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Manchester Town Hall, a Case Study in Structural Glass Reliability and Robustness

B. Eckersley [a], G. Coult [a] & P. Lenk [b]
[a] *Eckersley O'Callaghan, UK, graham@eocengineers.com*
[b] *Formerly of Eckersley O'Callaghan, UK*

The design of the structural glass for the Manchester Town Hall Link (completed 2015) was carried out by engineering consultancy Eckersley O'Callaghan (EOC). The glass facade forms an enclosed shell which supports a steel roof and acts monolithically to resist lateral loads. The project collates the latest advances in glass technology combined with innovative design methods. In the absence of explicit codes of practice for structural glass, EOC performed a first principles approach in using empirical data, acquired through previous projects, and analytical methods, including FEM and parametric modelling, to justify an elegant and efficient structure which was approved by building control authorities. The result has reduced glass joints and less visible metalwork to meet the architects' aspirations. This paper describes the approaches and innovations in designing the structural glass for this project.

Keywords: Glass, Robustness, Buckling, Material Properties

1. Introduction

The Library Walk pavilion is a link between the remodelled Grade II-listed extension building of the Manchester Town Hall and the adjacent Central Library, designed by Simpson Haugh and Partners.

The 175m^2 pavilion uses frameless structural glass panels to support a 30-tonne, stainless steel roof structure. The distinctive shape of the roof was form-found using mathematical algorithms designed to create a smooth, organic but 'rational' undulation in the soffit, based on spherical distortions of a flat surface. This undulation responds to the arched windows of both extensions.

Fig. 1 Manchester Town Hall Link Building

The roof consists of polished stainless steel monocoque construction consisting of top and bottom doubly-curved structural surfaces contributing to the stiffness and strength of the roof, allowing it to span 15m across a column-free space. These external exposed surfaces are welded to an internal armature of stiffeners, creating a rigid structure. This monocoque construction was engineered and fabricated by specialist manufacturers, CIG, using automated bending techniques combined with a manual crafted polishing of the surface. Large segments were prefabricated in the factory and bolted, welded and polished on site at the seams.

The structural glass façade collates the very latest in glass technology coupled with cutting-edge analysis and design to refine the visual simplicity and transparency. As there are no vertical fins, full-height glass panels are utilised; this innovative design has fewer glass joints and less visible metalwork to exceed the architect's aspirations. The façade includes both straight and curved panels. The 7.2m tall, base supported panels support the roof and provide lateral stability to the whole pavilion.

The simplicity and purity of this building is achieved by the simple combination of the two structural elements of the roof and the vertical glass façade, which are rigid in virtue of their form. In other words, the structure is the form, the enclosure as well as the supports.

Library Walk was an opportunity to experiment with new design tools, technologies and fabrication methods to allow the pure expression of structure in sculptural forms. In both the roof monocoque, delivered using innovative method of fabrication borrowed from the shipbuilding industry and refined to new levels of precision, and in the structural glass façade, taking the latest in reductive engineering to its zenith; form, enclosure and structure have been merged into a single integrated system.

As well as being a playful, sophisticated and sensitive sculptural piece, the project opens new opportunities within a more integrated approach to the design and fabrication of structural and cladding systems.

This paper will discuss the design approach adopted to ensure robustness and reliability absence of explicit codes of practice.

1.1. Company Introduction

Eckersley O'Callaghan (EOC) is recognized as one of the leading authorities in the structural design of glass through their notably high profile work with Apple Inc. amongst others. These structural glass projects are frequently referenced as the cutting edge in both glass design and ambition. Further, and perhaps most relevantly, these structures have been designed and justified in many different countries through many different building control authorities. EOC's expertise is therefore more than just the successful design of these glass structures, it is the successful track record they have had in obtaining permissions for over 40 major glass structures in over 10 different countries (including the UK) and cultures in the last 10 years. Specifically, in the UK EOC has designed and had approval for glass structures in London, Bristol, Exeter, Milton Keynes, Glasgow, Belfast, Liverpool and Manchester.

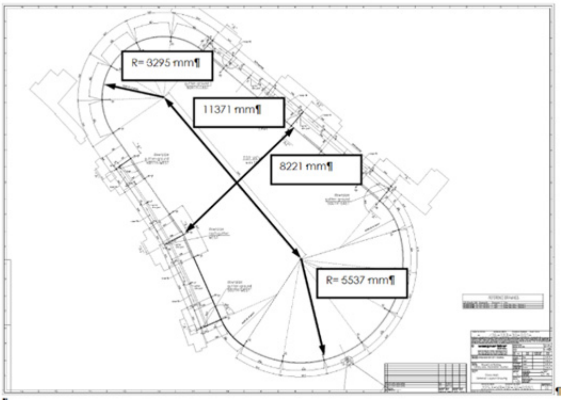
1.2. Eckersley O'Callaghan's Role

Eckersley O'Callaghan were the structural engineers for the detailed design of the façade. Both the roof and substructure were designed by others. The glass panels act as the interface between the two structures; careful co-ordination of both deflections and load transfer paths were required throughout the design process to ensure a cohesive structural system.

