Drift Performance of Point Fixed Glass Façade Systems

S. Sivanerupan1,*, J.L. Wilson1, E.F. Gad1 and N.T.K. Lam2
1Faculty of Engineering and Industrial Sciences, Swinburne University of Technology, Victoria 3122, Australia
2Melbourne School of Engineering, the University of Melbourne, Parkville 3010, Australia
(Received: 9 April 2013; Received revised form: 25 September 2013; Accepted: 19 May 2014)

Abstract: Glass façade systems in buildings are subject to racking actions caused by inter storey drifts from earthquakes and wind action. The performance of façade systems is dependent on the amount of imposed drift and the interaction of the glass panels with the façade structural support frames. There are two major concerns related to the glass façade system performance during and immediately after a seismic event; hazards to people from falling glass and the cost associated with building down time and repair. It was observed that earthquake damage to glass façade systems resulting from in-plane racking actions is increasingly common and yet there has been limited research published in this field. The research completed to date has mainly focused on traditional framed glass façade systems; however, the racking performance of point fixed glass façade system (PFGFS) is likely to be quite different. Therefore, the aim of the research presented in this paper is to assess the in-plane racking performance of PFGFS which is a façade system gaining popularity worldwide.

Two unique full scale in-plane racking laboratory tests on typical PFGFS with different types of connections were conducted and specific racking mechanisms were identified. Sophisticated non-linear finite element models (FE models) were developed and benchmarked against experimental results with excellent correlation. Further detailed FE analyses were conducted to evaluate the individual drift contributions of each racking mechanism such as rigid body translation of the glass panels at the oversize holes for construction tolerance, spider arm rotation and spider arm deformation. It was found that most of the drift capacity is attributed to the rigid body translation at the oversize holes. In this paper, the laboratory test setup and the experimental results are discussed together with the confirmatory FE analysis results to assess the in-plane racking performance of the PFGFS.

Key words: point fixed glass façade system, in-plane drift capacity, façade systems, earthquakes.

1. INTRODUCTION

Glass façade systems in buildings are subject to racking actions caused by inter storey drift from earthquakes and wind action. The performance of façade systems is dependent on the amount of imposed drift and the interaction of the glass panels with the façade structural support frames. There are two major concerns related to the glass façade system performance during and immediately after a seismic event; hazards to people from falling glass and the cost associated with building down time and repair.

Glass façade systems can be classified into two types namely, framed glass façade system and point fixed glass façade system (PFGFS). It was observed that earthquake damage to glass façade systems resulting from in-plane racking actions is increasingly common and yet there has been limited number of research published available in this field. The research conducted...
to date has mainly focused on traditional framed glass façade systems; however, the in-plane racking performance of PFGFS is likely to be quite different. In the structural design of glass façades both out-of-plane and in-plane actions are considered by the façade engineer. Self-weight, thermal expansion, spandrel beam deflection and in-plane building movements due to wind and seismic loadings are considered for in-plane design whilst wind load is the main design action for out-of-plane performance of façade systems (Sivanerupan et al. 2011).

A substantial number of laboratory and analytical studies related to the simulated seismic performance of framed glass façade systems have been performed over the past few decades. The most extensive testing programs were performed on framed glass façades at the University of Missouri-Rolla (UMR) and University of Pennsylvania (Behr 1998; Behr and Belarbi 1996; Behr et al. 1995; Memari et al. 2003, Memari et al. 2004). ASCE7-10 (2010) provides a general expression based on these tests for assessing architectural framed glass façade systems under in-plane loading as expressed by Eqn 1. The drift capacity ($\Delta_{\text{fallout}}$) should exceed the drift demand which is a function of relative seismic displacement ($D_p$) and the occupancy importance factor ($I$)

$$\Delta_{\text{fallout}} > 1.25ID_p \text{ or } 13 \text{ mm whichever is greater} \quad (1)$$

However, the seismic performance of a PFGFS is different from conventional framed glass façade systems and there are no standards or design guidelines available to evaluate the in-plane drift capacity of PFGFS. Spider arms are used in PFGFS to connect the glass to the support structure whilst the glass to the spider arms are connected using special bolt fittings as shown in Figure 1. Despite their growing popularity, there is very limited published research on the behaviour of PFGFS under in-plane racking action.

