Amplification of seismic demand to a glass facade attached to a RC structure: an experimental simulation against a real earthquake

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ABSTRACT
In modern architecture glass curtain walls are becoming one of the most used building envelope, both for commercial and residential buildings. However, recent earthquakes have highlighted the vulnerability of such non-structural components, whose damages could represent a significant economic loss, due to downtime and repairing, as well as an important threat to pedestrians and occupants of the injured building. Nevertheless, limited code prescriptions are provided regarding seismic design of these systems and, in addition, experimental tests are not sufficiently developed. Based on these considerations, the present paper presents the analysis of the experimental results of a dynamic racking test in which a real earthquake has been simulated. In particular, a full-size glass façade has been subjected to the displacements registered at the third floor of a monitored school building located in Norcia (central Italy). This seismic event (of moment magnitude Mw=6.6) struck Norcia at 07:40 local time (06:40 UTC) on October the 30th. The seismic demand of the glass façade have been evaluated and discussed. The response of the tested non-structural component showed an amplification in terms of peak accelerations compared to peak floor acceleration of the main building structure. Finally, through the formulation suggested by the Eurocode 8, that permits the evaluation of the floor spectral acceleration, the fundamental period of the tested facade was calculated and compared with that obtained by performing a modal analysis in Sap2000 software.

1 INTRODUCTION

In the last years, the seismic behaviour of non-structural elements is becoming increasingly important. The damage to the building parts, which are considered not belonging to the main structure and, so, traditionally considered of low importance in the seismic design of buildings, can be, indeed, very significant in terms of human life and economic losses. In fact, also after moderate earthquakes the major economic losses and safety threats are often attributable to these components.

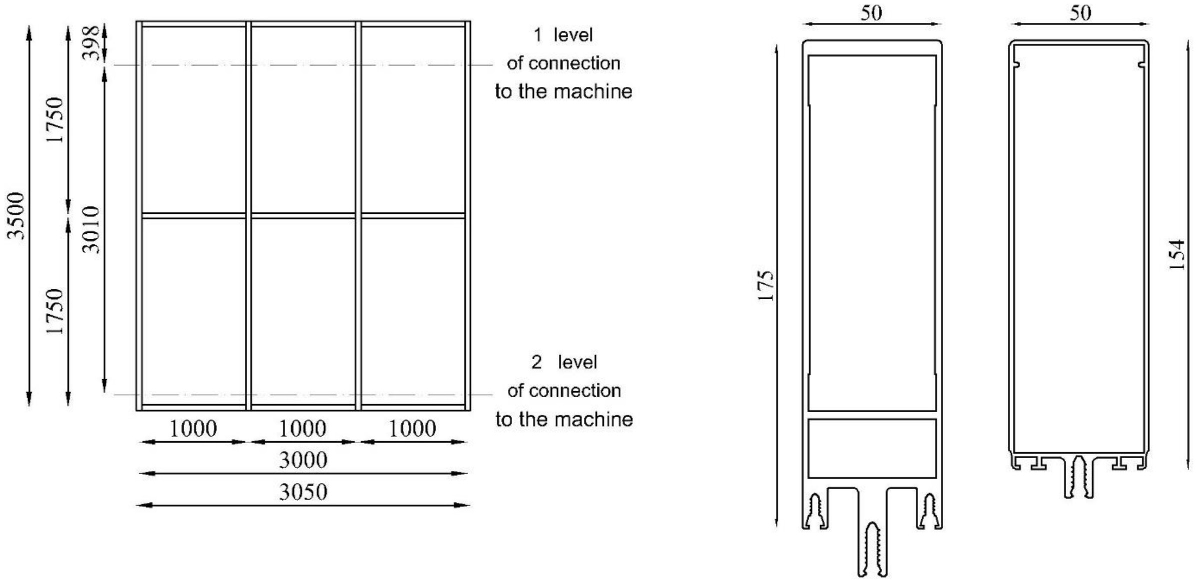
During recent earthquakes, glass curtain walls, like other non-structural elements, have shown poor performances (e.g. glass panel fallout or functionality losses) (Sucuoglu and Vallabhan 1997), (Andrew 2007), (FEMA E-74 2011). Evans and Ramirez (1989) observed that during the Mexico city earthquake the 17% among 510 damaged and undamaged buildings experienced glass façade failures.

