Challenging Glass 6 - Conference on Architectural and Structural Applications of Glass
Louter, Bos, Belis, Veer, Nijsse (Eds.), Delft University of Technology, May 2018.

ISBN 978-94-6366-044-0, https://doi.org/10.7480/cgc.6.2199

Hybrid Glass Structures – Design Philosophy and Selected Checking Procedures

Peter Lenk [a], Chris Noteboom [b], Mark Arkinstall [a],
[a] Arup, London, United Kingdom, peter.lenk@arup.com
[b] Arup, Amsterdam, Netherlands

In this paper authors will summarise current design approaches for hybrid glass structures. Our past and present projects will be used to demonstrate how such structures can be justified using finite element analysis, analytical solutions in context of prescriptive national and international regulations, industry guidance's developed and accumulated over past decades. General methods will be distilled, relevant for future hybrid glass structures.

Keywords: Glass, Hybrid, Design

1. Introduction

Many engineers are using glass components in their structures to strive for bespoke designs driven by either an architectural vision, technological reason or economical need. Glass can be applied as merely infill, but also as secondary or even primary load bearing element. In combination with other materials hybrid glass structures can be created that may offer benefits over schemes comprising from single material applications.

Main themes are connections, uncertainty, robustness, materiality, checking procedures and execution. Connections are often combinations of mechanical (i.e. bolted) and adhesive connections (i.e. structural silicone). Stiffness of supports is uncertain and sensitivity studies should be carried out to determine structural behaviour. As glass is a brittle material an assessment of robustness is required to assess the consequences of possible failure. Materials usually combined with glass are (stainless) steel or wood. Checking of serviceability limit state is often done with relative simple models with reasonable parameters. Ultimate limit state checks are done by checking codified stresses in global or local models and investigating the influence of variation of parameters. How analysis models are created and reviewed is of great importance. The execution of a hybrid glass structure can be more complex than a regular structure in particular when connections are relying on site construction. Adhesive connections require particularly strict conditions for application, curing and on-site monitoring. Replacement strategies of glass elements need to be reviewed and convincing method statements developed along with the actual design.

2. Design Concepts

In current practice, designs are very often optimised where economical and more slender solutions are being proposed to achieve desired architectural intend, while keeping budgetary considerations in mind. It is common practice, for structures comprising of typical structural materials as steel or timber, that such structures should be designed to lower overall deformations if they are over clad with brittle materials such as glass. Obviously, this usually contradicts with the architectural vision of 'lightness' associated with glass. More comprehensive structural concepts are expected to be developed pushing boundaries of engineering knowledge and code acceptance. As proved by many projects, glass can enhance structural performance of the primary structure. The authors recognise the following design strategies structural engineers may consider:

- Strength and stability of the original structure (ultimate limit state);
- Stiffness, vibration and visual performance (serviceability limit state);
- Robustness, redundancy, damage tolerance, risk mitigation, and post failure performance.

This paper will outline possible cons and pros of the above design strategies in the following text in more detail.

2.1. Strength and stability

The ultimate and most challenging design consideration is where the primary structure completely depends on the performance of the glass component to safely transfer all loads over a prolonged period of time. In this scenario the structural system will not be capable to safely withstand design loads without the glass component and as such a complex justification which will be outlined in the next chapter shall be carried out. This may trigger full scale testing and extended negotiation with building authorities as well as with specialist contractor. The extend of the justification is determined by the scale of the primary structure. If a façade or glass enclosure is seen as primary structure, investigation will be less extensive compared to, for example, hybrid glass columns that support floors. It is still unlikely that in the near future, primary structures reinforced with structural glass will grow beyond single story pavilions, bespoke façade system, staircases and art sculptures.

2.2. Stiffness, vibration, visual performance

Very often the primary structure is capable to safely transfer design loads without need of contribution of a glass components. However, it might be relatively flexible and therefore not meeting serviceability code requirements. It could be argued that in some specific building types those limits might be strict based on, for example, typical cladding detailing, which can be reviewed by designers to allow accommodation of movements and tolerances specific for a particular application. Other situations, which need a more sensitive approach is where the occupant comfort or public perception of safety might be of concern. A good example might be the Kempinski hotel, Munich Airport, where the cable façade is designed to allow for relatively large deformations. Certainly, the façade proved for decades that it was fit to purpose, where the above mentioned careful detailing methodology was successfully adopted. However few years after competition, the general public perception of such movements were causing discomfort to some. In other projects where this might be a problem, glazing panels may be added in to the structural system to evaluate contribution to the global stiffness. Connection stiffness and accommodation of tolerances shall be considered in such analysis to prevent glass overloading and subsequent failure of glass elements in serviceability loading conditions. In ultimate limit condition glass components are neglected and possibly glass components fail. It shall be noted that glass failure should be well defined and controlled, with no risk to the occupants. Use of laminated heat strengthened glass with sufficient glass edge restrains is recommended. However, it's also important to note that as the real strength of glass is often much higher than the assumed strength in calculations, the chance of glass failure is generally quite low.

2.3. Robustness, redundancy, damage tolerance

Typically, structures are organized in a hierarchical manner where load paths can be clearly followed and failure mechanism may be predicted. It shall be noted that progressive collapse of the structural system is limited proportional to the cause, in the current regulations and it is characterized by a distinct disproportion between comparatively minor cause or local failure of few structural elements and the resulting wide spread collapse.