2. Structural System

2.1. Gravity System

The roof structure is supported on 20 laminated glass wall panels. The panels are constructed from 3 fully-tempered 12 mm glass lites, laminated with 1.52 mm SGP interlayers. The geometry of the building was optimized from the design intent resulting in a simpler form that is divided into regions of flat glass panels and curved glass panels of 2 different radii with negligible geometric difference. Flat glass panels are stiffened by perpendicular glass panels, which serve to connect the new glass building to the two existing buildings. There are 6 sliding glass doors within the façade. Portal steel frames around the doors are design to carry the weight glass panels above the door and provide lateral restraint to the neighbouring glass panels. To minimize the door frame steel size, the roof does not transfer load to the glass panels over the door frames, instead the roof spans over the openings and loads the glass panels as shown in figure 2b.



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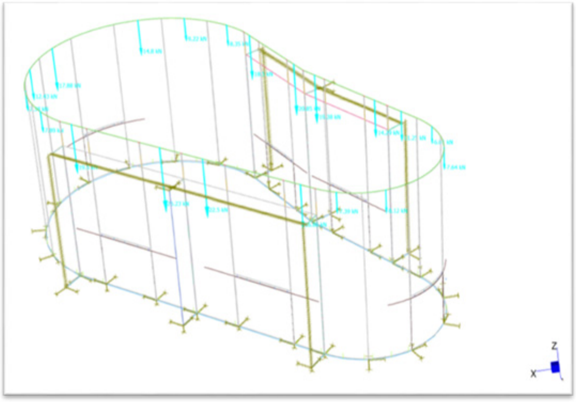
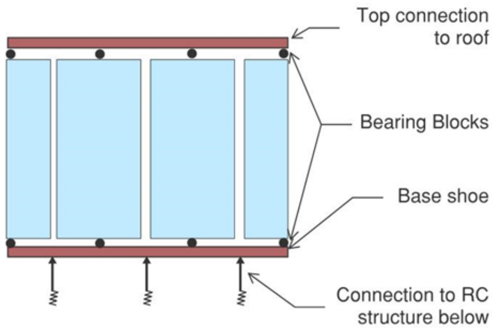
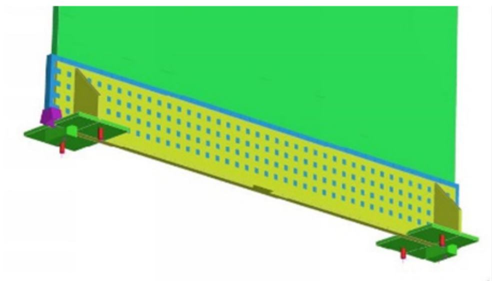


b) Global Analysis Model

Fig. 2 Gravity Load Resisting System a), Panelization b) Global Analysis Model

Each glass panel is base supported within a semi-rigid steel shoe, and restrained laterally at roof level. Gravity loads are transferred to the steel shoe by a central bearing block, as shown in figure 3. The steel shoe is supported on a reinforced concrete ring beam at the panel corners. Roof loads are transferred to the glass panel via a central bearing bloc only.





b)

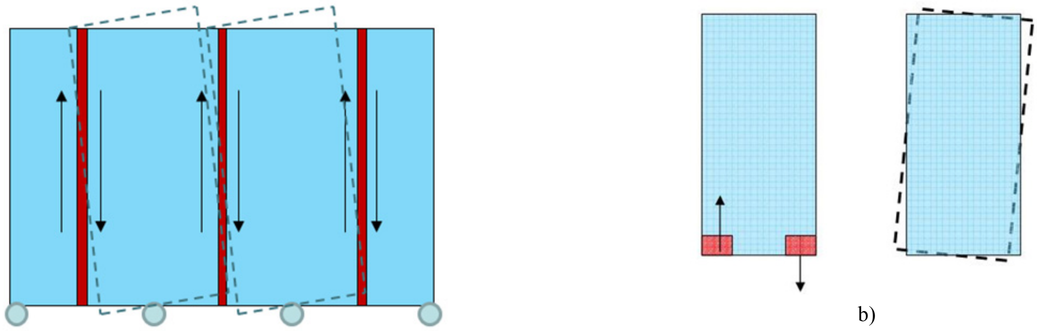
a)

Fig. 3a) Panel Loading and Support Points, and b) Semi-rigid shoe

2.2. Lateral System

The global stability is primarily provided via shear transfer at vertical silicone joints between glass panels (Figure 4a). The roof acts as a stiff diaphragm connecting all glass panels together; the bonded glass walls then form a stiff loop which resists overturning moment by the second moment of inertia of the entire building. This significantly reduces internal glass stresses from stability loads.

A secondary stability system is also provided by silicone in the base shoe connection detail (Figure 4b). This system resists panel overturning by a push-pull action at panel corners. This system acts as a secondary, redundant system to the primary vertical silicone joint system, and is utilised in the event case of failure a glass panel.



b)

a)

Fig. 4a) Primary Stability System, and b) Secondary Stability System.

