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# DEVELOPMENT OF BLAST RESILIENT GLASS FACADES FOR WORLD FINANCIAL CENTRE IN NEW YORK

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**Abstract:** The World Financial Centre (WFC) in New York has a ground level shopping area known as the Courtyard. A replacement glazed facade to the Courtyard Entrances was to be provided and, given the high profile nature of the building, the design was required to be blast resilient. This paper demonstrates the design process and philosophy utilised in developing the blast resilient design and highlights how various design issues were investigated and resolved. The main challenge was the demonstration that long-span glass fins columns would not break under blast loading, particularly due to lateral torsional buckling during dynamic rebound. Dynamic interaction between various components of the façade was an important consideration while studying the dynamic response. The design of the glass fin friction grip splice connections was particularly challenging and required testing to be undertaken by Pilkington.

## Introduction

The World Financial Centre (WFC) in New York has a ground level shopping area known as the Courtyard, which was scheduled for refurbishment. The refurbishment included installation of a replacement glazed facade to the Courtyard Entrances. In addition to normal glass design requirements, the specification required that the building façade satisfied prescribed bomb blast criteria.

W&W Glass LLC won the contract to undertake the refurbishment with support from Pilkington Architectural. Pilkington appointed MMI Engineering Limited (MMI) to act as blast engineering consultants. MMI’s design work was subject to peer review by Wiedlinger Associates Inc (WAI).

In order to develop a blast resilient design, hand calculations, linear and nonlinear dynamic analyses have been carried out by MMI. This paper demonstrates the design process and philosophy utilised in developing the blast resilient design and highlights how some of the main design issues were investigated and resolved.

## Façade description

The Courtyard has two main entrances, one from the North side on Vesey Street and the other on the South side from the Marina. The Vesey Street entrance required a replacement glass façade measuring approximately 21m wide by 17m high. On the Marina side, the replacement facade measures 27m wide by about 12m high. An overview of the Vesey Street façade during construction is shown in Figure 1.

The main components of the glass façade are as follows:

- Insulated Glass Units (IGUs) span vertically and are analysed as one-way slabs, simply supported. Distance between the horizontal mullions is about 1.4m. The insulated units are composed of 9.5mm fully toughened (FT) outer lite and 12.5mm laminated heat strengthened (HS) glass inner lite (6.4mm HS – 1.52mm PVB – 6.4mm HS) with a 12.5mm air gap between the two lites.

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- Horizontal T-mullions span horizontally 3m between vertical glass fins. The mullions are solid steel sections 95mm deep by 38mm wide with T-shaped aluminium profiles screwed into the outer face.
- Laminated glass fins support the mullions at 3m centres. The fins are about 1.0m deep by 36mm thick on Vesey Street and 0.9m deep on the Marina Entrance and span vertically over the full height of the façades. Due to limitations on the length of laminated glass fin which can be supplied, these fins are in two or three lengths with bolted splice connections (see Figure 1).


*Overview of the façade under construction* *Glass fin with splice connections*

**Figure 1: Photographs of the WFC façade during construction**

**Blast Loading and Required Performance**

The blast loads for the design of courtyard façade were taken from the design specifications and was prescribed in terms of pressure/impulse and was considered to be uniformly applied to the façade.

The following performance criteria were established for the blast loading case:

- The IGUs could shatter but not become detached from the supporting structure
- The aluminium glass carriers, steel mullions and fixing brackets were permitted to undergo controlled plastic deformation
- Glass fins were not allowed to break
- Slippage at friction grip connections was not permitted
- Connections were required to remain elastic with suitable safety factor

**Design Strategy**

In order to demonstrate that the above performance criteria were satisfied, a design strategy was established to define how each component would be assessed and what potentially critical modes of behaviour needed to be investigated.

It was concluded that dynamic response of steel/aluminium sections and the assessment of plastic deformation was conventional and could be readily demonstrated using single-degree-of-freedom (SDOF) hand calculations. The details of this are not covered in this paper.

