.61). Traditional putty has to be repainted regularly to prevent it from drying out. Once the linseed oil dried out, the putty starts cracking (e.g. due to differential thermal dilatations) and cannot maintain the water tightness of the connection, thus needs local replacement.

Preserving the traditional connection method and traditional materials (e.g. linseed oil putty, copper hooks) during a restoration campaign is preferred. In some cases (e.g. due to compatibility issues between putty and laminate resin, due to increase of weight of glass as a results of the lamination, due to a lack of accessibility of the construction, etc.), larger adjustments are necessary. The linseed oil putty can be partly or as a whole replaced by a modern sealant or adhesive material, so that the overall joint geometry is preserved (Figure 7-10). Modern sealants and adhesives however are not always accepted in protected monuments. For instance, problems with diffusing silicone oils through natural stones in the Cologne cathedral after restoring stained glass using silicone sealants, increased the scepticism about modern sealants and adhesives for heritage.

Changing the principle of the connection between the iron glazing bar and the glass plates often has the aim to exclude the use of any sealant or adhesive. It is another way of avoiding the use of traditional linseed oil putty to reduce the maintenance cost after the renovation. From a structural point of view, the stiffness in the different directions can in this case be defined more independently than for an adhesive connection. During the 1993-97 renovation campaign of the King's and Queen's Gallery of the Saint-Hubertus Galleries, the original putty connection was replaced by covering sections in stainless steel (Figure 7-6). The limited visibility and the difficult accessibility of the glass roof due to its high height, presumably contributed to the acceptance of this intervention.

6. Interventions to increase the structural loading capacity of the construction

To strive for minimal interventions like specified by the international charters, a first step in the structural assessment of iron and glass roofs is to include the glass plates into the structural model. The impact of this integration was discussed in paragraph 1. When the extended structural assessment points out that it is necessary to strengthen the iron frame, different intervention strategies are possible.

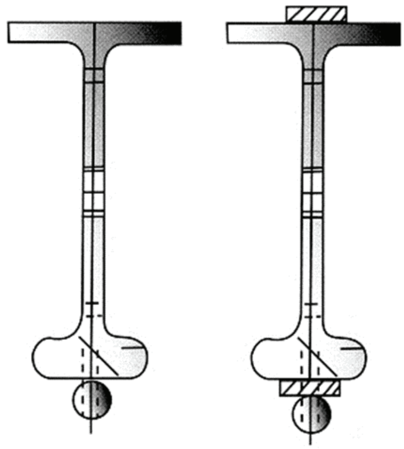

Figure 7-14: Palm House at Kew Gardens: adding sections to original cross-section to strengthen the wrought-iron arches during the 1985-88 restoration campaign[13]

Figure 7-15: Palm House at Kew Gardens: new section in top of wrought-iron I-section[14]

The span of the primary iron components can be lowered by inserting additional main members in between existing members. This is a (partly) reversible and at the same time recognizable intervention. However, for glass roofs the applications are limited due to the large visual impact on the transparency of the glazed building envelope.

The original iron components can be strengthened themselves by adding extra material (e.g. bolting extra sections on top or adding a fibre reinforced laminate). The rectangular sections that were added to the main wrought-iron arches during the 1985-88 restoration campaign of the Palm House in Kew Gardens, London (Chapter 1 paragraph 1.2 p.19), are an example of how the load-bearing cross-section can be increased (Figure 7-14 and Figure 7-15)[15]. Connecting these new sections with the existing ones can be executed by welding (when the historic iron is weldable) or by

[13] Nieuwmeijer 2008.
[14] Ibid.
[15] Guthrie, Allen, and Jones 1988, p.1161; Minter 1990, p.168.

screwing or bolting (with the consequence of damaging the historic fabric by adding holes).
Another option is to change the stress regime of the structure, e.g. by adding tie rods to an arched roof or by changing the supports of the structure. For example during the renovation campaign of the Saint-Hubertus Galleries in 1993, the option of strengthening the hinged supports towards fixed supports was included[16]. A calculation was performed on a two-dimensional model of one frame and concluded that the stresses in the arch were lower for fixed connections. During the renovation however, the connections were not altered. The real connection detail will probably lie somewhere in between a hinge and a fixed connection: it is clearly designed as a hinge, but due to corrosion the resistance against rotation is probably increased.