2.3. Design Approach

Our design approach is based on a combination of published codes, in house testing data, published papers and journals, research and development, prior project experience and test results, published guidelines where they remain current and the design experience developed from over 20 years of active involvement in the field of structural glass. Due to the lack of a valid European code for glass strength and its limitations in the application to structural glass an approach based on the American ASTM E1300 (2012) was adopted for the design of glass elements. Where relevant BS EN codes exist, (including the draft standard prEN 13474-3 (2008) and now published IStructE guidance (2015)) their guidance was followed, and figures were justified with in-house testing when necessary. In order to ensure that the Town Hall Link Building met the Eurocode-defined minimum level of structural reliability, a first order reliability analysis was carried out. In order to ensure that the material data assumed within the design process was matched by the finished construction a stringent set of quality control tests was defined, and carried out by a third party.

Structural analysis was carried out using both standard analytical methods and numerical methods and checked using simplified approaches. Panels were subdivided into 4 design groups: 2 flat and 2 curved. For each group the critical panels were identified using the global analysis model.

3. Design Requirements

3.1. Collaboration at Interfaces

The glass panels span between a reinforced concrete slab at base level and the steel roof. Both the roof and the floor were designed by others, and have very different stiffness properties. Calibration was required at both levels in order to accurately predict the load distribution from the glass roof to the glass panels. An initial assumption of even distribution of roof weight to each panel was taken, which provided an initial estimation for the spring stiffnesses at the supports. The spring stiffness at the base was refined from deflection plots provided by URS. This provided a more accurate distribution of roof loads within the glass panels.

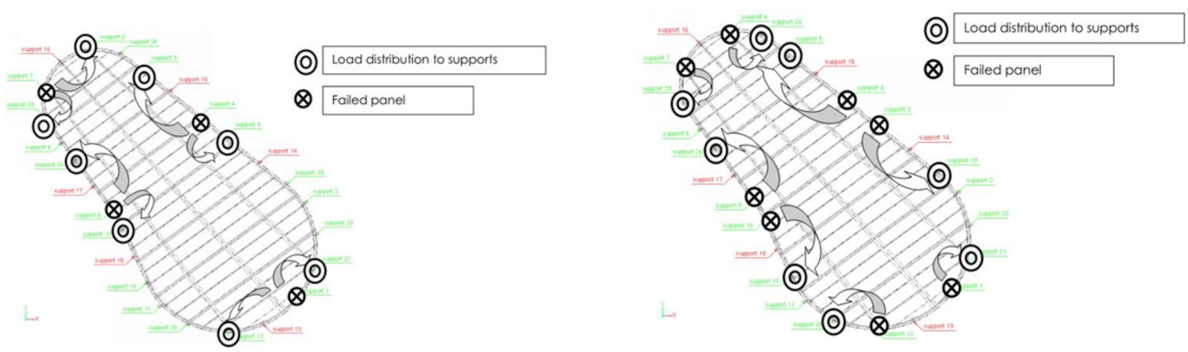
Additionally, thermal differential movements between the roof and the RC slab needed to be accommodated within the glass walls. Lateral expansion of the roof was accommodated by allowing the panels to tilt, rotating around the minor axis. Internal temperature changes cause hogging and sagging in the steel roof, which in turn causes the distribution of roof loads in the panels to vary.

3.2. Robustness and Redundancy

Redundancy has been provided for both the gravity and stability force resisting systems. In the event of accidental damage, three different redundant design conditions were identified:

a) Failure of a single ply
b) Failure of critical glass panel
c) Failure of 2 critical glass panels

If a complete panel is broken, the roof is designed to span over the soft spot and redistribute forces to the remaining glass panels. Additionally, the roof acts to tie the remaining glass panels together. The distribution of loads to the remaining intact panels was assessed, as shown in figure 5 for redundancy conditions b and c. Reduced partial safety factors were utilised for the accidental design.



a) b)

Fig. 5a) Failure of critical panel within a design group, and b) Failure of two critical panels within each design group.

Global stability after accidental breakage was ensured by the secondary shear connection at the panel base, as shown in figure 3.

3.3. Design Reliability

Eurocodes 1991 - 99 ensure that all Eurocode compliant structures have a minimum level of structural reliability. Annex B and C of BS EN 1990 (2002) specify reliability in the form of a reliability index *(β value)*. The defined β values vary with building consequence class.

Partial factors for material safety and applied loads are defined within individual Eurocodes in order to achieve a minimum reliability of β=3.8, at ultimate limit state for a 50-year life span. This is the β value associated with medium consequence class buildings. In the absence of a valid European Code for structural glass, a global factor of safety is required in order to meet the minimum structural reliability defined in BS EN 1990.

A global factor of safety for buckling has been calculated according to the rules established by the Joint Committee on Structural Safety (JCSS). For a specific design condition comprising dead and snow load, a reliability assessment was carried out. For this, a first order reliability method (FORM) was adopted through the use of calculation software CodeCal developed by Faber et al. (2003). A Weibull distribution was taken for the glass material strength, with a coefficient of variation of 15% for fully tempered glass that was to be corroborated by testing during fabrication quality control. For the required load combination, a global factor of safety on buckling was calculated as 1.8 at ULS, for the 50-year design life.