However, it is important to note that these failures can cause an important hazard to people, a huge repairing or replacing expense and an inconvenience due to the interruption of the normal activities that usually taken place in the struck building.

The research on this topic is still not very extensive and accordingly there is a lack of regulations about their seismic design.

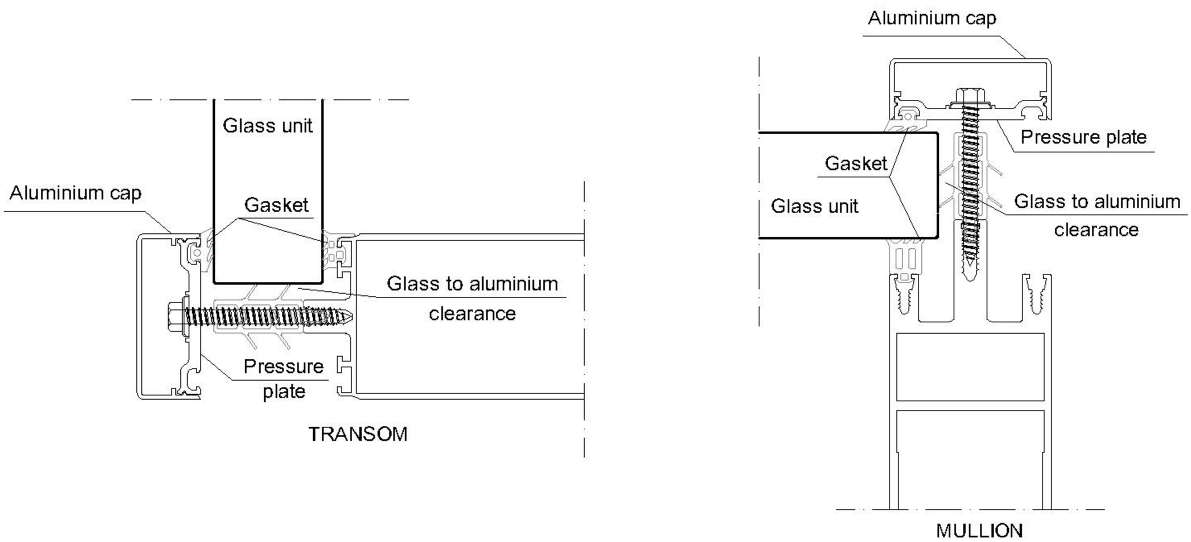
During the last year several experimental studies have been carried out (e.g. (Bouwkamp and Meehan 1960), (Bouwkamp 1961), (Lim and King 1991), (Thurston and King 1992), (Pantelides and Behr 1994), (Behr et al. 1995), (Carre´ and Daudeville 1999), (Behr 2001), (Memari et al. 2003), (Eva Hutchinson 2011), (Memari et al. 2018), (Caterino et al. 2017), (Aiello et al. 2018)).

For most experimental campaigns, the in-plane racking displacements have been considered the dominant cause of damage (as observed also in Wright (1989)) and therefore the performed test are focused on the in-plane behaviour of glass curtain walls.



(a) (b)

Figure 2. Glazed stick-built system: (a) front view, (b) cross-sections. Dimensions in mm.



(a) (b)

Figure 1. Glazing details: (a) transom; (b) mullion.