7. Conclusions

To preserve our built heritage, the aim of a restoration campaign of a 19th-century iron and glass roof should be minimal interventions. The simulation of the structural behaviour by including the glass plates in the calculation model, can lead to a reduction of the necessary interventions to fulfil the modern standards on structural integrity. The overall geometry of the roof defines the possible contribution of the glass cladding: the contribution in the plane of the glass plates is considerably higher than in the other direction, while at the same time some local overloading phenomena or local buckling can be less qualifying for the structural assessment.
Apart from the structural assessment, interventions on an iron and glass roof can also be guided by the condition of the structure, by modern standards on safety, and/or by reducing the maintenance intensity. The necessary interventions depend on the typology of the glass roof: a double-walled roof separates the requirements for weather-proofing from the decorative aspects; the height of the roof can define the visibility of the interventions; the accessibility determines the ease of maintenance works; the heritage value of the glass plates and safety requirement determine whether laminated glass will be applied; etc. Striving for minimal and reversible interventions is however crucial.

[16] T.C.A., CEWAC, and OREX 1996, Etude de stabilité, p.2.

Chapter 8

Conclusions and further research

This research focused on the renovation of 19th-century iron and glass roofs and which contribution the glass cladding could have to the structural assessment, taking into account the heritage value of the roof and its components. The development of the iron and glass architecture started in the beginning of the 19th century with the large scale production of Leblanc soda (invented in 1787-1793 by Frenchmen Nicolas Leblanc) and the invention of the wrought iron glazing bar by John Claudius Loudon (first published in 1817). The studied period ends in the beginning of the 20th century, with the start of mechanical production of glass (with the first Belgian patent of Fourcault in 1901), the First World War which disrupted the iron industry, and the start of writing national standards for the building industry (from 1919).

When renovating 19th-century iron and glass roofs, they have to fulfil the modern standards of comfort, safety and structural integrity. The glass cladding has an important role to play: the glass cladding composition could have an influence on comfort and safety (with the use of double and/or laminated glass), and the glass could have a contribution to the overall structural behaviour. To account for the latter, calculation models have to simulate the composite action of both iron and glass. Including the glass plates in the simulation demands some time investment. This research intended to identify for which cases and under which conditions this time investment is appropriate.

1. The structural behaviour of 19th-century iron and glass roofs

The overall structural behaviour of a 19th-century iron and glass roof was simulated and a parameter study was performed. The geometry of the Saint-Hubertus Galleries served as a basis for all calculation models.

Including the glass plates into the structural model showed to be appropriate when only slight overloading was examined. For structures that exceed the allowable stresses and deflections to a higher extent in all iron components (due to an increase in the imposed loads, a change in the support conditions, reduced sections due to corrosion, etc.), including the glass plates in the model could limit the necessary interventions. Replacing the putty by a modern adhesive with higher stiffness could help reduce some specific local overloading problems (e.g. stresses in the longitudinal iron components). Replacing single by laminated glass had only minor influence, however the glass weight is a parameter to watch carefully. When the calculation of the iron frame highlights some local overloading, including the glass plates in the model could make a major difference. Examples are the local buckling of the iron frame or stress peaks in the longitudinal iron components. The contribution of the glass plates was also higher for loading in the plane of the glass plates (under the horizontal load case).

The effect that the glass plates can have on the structural behaviour depends on the geometry of the glass roof. Flat or inclined roofs, pitched roofs, single or double curved roofs all behave differently considering their structural behaviour. The Saint-Hubertus Galleries are an example of a barrel vault. Further research should clarify if the conclusions from the Galleries can be extended to other geometries: is the effect for flat roofs negligible, how can the glass contribute to the longitudinal stability of pitched roofs, and to what extent is the three-dimensional stiffening effect of cupolas also applicable to the glass cladding?