Nonhierarchical structural systems are on the other end of the spectrum. The structural load path is difficult to follow where many alternative loads paths exists in parallel. It is less common to design structures comprising components from structural glass with nonhierarchical structural system due to the uncertainty in the identification of the load path and particularly unpredictability of the 'locked in stress' due to the substructure movement and thermal load. However in nonhierarchical systems, collapse resistance could be higher and susceptibility to the progressive collapse could be smaller than in the hierarchical structural approach.

Failure mechanism are dependent on the material used, magnitude, duration and nature of the loading as well as type of the structural element and its position in the structural system Xin and Blockley, (2007). Collapse resistance is a property that is influenced by numerous conditions including both structural features and possible causes of initial failure. Robustness is a desirable property of structural systems which mitigates their susceptibility to progressive collapse. It is defined as the insensitivity of a structure to local failure. To examine a structure in terms of its robustness, a quantitative description by means of a measure would be useful. Some simple formulations of stiffness, damage, or energy-based measures of robustness developed are presented in Starossek and Haberland (2010).

Lenk and Honfi (2016) proposed an advisable strategy for designing structural glass by not simply increasing robustness through building glass 'fortresses', but to identify a balance of providing alternative load paths and sacrificing plies/elements/subsystems in a way that functioning of the building should be disrupted as little as possible and risk of injury from glass failure mitigated to ALARP. One possible alternative investigated in this paper could be use of hybrid glass structures.

Comprehensive study to define safety in the structural glass design can be find in Bos (2009) where damage sensitivity, relative resistance, redundancy and fracture mode is combined in an element safety design diagram (ESD). In this work a variety of glass beams was experimentally tested to investigate the safety performance. Hybrid glass applications are discussed and it is mentioned that these are often created for post-failure resistance. However, for quite some applications full post-failure resistance is not necessary. The ESD can be used to determine the minimum requirements.

At last, the Dutch code NEN 2608+C1:2012 also provides good guidance on the safety evaluation of glass structures. Appendix D offers a risk assessment methodology commonly known as the Fine/Kinney method, since it is based on early work of Fine (1971), which then was later revised by Kinney (1976).

2.4. Levels of performance requirements of the hybrid glass structures

In terms of hybrid glass structures the following performance levels are proposed:

- Class 1 - Infill glass panel with sturdy structural member – lower boundary without significant glass contribution to the overall capacity of the structural system. Typical design approach. The stiffness of the primary structure shall consider requirements of brittle cladding components.

- Class 2a - Structural glass with slender structural member - Non -brittle elements are capable of safely transferring all ultimate limit state loads and accidental loads. However the structure or it's component is not sufficiently stiff enough if subjected to the serviceability loads in the case of entire or portion of the glass element is not present, removed or damaged. Structural glass shall transfer all serviceability loads without damage; in ultimate limit state damage is acceptable but safe failure shall be ensured.

- Class 2b - Structural glass with slender structural member – Non brittle elements are capable to contribute to the structural capacity but are not strong or stable enough to carry ultimate limit load without contribution of structural glass. Non - brittle elements shall be capable of safely transferring accidental loadings without promoting progressive collapse in the case of entire or portion of the glass element is not present, removed or damaged. Structural glass shall transfer all ultimate limit state loads without damage; in accidental loading scenarios damage is accepted but safe failure shall be ensured.

- Class 3a - Structural glass with slender nonstructural member – Non - brittle element is transferring load between glass plates, predominantly acting as a connection element. Additional function of such element can be increase of the damage tolerance of the structural system, and in case of accidental design situations provide alternative load path and retention to the damaged structural glass components. Non brittle element do not contribute significantly to the capacity nor stiffness of the overall structural system. Structural glass shall transfer all ultimate limit state loads without damage, in the accidental loading scenarios damage is accepted without promoting progressive collapse and safe failure shall be ensured.

- Class 3b - Structural glass – Upper boundary without any additional non brittle element where structural glass is the only material forming load bearing structure. Structural glass shall transfer all ultimate limit state loads without damage, in the accidental loading scenarios limited damage is accepted without promoting progressive collapse with sufficient post failure capacity and safe failure shall be ensured.

Above performance levels will be explained on the project examples in the following chapter.

3. Project examples

In this chapter a variety of past and present projects will be used to demonstrate structural principles and performance level classification as outlined in the previous chapter.

A good project example to start with is an historic greenhouse in southern England. Bicton house botanic gardens palm house built in 1820's with a lattice iron glass composite shell as presented in Figure1a. This structure benefits from curvilinear iron bars closely spaced, clad with small cast glass sheets. As with all historic structures' structural systems, hierarchy and element utilization is difficult to assess. However, it is very likely that the glass skin is carrying some of its own weight as well as providing global stability to the entire building. Iron ribs might be not able to carry all of its vertical load and certainly some global buckling instabilities may occur if large portion of glazing are removed. As per the above classification, this project would achieve class 2b.