Recently, a PFGFS with large horizontal slotted holes in arms was designed and constructed in a high seismic region of California and the in-plane racking performance was verified using mock-up tests. Desai et al. (2005) and Gowda and Heydari (2009) assessed the seismic performance of PFGFS with different types of structural support frames for use as cladding facades of buildings in areas of high seismicity. Mock-up tests were conducted to address the drift limit criteria of 2.0% to 2.5% for cladding systems as per the seismic provisions of the California Building Code (CBC 2002). The function of the proposed façade systems is to isolate the glass from the structural support frame for in-plane deformations and loads while supporting the system for vertical loads and for out-of-plane loads. Specially designed spider arms with large horizontally slotted holes (Figure 2) were used to accommodate the drift by allowing isolated horizontal translation of glass panels. The sizes of the slotted holes were calculated according to the height of the glass façade and the drift demand from the building.

A unique research project has been undertaken by the authors to assess the racking performance of contemporary PFGFS for regions of lower seismicity to overcome the paucity of data in this area. The research involved laboratory experimental testing and analytical modelling of PFGFS with toughened glass panels. Two unique full scale in-plane racking laboratory tests on typical PFGFS with different types of spider arms were conducted (Test #1 and Test #2). Specific racking mechanisms were attributed to the in-plane drift capacity in each test. Sophisticated non-linear finite element models (FE models) were developed and conservatively benchmarked against experimental results with excellent correlation. Detailed FE analyses were carried out to evaluate individual contributions from each racking mechanism. The analyses showed that a significant amount of the drift capacity was attributed to the rigid body translation at the oversize holes provided for construction tolerance. This paper provides an overview of the racking performance of PFGFS systems and describes the laboratory experimental test setup (Section 2), experimental results (Section 3), confirmatory finite element modelling (Section 4) and conclusions (Section 5).
2. LABORATORY EXPERIMENTAL TEST SETUP

Two unique full scale in-plane racking laboratory tests (Test #1 and Test #2) representing typical point fixed glass façade systems (PFGFS) were conducted. The tests utilised contemporary connections to attach the glass panels to the structural support frame consisting of spider arms and special bolt fittings. The in-plane racking performance of PFGFS was dependent on three main components: the glass panels, the connection details and the structural support frame. In these tests the structural façade support frame representing the building superstructure was articulated so that the racking performance of the glass panels and the connection details could be directly assessed. The spider arms were configured as pinned X-type with countersunk bolt fittings for Test #1 (Figure 3) and fixed K-type with button head bolt fittings (Figure 4).

2.1. Test #1 Specimen Description

Test #1 was conducted on a typical PFGFS as shown in Figure 5, which consisted of four 1200 mm x 1200 mm toughened 12 mm thick glass panels joined with 8 mm thick silicon weather sealant. A structural façade support frame was designed to support the glass panels through the spider arms. It was fabricated using 180PFC sections and bolted together using M24 bolts which were snug tightened to allow a racking mechanism to develop. The flanges of the vertical PFC were removed at each end to facilitate pin connections between the vertical webs and the horizontal PFC members. The bottom flange of the horizontal PFC at the floor level was rigidly connected to the laboratory strong floor using M24 bolts. X-type spider arms and countersunk bolt fittings were used in this experimental test. The X-type spider arms were connected to the structural façade support frame using a single snug tightened M10 bolt to allow in-plane rotation of the glass panels at the spider arm-to-structural support frame connection, as typically occurs in industry.

