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An Experimental Study of Reinforced Concrete Façade Panels and Their Fixing Assemblies Subjected to Blast Loads

Tung Pham
Department of Civil and Industrial Engineering
National University of Civil Engineering, Vietnam

Tuan Ngo
Department of Infrastructure Engineering
University of Melbourne, Australia

ABSTRACT: This paper presents experimental studies of concrete façade response to blast pressures. The experimental program also aims to establish the influence of fixings assemblies on the performance of pre-cast concrete façade panels. The current standardized approach of pre-cast concrete design as defined in UFC3-340-02 limits the support fixity of pre-cast wall panels to be a simply supported system. The restriction to a simply supported system in a pre-cast wall panel would lead to inefficient design due to non-optimized use of concrete panel thickness. The experimental results showed that varying fixing assemblies may lead to a partial fixity condition, which is not accounted for in the UFC3-340-02 design guideline. The partial fixity conditions observed in the experimental results indicate that there are residual capacities in the pre-cast façade systems. The efficacy of the façade panel system could be improved by taking into account partial fixity in the support system. Similarly, premature shear failure could also be prevented by understanding the additional contribution of the partial fixity on fixing assemblies.

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1 INTRODUCTION

In recent years, the rise of terrorist attacks has been of major concern around the world. Threats of terrorism may come from gas or chemical explosions, a car bomb, or the impact of a missile or an aircraft. Lessons learnt from recent events such as the Oklahoma Murrah Federal Building bombing [1], the New York World Trade Centre collapse [2], and the Australian Embassy attack in Jakarta [3] urged that special attention must be given to the behaviour of structural elements in order to improve their redundancy, toughness, and ductility under extreme impulsive loading. In response to the threat of terrorist attacks around the world, structural engineers are seeking cost-effective protective technologies for mitigating damage caused by severe impulsive loads to building structures.

An optimally designed reinforced concrete structure is generally capable of absorbing a large amount of energy [4], which indicates that the material can be effectively used in protective structures. Although the reinforced concrete façade systems have been designed to withstand the normal service loads, such as live load, wind load and severe weather conditions, these elements have rarely been designed to withstand loads resulting from explosions. There is a considerable amount of research conducted on concrete and reinforced concrete elements subjected to blast loading (to name a few), however, only a few studies have observed and quantified the response of the cladding panels and fixing assemblies [5-9].

Most researchers [5-7, 9] have agreed that the stiffness of the panel has a significant effect on the forces transferred to its fixings. Pan and Watson [6], considering only out-of-plane fixings, concluded that the panel response and the forces transferred to its fixings depended mainly on the panel stiffness but were not profoundly dependent on the stiffness of fixings. Pan, et. al. [7] studied the interaction between the panel and its fixings for both out-of-plane and in-plane types. Again, the dependence of forces transferred to the fixing based on the panel stiffness was highlighted. However, this study only mentioned the effect of restraint types, i.e. fixing types on the panel response, not the effect of fixing stiffness.

Starr and Krauthammer [9] conducted a number of precision impact tests on reinforced concrete beams. The objective of this study was to evaluate the load transferring mechanism from the facade structure to the main structure through the fixing systems and so the impact load was varied to provide several test conditions. From the experimental data, a reduction of 25 to 50 percent was observed in total peak forces transferred to the fixing assemblies with the impact force.

The evidence observed in the literature appears to indicate that the fixing assemblies of a pre-cast panel would affect its response. However, the standardized approach established in UFC3-340-02 limits the design of concrete façade panels to simply supported fixings assemblies [10]. The connection details required to achieve a perfectly simply supported system in a typical wall panel system are often impractical. Hence, in practice, it is likely that the fixity assemblies would provide partial moment resistance. This would result in a conservative design, which is acceptable in a conventional design.