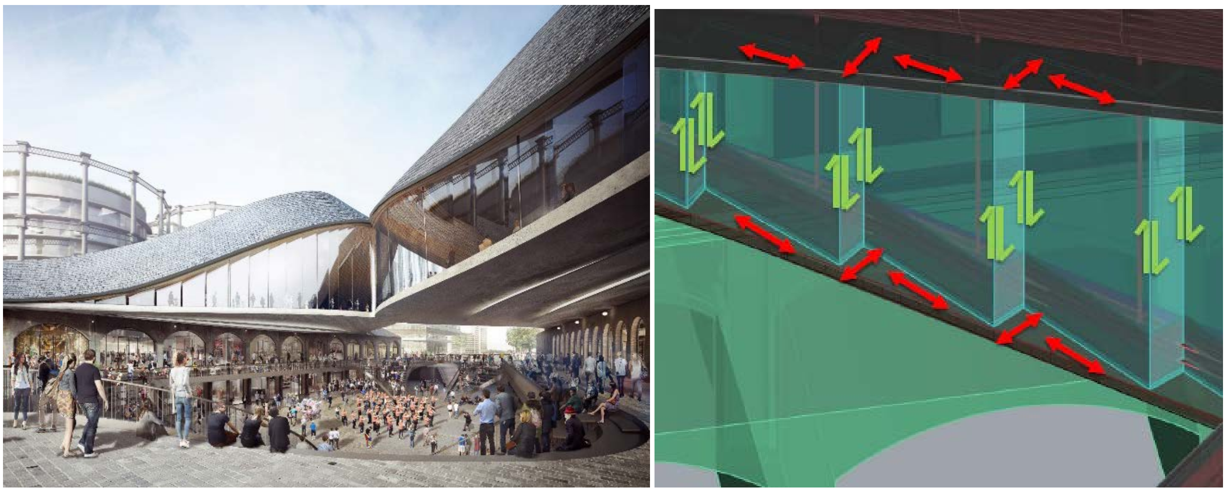
Another more recent example of a hybrid glass structure is the Bombay Sapphire greenhouse, a folded glass plate structure. The general design philosophy in Bombay sapphire was for the steel to carry the load vertically to the ground and for the glass to provide the structure's shear stiffness and buckling restraint to the steel. This can be assumed as typical hierarchical structural arrangement and this follows recommendations for class 2b. As outlined in the previous chapter, in an extreme design situation considered in the early stage of this project, it is also possible to design steel elements only as a medium to transfer load between glass panels and to provide additional robustness and damage tolerance to the system (class 3a).

a.) b.)

Fig. 1 a.) Bicton House, b.) Bombay Sapphire (Heatherwick studio)

The idea of designing class 3a structures in more experimental projects during concept stage to investigate all possible alternatives for clients and collaborators has been trialed. Some of our projects are well progressed in to construction stage, others are in concept stage waiting for a brave contractor or client to move to the next project phase.

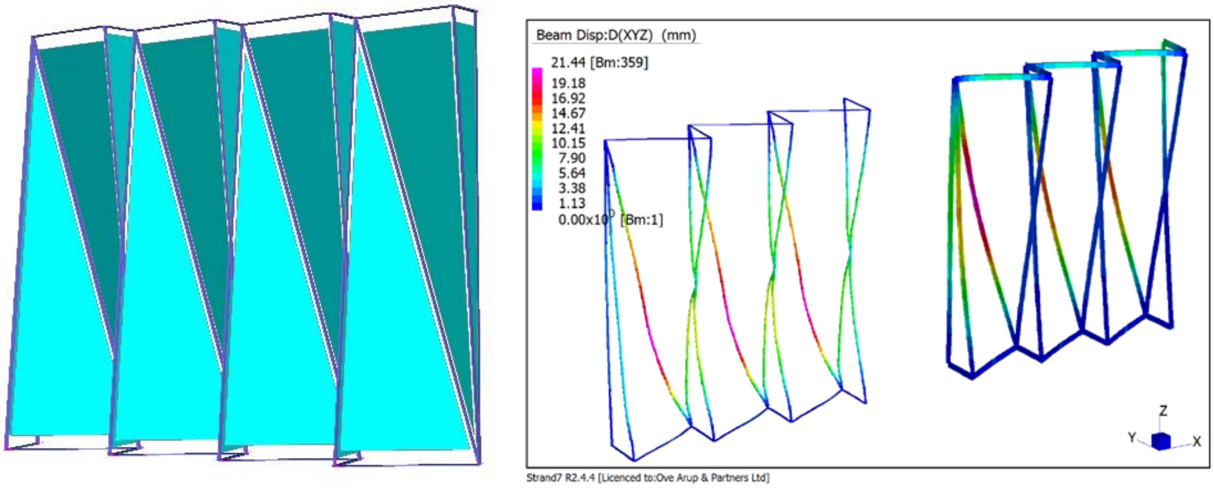
In figure 2a, an example of a saw tooth glass façade for a London shopping mall is presented. This project is on site and completion is expected in late 2018. Our team took advantage of the proposed geometry, where the return glass panels are acting as a support to the face panels and vice versa. Behaviour of the folded glass structures were studied by Marinov, et al (2016) and Marinitsch, et al (2015) in substantial detail. This project is a typical structural glass project, with as glass - to - glass structural silicone joints transferring load between glass panels with no steel elements. As per the classification, this is class 3b.



a.) b.)