4. Analysis Approach

Three levels of numerical modelling were used to analyse the structural system. At the global scale fittings are modelled as simple beam elements, façade panels as plate elements and fins as beam elements. These models use appropriate techniques to reflect the relative stiffness of the structural system such that load paths are representative. From this model the critical areas and the load path can be identified.

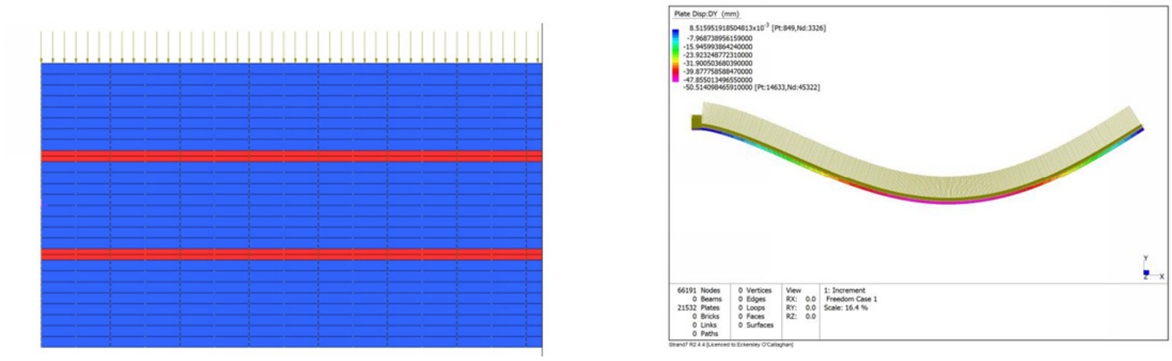
Identified critical panels, restraints and connections are subsequently modelled in greater detail by 2-dimensional analysis models. After validation of these models by simple hand checks, detailed volumetric models of key fitting connections or areas of interest are created.

For the two dimensional models, the complex behaviour of laminated glass is approximated with an effective thickness approach. For detailed stresses, for example around fittings, full 3D volumetric models are used with the bulk interlayer modelled.

4.1. Effective Thickness Modelling

Both analytical and numerical modelling techniques were used to establish the effective thickness of the three ply build-up.

The propped cantilever arrangement was modelled as a 2D-plane strain model with the laminate build up as shown in figure 5. The deflection under out of plane loading was calculated, from which the effective thickness of an equivalent glass monolith was determined.



a) b)

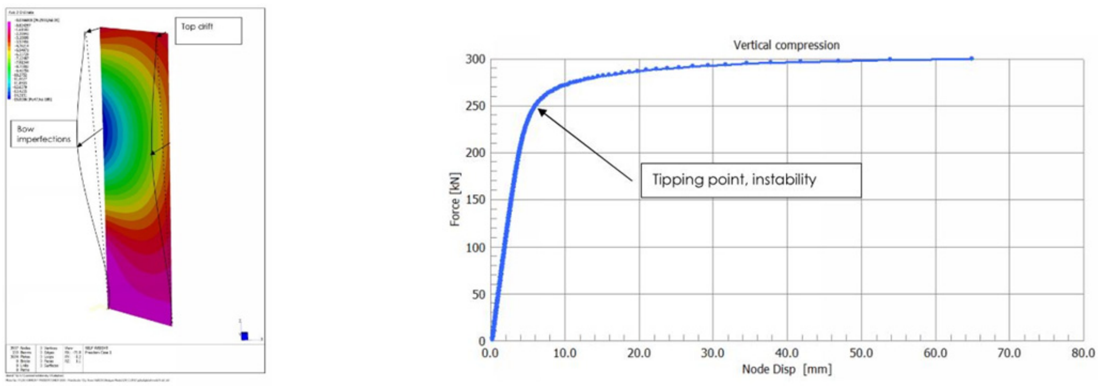
Fig. 6a) Failure of critical panel within a design group, and b) Failure of two critical panels within each design group.

Additionally, the effective thickness has been calculated using the approach presented in prEN13474-3 (2008) for comparison. The lesser of the two effective thicknesses was used for design.

4.2. Buckling Analysis of Glass Panels

Non-linear finite element buckling analysis of both curved and flat glass panels was carried out using Strand 7.

Different combinations of initial fabrication imperfections, out of plane wind loading, and lateral roof movement were investigated for each panel type. For a combination of initial bow imperfections of L/1000, wind induced deflection of L/1000 and a horizontal roof movement of 3.6 mm (assessed based on the thermal expansion of the roof), the buckling response is shown in figure 7.

