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ASSESSMENT AND MODELLING OF SEISMIC BEHAVIOR OF GLAZED CURTAIN WALLS

N. Caterino[(1,5)], M. Del Zoppo[(2)], G. Maddaloni[(3,5)], A. Occhiuzzi[(4,5)]

[(1)] *Assistant Professor, University of Naples "Parthenope", nicola.caterino@uniparthenope.it*
[(2)] *PhD student, University of Naples "Parthenope", marta.delzoppo@uniparthenope.it*
[(3)] *Associate Professor, University of Sannio, giuseppe.maddaloni@unisannio.it*
[(4)] *Full Professor, University of Naples "Parthenope", antonio.occhiuzzi@uniparthenope.it*
[(5)] *Construction Technologies Institute, National Research Council (CNR), Viale Lombardia, 49, 20098, San Giuliano Milanese (MI), Italy*

Abstract

Recent earthquakes confirmed the large occurrence of non-structural components failure, so as furnishings, equipment, and architectural, electrical and mechanical fixtures, which causes economic losses and represents a risk for human life safety. The glazed curtain wall systems are non-structural elements increasingly used in new mid and high-rise buildings. These systems, often realized with aluminium/steel frames and glass panels, are hung to the main structure and generally cover the entire facade. This study investigates the behavior of stick curtain wall systems under seismic excitation, through an experimental campaign conducted at the laboratory of the Construction Technologies Institute of the Italian National Research Council on a full-scale aluminium/glass curtain wall test unit. Then, a FE model has been developed aiming at reproducing the results of the experimental test and at suggesting the most suitable ways for predicting the seismic response of such kind of non-structural elements. The interaction between glass panels and aluminium frame, the gasket friction and the stiffness degradation of aluminium-to-glass connections due to the high deformations are taken into account in the model.

Keywords: non-structural elements, glazed curtain walls, stick systems, seismic behavior, FE modelling.

1. Introduction

The seismic behavior of non-structural elements has been investigated from several researchers in the last decades. In particular, the large use of glazed curtain wall systems as external enclosures in multi-storey buildings pushed the researchers to investigate the response of such non-structural elements when a seismic event occurs, in order to guarantee the pedestrians and occupants life safety and to avoid damages to structural members and unsustainable economic losses.

During earthquakes, the glazed curtain walls are subjected to both in-plane and out-of-plane actions. However, the out-of-plane behavior of those components is mainly governed by the wind action whereas the in-plane behavior strictly depends on the seismic response of the main structure. Indeed, the external glazing systems are directly hung to the main structure's beams/slabs and the interstorey drifts often cause damages on the curtain walls systems during the seismic event [1- 4].

First experimental studies on the behavior of window glass panels under static in-plane loads have been conducted in '60 [5, 6]. Decades after, several experimental in-plane tests on full-size single glass test units with aluminium framed mullions/transoms have been performed [7-14], investigating different structural glass typologies, clearance values and anchorage systems. Common mechanisms of damage recognized are gaskets dislodging, glass crushing at corners, glass cracking and glass panels fall-out. Bi-directional experimental tests have also been performed on plane and corner configurations (i.e. "L" shaped units) [8, 11], in order to observe both the in-plane and out-of-plane response. However, a lack of experimental tests on the seismic behavior of full-scale glazed curtain walls (i.e. with more than one glass panel) has been recognized in literature.

For supplying at the experimental tests and making general predictions about the behavior of new and existing glazed curtain wall systems, a finite element (i.e. FE) model has been developed in [15, 16] and a calibration on experimental results has been proposed.

The present paper focuses the attention on stick wall systems. The results of the experimental tests on two full-scale aluminium/glass curtain wall test units are presented and discussed. Then, a FE model developed within the software SAP2000 [17] is described and calibrated above the experimental results. The comparison between numerical simulation and experimental evidences confirms the reliability of the FE model for the prediction of the lateral response of stick curtain walls.

2. Experimental behavior of full-scale stick curtain walls

Two full-scale stick curtain walls has been tested by performing full-size in-plane cyclic tests. The two typology of stick curtain walls tested, actually produced and commercially available, are both made of an aluminium transom-mullion frame and insulated glazing systems. The two specimens are referred to in the next as "façade A" and "façade B".

2.1 Test setup and load protocol

The test facility consists in a steel frame 5,720 x 7,370 mm with three rigid beams, which simulate the principal structure's beams or slabs. The beams can be installed at different heights so as to adapt to different geometric configurations of the building where the curtain walls will be installed. They can be moved in the horizontal direction to simulate seismic induced lateral displacements.