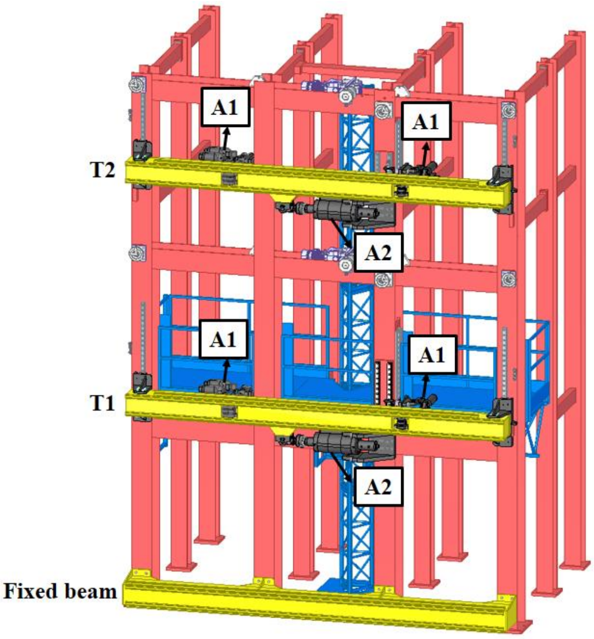
Among the different load protocols that have been developed and used to perform the abovementioned experimental tests the *crescendo test* (Behr and Belarbi 1996), (Behr 1998) is surely the most commonly adopted. It is characterised by a concatenated series of sinusoidal cycles at increasing amplitude and it has been included in the AAMA 501.6 recommendations (2001) to assess the horizontal racking displacement amplitude of glazing system frame, which might provoke fallout of glass panels.

However, few activities have been conducted to study the perfomance of these systems under real earthquake motions (i.e. seismic capacity) and to compare this last with the seismic demand in terms of acceleration and deformation (Huang et al. 2017).

The seismic force trasferred from the main structure to the curtain wall system is the floor response, which produce both interstoy drift and inertia force to the façade causing its probable failure (Lu et al. 2016).

The scope of the present work is to investigate the response of a glass façade under an earthquake motion. In particular, an in-plane racking test has been performed at the laboratory of Construction Technologies Institute (ITC) of the Italian National Research Council (CNR).

During this test the displacements recorded at the second and third floors of a monitored building



(a)

(b)

Figure 3. ITC-CNR test facility for façades. a) Schematic representation; b) tested façade mock-up.

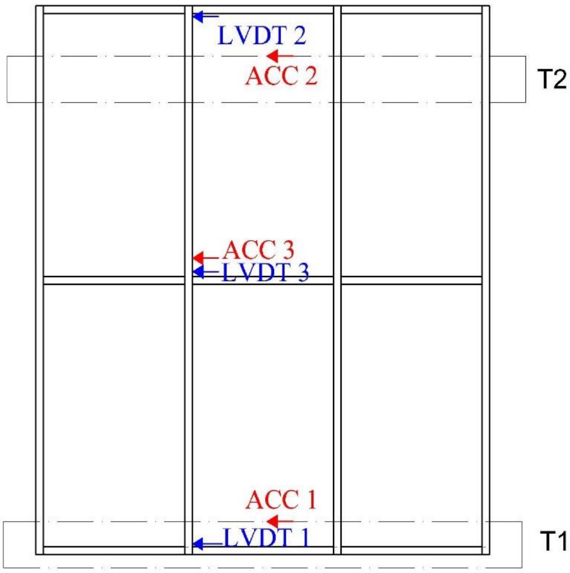


Figure 4. Location of the potentiometric displacement transducers (LVDTs) and of the accelerometers (ACCs) on the glass façade.

school located in Norcia (Italy) has been applied to the tested façade.

The corresponding response registered by the accelerometer located on the specimen during the test has been evaluated and compared with those calculated through European code formulation.

2 SPECIMEN AND TEST SET-UP

The tested sample is 3.50 m high and 3.05 m wide and covers one storey of the building under exam (Figure 2a). The structure is a grid system made of three mullions and three transoms of extruded aluminium profiles (alloy EN-AW 6060, supply type T5), whose cross sections are shown in Figure 2b.

The insulating glass units (all of the same size 1.70 x 0.95 m) consist of two laminated glass plates, obtained by interposing a PVB interlayer (0.38 mm) between two 4 mm thick glass plates. This last are separated by a 16 mm thick air space. The total thickness of the glass panel is therefore about 32.8 mm (4/0.38/4/16/4/0.38/4). The clearance between glass panels and aluminium frame is about 5 mm (Figure 1).

The transoms are fixed horizontally to the mullions by specific connecting bolts (button support) and stainless steel screws.

The insertion of the glass panels in the respective seating is carried out from the outside. The panels are then blocked by a presser profile (pressure plate in Figure 1), which is fastened to the mullion by stainless steel self-tapping screws.