The dynamic response and breaking behaviour of the IGUs was more complex because of the air gap; while SDOF approaches gave a reasonable indication, it was concluded that a 2-

degree-of-freedom (2DOF) approach would be more robust. This was implemented following the methods described in [1].

Because the glass fins were required to remain elastic, their dynamic analysis was potentially relatively straightforward. However, it was identified that there were several design aspects which required investigation. Selected aspects are summarised below:

- **Dynamic interaction** –oscillation of the IGU and mullion assemblies could tune in with the fin response leading to dynamic enhancement of forces and displacements.
- **Lateral torsional buckling (LTB)** – the resistance to reverse loading, when the fin becomes an outstand acting in compression, was a potential design concern. Since no lateral ties were permitted by the architect, it was necessary to demonstrate that LTB under dynamic rebound did not result in failure of the fin.
- **Splice design** – the design relies on the ability to transfer moments and shear forces between glass and steel components. The technology of such connections requires the use of adhesive to generate a coefficient of friction combined with bolting to provide an external clamp force. Slippage was to be avoided.

Composite action between the fin and the façade was identified as a significant structural behaviour. Hand calculations initially considered this and were confirmed through subsequent FEA. This aspect is not covered in detail in this paper.

In addition, the physical proximity of the glass fins to other building elements meant that its dynamic response needed to be understood and sufficient clearance provided to prevent impact with hard points. This aspect is not covered in detail in this paper.

In order to demonstrate the above design issues with confidence it was concluded that a range of hand calculations, FEA and supplementary testing in cooperation with Pilkington was required. Each of the above topics is discussed in the following sections.

### Dynamic Interaction

In order to understand the importance of dynamic interaction in the design of these façades, hand calculations and FEA were employed.

<u>Hand Calculations</u>

For the hand calculations, a step-by-step SDOF approach was used where it was assumed that there would be no dynamic interaction between the glass, mullion and glass fin and therefore each component in the load path could be analysed using the transferred reactions/loads.

The hand calculations were based on the assumption that the IGUs are one-way spanning and simply supported along the long edge and decoupled from the mullion. The response of the double glazed unit was computed using a two degree of freedom model, with each pane modelled separately as a SDOF system following the method suggested by [1].

The ductility demand on the steel mullions was estimated by treating them as SDOF systems subjected to the reactions from the IGUs.

The mullions are connected to the glass fins by stainless steel brackets. The stiffness of these brackets determines the boundary conditions at the mullion support on the glass fin. For hand calculations, two extremes of boundary conditions were considered for the mullions namely fully fixed and pinned supports. The glass fin brackets connecting the mullion are secured by a pair of bolts at several locations over the height. The dynamic reactions from the mullions were applied to the glass fin.

Laminated glass fins are connected to the primary structure at the top end by a steel pin and sit in shoes cast into the structural slab at the bottom end; the shoe is provided with a neoprene lining to allow thermal expansion/contraction. For the glass fin hand calculations, a pinned boundary condition at the top and roller support at the bottom was assumed.

Friction-grip splice connections are used to connect adjacent glass fin sections. The splice connections were not considered explicitly in the hand calculations. It was initially assumed that splice connections would be able to withstand the developed moments and therefore no slippage would occur at these locations. The fin was therefore considered as a constant section beam and the moment developed in the fin was used to separately design the splice connections.

Based on the use of Heat Strengthened glass it was found that the fins were only working up to 40% of yield. The stiffness and ultimate resistance of the fin was based on the full section of the glass layers acting together, with the PVB effectively behaving in a rigid manner under very short-duration loading.

It was observed that the natural frequency of each component in the system was similar. Furthermore, the degree of flexibility offered by the glass fin to the mullion varies with height. It was concluded that the decoupled step-by-step approach was potentially non-conservative and that a coupled MDOF analysis was required to investigate possible dynamic interaction.

Finite Element Analysis

Supplementary FEA of the curtain wall was conducted to investigate dynamic interaction between different components.