In this research, the stiffness of the connection detail was estimated by an approximate analytical method. Further research could focus on a more accurate method to determine the stiffness of connection details between iron glazing bar and glass plates. By experimental investigation, the stiffnesses in all directions could be determined, for example for a puttyless glazing system.

The simulations proved that the contribution of the glass cladding to the overall structural behaviour can reduce the necessary interventions on 19th-century iron and glass roofs. The other parameters that define the restoration strategy of a 19th-century iron and glass roof, can now be looked at more in detail.

2. Mechanical properties of adhesive connections in historic structures

In 19th-century iron and glass roofs, glass plates were placed on the iron glazing bars using traditional linseed oil putty. Experimental research showed that linseed oil based putty can have significant compression stiffness, so forces can be transmitted between iron glazing bar and glass plate. Modern sealants are developed by the adhesive manufacturers that have comparable viscosity and texture parameters as linseed oil putty, but stay flexible for a longer time. These sealants can have higher shear stiffness than linseed oil putty, but the reliability of the results is also higher. The influence of parameters specific for the on-site execution of adhesive bonding on historic iron was investigated on modern adhesives. Adhesives are explicitly developed for structural applications in contrast to sealants, therefore the shear stiffness of adhesives is in general higher. Grit blasting the substrates, resulting in different surface roughnesses, showed to have only minor influence on the shear strength of the adhesive. The same conclusion can be drawn for the application of an epoxy paint layer. However, the shear strength of a bond on mild steel exposed longer to air was lower than the shear strength when the paint layer or adhesive bond was applied within a short time period.

The structural calculations demonstrated that the traditional putty achieves sufficient stiffness to let the glass cladding contribute to the overall structural behaviour. Further research about the traditional sealant materials and their mechanical properties would broaden the technical knowledge on historic materials.

In a renovation context, it is barely impossible to apply a paint layer or an adhesive bond quickly after cleaning the surface by grit blasting. Further research on the influence of surface corrosion on the adhesion strength to historic iron and steel would complement the results obtained from this study. Primers could have a major contribution to the adhesion on historic iron and steel (e.g. paint varieties that are developed that can be applied on heavily corroded surfaces). The parameters of the specific heritage context could be expanded to include the influence of adhesive bond thickness and its non-uniformity. Stiff adhesives, traditionally limited by their manufacturers to thin adhesive bonds, are also interesting to investigate for their properties in thicker bonds.

3. Heritage value of construction techniques

The construction techniques of 19th-century iron and glass constructions contribute to the specific historic value of these buildings. The slender iron glazing bars covered with a high number of small overlapping glass plates define its distinguishable look. The construction techniques were investigated based on manuals and course books from 1847 until 1919, giving an overview of the knowledge diffusion via printed publications on how to construct these iron and glass buildings. In the 19th century, glass was sold by its weight per square meter, which can be roughly translated to the thickness of the glass plates, and names were assigned per weight class. *Verre double* was prescribed for use in glass roofs, defined as glass of 3 to 4 mm thickness. The definition of *verre double* did not change over the investigated period, probably due to the sufficient thickness needed to resist weather conditions for glass in roofs. In Belgium, this glass was placed on T-shaped or compound iron glazing bars. The glass was then sealed to the glazing bars with linseed oil putty. Systems for puttyless glazing were developed to avoid the intensive maintenance of putty connections, however were only rarely used in Belgium. The glass plate thicknesses, the geometry of the iron glazing bar and the use of putty for sealing the connections was confirmed by all investigated Belgian case studies.

This overview of 19th-century construction techniques makes it possible to evaluate the heritage value of the connections in a 19th-century iron and glass roof.