The grid is completed externally by snap-on coverings of 50mm in length (aluminium cap in Figure 1).

Finally, various accessories in EPDM ensure the non-infiltration of water (gasket in Figure 1).

The test facility to perform static and dynamic tests on curtain wall systems is located at the Institute for Construction Technologies (ITC) of the Italian National Research Council (CNR) in San Giuliano Milanese (Milan, Italy) (Figure 3).

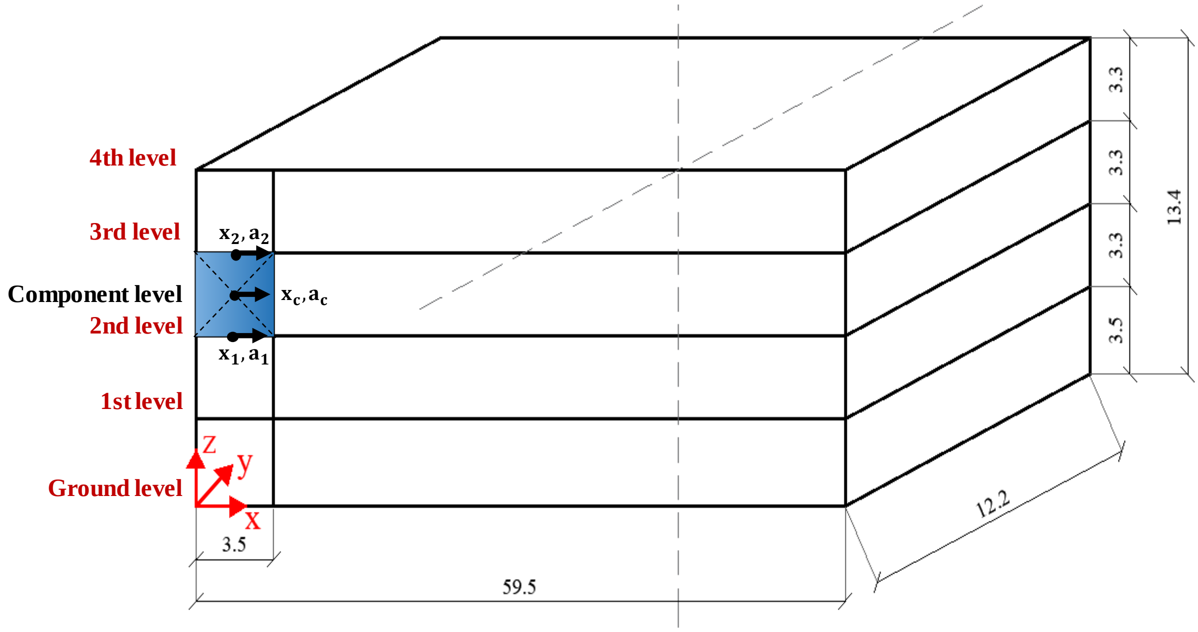


Figure 5. Schematic representation of the case study building school (dimensions in m).

The facility is able to test the primary performances expected from a façade system in terms of resistance against wind pressure, static and dynamic water-tightness and air leakage. The structure was then recently improved to reproduce also in-plane and out-of-plane displacements, in order to simulate earthquake induced horizontal movements. The test facility is able to accommodate full size façades up to 6.3 m wide and up to 8 m high which can be anchored to the steel support structure at three beam levels: one fixed beam at the bottom and two moving support beams at the second and third level (T1 and T2 in Figure 3a). All beams are provided with anchor channels to connect various type of façade systems, walls, partitions through commercial typical connections.

The moving beams are supported on low friction rollers and connected to a dynamically controlled hydraulic actuators system. A mechanical lift system for the moving beams allows for various inter-storey heights.

Racking movements are produced with a dual stage hydraulic actuator (on each moving beam) with a load capacity of 200 kN, a maximum displacement of ± 85 mm and a maximum frequency of 30 Hz in the in-plane direction (A2 in Figure 3a). Out-of plane movements are produced with two dual stage actuators (on each moving beam) with a load capacity of 100 kN each, a maximum displacement of ± 85 mm and a maximum frequency of 30 Hz (A1 in Figure 3a), allowing high frequency load protocols.