The FE model of the façade was limited to one bay which is representative of the central section of the façade. It was assumed that the central section would be conservatively subjected to uniform blast loading and would respond symmetrically.

The FE model of the Marina façade is shown in Figure 3. A similar model was also developed for the Vesey street façade. The FE models comprise the following parts:

- Vertical spanning IGUs were modelled as shell element with equivalent thickness to represent the actual stiffness of the glass and density adjusted to take proper account of the mass. Based on the hand calculations no cracking was expected in the glass so the PVB layer was not explicitly modelled.
- The T-Mullions were modelled as beam elements. An elastic-perfectly plastic model for the steel was assumed, ignoring strength increase factors (SIF) and dynamic increase factors (DIF) and strain hardening effects, in the interests of maximising the mullion response. Sensitivity studies with higher reactions were also considered.
- The glass fins were modelled as elastic beam elements or shell elements when a more detailed investigation was required.
- The glass fin brackets were modelled as shell elements. Nonlinear stress strain curve for 316L grade stainless steel was used.

The blast analysis was performed by applying the prescribed blast load on the glass façade. The boundary conditions imposed on the system are illustrated in Figure 3. It was also assumed in the analysis that the connection between the glass and the mullion is pinned based on the use of thin bonded aluminium carriers, which are simply screwed to the T-bar sections.

The results of hand calculation and FEA for both façades are summarised in Table 2. It can be seen from the comparison that the Marina entrance fin response predicted from FEA agrees well with the hand calculations in terms of both the displacements and bending moments. However, for the Vesey Street entrance façade, there is a relatively large discrepancy between the fin response predicted from hand calculations and FEA, with the hand calculations actually showing a larger response. This can be attributed to dynamic interaction between the various components of the façade which is not captured in the hand calculations.

