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Considerations on efficient procedural steps for seismic capacity assessment and diagnostics of historic structural glass systems

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ABSTRACT

Structural designers are nowadays supported by standards and guidelines to optimize the mechanical performance and safety level of glass load-bearing elements and assemblies in buildings. Key role is given to deformation and stress check, and post-breakage verification, for newly designed glass systems. Performance limits are provided also for extreme events, such as earthquakes. Besides, major risk for people could derive from existing/historic glass components and systems that have not been conceived to offer minimum structural capacity against operational loads and accidents. No technical recommendations are available to quantify their actual vulnerability and capacity, thus retrofits and mitigations are also hard to plan. In this paper, possible procedural methodology steps towards are addressed. The attention is given to present technical guidelines for structural design, on one side, and on the optimized use of non-destructive methods (like Operational Modal Analysis (OMA)) and numerical methods to facilitate diagnostic steps and the mechanical characterization of components, as well as to realistic description of boundaries and even damage. Practical examples are discussed for a case-study glass-steel envelope built in 60s in Italy, which is analyzed with the support of Finite Element (FE) numerical models against in-plane and out-of-plane seismic actions.

1. Introduction

Glass material finds place in buildings in several forms [1]. Specific calculation methods and performance indicators have been elaborated for various loading and boundary conditions and can be adopted by structural designers to ensure appropriate performances and safety levels in the framework of well-known Limit State concept in use for buildings [2–4]. Both for novel and existing constructions, even “secondary” structural glass systems are frequently realized to interact with primary components of different materials [5]. In most cases, for newly designed glass systems, the structural challenge due to the intrinsic brittleness and vulnerability of material can be efficiently controlled with the use of laminated (LG) multi-layer sections. However, further potential risks for people should be properly minimized, like for example, in the presence of extreme loads like earthquakes [6,7]. Special design attention must be hence spent in design stage also for restraints and fixings, given that they have a critical role in preventing stress peaks in glass and minimizing maximum deformations [8–10]. Novel dissipation system can be also used to replace traditional restraints [11–13].

On the other side, existing (and even historic) glass elements and systems that are part of buildings, especially if realized before the definition of structural design concepts and rules, are not specifically conceived to sustain design loads, and thus can represent potential risk for people under accidental/extreme events (Fig. 1). Mitigation and retrofit interventions can be planned to reduce risk, like the use of anti-shatter films [14]. However, their efficiency depends on loading and operational conditions [15]. Also, engineering knowledge of current load-bearing capacity and structural demand for glass systems to retrofit is first required.

In this paper, a case-study glass envelope is taken into account and investigated for the assessment of possible procedural methodological steps towards the characterization of actual mechanical properties and load-bearing capacities for seismic (and multi-hazard) retrofit purposes. The selected system is used to extend some considerations about a possible methodology (and gaps) for seismic capacity assessment and diagnostic investigation on similar assemblies. Examples can be found for different loading and boundary conditions in Refs. [16–18], The presently investigated glass-steel envelope, designed in 1961 and built in 1962 in a museum in Italy [19], should not represent a source of

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(a)



(b)

Fig. 1. Examples of historic glass systems: (a) curtain wall for Hallidie Building in San Francisco (US), completed in 1918 (designed by Willis Polk, photo © Patricia Chang); (b) roof of Vittorio Emanuele II Gallery in Milano (Italy), completed in 1867 (designed by Gaetano Mengoni).

Table 3.9. Classification of structural glass elements according to conditions of use. By definition non-structural construction products fall under class 0.

Type	Class for pre-glass-failure assessment ¹	Class for post-glass failure assessment
Vertical elements ^{2,3} with continuous edge restraint	1	I/NA ^{4,5}
Vertical elements ² with fixing points	2/1	I/NA ^{4,5}
Horizontal roofs ^{2,3}	2	2/1
Railings with danger of fall	2	2/1
Reinforcement fins	2	2/1
Floors, load-bearing beams	2	2
Pillars	3 (specific analyses with Level II or Level III methods)	2 (with pre-failure action)

Notes:
⁽¹⁾ Within the same category, the choice of the most or least restrictive class of assessment depends on the importance of the work, the degree of danger arising in the event of failure of the glass, and whether safety countermeasures can be immediately implemented in order to reduce the consequences of failure (e.g. propping, protection measures, fences, etc.).
⁽²⁾ An element is considered vertical if the angle of its plane from the vertical is less than 15°. An element which does not fall under this definition is considered horizontal.
⁽³⁾ NA stands for "No Assessment". When failure of the glass produces negligible risks in terms of damage or human life, post-failure assessment may be omitted.