For the specific tests herein described, only the intermediate steel beam has been pushed in the horizontal direction, while the upper and lower beams were firmly fixed to the external frame, in order to simulate the seismic interstorey drift. Cyclic force control tests have been performed pushing over, in the horizontal in-plane direction, the external mullion, in correspondence of the mullion-beam connection, by means of a hydraulic jack

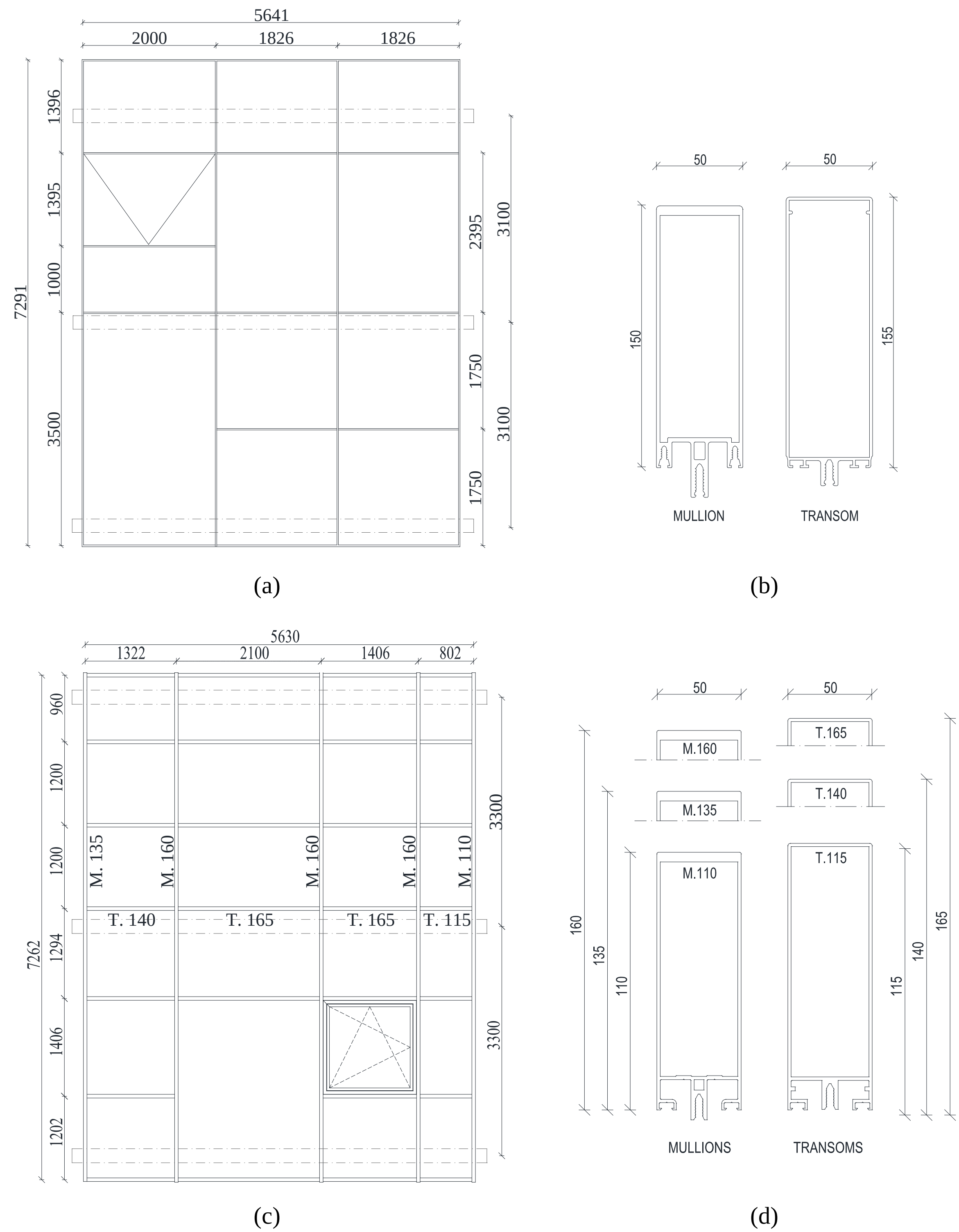


Fig. 1 – Façade A: geometry (a) and transom/mullion cross-sections (b); Façade B: geometry (c) and transom/mullion cross-sections (d), dimensions in mm

connected to an external rigid reaction system. A further description can be found in [18].