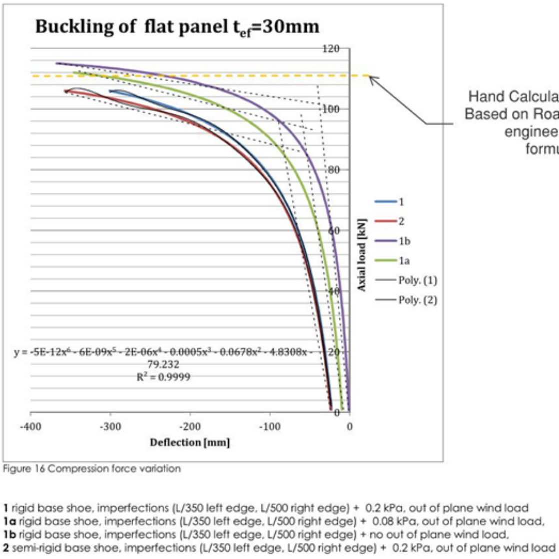


a) b)

Fig. 7) Non-linear buckling analysis of flat glass panel, a) Glass stresses under vertical load for defined initial imperfections, b) Force displacement response for critical node

In order to validate the non-linear analysis a series of increasing complexity models were produced. The theoretical buckling capacity was calculated analytically assuming each panel to act as a propped cantilever with axial point loads at the tip and mid-height.

A linear buckling analysis was then carried out in Strand 7 for both flat and curved panels. A variety of base support conditions and initial imperfections were considered in order to calibrate the numerical model. The numerical model was found to converge well to the theoretical solution. Figure 8 shows the analysis of a single flat panel. The applied loads in figure 8b are extracted from the global analysis model shown in figure 2b. In figure 8a the buckling response under different boundary conditions is shown alongside the theoretical buckling response.



a)

b)

Fig. 8a) and b) Linear Buckling Analysis of flat glass panel

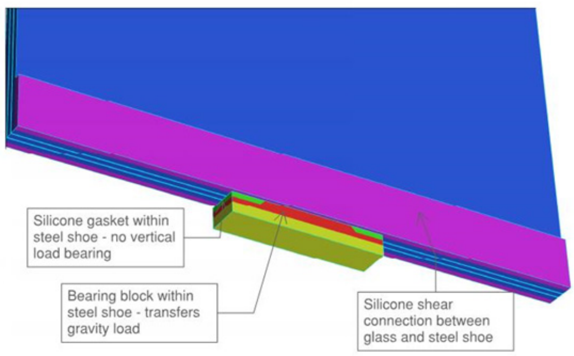
4.3. Detailed Sub-Modelling of Connections

Detailed modelling of connection elements was carried out in Strand 7. Volumetric models were constructed of key elements including:

- Vertical silicone joints
- Base silicone connection
- Base shoe
- Polymeric bearing blocks at floor and roof level

The first two models were used to confirm the silicone stresses under lateral loading. The base shoe model was used to determine the support conditions for the glass panels, and lastly the bearing block models were used to confirm contact stresses within the glass panel.

A variety of joint shapes were experimented with for the design of the bearing blocks in order to optimise the bearing stresses in the glass panel. A solution that utilized a tapered bearing block, as shown in figure 9a was found to produce an even stress distribution within the glass panel, without large peak stresses.



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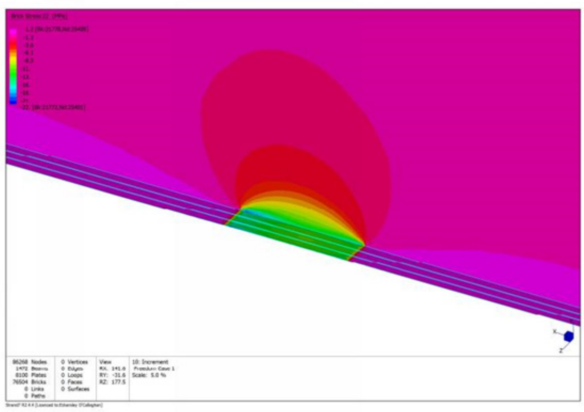


b)

Fig. 9 Bearing block local model a) Tapered bearing block b) Glass stress distribution under gravity loading

5. Material Properties

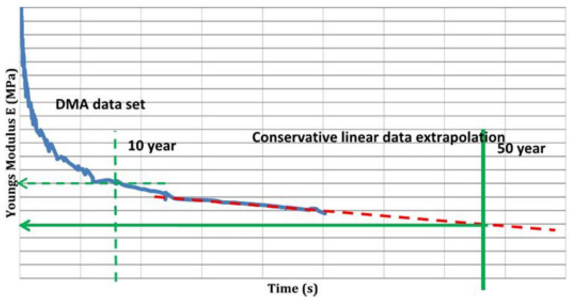
For all analysis, numerical or analytical, accurate solution depends on accurate material data. This requires material knowledge of the glass, interlayer and silicone joints.

5.1. Glass Material Strength

In the absence of Eurocode defined material strength of glass, design data has been adopted from the American standard ASTM E1300. Three different load durations have been specified and the corresponding material strengths used in design.