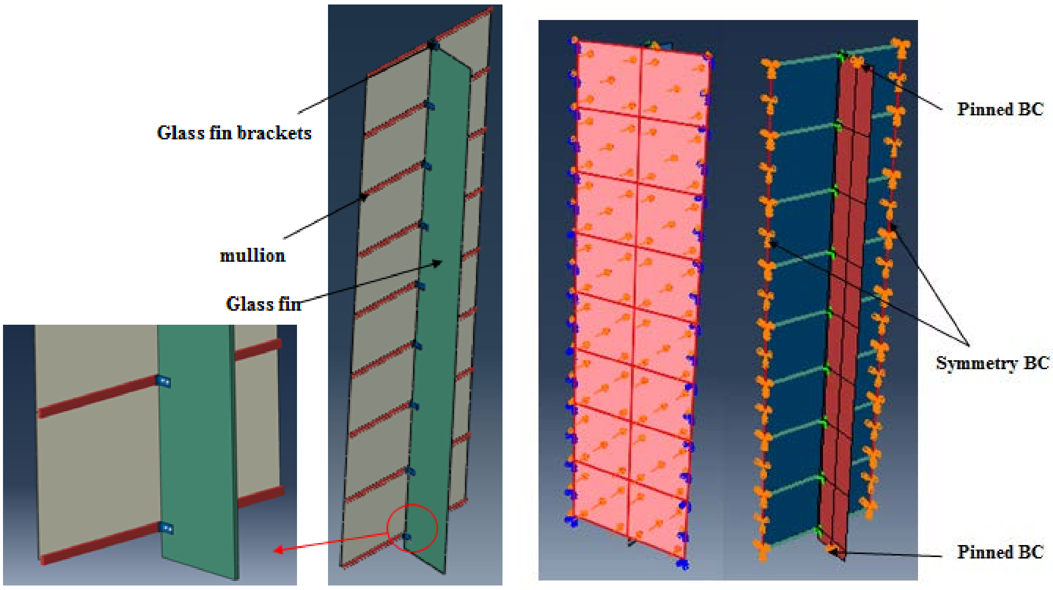
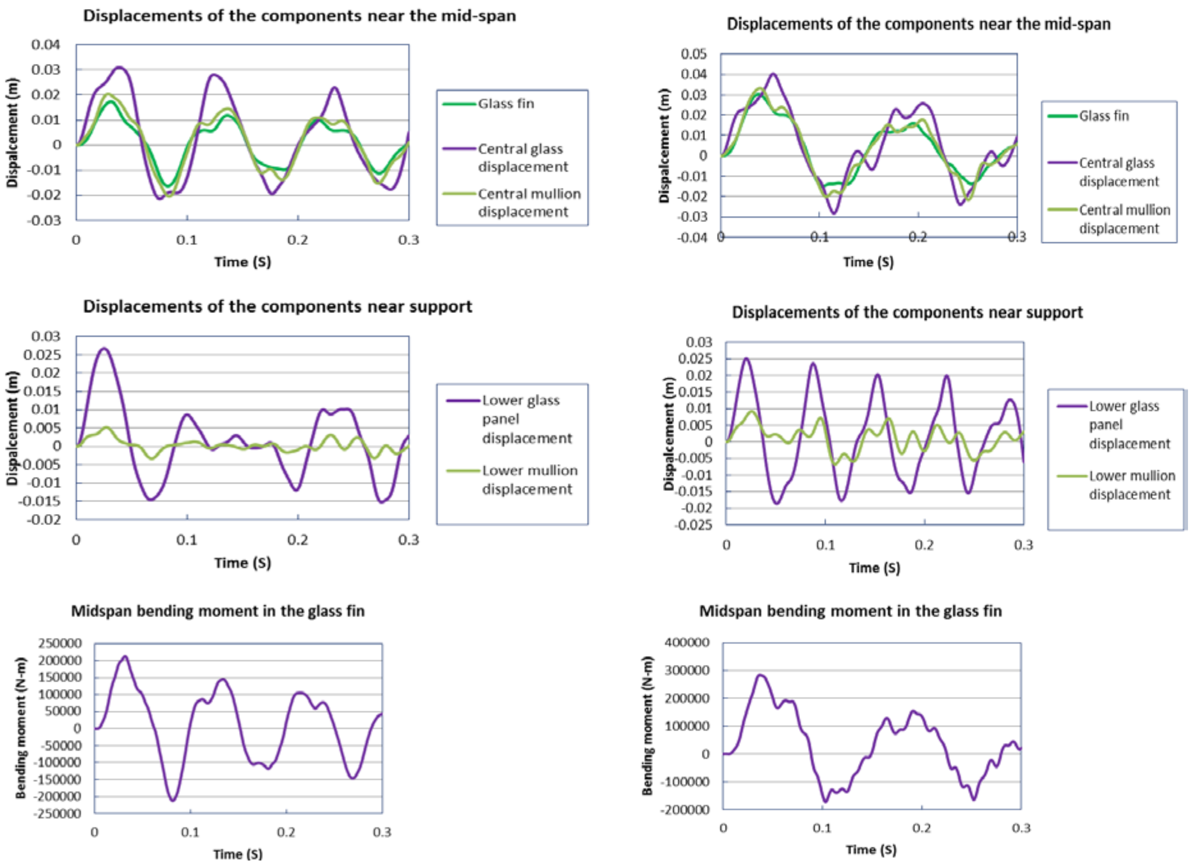



Figure 2: FE model of the curtain wall system

Table 1: Comparison of hand calculations with FEA

| Analysis Parameter | Marina Façade | | Vesey Street Façade | |
|---|---|---|---|---|
| | FEA (including dynamic interaction) | Hand calculations (excluding dynamic interaction) | FEA (including dynamic interaction) | Hand calculations (excluding dynamic interaction) |
| Main bending frequency of the system | 10.6Hz | 9.8Hz | 6.9Hz | 5.4Hz |
| Maximum IGU displacement | Absolute: 30mm Relative to mullion: 19-22mm | 20-23mm | Absolute: 40mm Relative to mullion: 17-22mm | 20-23mm |
| Maximum mullion displacement | Absolute: 20mm Relative to fin: 5mm | 36mm-104mm | Absolute: 33mm Relative to fin: 9mm | 36mm-104mm |
| Maximum glass fin displacement | 17mm | 16-17mm | 30mm | ~45mm |
| Glass fin maximum bending moment | ~224kNm | ~227kNm | ~285kN-m | ~420kN-m |
| Bending moment at the splice location | ~224kNm | ~225kNm | ~260kN-m | ~364kN-m |