The construction techniques described in this research were extracted from printed publications. However, further research of archives of contractors, patents and building specifications can contain information on the construction techniques in general and the putty recipes and puttyless glazing specifically, that was not found in the printed publications.
The literature research was also aiming at finding information about the historic materials that were used and their mechanical properties. Technical data on the glass plates were dealing mainly with the dimensions and thickness. Nevertheless, information on the strength of 19th-century glass (incorporating the effect of weathering and age of the glass plates) would broaden the application of the structural calculations: the strength of the glass plates could then be incorporated in the model. However, almost no information was found on the glass strength in the course books and manuals. Further research is needed to verify other written sources (e.g. archives of glass manufacturers), complemented by experimental research on preserved historic glass.

4. Heritage value of historic glass

The replacement of broken glass panels is often considered as standard maintenance intervention, which is therefore often not reported in conservation reports. However, assigning a date to the glass plates is essential to evaluate the heritage value of the glass plates. During the restoration studies, the glass is often neglected and thus not preserved. However, glass that was installed in a later period, although not original, can also have historic value.

The evolution of glass production processes can be used to date historic glass using chemical analysis, a methodology that is already developed in England. The Belgian situation however was different:

- the economic conditions were different in Belgium (e.g. the taxes on window glass in England);
- the major production process was different in Belgium (cylinder glass in Belgium versus crown glass in England);
- and the resources were not comparable between the two countries (e.g. the good quality sand from Mol in Belgium).

Therefore a literature research on the Belgian window glass production was carried out. A timeline was extracted which gives an overview of the evolution from 1790 until 1915 of the economic situation, the maximum glass plate dimensions, the raw materials, the melting furnaces and the processing technology. Belgian cylinder glass was exported all over the world, which makes this timeline relevant for a broader application.

The conclusions from the literature research on 19th-century Belgian window glass production need to be validated against other information sources. Further research can go into the archives of glass manufacturers and patents from the investigated period to reveal more details on the evolution of the Belgian cylinder glass production. An extensive experimental research is essential to link the written sources to the chemical composition of the actual applied glass.

The course books and manuals also mention information on textured glass tiles (often used for floor tiling), colouring glass by adding oxides, cast glass products, etc. Further research on the application of these other glass types in the built heritage would be a complement to this research on colourless roof glazing.

5. Renovation strategies combining modern standards and heritage value

The importance of iron and glass architecture in the construction history of the 19th century is clear. The question arises how we can preserve this built heritage but make the construction fulfil the modern standards on safety and structural integrity. The heritage value of the whole construction and of the separate components should establish the boundary conditions in which possible interventions are defined. Possible renovation strategies were illustrated by three case studies.

Modern standards on safety prescribe the use of laminated glass for overhead glazing. The library of the National Bank in Brussels was added as an example of laminating historic glass to obtain a sufficient safety level to open the library to the public. For the Kibble Palace in the Glasgow Botanic Gardens, a deliberate choice was made to install single glass panes above the planted zones of the glasshouse, with a reduced safety level as a result (with the consequence of limited accessibility during bad weather conditions).

Modern standards on structural integrity are written for modern constructions. Research on modern glass constructions already investigated the buckling behaviour of single and laminated glass plates. The structural possibilities of glass plates to stiffen building envelopes were proved for these modern applications. Furthermore, the high number of small glass plates in 19th-century iron and glass roofs ensures a high redundancy: if a glass plate breaks, loads can be transferred via a number of other load paths through the structure, due to the high redundancy of the structure. However, it is important to incorporate aspects that are specific for these 19th-century constructions in the structural simulations. For example, the Saint-Hubertus Galleries in the Brussels city centre were calculated taking into account the corrosion damage of the iron frame.

Different approaches are possible for the renovation strategies of separate components. This research gave examples on all levels: sandblasting and painting the iron components, the influence on both safety level and structural performance of the installation of single or laminated glass, the choice of replacing linseed oil putty by a modern sealant due to maintenance issues, etc. The interventions have to be chosen based on their influence on the heritage value of the building as a whole and its components, on the connection details of the construction, on the safety level of the glass roof, and on the structural integrity of the whole construction.