5.2. Interlayer Material Properties

SGP is viscoelastic, a different shear modulus is required for different durations of loading. Design data is available from Kuraray (then DuPont) (2014) for a variety of different temperatures and load durations. SGP bulk material properties were acquired by Dynamic Mechanical Analysis (DMA) carried out for a previous project by an independent body. The results were comparable to those published by Kuraray but enabled us to extrapolate beyond the table of published values (see Figure 10).



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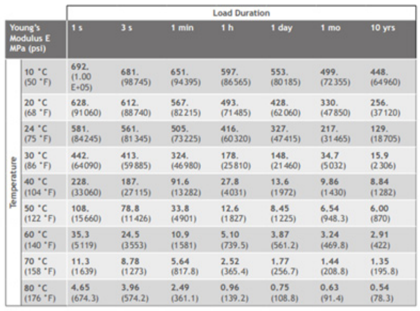

Young's Modulus E MPa (psi)	Load Duration 1 s	3 s	1 min	1 h	1 day	1 mo	10 yrs
Temperature 10 °C (50 °F)	692. (1.00 E+05)	681. (98745)	651. (94395)	597. (86565)	553. (80185)	499. (72355)	448. (64960)
20 °C (68 °F)	628. (91060)	612. (88740)	567. (82215)	493. (71485)	428. (62060)	330. (47850)	256. (37120)
24 °C (75 °F)	581. (84245)	561. (81345)	505. (73225)	416. (60320)	327. (47415)	217. (31465)	129. (18705)
30 °C (86 °F)	442. (64090)	413. (59885)	324. (46980)	178. (25810)	148. (21460)	34.7 (5032)	15.9 (2306)
40 °C (104 °F)	228. (33060)	187. (27115)	91.6 (13282)	27.8 (4031)	13.6 (1972)	9.86 (1430)	8.84 (1282)
50 °C (122 °F)	108. (15660)	78.8 (11426)	33.8 (4901)	12.6 (1827)	8.45 (1225)	6.54 (948.3)	6.00 (870)
60 °C (140 °F)	35.3 (5119)	24.5 (3553)	10.9 (1581)	5.10 (739.5)	3.87 (561.2)	3.24 (469.8)	2.91 (422)
70 °C (158 °F)	11.3 (1639)	8.78 (1273)	5.64 (817.8)	2.52 (365.4)	1.77 (256.7)	1.44 (208.8)	1.35 (195.8)
80 °C (176 °F)	4.65 (674.3)	3.96 (574.2)	2.49 (361.1)	0.96 (139.2)	0.75 (108.8)	0.63 (91.4)	0.54 (78.3)

d)

Fig. 10 SG5000 material properties a) DMA data b) Published Kuraray Data (2014)

5.3. Structural Silicone Material Properties

A Mooney – Rivlin material model was determined by dynamic mechanical analysis carried out for a previous project. Material constants were calculated directly from test data within Strand 7 (Figure 11, Table 1)

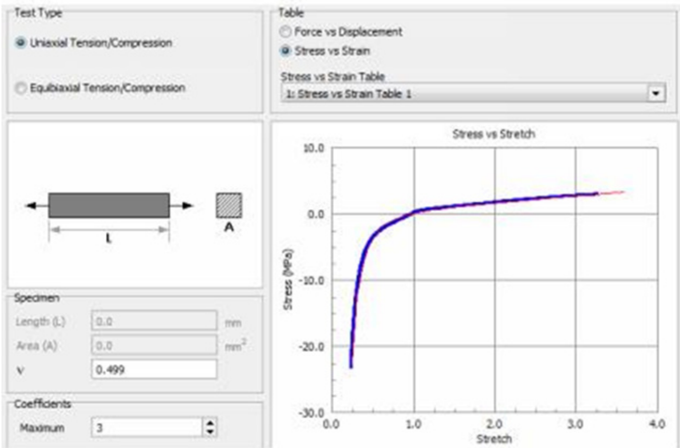

Fig. 11 Curve Fitting to establish Mooney-Rivlin Material Properties

Table 1: Structural Silicone Material Properties.

Material	Density [kN/m^3]	Young's Modulus [MPa]	Poisson Ratio	Tensile Strength [MPa]	Design Strength [MPa]	Thermal Expansion [μm/m/K]
Silicone	0.98	1.4	0.499	1.2	0.138	9.2

6. Validation and Quality Assurance

Design assumptions were validated through a combination of material testing and numerical modelling. Additionally, a stringent set of quality control tests were specified and carried out to verify the data taken for analysis.

6.1. Validation of stability system.

In design it was assumed that the vertical silicone joints provided the primary stability system, with silicone in the base shoe providing a secondary system in the event of panel failure. This assumption was tested by comparing the relative stiffness of the two systems. Global analysis models of the structure were defined with 1) vertical silicone joints only, 2) base silicone joints only, and 3) both sets of joints. The vertical system was found to be 30% more stiff than the base joints (Figure 12). Both systems were found to be capable of resisting lateral movements independently.