2.2. Test #2 Specimen Description

The 2x2 glass panel test specimen was constructed with K-type spider arms and button head bolt fittings is shown in Figure 6. The specimen description and the experimental setup was the same as described in Section 2.1 for Test #1. However, the structural façade support frame was modified by welding some ‘T’ sections thus creating fixed cleats, to which the K-type spider arms were connected with two snug tightened M10 bolts. This replicated a fixed spider arm connection that did not
allow the glass panels to rotate at the spider arm-to-
structural support frame connection but allowed sliding at
the base slotted hole connection in the vertical direction
which is often used in practice as shown in Figure 1.
The spider arm base had two vertical slots measuring
14 mm x 25 mm for the M10 bolts [Figure 7(a)] to allow
for construction tolerance. In addition, the representative
oversize holes for construction tolerance were
provided by drilling 20 mm diameter holes in
the cleat to accommodate the M10 bolts as shown in
Figure 7(b). This provided ±7 mm horizontal (in the out-
of-plane direction of the glass panels) and ±17.5 mm
vertical gaps for the M10 bolted connections. The K-type
spider arms to cleat connection details are illustrated in
Figure 8, which is representative of industry practice.

2.3. Instrumentation and Test Procedure
The reaction frame and hydraulic jacking system was
capable of applying 100 kN in-plane lateral load and more
than 150 mm in-plane displacement. The structural façade
support frame was prevented from moving in the out-of-
plane direction by four sets of rollers mounted at the top
and the rollers ensured that the structural façade support
frame was aligned with the loading direction. A schematic
of the test setup and the arrangement of the instrumentation
are illustrated in Figure 9. Once the structural support
frame was assembled, glaziers fixed the spider arms, glass
panels and applied the weather sealant and a special
transparent adhesive film was applied to the glass panels to
prevent the glass fragments scattering following any glass
fracture (Figures 4 and 5). In addition, the test area was
enveloped with nets to capture any flying glass fragments
following fracture and to ensure safety in the lab.

The racking test procedure and instrumentation setup
were as follows:
• A lateral load was applied to the top right hand
corner in a step by step manner with
displacement increments of 5 mm until failure.
• Two systems of displacement measurement
were recorded to measure global and local
movements:
  • Linear Voltage Displacement Transducers
    (LVDTs)
  • Photogrammetry

Figure 4. Test #2 - Fixed K-type spider arm with button head bolt
fittings

Figure 5. Test #1 specimen - glass panels with transparent
adhesive film

Figure 6. Test #2 specimen - Glass panels with, transparent
adhesive film and photogrammetry targets
Deflections were measured at 11 locations with the LVDTs (horizontal, vertical and out-of-plane) whilst the applied load was measured using a load cell (Figure 9).

Photogrammetry provided displacement data for the target points that were tactically positioned and marked with retro-reflective adhesive labels (Figure 6). Photographs of the targets were taken before and after a sequence of loading and the relative movement in their positions were interpreted using software based on the principle of triangulation. The Photogrammetry measurements provided movements in all three directions (x, y and z).

3. EXPERIMENTAL RESULTS AND DISCUSSION

3.1. Test #1 - Experimental Results and Discussion

The load versus in-plane drift curve measured at the top of the structural support frame at a relative height of 2.72 m from the floor is shown in Figure 10. It indicates that the structure performed approximately linearly until failure. The slightly jagged nature was
reflective of the initial stiffness of the sealant relaxing together with the difference between the dynamic and static frictional movement and rotation at the connections (structural support frame to spider arm, spider arm to bolt fitting and bolt fitting to glass connection). A maximum in-plane lateral displacement of 58 mm was measured with a corresponding 16 kN racking load before failure. Surprisingly, this resulted in a significant 2.1% in-plane drift capacity for the system with minor damage to the sealant and yielding of the spider arms, before catastrophic failure of one of the glass panels. The failure of the system and the glass panel are shown in Figure 11. The adhesive film was very advantageous.
in preventing the shattered glass fragments from spreading all around the laboratory.