Actuators are commanded by a digital controller, where the control panel allows applying the desired displacement amplitudes, frequencies, displacement waveforms, and number of cycles.

In particular, the test facility is composed of:

- steel frame support structure, stiff enough to counteract the actions induced by the moving beams;
- moving beams, 2 steel beams supported on low-friction rollers for smooth sliding,
- controlled by the dynamic hydraulic actuators system (T1 and T2 beams in Figure 3a);
- mechanical lift system connected to the moving beams to allow different inter-story heights;
- controlled hydraulic actuators system, consisting of:
  - 2 type A2 actuators (in-plane movements): hydraulic Ø157 x Ø110 cylinder with maximum static force of 200 kN, full stroke of 170 mm, frequency up to 30 Hz;
  - 4 type A1 actuators (out-of-plane movements): hydraulic Ø102 x Ø063 cylinder with maximum static force of 100 kN, full stroke of 170 mm, frequency up to 30 Hz.
- Control and data acquisition system, consisting of the following force and displacement transducers:
  - for actuators A2, a 200 kN load cell, a ±100 mm displacement transducer, 0.2% accuracy;
  - for actuators A1, a 100 kN load cell, a ±100 mm displacement transducer, 0.2% accuracy.
- Data acquisition device and controller.
- Purpose-developed computer program.

Three linear potentiometric displacement transducers (LVDTs), with range ±100 mm, and three accelerometer were placed on the façade, as illustrated in the Figure 4, to measure horizontal displacements and accelerations. Furthermore, load and displacement data were also continuously recorded during the test through the data acquisition system located on the actuators.

3 TEST PROTOCOL AND EXPERIMENTAL RESULTS

In order to evaluate the seismic performance of curtain walls, a full-size dynamic in-plane test has been performed at the laboratory of the Construction Technologies Institute (ITC) of the Italian National Research Council (CNR). In particular, the tested façade has been subjected to a real earthquake ground motion, registered at the basis of a school of Norcia (Italy) at 07:40 local time (06:40 UTC) on October the 30th. The data related to the building and the earthquake registration have been taken form the website of the seismic observatory for structures, Italian Department of Civil Protection (http://www.mot1.it/iss/), where the school is indicated as “15SNO”.

The specimen is supposed anchored to the second and third floors of the building as shown in the Figure 5.

The building in exam is monitored by a sensor system, composed of eighteen accelerometer.

In particular, for the performed experimental campaign, the accelerogram in the plane of the façade has been considered (Figure 7). The peak ground acceleration is equal to 0.55 g.

Two linear direct-integration time-history analysis in displacement control have been

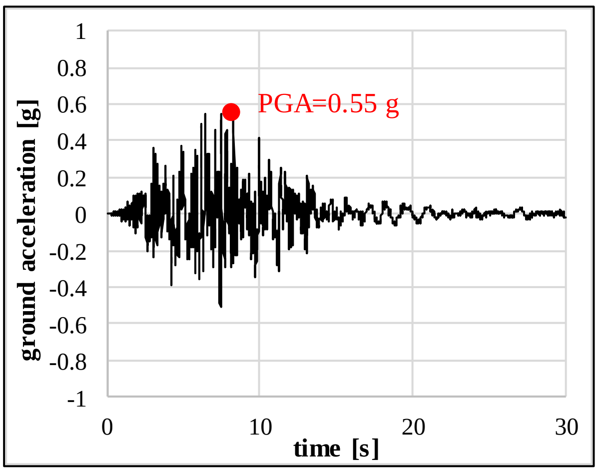
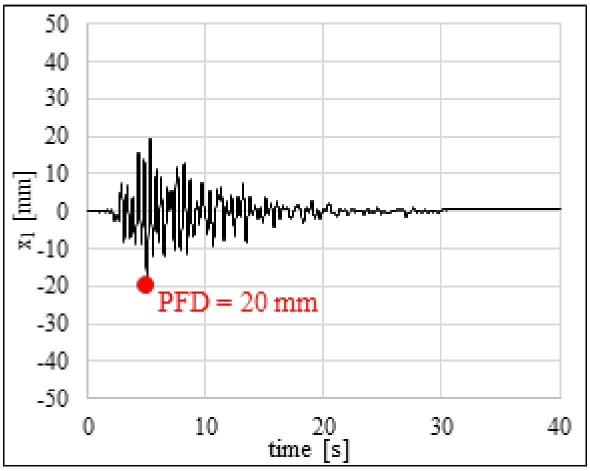
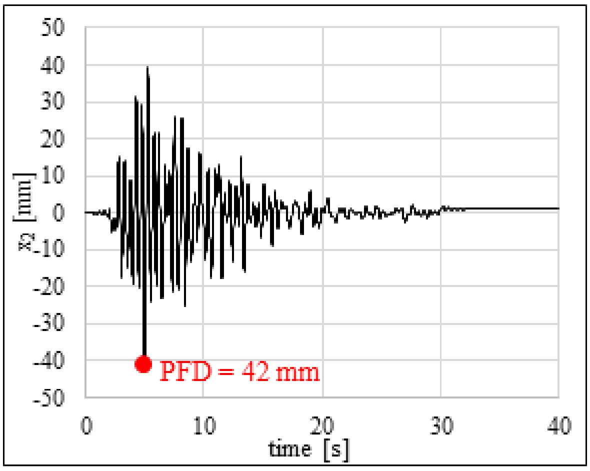