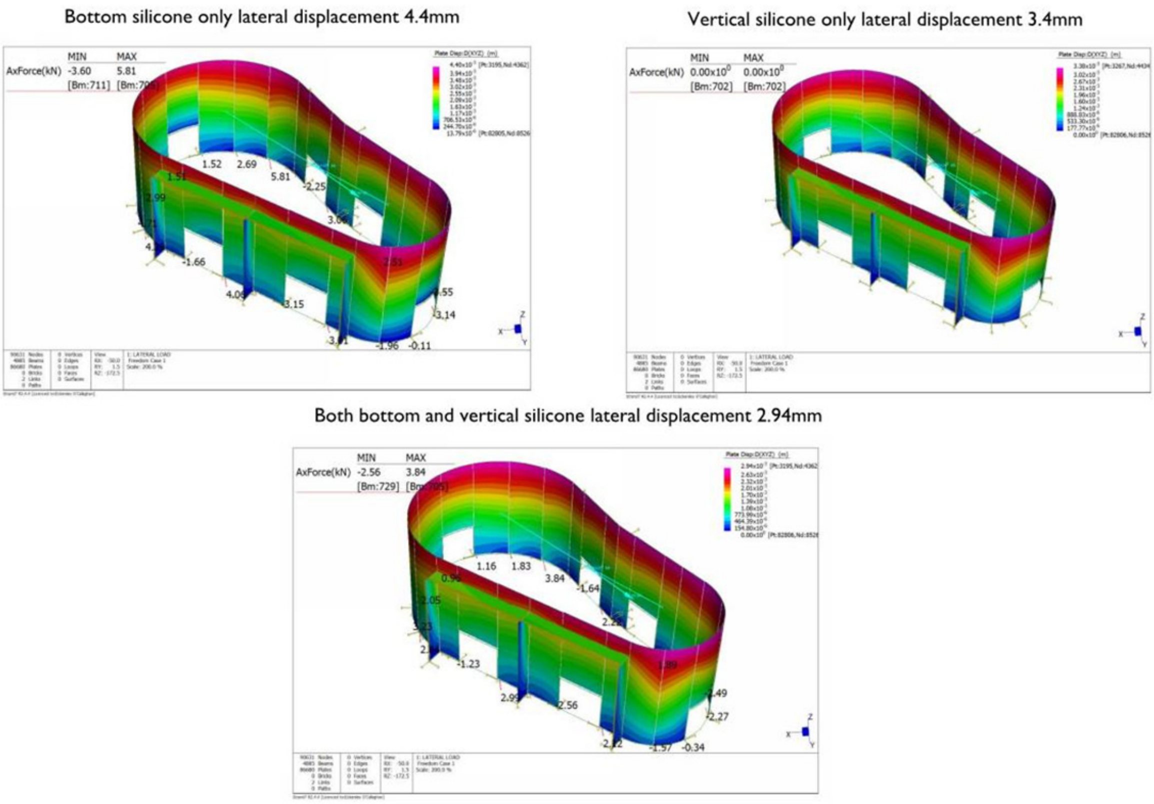

Fig. 12 Global lateral deflection of independent stability systems

6.2. Validation of silicone design assumptions

Non-linear shear properties of structural silicone are not yet well understood. In order to confirm the linear behaviour of the structural silicone in shear, a combination of material test data, bulk finite element models and published journal articles were used. Dias et al. (2012) and Bondi et al. (2009) have both shown that silicone non-linear behaviour only occurs beyond strains of approximately 60%; volumetric finite element models of the structural silicone joints, based on material properties established by DMA showed that for the geometry and loading conditions considered here, strains of 13% were observed. The silicone is well within the linear range of behaviour.

Additionally, project specific mock-ups of the base shoe were tested in tension as part of the quality control procedures. Large scale pull-out tests, as shown in figure 13, were carried out. A tensile force was applied to the glass-to-shoe connection, and the deformation and failure capacity recorded. The results were used to validate the detailed volumetric finite element models of the base connection shown in figures 13e) and f) an also the curing of the deep joint. Additionally, the finite element modelling assumed that silicone deformation and failure occurred within the bulk silicone material and not the silicone-glass or silicone-steel interface. Pull out tests were performed to failure, and the failure mechanism recorded. When adhesive failure occurred the joint preparation specification was adjusted to ensure cohesive behaviour.