2.2 Specimens description

Façade A covers two storeys (7,300 mm height and 5,650 mm width), with 3,100 mm interstorey height. The test unit is characterized by four mullions and four transoms (see Fig. 1a). Extruded aluminium profiles are used for mullions and transoms and their cross-sections are shown in (Fig. 1b). All mullions have same cross-section, so as the transoms.

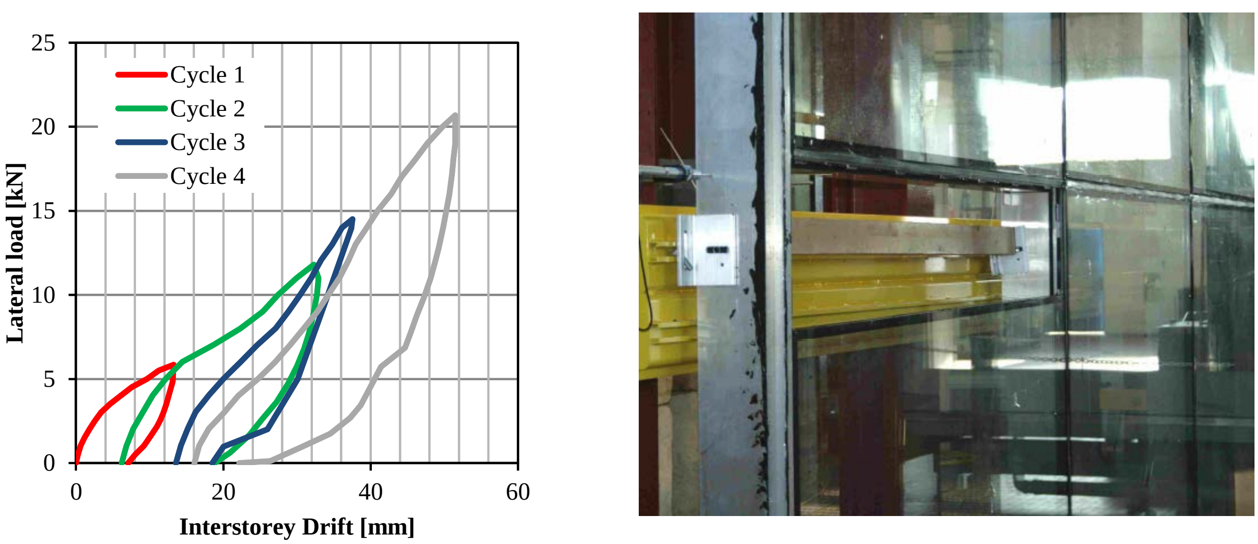
The insulated glazing units are made with fully tempered glass panels, thickness 8+16+8 mm, linearly supported on the edges trough silicone gaskets. A single openable window is present, whereas the other glass panels are fixed. The mullions are continuous along the entire façade height and the transoms are supported between the mullions.

Façade B is 7,200 mm high and 5,600 mm wide, with 3,300 mm interstorey height. Differently from the previous specimen, the test unit B has five mullions and six transoms (Fig. 1c). Three different profiles have been used for mullions, so as for transoms (Fig. 1d). In particular, it should be noted that transoms of the side spans are characterized by cross-section with lower inertia respect to central spans. The insulated glazing panels thickness is 8+8.2+16+6 mm, linearly supported on the edges trough silicone gaskets. Also in this specimen, just one glass panel could be opened.