It was observed during the racking test that the glass panels and the spider arms all translated as rigid bodies whilst the sealant deformed at the interface followed by spider arms deformation and yielding at one location. The X-type spider arms used in this experiment had a frictional moment capacity (torsional) after which rotation would occur. Rigid body translation in both the horizontal and vertical directions was observed at the oversize holes for construction tolerance (gaps) at the connection details (structural support frame to spider arm, spider arm to bolt fitting and bolt fitting to glass connection).

Interestingly, the glass panels were also displaced in the out-of-plane direction relative to each other. This was because the support frame and the glass panels were translating in different planes resulting in the out-of-plane bending and deformation of the spider arms which induced bending stresses in the glass panel as shown in Figure 12. The differential out-of-plane movement of the spider arms was measured using the photogrammetry targets on the bolt head and the permanent photogrammetry targets on the floor. A maximum out-of-plane differential movement of 8.5 mm was recorded. This out-of-plane movement induced combined local bending and tensile stresses in the glass particularly around the bolt hole resulting in the initiation of cracking and catastrophic failure of the bottom right hand glass panel as shown in Figure 11. Further information on Test #1 experimental results and discussion are published in Sivanerupan (2011) and Sivanerupan et al. (2013).

3.2. Test #2 - Experimental Results and Discussion

The load versus in-plane drift curve measured at the top of the structural support frame (LVDT No 1 in Figure 9) for Test #2 is shown in Figure 10. It indicates that the façade system performed almost linearly up to failure. A maximum displacement of 143 mm was measured with a corresponding 38 kN racking load at failure. Surprisingly, this resulted in a maximum 5.25% in-plane drift capacity for the 2.72 m height system. At this high level of drift there was damage to the sealant and yielding of the spider arms, before catastrophic failure of one of the glass panels. The failure of the system and the glass panel are shown in Figure 13 with the adhesive film preventing the shattered glass fragments from falling.
It was observed during the racking test that the glass panels translated and the spider arms moved vertically within the slotted hole connection to the structural support frame cleat. Damage along the vertical silicon sealant was noticed at a 2.0% drift. Beyond a drift of 3.3% there was no capability for further rigid body translations as the spider arms were almost bearing on the edges of the circular and slotted holes and consequently the spider arms began to deform to accommodate further drift. This resulted in both in-plane and out-of-plane deformations of the spider arm fixings, and created excessive bending and tensile stresses in the glass, before catastrophic failure at a large drift 5.25%.

A simple truss analysis was carried out to determine the loading actions (tension or compression) in the panels as shown in Figure 14 (where ‘PA’ is panel ‘A’ and B4 is bolt 4) together with the possible vertical movement direction of the spider arms. The initial (red) and the final (blue) locations of the panels are shown to scale in Figure 15 and demonstrate the translations and rotations that occurred in the glass panels before failure. The vertical displacement of the spider arm to glass bolted connection is plotted in Figure 16 whilst the relative vertical measurement of the internal spider arm is shown in Figure 17 after failure of the system. The out-of-plane deformation of the glass panel is plotted in Figure 18 and illustrated in Figure 19 whilst the damage and yielding of the spider arm fixings at failure is shown in Figure 20.

The racking load applied to the structural support frame was transferred to the glass panel via the spider arms resulting in diagonal axial tension and compression forces as shown in Figure 14. Similar to Test #1, the support frame and glass panels were translating in different planes, which caused the out-of-plane deformation of the spider arms which then induced bending stresses in the glass panels in the vicinity of the point fixed connections. The induced combined local bending and axial tensile stresses in the glass panels, particularly around the bolt holes, caused the catastrophic failure at a large drift 5.25%.