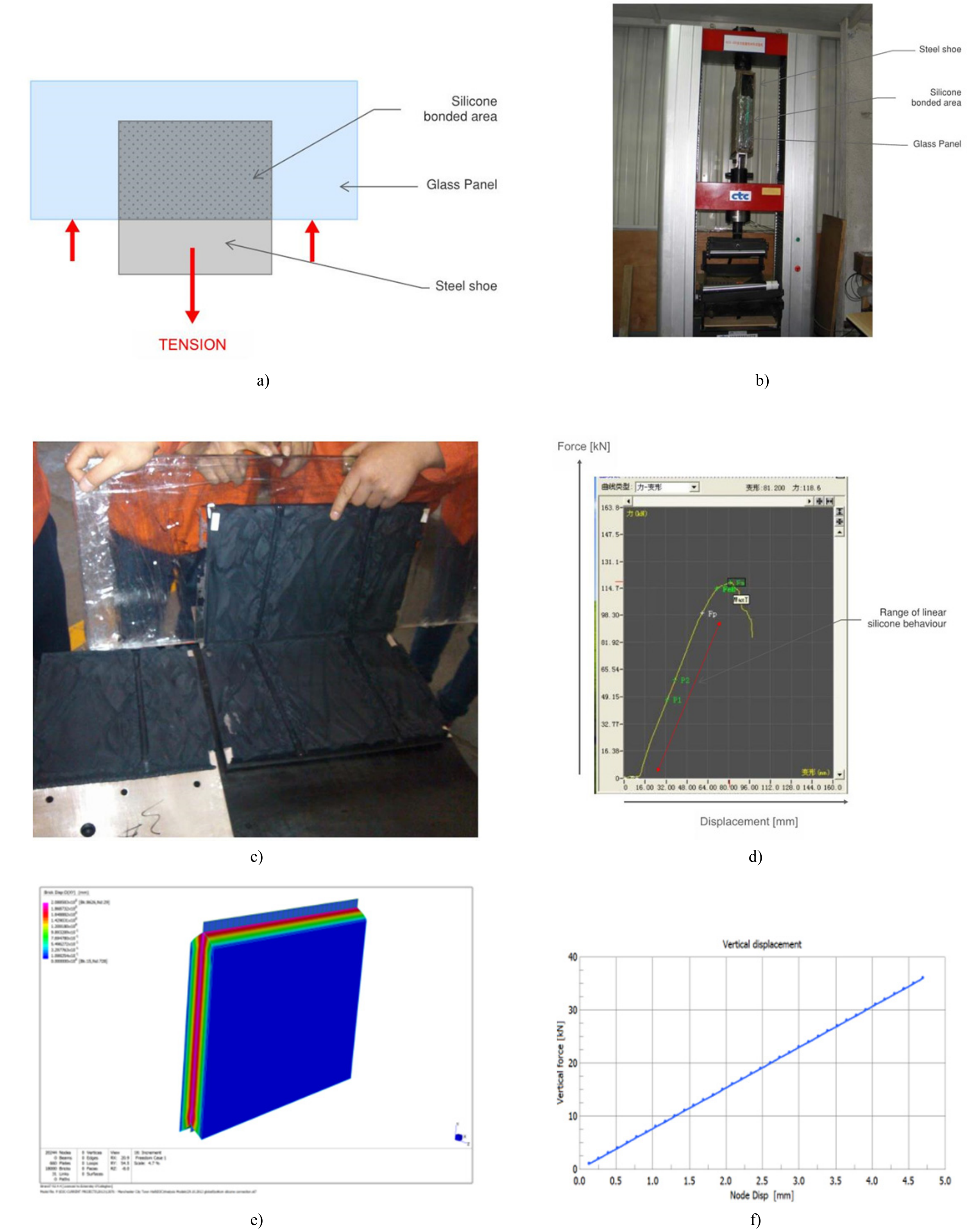

Fig. 13a) Schematic of base shoe pull-out test procedure, b) Test performed by independent testing authority, c) Cohesive failure in the silicone joint, d) Test results highlighting range of linear material properties e) Volumetric finite element model of shoe connection f) Numerical analysis response showing linear behaviour of silicone joint

6.3. Quality Control Procedures.

A stringent quality control specification was defined to ensure the glass material strength was as expected. Small scale fragmentation and mechanical strength tests were carried out to EN 12150 (2004). Tests were performed to ensure surface compression met the minimum requirements for fully tempered glass as defined in BS EN 14179 (2005). Prior to testing, surface residual stresses were also measured using the scattered light polariscopic (SCALP) procedure.

Full-scale fragmentation tests were carried out on the bent glass panels. Panels were fractured using an impactor, and the fragmentation size recorded. Quality control tests were performed for each batch of panels manufactured, with non-conforming panels rejected.

7. Conclusions

In structural glass design a simple structure is more deterministic; load paths can be clearly understood and the requirement for complex modelling is reduced. Complex structures constructed of many components of varying stiffnesses are difficult to analyse globally, and load paths may be incorrectly interpreted.

Simple glass structures however have an increased requirement for robustness, and post-failure capacity. If only one load-path exists, there is a greater requirement to ensure a residual capacity in the event of failure. Careful analysis of the load-redistribution after glass failure must be performed, and multiple failure modes considered.

In the absence of codified rules for the design of structural glass elements, an emphasis must be placed on ensuring the design meets the minimum reliability requirements of BS EN 1990, and that the material assumptions used within the reliability analysis are confirmed by quality control testing procedures.

Structural glass design is still a very young industry. The complexity of design is not always apparent when reviewing a given scheme. It is important for designers to consider the load paths at every scale of the project, from an initial global analysis level through to small scale connections. Although more and more guidance is available, it is important for designers to not impart on the design of schemes beyond their understanding. In addition to a good understanding of engineering principles, designers need an understanding quality assurance procedures, robustness requirements, reliability analysis and validation of analysis procedures by project defined testing.

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