Fig. 2 a.) Coal drops saw tooth structural glass façade (Heatherwick studio) b.) Structural principles

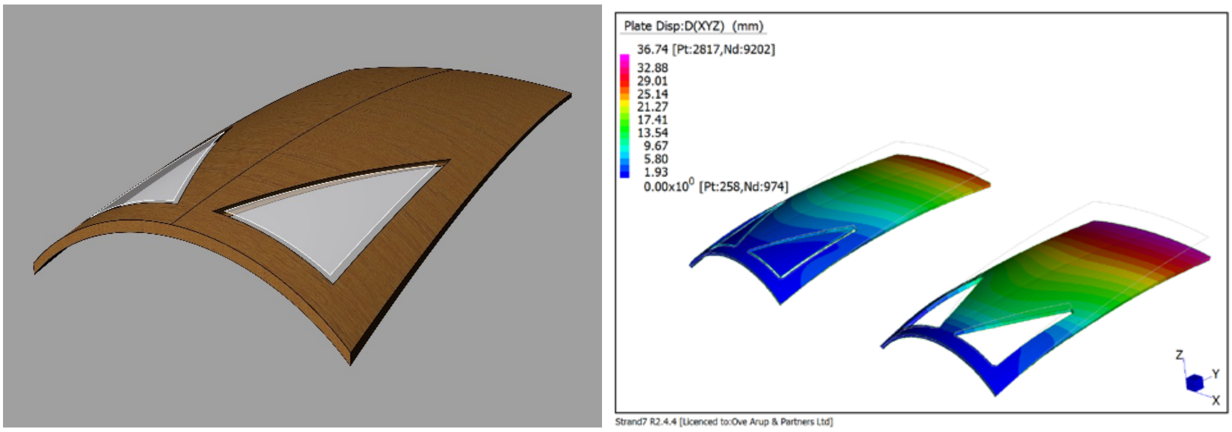
A similar concepts includes a folded hybrid glass facade with steel trim between glass joints is presented in figure 3a.) The glass façade is formed from the planar triangular glass elements with maximum size of 8m by 2m. Laminated glass (2x8mm) was considered in this example as no insulated glass unit was required for this shop front. Two options for the stainless steel edge beam were presented to the client. In the first concept, a stainless steel plate of 50 x 10mm was considered. The main function of this plate is to transfer loads between glass elements. An additional function is to increase redundancy and damage tolerance of the system. This will follow requirements of class 3a. In the second option, a more traditional vertical mullion (structural Tee section of 140 x 60 x 10) was proposed. In this system glass elements are providing lateral restrain to the slender steel section and as such ultimate limit state justification had to be carried out and lateral torsional buckling capacity evaluated. This design will follow recommendations given for class 2b. Results of the steel element deformation subjected to the out of plane wind load only for both options is presented in the figure 3b.



a.) b.)

Fig. 3 a.) Folded glass façade consisting of complex geometry, b.) analysis results steel deformations

An example of double curved shell entrance canopy for a current London project is presented in the figure 4a. Cross laminated timber and fibre (glass and carbon) reinforced plastic is considered as the material palette for the primary structure. The canopy is cantilevering from a continuously supported back edge with clear span of 6m. It is 3m wide. Two glazed triangular panels of 2m by 1m are located at the back of the canopy. As the elastic modulus of the timber is 1/6th smaller than the elastic modulus of glass, we investigated an idea to include triangular glass components to the structural load path of the timber scheme. Two finite element models consisting of shell elements were analysed. The first model presented in figure 4b (left) is benefiting from structural glass while the second model presented on figure 4b (right) is using glass only as a load patch transferring out of plane loads to the primary structure. Analysis results are presented in the figure 4b below. As expected canopy with 2x10mm structural glass contributing to the primary structural load path in about 30% stiffer than option with the infill glass panels considering the same thickness of the shell elements for the cross laminated timber. While such results might be encouraging to an unexperienced or perhaps an enthusiastic engineer, it shall be noted that aspects as prolonged design and approval period linked together with additional testing and certain requirements of specialist glazing contractors, might negate initial cost or material savings made. As per our classification, the first option will be class 2a while second option will be class 1.