3.3. Ultimate Fracture Strength of Toughened Glass

During glass failure, the crack front propagates through the material, creating fracture features known as the mirror, mist, and hackle (Figure 21). The crack front initially produces the smooth mirror region. However, as the crack accelerates it becomes more unstable, creating a dimpled surface known as mist. This instability eventually causes the crack to branch out, producing the rough hackle region. The hackle region is characterised by elongated markings that proceed in the direction of crack propagation. The hackle markings point back to the flaw origin (Frechette 1990).

The ultimate fracture strength of glass can be measured using an empirical expression. The radius of the mirror is inversely proportional to the square of the stress when the mirror was formed and it may be used to calculate the stress at the instant of fracture. From fracture mechanics analysis, the radius of the mirror also relates to the critical size of the flaw and the time to
catastrophic failure under fatigue conditions (Shinkai 1994). The stress that initiates the fracture, $f_r$, can be found from the Eqn 2 (Shand 1959):

$$f_r = \frac{2.14}{\sqrt{r}}$$

In this equation, $f_r$ is the tensile stress in MPa and ‘$r$’ is the ‘mirror radius’ in metres. This equation has been found to be reasonable for a variety of sample sizes and surface conditions. Measurement of the mirror radius parallel to the surface of the sample has been found to produce the most reliable results (Brungs and Sugeng 1995). In Test #2 the broken glass panel was investigated and the fracture origin was identified. The ‘mirror radius’ was measured to be $r = 6$ mm at the glass hole where the crack originated as illustrated in Figure 22.

and the effective tensile stress at the edge of the hole where the fracture originated was approximately 28 MPa (Eqn 2). Conservatively, the minimum pre-compressive stress at the edge of the hole should be 69 MPa which is the minimum toughening stress given in AS1288 (2006). Therefore, the applied tensile stress due to the racking loads to overcome the pre-compression and cause tensile failure could be estimated to be around 97 MPa. This was approximately equal to the 94 MPa nominal strength of 12 mm thick toughened glass panel at the bolt hole specified in the Australian standard AS1288 (2006).

4. ANALYTICAL MODELLING

Detailed three-dimensional non-linear finite element models (FE models) were developed using ANSYS finite element software and benchmarked against the laboratory experimental results. FE models were created (Figure 23) with a number of features to represent the in-plane racking behaviour, including:

- The structural façade support frame, spider arms and M10 bolts to connect the spider arms to the glass panels were modelled using beam elements,
- Non-linear spring elements were used to model the frictional torsional moment versus rotation at the spider arm connection,
- The translations between (a) bolt fittings and spider arm and (b) spider arm and support structure were modelled using non-linear springs,
Spring properties were specified with non-linear load versus deflection curves to represent the oversize holes for construction tolerance and the bearing effects (gaps closed).

Figure 18. Test #2 - Differential out-of-plane movement of the spider arms measured

Figure 19. Test #2 - Out-of-plane deformation of the spider arm PAB3 and PCB1

Figure 20. Test #2 - Spider arm deformations

Figure 21. Schematic diagram of a typical glass failure or crack origin (Castilone et al. 2002)
• The glass panels were modelled using shell elements and finely meshed around the glass holes to model the stress concentration and better predict the failure stresses,
• A failure strength of 94 MPa for 12 mm thick toughened glass at drill holes was used in accordance to Australian Standard – Glass in building (AS1288 2006) which is comparable with the experimentally estimated failure strength 97 MPa described in Section 3.3.
• The countersunk bolt head and button head bolt heads were modelled conservatively as 20 mm diameter cylinders and modelled using shell elements,
• The spider arms were modelled to behave linearly until the failure of a glass panel,
• Manufacturers recommended material properties were used for the spider arm, bolt connections, glass panel and silicon sealant,
• Incompressible behaviour of the silicone sealant was modelled using a material model specified in the ANSYS called Blatz and Ko (Bondi and McClelland 2009) which is a one parameter model to represent hyper elastic material.