Modern standards on structural integrity prescribe the use of specific climatic loads like snow and wind loads. These standards predict the loads for a structure that still has to be built, by using simple geometries and making assumptions on boundary conditions. However for the built heritage, no uncertainties (e.g. on neighbouring conditions) exist. The simple geometries from the standards often do not suffice to describe the known geometry of the construction. Further research could clarify for example how realistic wind loading conditions can be assessed (by computer modelling, wind tunnel testing, or on-site measurements, etc.).

6. Iron and glass heritage

The evolution of the production processes of both iron and glass underwent major changes during the 19th century. New furnaces were the basis of the development of cast iron, wrought iron, and mild steel production processes. The application of iron and steel in building construction expanded and an architectural vocabulary specific for iron construction was formed. Glass manufacture however stayed a traditional process until the beginning of the 20th century. By combining improvements originating from different scientific disciplines (furnace technology, raw materials, working tools, etc.), the production volume and the quality of the produced glass increased. The great innovation of the 19th century however was the application of glass in architecture in combination with iron. The slender iron frames cladded with glass allowed light to penetrate to the core of the buildings. New building types, originating from the Industrial Revolution like railway stations, exhibition buildings, shopping galleries, and glasshouses, used this possibility to a great extent.
In this research, an overview was given of the international evolution of iron and glass architecture. Belgian cases were added to illustrate this evolution.

The previous conclusions all depend on the roof typology that is being assessed. The structural behaviour depends on the overall geometry of the iron and glass roof, historic glass has a higher chance to be preserved in double-walled than single-walled roofs, the performance requirements for the putty connection depend on the climatic loads that are acting on the roof, etc. The structural calculations demonstrated that the structural contribution of the glass cladding can be significant, but depends on these typologies. Further research should therefore be performed on the available roof typologies: an inventory of the preserved 19th-century iron and glass roofs in Belgium, an overview of the roof typologies and their performance requirements for the glass cladding, etc.

7. Conclusions

The structural assessment of 19th-century iron and glass roofs can be dealt with in two directions. The structural consequences of specific interventions can be calculated so that their structural influence can be part of the evaluation of the proposed renovation strategy. At the same time, the structural behaviour of a preserved structure can be simulated so that the need for interventions can be assessed. In both cases, this research provides information to make a multi-disciplinary assessment of 19th-century iron and glass roofs taking into account the heritage value, the construction techniques, the structural behaviour of the whole construction, and the modern standards.

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Appendices

The following appendices are added digitally on the attached CD-rom.

1. Example script of the calculation model

The python script of the 4a_putty model under asymmetric wind load is added to illustrate the method of constructing the model.

2. Results of calculation models

This appendix contains the results of all calculation models discussed in Chapter 6. Tables with the nominal results and graphs showing the influences are reported for each of seven load cases.

In the 19th-century architecture, the use of iron and glass roofs made it possible to bring light to the core of buildings. Together with the social context of the Industrial Revolution and the improvements in heating techniques, the creation of a “Garden of Eden” became popular. Building typologies like railway stations, market halls, fabrication halls, exhibition halls and greenhouses exploited this possibility at full extend.

When renovating the 19th-century iron and glass roofs, both the heritage value and modern standards on safety and structural performance have to be taken into account.

A first part of the research, which is based on literature study, goes into the construction history of iron and glass roofs to define the heritage value of the roof (and its components) to be restored.

The second part puts forward a methodology to recalculate the performance of the original structure by taking into account iron and glass. Lab tests were performed to deliver data that was still missing to do so. Tests on historic adhesives, namely putty, were carried out. And modern adhesives were applied on elements that show effects of deterioration, present in renovation projects.

Finally, a parameter study was carried out to determine the effect of renovation options which are applied nowadays: replacing historic putty by modern adhesives, replacing single glass by laminated glass, etc. Simulations showed that taking in account the original single glass panels in a calculation (even when connected with putty), has a positive impact on the overall behaviour.

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advisors: prof. dr. ir.-arch. Ine Wouters
prof. dr. ir.-arch. Jan Belis