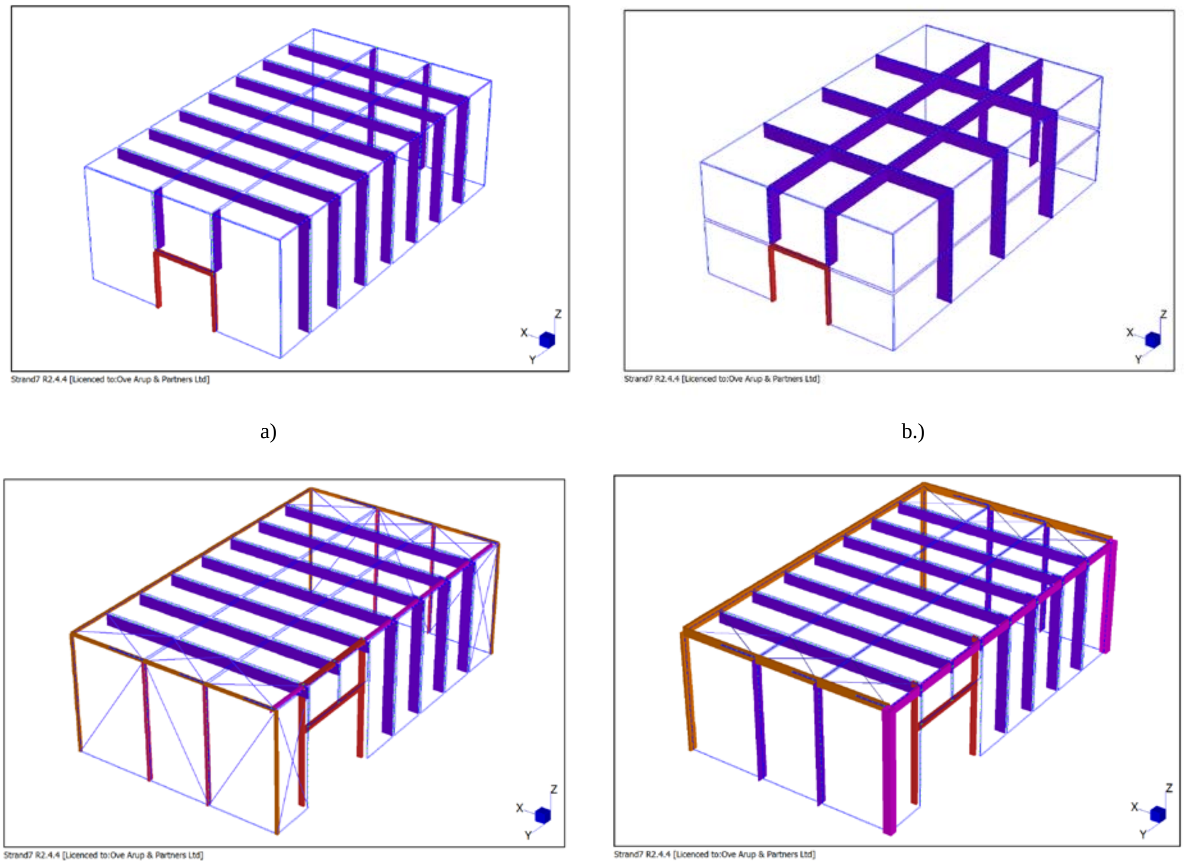
b.) b.)

Fig. 4 a.) Double curved canopy, b.) Analysis results, serviceability limit state

Another project presented in this paper is a typical structural glass project with a very traditional rectangular box geometry of 13.5m x 8m x 4.5m. Particular challenges with this project include its location, which is on the 36th floor terrace of a high-rise tower in Melbourne. This presented the design with challenges related to the installation, access and maintenance. The Atrium glass box comprises three glazed walls and a glazed roof which interfaces with the trunk curtain walling of the main tower. The box provides access to the terrace and daylighting to the office space below. At the beginning of the concept stage we developed fully glazed structural glass solutions presented in the Figure 5a, b. as per initial client brief. The fully glazed box included structural glass beams and fins, structural double glazed units which act compositely for the stability of the structure and allowed for a steel –free design with desired glass corners and edges. This will follow our classification of class 3b.

After initial costing exercise of all glass schemes and alternative more traditional designs, aligned with industry norms for stability and support of glazing as presented in the Figure 5 c.),d.) were developed. Those designs benefited from perimeter steel brace frame or, alternatively, portal frames to provide stability to the box and as such removing the requirement for full height structural double glazed units. Savings were provide in the glass build ups, testing and approval procedures required to verify bespoke design. This will follow recommendations as specified for class 1 project.

Obviously, the size of steel elements or need for cross bracing was not fully appreciated by neither architect nor client. Detail design work to optimise and reduce steel cross sections was carried out. Eventually, it was concluded that the best way to reduce the size of steel elements was to adopted a class 2a approach, where steel elements are able to carry ultimate limit state loads but require help from the façade panels to reduce the sway deformations to the acceptable limits.



a) b.)

c.) d.)

Fig. 5 Glass box, design stages a.) Structural glass option 1, b.) Structural glass option 2, c.) Hybrid glass steel braced frame d.) Hybrid glass steel portal frame

4. Codified checking procedures

4.1. Compression buckling capacity of steel mullions

In all the examples mentioned above, the steel mullions are quite slender and as such they are susceptible to buckling. However, the buckling performance of the mullions is significantly improved by intermittently connecting the glass panels to the steel mullions in a way that provides lateral restraint. This enabled significantly higher stress to be taken through the mullions to ground. The compression buckling loads of the mullions were determined by linear buckling analysis of all load combinations.

Each unique buckling mode can then be used to back-calculate an effective length of the mullion to use in a standard compression design check for codes around the world that still use effective lengths. Eurocode 3 allows for the direct input of the elastic critical buckling force. It shall be noted that glass restraint shall be considered carefully and connections with fuse, and/or springs stiffness should be added to the details to protect the glass from overloading.

Glass restrains can effectively increase buckling lengths however, and accidental conditions should be considered in the design accordingly.