Table 4.7. Performance levels required against seismic actions.

Name	Description
ND – No damage	It is assumed that the system is free of any damage that requires the glass panes to be replaced for the functionality of the building. Specifically, façade and roof elements must maintain their requirements of impermeability to wind and to rain.
SD – Slight damage	It is assumed that the system may suffer loss of functionality of some elements, the rapid replacement of which does not involve particular technical details, with the building remaining usable overall. There is no risk for users connected with partial failures.
HD – Heavy damage	The system suffers heavy damage, with a high degree of loss of functionality, with high costs in terms of re-establishing functionality, but without risks of falls of material which could potentially lead to high risk situations.
F – Failure	The system suffers severe damage and exhibits extended evidence of failure. Any falling material would cause risks comparable to those caused by other elements such as cornices and external cladding.

Table 4.8. Performance levels for the "glass" system

Level	Importance class			
LS	I	II	III	IV
SCO	-	-	ND45	ND60
SLD	SD15	SD10	SD75	SD100
SLP	HD133	HD175	HD713	HD910
SLC	-	-	F1463	F1950

Fig. 2. Selected relevant seismic parameters for newly designed glass elements (CNR-DT 210/2013 [5]).

possible risk for museum visitors. The attention is focused on the performance demand evaluation, based on a standardized protocol to possibly account for existing glass envelopes and systems. The procedure, as shown, results from the application of actual technical regulations for structural glass design and from the integration of in-field Operational Modal Analysis (OMA) experimental techniques for the Finite Element (FE) numerical analysis of state-of-art performance under ordinary and extreme design actions. In this manner, possible uncertainties on geometrical and mechanical properties are minimized by OMA output and FE numerical models are extended to possible critical operational scenarios. Further, degradation of materials (like in boundaries) is also efficiently taken into account.

2. Design of glass element under seismic loads

2.1. General requirements in the Italian framework

Technical guidelines and standard documents are nowadays conceived to support structural designers in the realization of load-bearing glass elements with primary or secondary, or even non-structural role in buildings, under the effects of various ordinary or accidental design actions [20].

Provisions of relatively recent elaboration for seismic demand

assessment of primary or secondary glass components – including also the definition of class of importance and class of consequence for the building context – are available since few decades only, see for example the CNR-DT 210/2013 technical document [5]. In the Italian context, it provides in fact extended specifications for the seismic analysis and verification of building elements (and systems) made of structural glass.

The analysis against stress peaks and deformations should be carried out at the component level. Design detailing against seismic action depends on multiple structural and building parameters, as also schematized in Fig. 2. In doing so, the use of double (or more) LG elements is privileged due to the additional plastic and dissipation role of bonding interlayers, hence resulting in improved post-failure mechanical performance and enhanced safety levels for customers.

From a structural point of view, during the last century, all the design calculations and assumptions for constructions in Italy were based on the Regio Decreto L.2229 (16/11/1939) [21], which included basic design regulations for reinforced concrete load-bearing members only. Until 1972, with the publication of L1086/1971 standard for reinforced concrete and steel load-bearing members [22], the Regio Decreto represented the univocal technical support and regulation for structural designers. The standard document L64/1974 [23], finally, introduced some fundamentals for structural design of constructions under seismic events. Presently, the NTC2018 standard for structural designers [20]

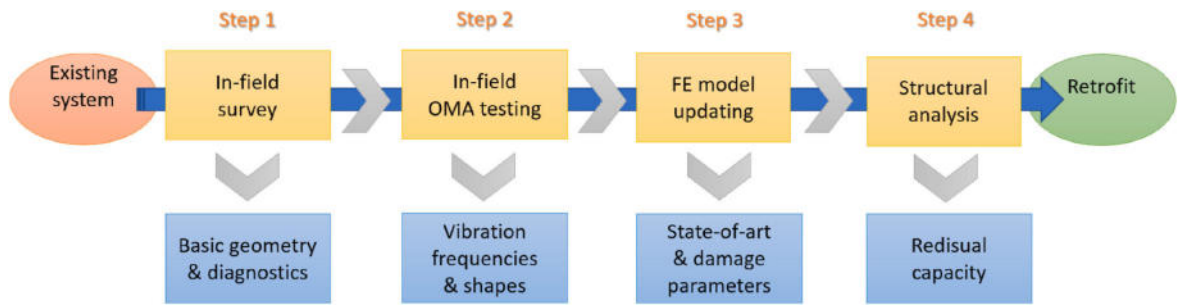


Fig. 3. Possible procedural steps of operational methodology for structural assessment of existing glass systems.