Similarly, in a blast design to resist the effect of blast pressures, the additional capacity would also be beneficial if inefficient in most cases. However, in some cases where the shear resistance or the fixing assemblies are already designed to their limit according to the flexural resistance of a simply supported façade panel, the additional capacity may lead to shear failure of the panel, or failure in fixing assemblies. The issues of additional weak-links on the system highlights the importance of understanding the actual behaviour of fixing assemblies in a pre-cast reinforced concrete façade systems.

This paper reports an experimental investigation of six full scale blast trials carried out on six reinforced concrete cladding panels with different connection stiffness schemes. All the panels, which were scaled down models of real size cladding panels with a scale factor of 0.5, were tested with 5 kg of Ammonite with a standoff distance of 2 m. Air blast pressure history, maximum inward and outward deflection at the mid span were recorded. After each blast trial, crack pattern and crack widths of each specimen were examined. These details are presented in this paper and the conclusions are drawn based on connection stiffness and safety of both panel and connection.

2 EXPERIMENTAL SETUP

2.1 Test specimens

The sample contains six reinforced concrete cladding panels as with the following details:

Clear span = 1500 mm
Width = 1000 mm
Thickness = 80 mm

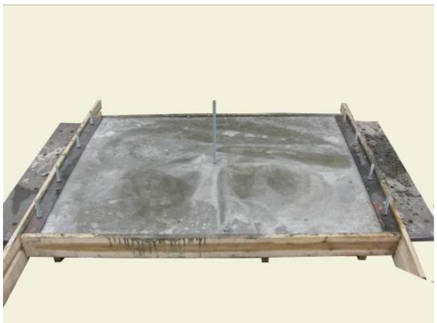
All six panels were cast using the same mix proportions of 475 kg of cement, 775kg of river sand, 1050kg of crushed rock of 10 mm, 171 kg of water and 4.75 litres of superplasticiser,Glenium-ACE388. However, different cast days of two panel types resulted in different compressive strengths. The compressive strengths of test panels along with the material properties of reinforcement, bolts and steel sections are presented in Table 1. The compressive strength tests were carried out in the LAS-XD 460 laboratory in Vietnam.

Table 1. Material Properties

Material	Property	Strength
Panel Type A	Compressive strength	f_{ck} = 46 MPa
Reinforcement	Yield strength	f'_y = 630 MPa
	Tensile strength	f_{ut} = 660 MPa
	Ultimate elongation	14.4%
Bolts	Tensile strength	f_{ut} = 800 MPa
Steel section	Yield Strength	f_y = 300 MPa
	Tensile strength	f_{ut} = 360 MPa

The panels are divided into two major types. Type A, which represents the baseline specimen; and type B, which represents varying fixing assemblies. The exact length of the panel varies slightly as panel type B requires extra 200 mm length to maintain an approximate equivalent clear span to typeB specimens. Both panel types were reinforced with two layers of N5 steel reinforcement mesh having a minimum cover of 10 mm. The reinforcement mesh spacing was set to a constant value of 100 mm. The reinforcement details for two panel types are shown in Figure 1 and Figure 2 respectively.

(a) Reinforcement detailing



(b) Casted panel

Figure 1. Details of Type A panel

Fixing details for both panels were based on fixing assemblies in realistic concrete facade systems

[11]. The selection of fixing type was based on the commonality and the frequency of usage. The schematic diagrams of the two fixing types used for this study are given in Figure 3. The fixing type 1 referred to as dowel type is commonly used for bottom fixing, is utilized in panel type A and type 2 referred to as angle cleat type fixing, which is used in both bottom and top fixings, is utilized in panel type B. The versatility of angle cleat type fixing resulted in its selection for more tests.