2.3 Experimental results for Façade A

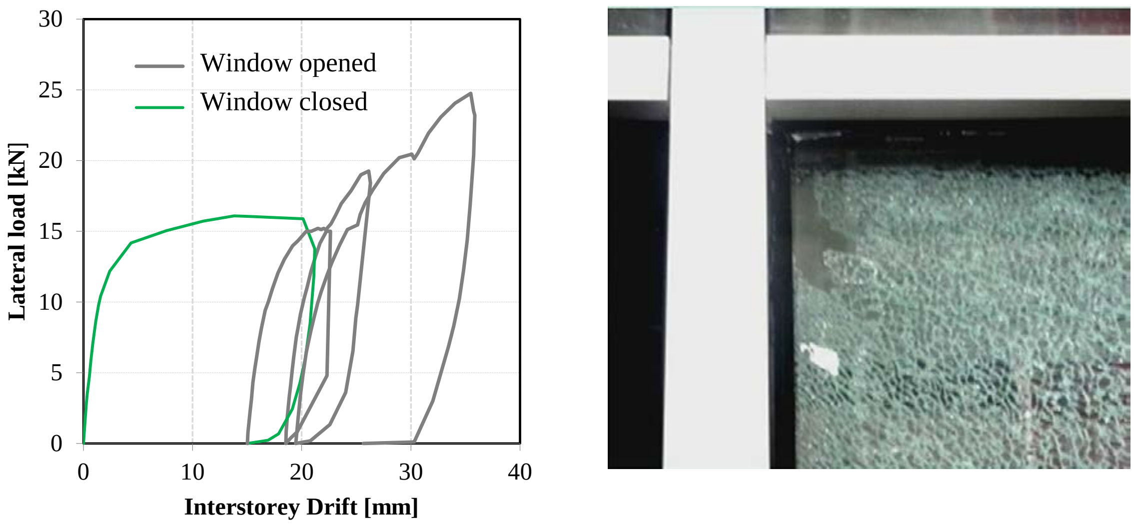
The experimental test has been carried out increasing the force applied at the central sliding beam until a specific value of relative displacement has been reached at the controlled point. Then, the force decreases up to zero, revealing the plastic deformations. Other three cycles of load-unload have been performed on the façade.

The results of the in-plane test are shown in Fig. 2a. During the first cycle, a maximum displacement of 13.07 mm was attained, corresponding to a lateral force of 5.9 kN. Removing gradually the load, a residual displacement of 6.2 mm (47.7% of the cycle peak drift) has been recorded. During the second cycle the maximum displacement achieved was 32.27 mm, corresponding to a load of 11.8 kN. Removing the load, the residual displacement was 13.52 mm (41.9% of the cycle peak drift). At the third cycle the maximum displacement attained was 37.35 mm, corresponding to a lateral force of 14.7 kN. The residual displacement recorded removing the load was 14.5 mm (38.8% of the cycle peak drift).



(a) (b)

Fig. 2 – (a) Load-drift relationship and (b) failure mode on **façade A**



(a) (b)

Fig. 3 – (a) Load-drift relationship and (b) failure mode on **façade B**

At the fourth and last cycle, the maximum displacement achieved was 49.88 mm, corresponding to a 20.7 kN lateral force. Removing the load, the residual displacement recorded was 16.06 mm (32.2% of cycle peak drift). During the last cycles, large shear deformations involved the aluminium frame. Furthermore, residual plastic deformations between 30% and 50% of the peak displacement were recorded.

The failure was achieved during the unload phase of last cycle, when the larger glass panel fell out from its aluminium frame, due to the high deformation of the inferior transom (Fig. 2b). The specimen drift capacity was thus 49.88 mm, which corresponds to 1.6% of the interstorey height (3,100 mm).