4.1. Test #1 – Analytical Results and Discussion
The results obtained from the FE models were tuned and calibrated against the test data including; pushover curve, failure stress and out-of-plane deformation of the glass panels. A good correlation was developed between the experimental and analytical pushover curves as shown in Figure 25. The nominal failure tensile stress of 94 MPa was developed at the edge of the glass hole at bolt PCB4 from the FE model, as shown in Figure 26 and indicates a highly non-linear stress distribution associated with the out-of-plane deformation of the spider arms beyond a drift of 1.5% in Test #1.
The calibrated FE models were then extended to extrapolate and investigate the performance of the PFGFS with other panel dimensions and configurations. Experimental and FE analyses results confirmed that a significant amount of the drift capacity was attributed to the rigid body translation facilitated by the oversize holes for construction tolerance between the bolts and holes within the spider arm and structural support frame connections. The calibrated FE models were used to evaluate the drift contributions from each of the three mechanisms at the maximum drift of 2.1% which is summarised as follows:

a) In-plane rigid body rotation of the spider arms (0.4%).
b) Rigid body translation facilitated by the oversize holes for construction tolerance between the bolts and glass panels within the spider arm and structural support frame connections (1.2%).

c) Deformations and yielding of the spider arms (0.5%).

4.2. Test #2 – Analytical Results and Discussion

The Test #1 FE model was modified at the bolted connections to incorporate the translations and rotations provided in Test #2. The FE model was conservatively calibrated to the experimental results using the following parameters: (a) displacement versus in-plane drift, (b) failure stress of 94 MPa, (c) in-plane differential movement of glass panels and (d) deformations of the silicon sealant. The in-plane racking action of the façade system and vertical sliding of the spider arms from the FE results are shown in Figures 23 and 24 which strongly agrees with the experimental results.

The maximum tensile stress developed in the glass panels is illustrated in Figure 26 and shows the rapid increase in stress due to out-of-plane bending in the final
25% of the drift capacity. Interestingly, as observed in the experimental test, the maximum tensile stress occurred at PCB4. The failure stress in the glass panel developed at a lateral displacement of 114 mm (4.75% drift) compared to 5.25% drift in the experiment with a lateral force of 35 kN as shown in Figure 25. Further detailed FE analyses were conducted to proportion the racking capacity from each mechanism.

The FE results showed that the applied racking drift of 4.7% was accommodated by three mechanisms:

a) Rigid body vertical translation of the spider arms at the base slotted hole connection to the support frame (2.9%).

b) Rigid body translation facilitated by the oversize hole connections between the spier arm and the glass panels (1.3%).

c) Deformations and yielding of the spider arms (0.5%).

The FE results showed that the major contribution of the drift capacity was obtained from the vertical tolerance of the spider arm slotted hole connections to the support frame. This rigid body translation is equivalent to a rocking mechanism as shown in Figure 27. Trigonometric expressions were used to verify the drift capacities ‘γ’ and the horizontal displacements ‘Δ’ associated with the rigid body translation of the glass panels resulting from the vertical translations at the spider arm base slotted hole connections as follows:

\[ \Delta = h \cdot \tan \theta = h \cdot \frac{s}{b} \quad (3a) \]

\[ \gamma = \frac{s}{b} \times 100\% \quad (3b) \]

where, “h” is the height of the PFGFS, “s” is the maximum possible vertical tolerance in the spider arm base slotted hole and ‘b’ is the width of a glass panel.

The drift in Test #2 can be calculated from Eqn 3b:

\[ \gamma = \frac{35}{1200} \times 100\% = 2.92\% \]

A maximum 2.92% drift can be attributed to the rigid body translation at the spider arm base slotted hole which is very similar to the FE results of 2.9% drift. From Eqn 3b the drift contribution from the vertical translation at the spider arm base hole depends on the dimensions ‘s’ and ‘b’. Clearly, if wider panels are used with the same spider arm the drift capacity of the system will be reduced whilst if narrower panels are used the drift capacity can be increased. Further detailed parametric study and in-plane drift calculation procedures for the façade system using FE models are presented in Sivanerupan (2011).