The deflection and moment-time histories shown in Figure 3 illustrate this feature of the response. For the Vesey Street façade, it can be observed that at the time of the peak fin response, the glass near the mid-span is responding slightly out-of-phase with the fin. This reduces the maximum bending moment in the fin, explaining why the FEA has given a lower fin bending moment. For the Marina façade, however, the glass is responding fully in-phase with the fin and therefore the transmission of the forces through the load path in the FEA is similar to that assumed in hand calculations. For this reason, the agreement between the hand calculations and FEA is better in the case of Marina entrance façade.




*(a) Response of the Marina entrance facade*

*(b) Response of the Vesey street entrance facade*

Figure 3: Displacements and bending moment time-histories

**Lateral torsional buckling**

Lateral torsional buckling is a phenomenon which reduces the strength of beams in bending where the compression flange is laterally unrestrained. Under loading, the compression flange becomes unstable and will naturally try to sway to one side, inducing twisting into the beam. This reduces the strength and stiffness of the beam and can lead to a rapid loss of load carrying capacity.

Glass fins are particularly vulnerable to this because the minor axis strength is much less than the major axis strength. The glass fin is unrestrained on the interior edge and will be in compression during the dynamic rebound phase of blast response. This can make the glass fin unstable. One possibility of avoiding the problem of lateral torsional buckling is to tie the interior edge of the glass fins. However, this option was rejected by the project architects.

Lateral torsional buckling of the glass fin was studied by hand calculations as well as FEA.

<u>Hand Calculations</u>

Suitable codified guidance was researched and through discussion with Pilkington it was found that only a few standards and documents deal with lateral torsional buckling design of glass fins. Three codes were consulted to study the phenomenon of LTB, namely:

- Australian standard AS-1288 [2]
- Eurocode 3 [3]
- ANSI/AISC 360-05 [4]

The Eurocode and AISC standards discuss the issue of lateral torsional buckling in context of steel structures but it was concluded that the methodology can be applied to the glass fin. However, Australian standard AS-1288 provides a design method for glass fins in various structural configurations.

For a better understanding, the code checks were performed on a fully unrestrained and a partially restrained (i.e. tension flange restrained) glass fin, where the code methods allowed. The design moment computed for the fully unrestrained glass fin using different codes is summarised in Table 2. All the codes used here give similar design moment for the fully unrestrained glass fin. The benefit of the tension flange restraint is highlighted, significantly increasing the LTB resistance. For the Australian standard AS-1288 check a slightly enhanced factor of safety was included based on extensive research and testing undertaken by Pilkington.

Table 2: LTB moments for the fully unrestrained glass fin

| **Code** | **LTB Moment**<br>**Marina facade** | **LTB Moment**<br>**Vesey Street facade** |
|---|---|---|
| Eurocode 3 | 84kNm | 70kNm |
| AISC 360-05 | 96kNm | 73kNm |
| AS-1288 | 91kNm<br>248kNm (restrained fin) | 71kNm<br>247kNm (Restrained fin) |