Figure 7. Norcia earthquake (October 30th 2016, 06:40), ground acceleration registered by the accelerometer sensor in the x direction.



(a)

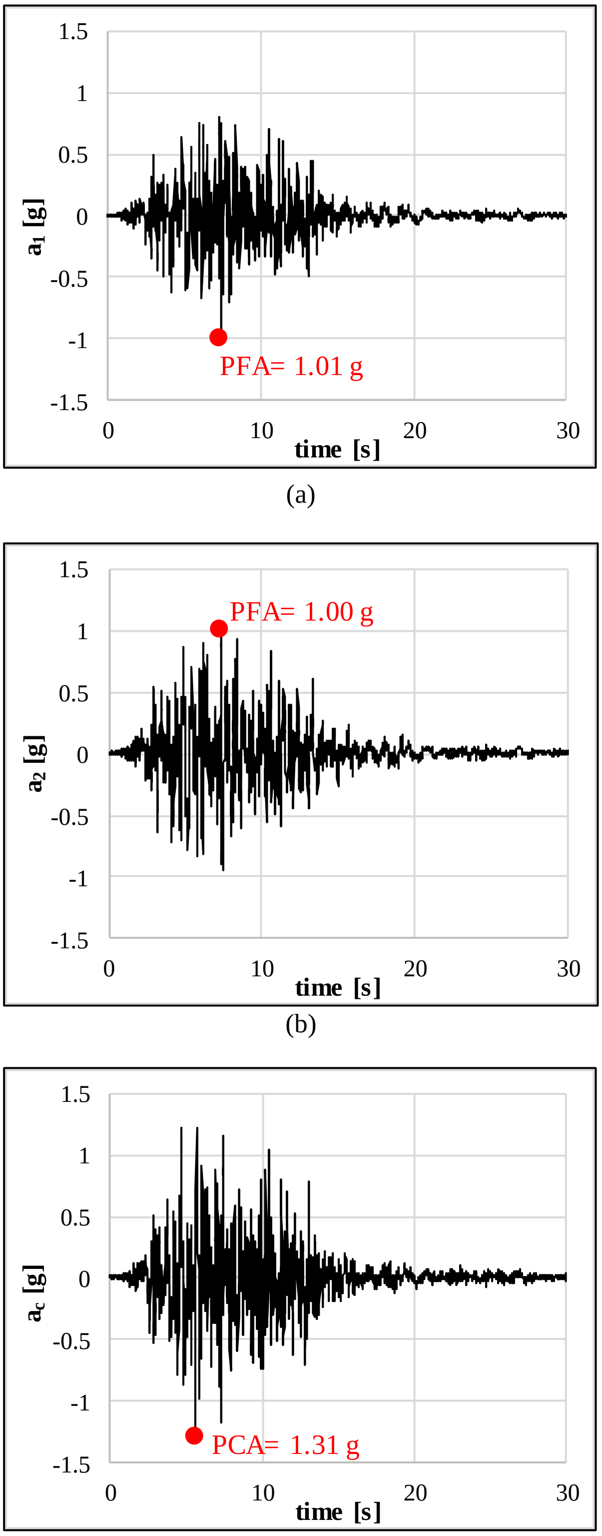


(b)