5. SUMMARY AND CONCLUSIONS

A unique research project has been undertaken by the authors to evaluate the in-plane racking performance of standard PFGFS in the lower seismic regions. The research involved both experimental and analytical racking studies of PFGFS with toughened glass panels to develop a push-over or load versus in-plane drift capacity for the system. The two tests completed indicated that the PFGFS with pinned X-type and fixed K-type spider arms had surprisingly large in-plane drift capacities caused by rigid body translation of the glass panels at the oversize holes for construction tolerance, spider arm rotation (Test #1), spider arm vertical translation (Test #2) and deformation of the spider arm components. Overall the drift capacities for Test #1 (pinned X-type spider arm) and Test #2 (fixed K-type spider arm) were 2.1% and 5.25% respectively.

In Test #1, the maximum drift of 2.1% was much larger than initially anticipated and demonstrated that the 2x2 system was surprisingly able to sustain high level of in-plane drift. The glass panel rigid body translation, spider arms rotation and deformations allowed the system to move laterally and created the increased drift capacity. A significant amount of out-of-plane movement was observed with a maximum differential movement of approximately 8.5 mm. This out-of-plane movement induced combined local bending and tensile stresses in the glass particularly around the spider arm bolted connection to the glass panel resulting in the initiation of cracking and catastrophic failure of a glass panel.
In Test #2, the maximum drift of 5.25% (143 mm) was much larger than initially anticipated and demonstrated that the 2x2 system had significant drift capacity. Damage along the vertical silicon sealant was noticed at 2.0% drift whilst the spider arms began to deform at 3.3% drift with the base plate of the spider arms commencing to yield. The system continued to deform until failure of a glass panel at 5.25% drift due to excessive bending stresses (from out-of-plane displacement of the spider arms) combined with the in-plane diagonal tensile stresses.

Sophisticated non-linear finite element models were developed and conservatively benchmarked against the experimental results with excellent correlation in terms of load-deflection behaviour, failure stress and out-of-plane deformation of the glass panels. FE results confirmed that in all 2x2 façade grid systems the applied racking displacement was accommodated by three mechanisms with the majority of the drift facilitated by the built-in oversize holes in the spider arm connections to the glass and the support frame. The calculated drift contributions from each of the three mechanisms for Test #1 at the maximum drift of 2.1% is summarised as follows:

a) In-plane rigid body rotation of the spider arms (0.4%).

b) Rigid body translation associated with the oversize holes of the spider arm connections (1.2%).

c) Deformations and yielding of the spider arms (0.5%).

The calculated drift contributions from each of the three mechanisms for Test #2 at the maximum calculated drift of 4.7% is summarised below:

a) Rigid body translation associated with the slotted holes of the connections to the support frame (2.9%).

b) Rigid body translation associated with the oversize holes of the spider arm to glass connections (1.3%).

c) Deformations and yielding of the spider arms (0.5%).

Trigonometric expressions were used to verify the drift capacities from the significant rigid body translation at the spider arm slotted hole connections to the support frame for Test #2. The expressions can be confidently extrapolated to undertake parametric studies and evaluate the in-plane drift capacity of other PFGFS systems with different panel dimensions and configurations. The performance of the façade system under extreme earthquake or wind loads can then be assessed by comparing the drift demand from analytical studies of the building superstructure with the performance drift capacity curves of PFGFS described in this paper.

ACKNOWLEDGEMENTS

Support funding by the Australian Research Council through DP0772088 ‘Collapse modelling of soft storey buildings’ is gratefully acknowledged. The authors are very grateful to the Australian Earthquake Engineering Society for a financial contribution towards the testing program, to the industry sponsors Australian Glass Assemblies and Viridian World Glass for supplying the spider arm fittings and the glass panels, to Dr David Heath for his assistance with the photogrammetry measurement and to the Melbourne Testing Services for assistance with testing.

REFERENCES