<u>Finite Element Analysis</u>

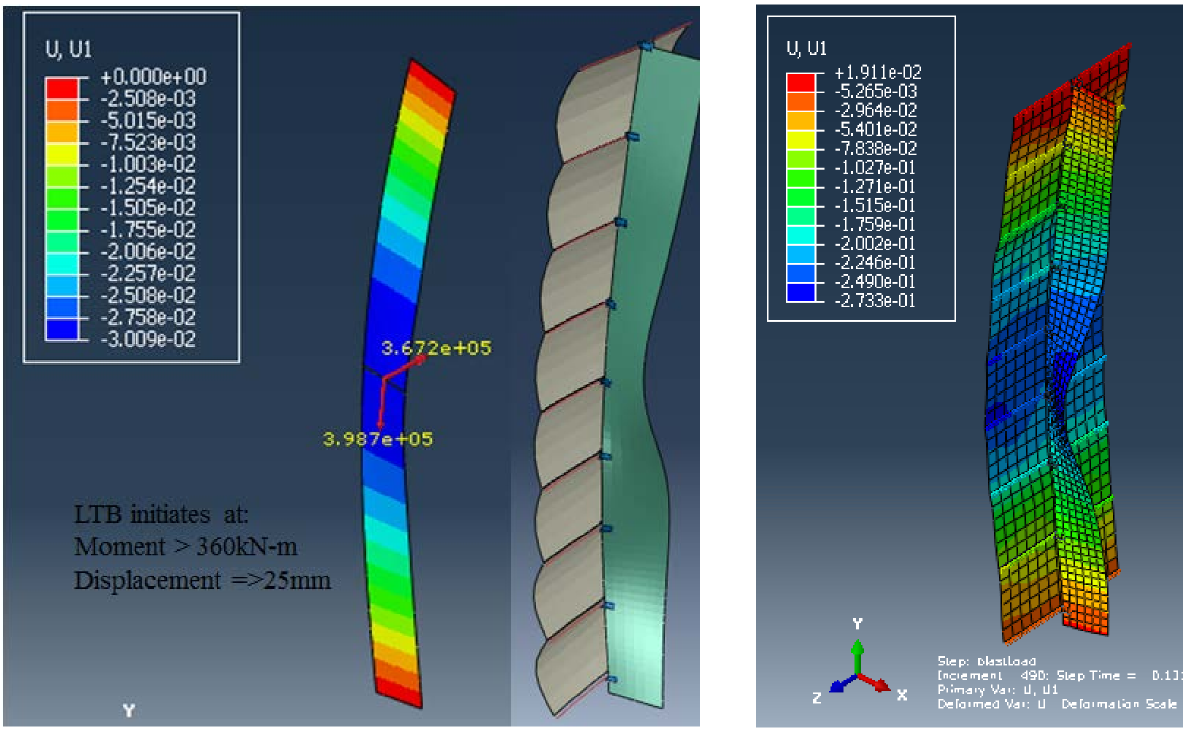
Hand calculations do not take the non-linear load carrying behaviour into account and are generally conservative in that they are based on a lower bound of design buckling moments. Therefore FEA was also conducted to investigate the lateral torsional buckling resistance of a glass fin and the likely dynamic behaviour.

Initially, static pushover analysis of the glass fin was conducted in the outwards direction to estimate the lateral torsional buckling resistance of a partially restrained glass fin to confirm the design moments computed based on AS-1288 codes. The restraint in this case was provided by the façade, connected to the tension face of the glass fin. A dynamic analysis of the glass façade was also undertaken to evaluate the response of the façade and assess the severity of lateral torsional buckling under dynamic rebound.

In all FE models used for investigating the phenomenon of lateral torsional buckling, initial imperfections were seeded in the glass fin to encourage buckling. This was achieved by defining imperfections as a linear combination of the buckling modes of the fin.

The maximum dynamic rebound moments developed in the Marina entrance façade fin and Vesey street entrance façade fin are 224kNm and 285kNm, respectively. These are less than the critical buckling moments of 360kNm and 300kNm predicted from the nonlinear buckling (static pushover) analysis of Marina façade and Vesey street entrance façades. For this reason, LTB does not occur in the dynamic response of the façade for the prescribed blast loading. It is noted that the critical buckling moments are comparable to but about 25-40% greater than the AS-1288 values. This is considered to reflect the conservative nature of codified guidance.