Euler buckling equation can be re-arranged in terms of effective length as:

$$L_e = \sqrt{\frac{\pi^2 EI}{P_{crit}}} \tag{1}$$

It is important to use the appropriate second moment of area, I_x or I_y. The engineer must consider which axis of the mullion is buckling and use the correct second moment of area for the direction that is buckling. The calculated effective length should match the visual buckling length of the mode shape in the buckling analysis. Once the effective length is calculated and visually confirmed from the buckling analysis the mode shape can be used in the calculation of a compression buckling capacity to code you are using based on its standard equations.

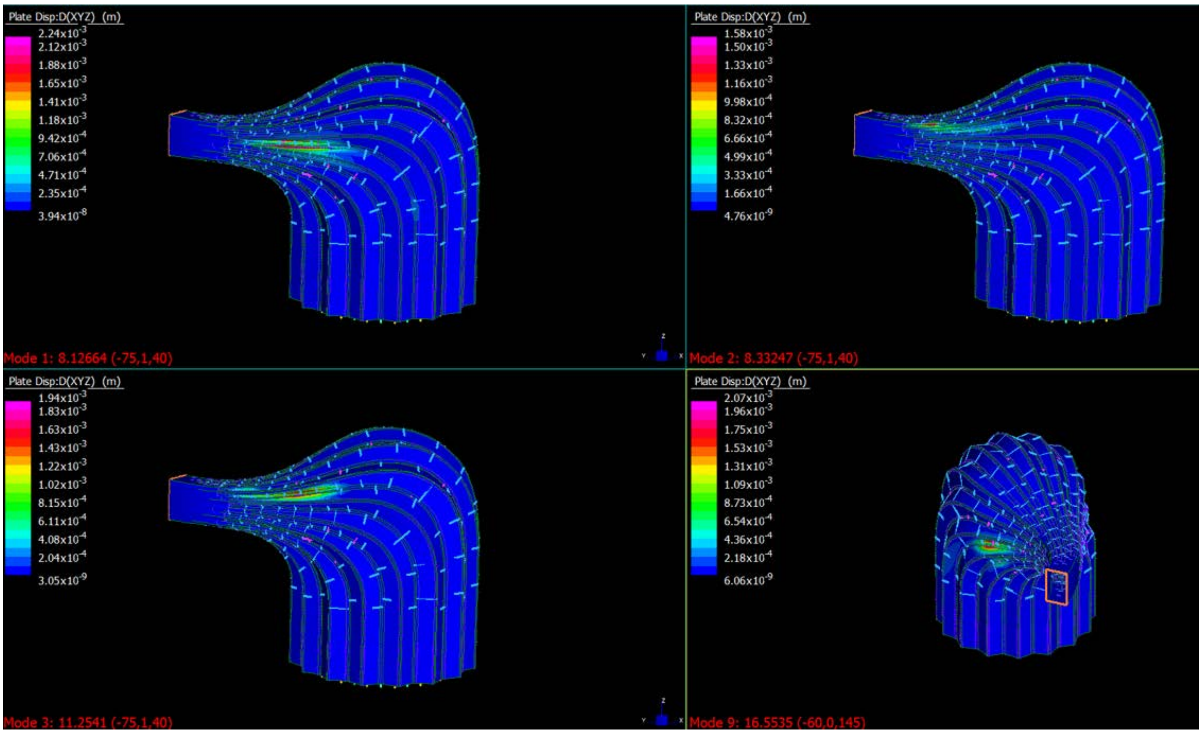


Fig. 6 First four buckling mode shapes Bombay Sapphire

4.2. Lateral torsional buckling capacity of steel mullions

The mullions of Bombay Sapphire were chevron shaped sections and as such would be susceptible to lateral torsional buckling. To reduce this susceptibility the glass panel connections to the mullions were designed to provide some rotational stiffness along with their lateral restraint to each end of the mullion chevron flange. To ascertain what effect this rotational stiffness restraint would have on LTB of the mullion, it was modelled numerically. Traditional beam element formulations are not able to capture lateral torsional buckling and so each mullion was modelled with shell elements in a second global model. The rotational stiffness of the glass to mullion connection was modelled between the mullion shells and glass shells with rotational springs. An elastic buckling analysis was then carried out for all load combinations. It was observed that the rotational stiffness of the glass connection was sufficient to stop all lateral torsional buckling effects. Therefore the lateral torsional buckling capacity was equal to the section moment capacity. However, if lateral torsional effects should be included, section 6.3.2 from Eurocode 3 can be used. The elastic critical moment can be determined in the finite element model for each load case by multiplying the moment in a profile by the buckling factor. The relevant section modulus should be used and formula 2 can be used to calculate the slenderness. The slenderness determines the reduction of moment capacity.