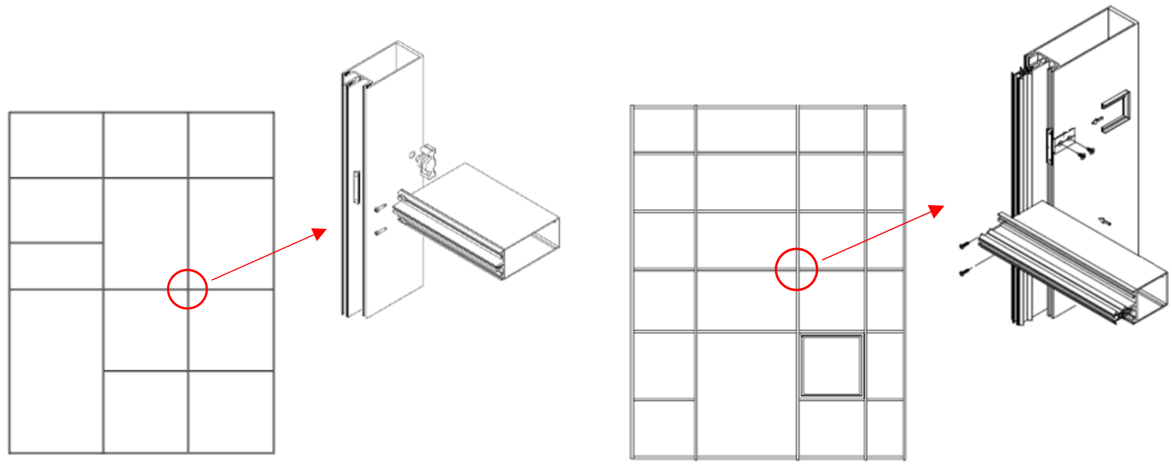
2.4 Experimental results for Façade B

Since the specimen had an openable window, the test was performed first on the façade with closed window (first cycle) and then on the specimen with opened window (subsequent cycles), since it was impossible to reclose it due to the high shear deformations of the aluminium frame. The force-drift cycles recorded are reported in Fig. 3a. During the first cycle (i.e. specimen with closed window), the maximum displacement achieved was 21.18 mm, corresponding to a lateral load between 15.9 kN and 16.09 kN. Removing the load, a residual displacement equal to 15.02 mm (70.9% of the peak value of drift) has been recorded.

The first cycle has been repeated on the specimen with opened window, in order to evaluate the incidence of a single glass panel on the façade global response. A lateral force of 15.9 kN was imposed to the specimen and the displacement attained was 22.64 mm. The curve stiffness is similar to that of first cycle, so the missing glass panel did not modify the façade behaviour. However, removing the load, the internal layer (thickness 6 mm) of the larger glass panel cracked at the upper corner (Fig. 3b), probably due to the redistribution of strength caused by the missing glass panel. A residual displacement equal to 18.54 mm has been recorded. Then, other two cycles were performed at maximum displacements of 26.29 mm and 35.86 mm. The peak forces achieved were 19.25 kN and 23.48 kN, respectively. No glass fall out has been recorded up to a maximum displacement of 35.86 mm, which corresponds to 1.08% of the interstorey height (3,300 mm).

3. FE modelling

A finite element model for the facade tested has been developed within the software SAP2000 [20] and non-linear static (i.e. push-over) analyses have been performed. In order to develop a generally valid model, the contribution of the aluminium frame, the glass-to-frame interaction and the gasket-to-glass friction have been taken into account.



(a) (b)

Fig. 4 – (a) Rigid and (b) deformable transom-to-mullion connections

3.1 Aluminium frame and transom-to-mullion connection

In the first phase of the analysis, the behaviour of the aluminium frame without glass panels subjected to lateral load has been considered. The aluminium frame is modelled using beam elements according to the facades geometry and the transoms/mullions real cross-sections have been properly modelled.

The effect of the three seismic beams has been modelled by inserting a rod constrain between the points of the mullions connected to the same seismic beam. This kind of constrain imposes a constant distance between the selected points during the loading phase.

The mullions are continuous elements along the entire facade hight, whilst the transoms are connected between two consecutive mullions. The transom-to-mullion connections behavior, in terms of rotational stiffness, is strictly related to the mechanics of the real transom-to-mullion joint.

If the connection between transom and mullion is sufficiently stiff, the transom could be considered fixed to the mullion (Fig. 4a). Vice versa, if the transom-to-mullion connection is concentrated at one side of the mullion, the connection should be modelled as an hinge (Fig. 4b).

The response of the aluminium frame subjected to lateral load is reported in Fig. 6, curve (a). The stiffness of the curve is a function of the transom-to-mullion connection rotational capacity.

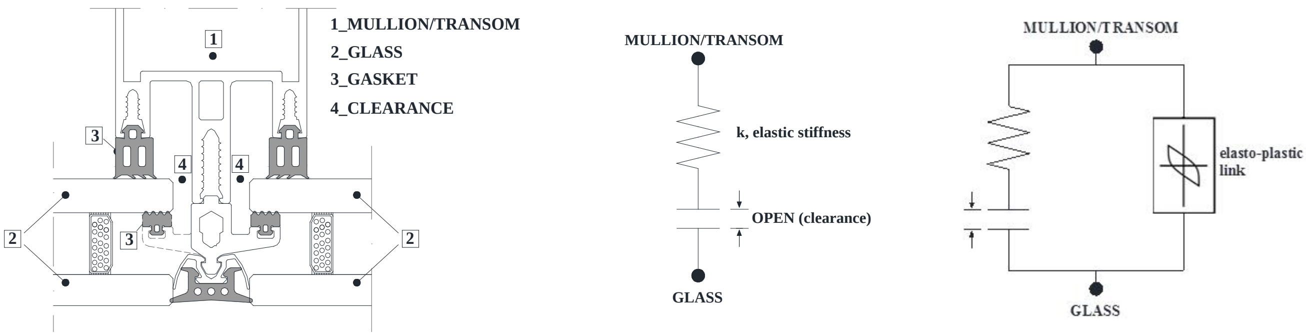
3.2 Glass-to-frame interaction

The glass panels have been inserted in the model as shell elements of global thickness equal to the sum of the single glass panes of the insulated system.