To confirm that the model was capable of capturing lateral torsional buckling under dynamic loading conditions, a dynamic analysis for a hypothetical load corresponding to 10 times higher blast pressure was conducted. This analysis found that LTB did occur under dynamic rebound, as shown in Figure 4.




*Static pushover analysis in reverse direction*　　*LTB due to 10 times blast load*

Figure 4: Lateral torsional buckling studies using FEA

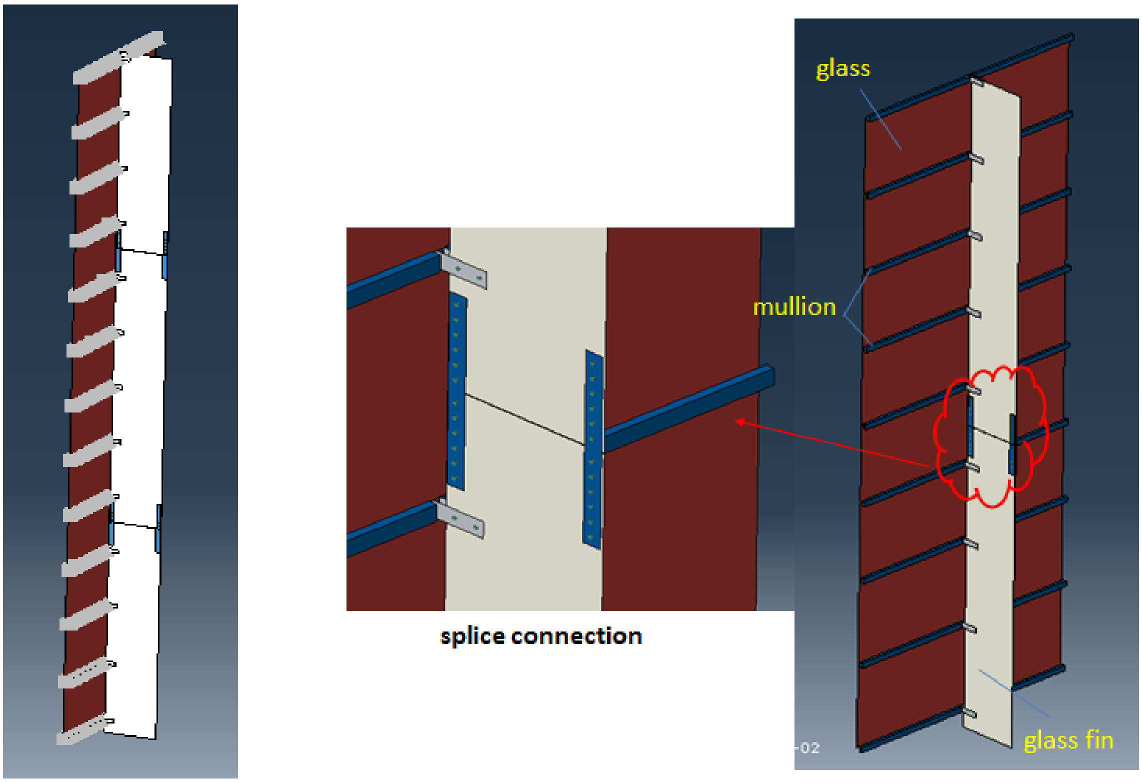
**Glass Fin Splice**

Due to height of the façade and limitations in the supply length of glass it has been necessary to provided bolted splices along the length of the fins. Following on from initial hand calculations, the splice connection design was based on FEA so that the demand on individual bolts could be calculated.

The strength of the splice detail depends on the number of bolts. Due to architectural requirements, the number of bolts was ideally to be limited to 5 bolts on each side of the splice plate. However, due to large bending moments in the fin under blast load, it was difficult to satisfy this requirement.