Figure 6. Displacement time history: a) for beam T1; b) for beam T2.

performed on SAP2000 software, applying to the structure, modelled in a previous study (Bergami and Nuti 2015), the abovementioned accelerogram in Figure 7. Therefore, the displacement outputs of the second and the third floor (Figure 6) have been applied respectively to the beams T1 and T2 of the testing machine (Figure 3). It is possible to notice that the displacements are amplified through the building floor: the Peak Floor Displacement (PFD) is about 20 mm at the 2nd floor and 42 mm at the 3rd floor.

The accelerations registered by the accelerometers ACC1, ACC2 and ACC3 (Figure 4) during the test have been filtered through SeismoSignal software (2004). In particular, to remove unwanted frequency components from each given signal, lowpass filtering, which suppresses frequencies that are higher than a user-defined cut-off frequency (40 Hz), has been applied. The classical infinite-impulse-response (IIR) filter Butterworth type has been chosen. It is advised for the majority of applications the use of



(a)

(b)

(c)

Figure 8. Acceleration registered during the test by a) ACC1; b) ACC2; c)ACC3.

a higher order filter (in the case in exam 8th order has been used).

Therefore, this filtered acceleration are shown in the Figure 8: the Peak Component Acceleration (PCA) is greater than both Peak Floor Accelerations (PFAs), so, the tested non-structural component glass façade amplifies the acceleration reached by each floor of the building structure.

According to the European code (EN 1998 2003) the floor spectral acceleration applicable to non-structural elements (seismic coefficient S_a) may be calculated using the following expression:

$$S_a = \alpha\, S \left[\frac{3\left(1+\frac{z}{H}\right)}{1+\left(1-\frac{T_a}{T_1}\right)^2} - 0.5 \right] \geq \alpha\, S g \quad (1)$$

Where:

α is the ratio of the design ground acceleration (PGA) on type A ground (stiff soil), a_g, and the acceleration of gravity g; S is the soil amplification factor; T_a is the fundamental vibration period of the non-structural element; is the fundamental vibration period of the building in the relevant direction; z is the height of the non-structural element above the level of application of the seismic action (foundation or top of a rigid basement); and H is the building height measured from the foundation or from the top of a rigid basement.

For the case in exam, considering an infinitely rigid non-structural component ($T_a = 0$), (being $\alpha = 0.55g$, $z = 8.45\ m$, $H = 13.4\ m$) S_a is equal to 1.07 g. This value is about the mean of the values PFAs (Figure 8a,b). Moreover, substituting in the equation (1) the value of the PCA (Figure 8c) it is possible to know the period of the glass façade, $T_a = 0.025\ s$, that is very similar to the one evaluated through a modal analysis performed with the SAP2000 software (2014).

4 CONCLUSION

This paper deals with the seismic assessment of glass facades. In particular, the seismic demand to a stick-system full-scale curtain wall, currently available on the market, caused by a real earthquake has been evaluated. The façade has been tested with an exclusive high performance facility at the laboratory of Construction Technologies Institute (ITC) of the Italian National Research Council (CNR). During the test, the displacement time-histories recorded during the 2016 Norcia earthquake have applied to the specimen.

It is noted that the accelerations of the floors of the main structure were amplified through the façade component, because the dynamic response of the supporting structure will filter different frequencies of excitation, amplifying the demand. The mean value of the floor accelerations has been compared with that suggested by the Eurocode 8 (EN 1998 2003) and they are very similar. Finally, the fundamental vibration period of the non-structural element glass façade has been evaluated according to the experimental results and to the abovementioned code provisions. It was alike the value calculated by carrying out a modal analysis in Sap2000 software.

Thus, further studies are needed to evaluate the displacement demand, in order to compare it with the displacement and the acceleration capacity of the component and with those that could be obtained by other current code provisions.

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