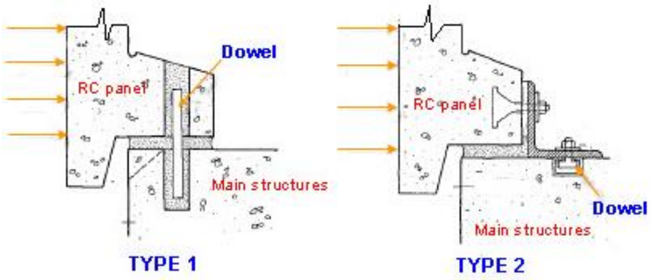
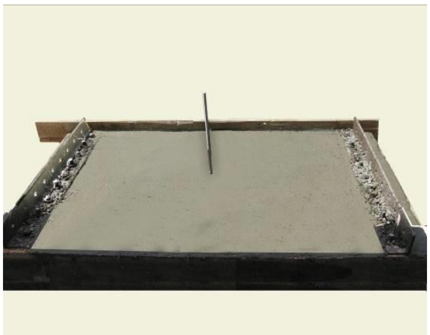


Figure 3. Typical types of fixings for concrete panels

(a) Reinforcement detailing



(b) Casted panel

Figure 2. Details of Type B panel

(a) Type A

(b) Type B

Figure 4. Connecting panels to test bunkers

Table 2. Panel fixing details

Panel Type	Panel ID	Cleat		No. of bolts	
		Type	Dim. (mm)	Horizontal	Vertical
A	1	Plate	1000x260x20	N/A	8M18
B	1	Angle	100x100x8	9M20	9M20
	2		100x100x10	9M20	9M20
	3		100x100x10	9M12	9M20
	4		100x100x10	3M12	9M20
	5		100x100x10	3M16	9M20

Panel type A was connected to the test bunker through a 20 mm thick steel plate and 8 M18 bolts. The angle cleat thickness, number of bolts and bolt sizes were varied to achieve different fixing stiffness in panel type B and are given in Table 2. Type A and type B connections are illustrated in Figure 4. Specimen B1 and B2 are defined to establish the influence of the angle cleat of the system, whilst specimen B2, B3, B4 and B5 are defined to establish the fixity influence of the bolts on the fixity of the system.

2.2 Instrumentation

Measuring instrumentation comprised of three different devices, namely, two 113B21 piezoelectric pressure gauges of PCB Piezotronics, a CDP-100, linear variable differential transformer (LVDT) by Tokyo Sokki Kenkyujo and a mechanical deformation measuring device to measure the permanent deflection at the centre of the panel. Two pressure gauges and the LVDT were connected to an automatic data acquisition system by HBM.

Two pressure gauges were mounted perpendicular to the test panel to obtain the reflected pressure history of the blast trials. The selected pressure gauges were capable of measuring transient pressures up to 6895 kPa with a resolution of 0.007 kPa [12]. The CDP-100 LVDT was mounted on the underside of the test panel to measure the displacement history of the panel. The CDP-100 is capable of measuring up to 100 mm of displacement and sensitive up to 100x10-6 strain/mm[13]. In addition to the LVDT, a mechanical displacement device was used to measure the maximum inward, outward and residual displacements of the test panel and its supports. In addition to these instruments, an ELE digital crack detection microscope, EL35-2505, was used for the crack identification and crack width measurements, after the tests.