$$\lambda_{LT} = \sqrt{\frac{W_y}{M_{cr}}} \tag{2}$$

4.3. Structural adhesives

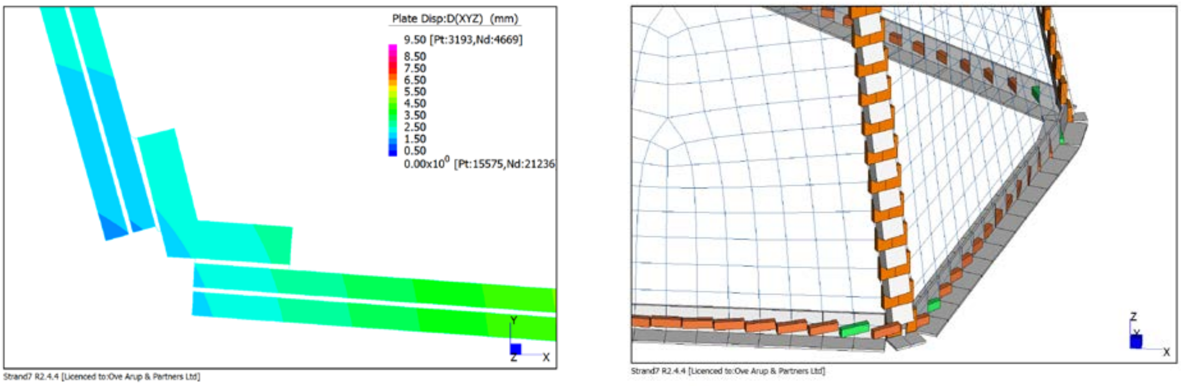
If structural adhesives are applied to create primary means of connection for a hybrid structure, guidance from manufacturers can be adopted. ETAG002 can be used as guideline. Both static (long term) and dynamic loading (short term) should be checked with the relevant design stresses. A combination of normal and shear stresses can be checked by using the formulas based on basic elastic material model (Mohr’s Circle) or any other advanced constitutive material models defining failure mechanism of a hyperelastic material.

5. Structural analysis and modelling procedures

5.1. Combination of global and local models

When glass panels provide the rotational stiffness restraint to the steel mullions at discrete points along the length of the mullions a combination of a global model and local sub models can be used to analyze the structure. The restraint of the mullion provided by the glass depends on the connection detail, connection stiffness of the materials and general fit and tolerances. The amount of the rotational stiffness could be determined from a detailed (local) finite element model of the joint show in Figure 7a below. Preferably, connection stiffness is confirmed by the experimental testing as this may influence force redistribution in the global model. This was achieved for the Bombay Sapphire project where a known force was applied at a known distance and the displacement and rotation of the joint measured. The calculated rotational stiffness was input into the global structural model at rotational spring elements that were positioned at the locations along the mullion where this joint was present.

In the global model the designer can test the influence of connection stiffness sensitivity and investigate nonlinear effects. Multiple connections should be integrated as for example spring supports of the mullion bases. Substructure can be modelled to reflect stiffness variations and as such potential load redistribution. Post failure analysis of key glass elements as well steel mullions can be assessed. In the global model it is good practice to model the glass panels as shell elements to provide realistic shear stiffness. However, if the glass would only act as global bracing this could be modeled in a global model with 1D elements in the early stages of project to reduce complexity, computation time and give engineer a chance to understand magnitude of forces and follow the load path more clearly.



a.) b.)

Fig. 7 a) Connection sub model; b) Connection in global model

5.2. Hybrid models

Sometimes it’s difficult to simplify the modelling of the connection in a global model. A combination of glass and other materials could be seen as one hybrid element that should be modelled accordingly. The interaction between glass and another materials can be modelled in various ways and is more complex than modelling just a single material.

For a double curved glass door, a similar principle to the one used in the car industry is considered, in order to reduce glass thickness (to simplify production of the curved element): this bonds a thin metal frame to the curved glass to reinforce the glass. Windshields are generally bonded by means of PU to add overall stiffness. Black screen prints are then applied to the edges of the glass to protect the adhesive from UV. For the door case study for one of our current bespoke facade project, which is unfortunately confidential, 2.7m tall and 1.2m wide finite element shell model was analysed. Two approaches to include an edge beam of the stiffened edge will be compared as shown in the Figure 8 below:

- Modelling type A: Glass is modelled with the shell elements as usual. Connection is modelled with volumetric elements and frame is modelled with shell elements.

- Modelling type B: Glass is modeled with shell elements as usual. Connection is modeled with plate elements and frame is modeled with beam element.

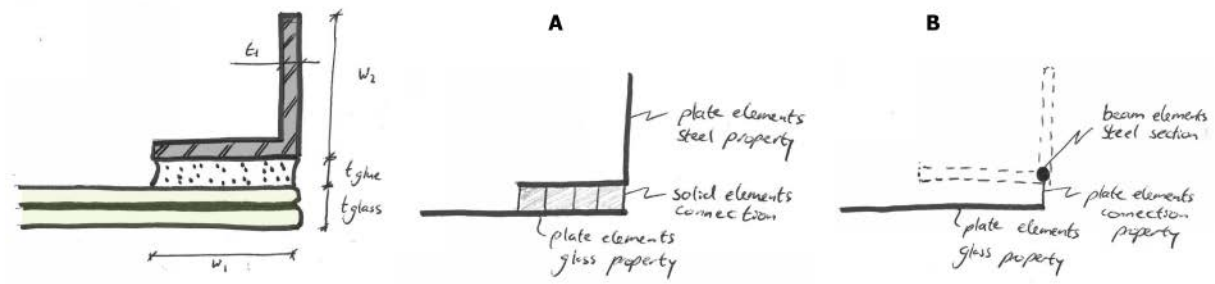
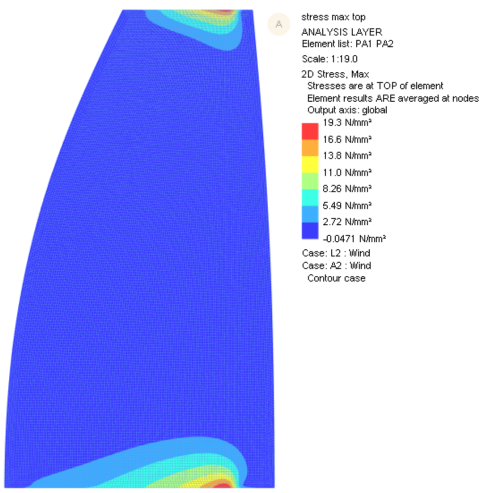
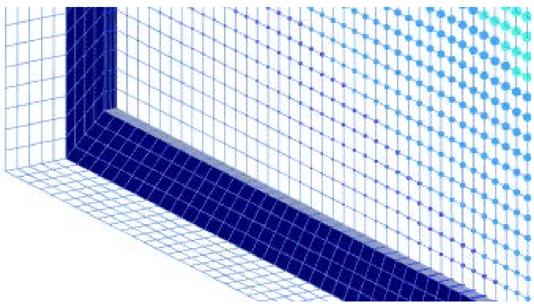