In stick systems, the glass panels are not directly and continuously connected to the aluminium frame. The connection between the two parts is obtained by elastomeric gaskets and a clearance of few millimetres is left between the glass and the transom/mullion (Fig. 5a).

In the model, the glass shells have been modelled with 200÷150 mm square meshes. Each external vertex of the shells is connected to the mullion/transom by non-linear gap links (Fig. 5b). The gap is an non-linear elastic element characterized by an elastic stiffness, k, and an open which works in compression only. This two-nodes link simulates the glass-to-frame interaction. Indeed, the gap open represents the clearance and the elastic stiffness represent the frame local deformability at the contact between glass and frame.

When the link is “closed”, the glass-to-frame contact happens and the glass participates to the global stiffness. Before the gap closure, the glass panel is free to move as a rigid body, because the rotation on the façade plane is free.



(a) (b) (c)

Fig. 5 – (a) Glass-to-frame connection, (b) gap element and (c) gap element in parallel with plastic Wen element

The contact between glass and frame is initially assumed perfectly rigid, so the elastic stiffness *k* assigned to the gap is assumed to be quite large, as depicted in Fig. 6, curve (b).

However, under the action exerted by the glass panels, the aluminium thin walls of the elements in contact develop local deformations, so the assumption of perfectly rigid behaviour seems unrealistic. Assuming a lower value for the gap elastic stiffness, the curtain wall response assume the trend of Fig. 6, curve (c).

The two curves (b) and (c) coincide with the curve (a) given by the frame only until the gap closure, as expected. After this moment, the glass panels are in contact with the frame and a stiffness increase is observed.

3.3 Gasket-to-glass friction

The connection between glass panels and aluminium frame is obtained by using elastomeric gaskets. These elements, during the loading phase, are mainly subjected to shear deformation. The friction at the interface between glass and gasket takes part to the global stiffness up to the detachment of the gaskets.

The gasket-to-glass friction has been modelled by inserting non-linear links which work in parallel with the aforementioned gaps (Fig. 5c). These two-nodes links are characterized by an elastic- plastic Wen behavior.

The comparison between the capacity curves with (d) and without (c) the gasket friction (Fig. 6) shows an increased initial stiffness due to the friction phenomenon. When the behaviour of the gaskets become plastic (i.e. detachment of gaskets), the stiffness of the curve (d) is reduced and becomes equal to the stiffness of the curve

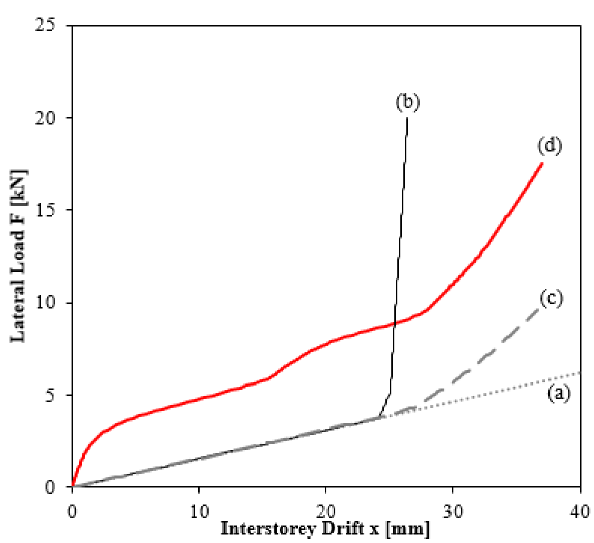


Fig. 6 – Force-drift curves for frame only (a), for frame + glass with perfectly rigid contact (b) and deformable contact (c), for frame + glass + gasket friction (d)