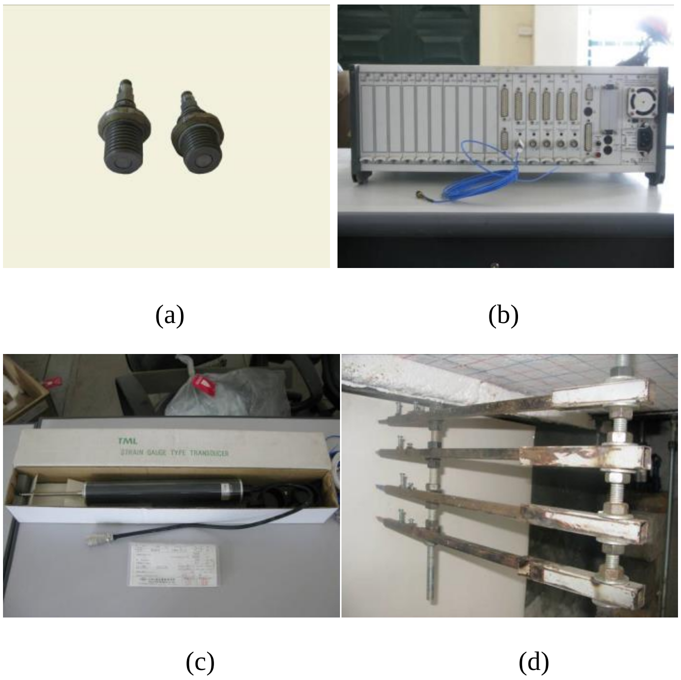


Figure 5. Measuring instrumentation: (a) Pressure transducers, (b) Data acquisition system, (c) LVDT (CDP-100), (d) Mechanical displacement measuring device

2.3 Charge details

The test program comprised of 5kg of Ammonite charges (Tri Nitro Toluene (TNT) equivalency of 1.1) at 2m standoff distance. The Ammonite charges were spherical in shape and 213 mm in diameter. The panels were tested in the sequence of A to B5. All the six charges used for the panel testing were identical in weight and shape. Panels were connected to the test bunkers horizontally and charges were detonated from a clear vertical distance of 2m from the centre of the panel.

3 EXPERIMENTAL RESULTS

3.1 Blast pressure history

As a verification measure, the experimental results from the trials are compared against the standardized approach to calculate the blast parameters, UFC3-340-02 [10]; and computational fluid dynamics modelling approach, Air3D[14]. The charge weight for UFC3-340-02 and Air3D was used as 5.5kg of TNT, using the TNT equivalency factor of 1.1 for Ammonite. Table 3 shows the comparison of peak reflected pressure and peak reflected impulse of the blast trial and Figure 6 illustrates the time history comparison of blast pressure for 5kg Ammonite.

It was observed that UFC3-340-02 parameters overestimated the peak reflected pressure, and Air3D underestimated the peak reflected pressure. However, the comparison of peak reflected impulse was in general agreement with the values recorded in the experiment, as illustrated in Table 3.

Table 3. Blast pressure comparison

W/S	Peak reflected pressure (kPa)			Peak reflected impulse (kPa.msec)		
	Exp.	Air-3D	UFC3-340-02	Exp.	Air-3D	UFC3-340-02
5kg/2m	2488	2381	3045	791	797	794

Note: W/S: weight/stand-off distance. Exp: Experiment

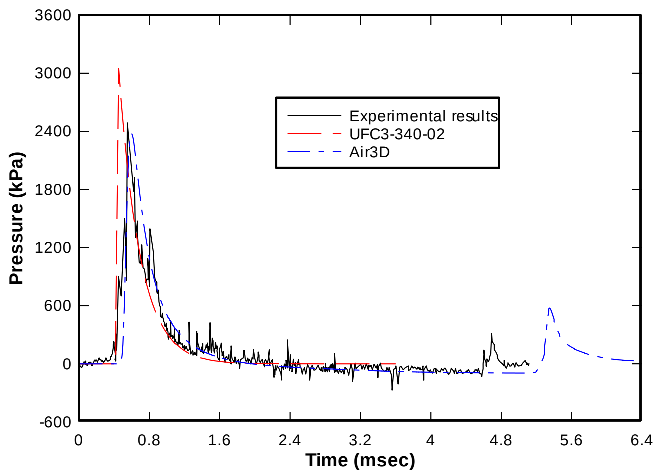


Figure 6. Reflected pressure time-history comparison

3.2 Observed displacement and crack pattern

Faulty data records were captured in the LVDT for panel A. Hence, for panel A, the only the maximum inward and maximum outward displacement obtained from the contingency mechanical measurement device will be presented. Failure on the fixings assembly was observed in specimen B4. This failure is expected considering that the only 3M12 bolts were used in the assembly. The results, although maximum inward and maximum outward displacements were recorded, the data should not be used to indicate the trends in the performance of the reinforced concrete panel. Otherwise, displacement history data on all other specimens were captured as planned.