Fig.8 Sketch of frame and two types of modelling

The advantage of option A is that the model is more accurate than option B as the frame is correctly coupled at one flange to the glass, however meshing is slightly more complex. Option B is applied regularly in the field as this is an easy way to see the influence of a stiffened edge and can be modelled in most software programs. It also has the possibility to show moments in the steel frame and to use the codified checking as mentioned in the previous paragraphs.

The glass pane is modelled as shown below with a steel edge profile w1 = w2 = 50mm and thickness t1 = 6mm. A glass thickness of 12mm is applied. A wind load of 1kPa is applied. The thickness of the bond is mode led as 10mm (width is w1) and the material stiffness is varied. It is shown that the stress near the (clamped) supports of the door are higher for increasing stiffness of the bond as expected and stresses in the centre of the door decrease. Most noticeable is the large difference in deformation (42%) of the door with bonded frame for high bond stiffness of 100 MPa. This is due to the difference in modelling of type A and B. Type A describes the composite action between glass and steel better due to the geometric nature of 2D elements compared to 1D elements and should therefore be applied in case of high stiffness bonding.





	A			B					
	3D elements for sealant			2D elements for sealant			Comparision		
	Glass			Glass					
	supports	center		supports	center				
E_bond	σ_top	σ_bot	δ	σ_top	σ_bot	δ	σ_top	σ_bot	δ
N/mm2	N/mm2	N/mm2	mm	N/mm2	N/mm2	mm	%	%	%
1	12.6	7.49	29.2	13.1	9.24	32.2	4.0	23.4	10.3
10	15.3	6.33	24.3	16.1	7.63	27.9	5.2	20.5	14.8
100	19.3	6.53	18.2	21.3	8.15	25.9	10.4	24.8	42.3

Fig. 9 Sketch of frame and two types of modelling

6. Conclusion

While structural glass solutions are still demanded by our clients, combination of glass and other materials could help to improve structural performance and if combined with low carbon material reduce carbon footprint of the structure. In this paper, we discussed design methodology and introduced performance classes. On our past and current projects we explained those performance requirements and demonstrated structural principles. Later in the paper we outlined on our project methods of analysis and checking procedures required to satisfy current design codes. We would like to continue with this research as we feel that this topic can be explored in more detail.

Acknowledgements

Acknowledgements to the Invest in Arup fund IiA , team members: Graham Dodd, James Griffith, Vladimir Marinov, Sara Clark, Luis Soares – Martins, Patrick Rogers, Mike Aurik, Adrian Roiz

References

Bijster, J., Noteboom, Ch., Eekhoult, M., Glass Entrance Van Gogh Museum Amsterdam, Glass Struct. Eng. (2016) 1:205–231 DOI 10.1007/s40940-016-0022-5

Bos, F.P., Safety Concepts in Structural Glass - Towards an Integrated Approach, PhD Dissertation, TU Delft, 2009

Dodd, G.,Structural Glass Sandwich Panels, In: GPD 2017, Finland (2017)

Fine, W.T.: Mathematical evaluations for controlling hazards, 1971.

Kinney, G.F.: Practical Risk Analysis for Safety Management. Naval Weapons Center, NTIS report number NWC-TP-5865, 1976.

Lenk P., Honfi,D. Resilience, Damage Tolerance & Risk Analysis of a Structure Comprising Structural Glass, In: engineered transparency, International Conference at glasstec, Düsseldorf, 20-21 September, (2016)

Marinov,V., Griffith, S.J., Marinitsch, S Teich, M., Structural Analysis of Folded Glass Plate Structures, In: Challenging Glass 5, Belgium (2016)

Marinitsch, S., Schranz, C. and Teich, M. (2015), Faltwerke aus Glas. Stahlbau, 84: 201–211. DOI:10.1002/stab.201590078

Starrossek, U., Haberland, M., Approaches to measures of structural robustness, p 625-631 DOI: 10.1080/15732479.2010.501562 (2010)

Xin, W., Blockley, D.I., Vulnerability of structural systems part2: Failure scenarios p319-333 DOI: 10.1080/02630259308970131 (2007)