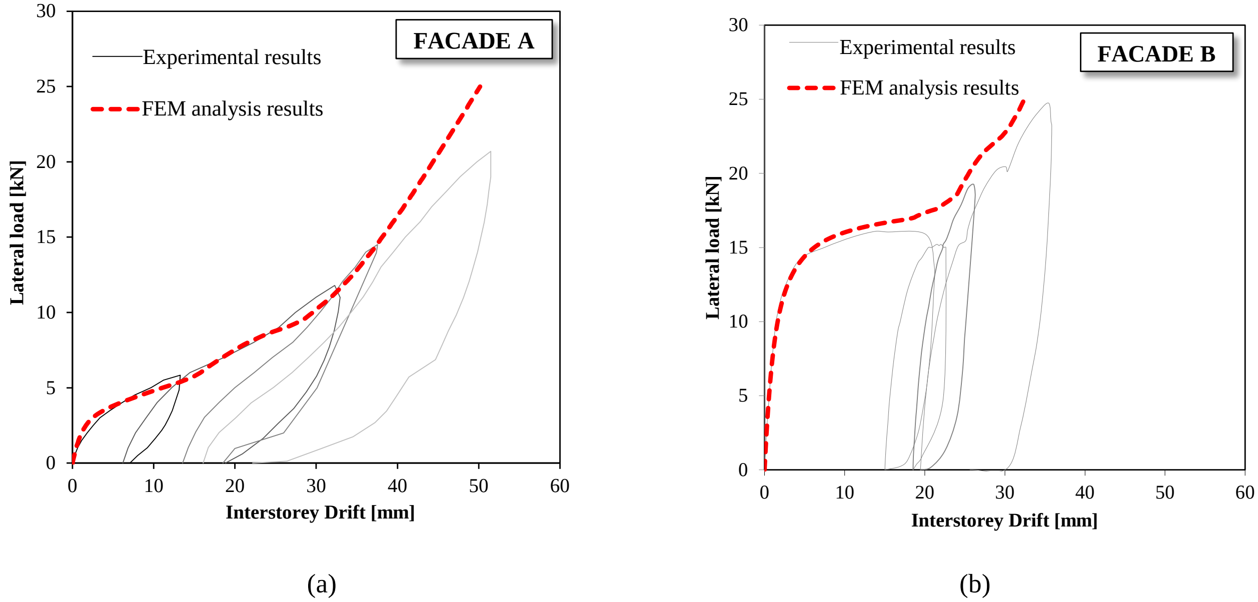


Fig. 7 – Comparison between experimental and numerical results: **façade A** (a) and **façade B** (b)

without friction (c). Then, when the contact between glass and frame happens, the stiffness increases suddenly in both (d) and (c).

4. Numerical vs. experimental results

Two numerical models have been developed for both façade A and B. For façade A, a rigid transom-to-mullion connection has been adopted. On the contrary, an hinged transom-to-mullion connection has been used for modelling the frame response of façade B.

The gap elastic stiffness and the plastic Wen initial stiffness and yield force of both models have been calibrated in order to best matching the experimental response.

It should be noted that the two façades have shown very different behaviours, both in terms of aluminium frame and gaskets response. The frame of façade B appeared less stiff with respect to façade A. On the contrary, the local stiffness after the glass-to-frame impact resulted higher for façade B if compared with façade A.

The silicone gaskets have a fundamental role in the initial responses of the two test units analysed. The mechanical behaviour of the silicone gaskets in terms of force-displacement relationship presents the same trend for both façades. Nevertheless, the gaskets effect provides a considerably high initial stiffness and strength for façade B.

The numerical and experimental force-drift relationships for façade A and B are depicted in Fig. 7a and b, respectively. The comparison between the calibrated FEM analysis and the experimental results shows a good match between the numerical models and the experimental curves for both the specimens analysed.

5. Conclusions

In the present work, the behavior of two full-scale stick curtain wall systems subjected to lateral loads compatible with a seismic excitation has been investigated.

The responses of the two test units analyzed showed a similar general trend in the force-interstorey drift relationships. In particular, the force-drift curves presented an initial non-linear behavior with decreasing stiffness and a sudden stiffness increase up to the glass failure. However, the strength/deformation capacity of the two specimens are not comparable, due to the high dependence of the lateral response from the geometric configuration.

Then, the lateral behavior of the stick system has been analyzed by means of FE modelling. The effect of aluminium frame and transom-to-mullion connection stiffness, clearance between glass panels and frame, local stiffness in the glass-to-frame interaction and gasket-to-glass friction have been analyzed and inserted in the model. The comparison between numerical and experimental curves confirmed the reliability of using such model for predicting the lateral response of stick curtain walls.

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