The peak inward and outward displacements at the mid-span of the specimens are summarised in Table 4, whilst the displacement histories recorded in the experiment is shown in Figure 7. Considering that movements were also recorded at the support, the maximum relative inward and outward displacements are also presented in Table 4. Relative displacement is defined as the difference between maximum displacement and maximum displacement at the support.

Table 4. Summary of displacements at the centre and supports of the test panels

Panel type	Panel ID	Displacements at the centre				Displacements at the support	
		Max inward	Max relative inward	Max out-ward	Max rel-ative outward	Max inward	Max outward
		mm	mm	mm	mm	mm	mm
A	1*	38.5	35	10	10	3.50	0.00
B	1	28.9	24.1	14.5	10.6	4.80	3.90
	2	24.4	22.7	22.5	20.5	1.70	1.00
	3	24.5	22.6	19.2	17.9	1.90	1.30
	4**	23.7	15.9	11.5	6.9	5.50	3.40
	5	25.7	21.9	15.7	12.5	3.80	3.20

Note: *Faulty transducer records
**Failed support

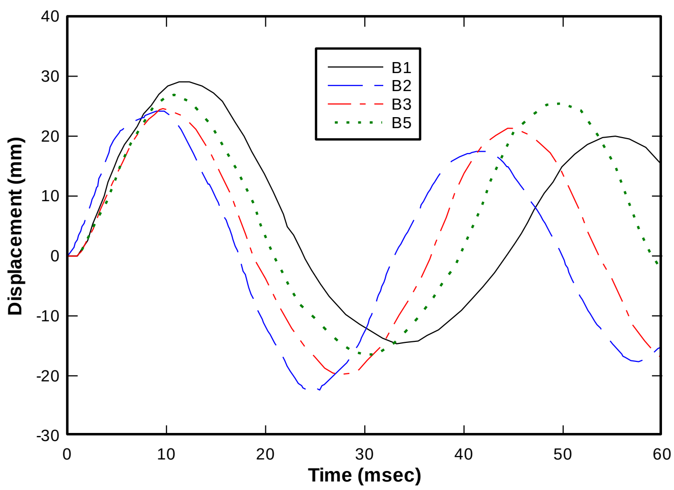
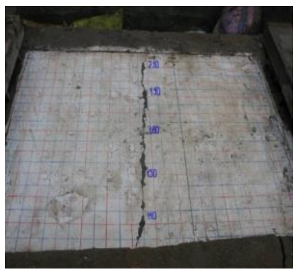
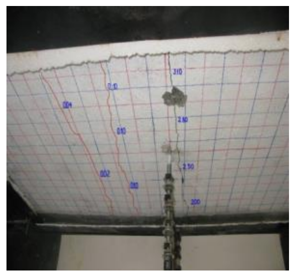
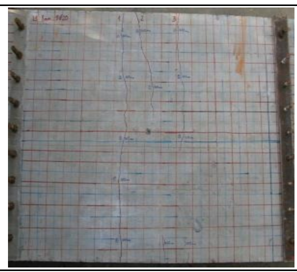
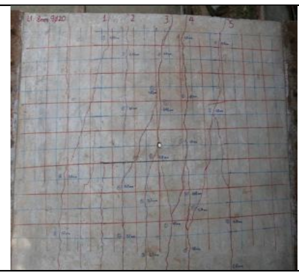


Figure 7. Centre displacement time-history of tested panels

After the test, the crack pattern on the specimens was recorded, and the crack development was measured using the EL35-2505. The results crack pattern and the crack width are as summarized in Table 5 and Table 6, respectively. The following patterns were observed in the specimens:

* Specimen A -Three crack lines were observed at the bottom surface, whereas only one crack line appeared at the top surface. The most severe crack lines at the top and bottom surfaces were located near the centre-line of the panel.
* Specimen B1 - There were five main crack lines at the bottom surface, whereas three crack lines appeared at the top surface. All three crack lines at the top surface were located near the centre-line of the panel. At the bottom surface, two critical crack lines at slight inclination to the centre-line were observed, whilst the remaining cracks occurred parallel to the centre-line of the panel.
* Specimen B2 - There were five main crack lines at the bottom surface, whereas three crack lines appeared at the top surface. The most severe crack line at the top surface was located near the centre-line of the panel. At the bottom surface, the critical crack line, overlapping the centre-line, was observed.
* Specimen B3 - There were five main crack lines at the bottom surface, whereas only one crack line appeared at the top surface. The crack line at the top surface was located near the centreline of the panel, with an average negligible width of 0.10 mm. At the bottom surface, the critical crack line was observed to be very close to the centreline of the panel.
* Specimen B4 - There were three crack lines at the bottom surface, whereas only one crack line appeared at the top surface. It is noted that that failure in the fixings assembly occurred in this specimen.
* Specimen B5 - There were four main crack lines at the bottom surface, whereas two crack lines appeared at the top surface. The most severe crack line at the top surface was located near the centre-line of the panel, with an average negligible width of 0.03 mm. At the bottom surface, two critical crack lines that slightly inclined with the centre-line were observed.