To minimise the number of bolts, the FE models were refined to explicitly include the friction-grip bolted splice connections. A contact surface between the glass and the splice plates was defined taking account of bolt pre-tensioning and the coefficient of friction of the adhesive applied between the glass and splice plates. The models developed for further investigations of the splice connections are shown in Figure 5.

Initial assessments found that some nominal slippage can occur in the splice connections on both façades. While these could be tolerated if enlarged holes in the glass fins are provided, to avoid the construction and installation difficulties associated with allowing for slippage in these connections, Pilkington wished to make the splice connections stronger so that slip does not occur.




*Vesey street façade* *Marina façade*

Figure 5: FE models developed to evaluate the response of splice connections

It was agreed that the connections would be enhanced through the following measures:

- Use of an increased coefficient of friction
- Increase the number of bolts from 5 to 6 on each side of each splice plate
- Increase the pre-tension in the splice bolts by about 8%

Pilkington supported the design of the splice connections with its experience of suitable friction coefficients. For the final design, the coefficient of friction was determined through testing conducted by Pilkington as illustrated in Figure 6. The test set-up comprised a 3m long cantilever glass beam with a bolted friction connection provided at the clamped end. The friction connection included adhesive which was applied between the glass and steel surfaces and then bolts were tightened to provide a friction grip.

The static tests were performed by slowly applying a load of certain magnitude and monitoring the displacement at the free-end as well as the relative displacement between the angle and the glass at fixed end. Using the magnitude of the applied load at the instance when the connection slips, the coefficient of friction was determined. It should be noted that this is a static coefficient of friction which has been used in dynamic calculations. It was therefore necessary to demonstrate that there was essentially no slippage for the calculations to be valid.

Recognising the risks associated with slippage failure it was agreed with the peer reviewers that a factor of safety was required. This factor of safety was applied to the blast loads. When the dynamic analyses were run it was found that extremely small slippage was predicted in a small number of the bolts but that generally there was no slippage. This gave confidence that the splice connections would behave in an acceptable manner.

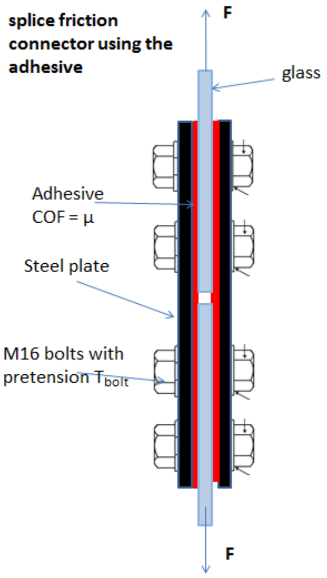




Figure 6: Static tests on splice connections to determine the coefficient of friction

## Conclusions

This paper has discussed some of the design issues addressed during the design of a glazed façade subjected to blast loads. To ensure predictable behaviour and avoid gross collapse of the façade it was a necessary requirement for the glass fins not to break when subjected to the blast loading. MMI worked collaboratively with Pilkington Architectural to achieve this. Three design issues were discussed within this paper.

Firstly, the potential for dynamic interaction between the various façade components was considered by comparing the results of coupled dynamic FEA to traditional step-by-step SDOF hand calculations. On this project it was found that strong coupling did not occur and that the hand calculations were generally bounding.

Secondly, the potential for lateral torsional buckling of the glass fins during dynamic rebound was considered both using codified procedures and dynamic FEA. It was found that the restraining effect of the front façade was beneficial, even though the interior edge of the fin was unrestrained. Dynamic FEA confirmed the hand calculations but indicated that the threshold for buckling was 25-40% higher than the code equations. This was considered to be due to the necessarily conservative nature of code requirements for buckling.

Lastly, the design of the splice connections was found to be a critical design aspect. To reasonably comply with architectural requirements, while it was necessary to increase the number of bolts, it was also necessary to provide enhanced clamping through increased bolt pre-stress and increased friction coefficients, as justified by testing undertaking by Pilkington.

## REFERENCES

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