Table 5. Summary of crack pattern of six tested panels

ID	Top surface	Bottom surface
A		
B1		

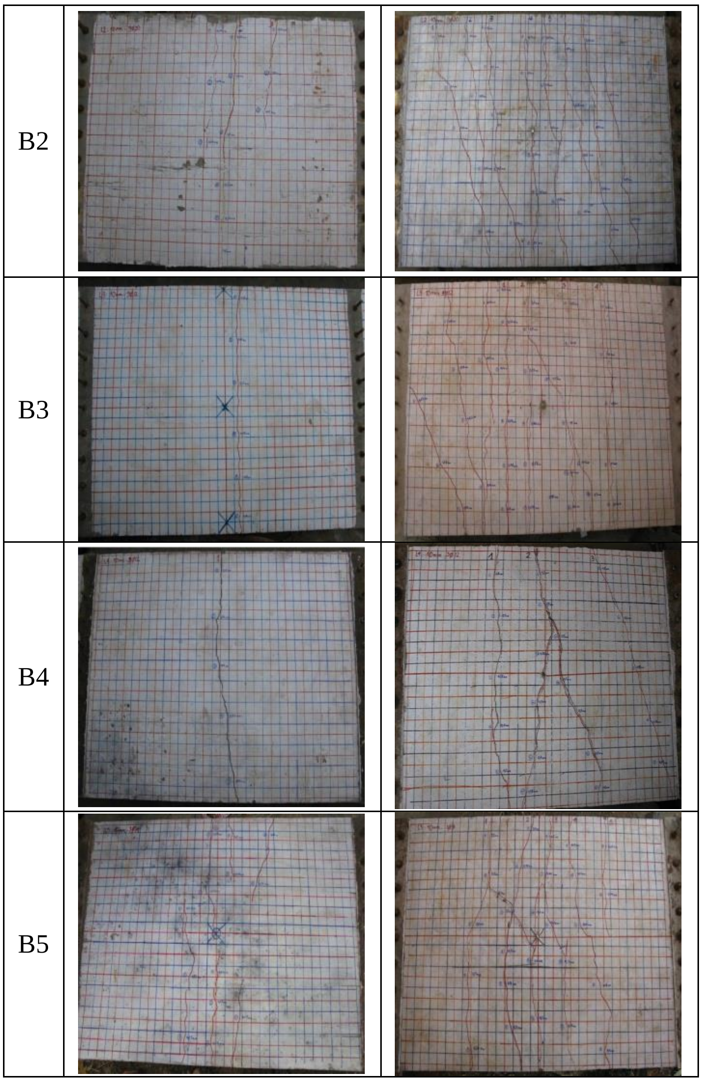


Table 6: Summary of major cracks

Surface	Panel	No of Cracks	Average crack width (mm)	Orientation (degree)	Distance to centre line (mm)
Bottom	A	3	2.6	0	27
	B1	5	0.87	0	111
	B2	5	0.77	0	0
	B3	5	0.73	0	-53
	B4	3	0.66	4.5	-265
	B5	4	0.74	12	-176
Top	A	1	1.72	0	0
	B1	3	0.02	12.8	0
	B2	3	0.18	0	0
	B3	1	0.10	0	0
	B4	1	0.02	0	0
	B5	2	0.03	0	0

4 DISCUSSION

It can be generalized that there are two flexibility parameters exhibited in the varying systems. The difference between panel A, B1 and B2, represents varying rotational rigidity, whilst different setup between panel B2, B3, B4 and B5 represents out-of-plane translational rigidity. These are broad generalization but it could be argued that the change in thickness of angle cleat and the change of fixity to embedded plate would represent varying rotational rigidity. Similarly, it could be argued that the variation of the number of horizontal bolts used in the assembly leads to the differing effective bolts cross-section area of the bolts. This is essentially a variation in vertical translational rigidity of the connection assembly. This can be observed in the recorded maximum inward and outward displacement at each supports, whereby the maximum inward and outward displacements increase with decreasing effective bolt cross-section area.

Intuitively, panel B2 will have a higher rotational stiffness compared to specimen A due to the rotational rigidity of the connections (Figure 4). Therefore, the inward displacement of specimen A as compared to specimen B2 is justified. The crack pattern on panel A exhibits a significantly larger crack width as compared to specimen B1 and specimen B2. The comparisons between specimens A, B1 and B2 have shown that the rotational rigidity of the fixings assembly influences the performance of the panel. Specimen A appears to have the least rotational rigidity in its fixings assembly, whereas specimen B2 appears to exhibit the most rotational rigidity in its fixings assembly. The most significant damage (observed based on the crack development) can also be observed in specimen A in comparison to specimens B1 and B2, whilst least damage was observed in specimen B2.

It is expected that panel B2 will exhibit a lower relative displacement compared with panels B3 and B5 due to its greater translational stiffness. Due to the reduction of stiffness at the supports via the removal of bolts from the connection, the central displacement of the panel is reduced while rebound at the supports is increased. The reduction of translational stiffness of support also appears to reduce the amount of damage on the panel. Although almost negligible, the improvement is evident in the reduced crack widths shown in Table 6. However, the reduction of the number of bolts leads to an increased risk of undesirable failure at the connection, and this occurred in test specimen B4.

5 CLOSING REMARKS

Several reinforced concrete façade panels with connections of varying translational and rotational rigidity were subjected to impulsive air blast loads. The results indicated that the configuration of the fixings assembly influences the performance of the panel due to decreased/increased translational stiffness in the support.

The test specimens assessed in this paper utilize commonly used fixing assemblies. It is important to note that a slight change in the configuration of the connection may lead to increased rotational rigidity of the panel. While the increase in rotational rigidity appears to improve the flexural performance of the concrete panel, it may also introduce more weak-links in the structure, mainly in the shear resistance and the connection of the system. From the crack pattern development observed in the experiment, a reduction on translational stiffness at the supports resulted in reduced damage of the panel. Although reaction forces are not recorded, it is possible that the more flexible system would lead to less reaction as well.

The findings of this experimental study show that the performance of a reinforced concrete façade system would be influenced by the fixings assembly configuration. The influence may present an improvement on performance in the form of reduction of damage, but may also adversely influence the performance by introducing another weak-link into the system. This highlights the importance of fully understanding the contribution of fixings assembly to the resistance of the overall façade system in order to establish an optimal design approach.

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