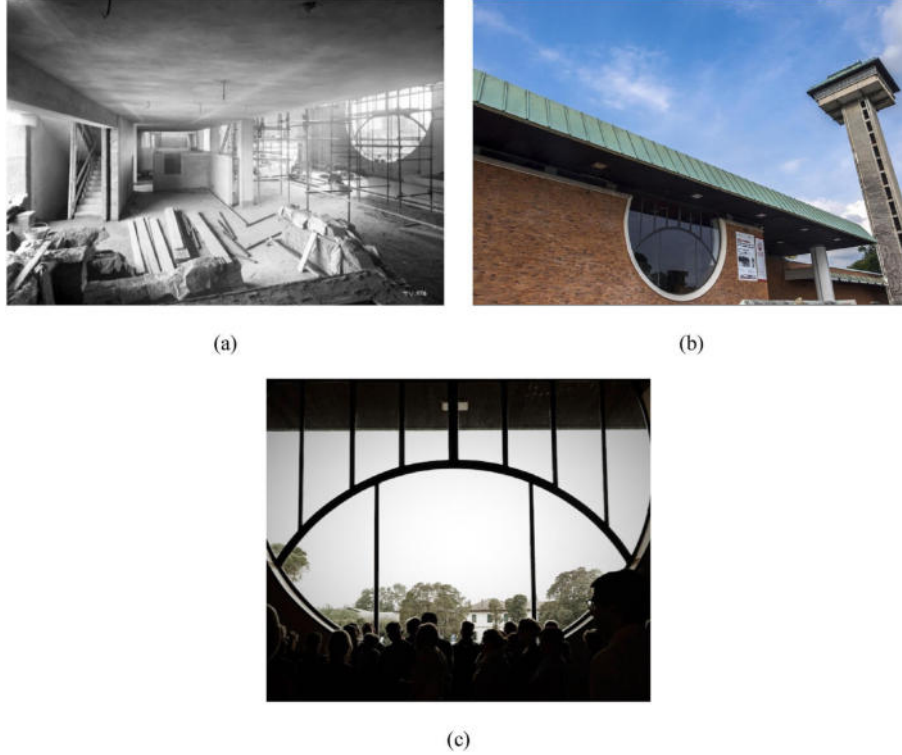


Fig. 4. Examined glass envelope: (a) building stage and (b) present external view (Southern side) of CID museum (designed by Cesare Pea); (c) envelope detail.

does not include specification for novel building materials, but suggests the use of consolidated technical documents like [5].

In such a kind of scenario and structural demand evolution, it is clear that a special consideration should be spent for existing (and even historic) glass components, which have been realized without specific consideration for minimum performance capacity against loading conditions of typical interest for buildings [6]. This issue is enforced by the fact that, often, technical drawings or documents and notes are not available for historic glass systems, and hence a vulnerability analysis of the state-of-art situation can result in rather complex assumptions.

2.2. Current parameters for seismic performance demand

According to Ref. [5], the interaction between a given glass structure and the primary building must always be considered, together with the local behaviour of the glass elements.

The defined performance levels can be reached using design choices and construction measures that involve 1) the use of suitable glass types and sizes; 2) the use of intrinsically ductile systems or connecting systems with a ductile behaviour; 3) the use of connecting systems which guarantee the glass elements a) rigid rotation/translation inside the

load-bearing structural system, b) suitable limitation of the stress level, towards the reference material strength.

The SLV (Limit State for the safeguard of human life) must make sure that the system capacity is not lower than the demand. For glass components, the performance demand is defined in terms of maximum stresses and displacements, therefore there are two types of verifications:

R1) resistance calculations, where it must be verified that the glass can support with acceptable stress peaks the accelerations induced by the assigned seismic event, that is:

$$\sigma_{Ed,max} \leq f_{g,d} \quad (1)$$

and.

R2) displacement compatibility assessment, where it must be ensured that the interaction between the glass element and the rest of the building (with inter-story height H) is compatible with the presence of glass. In general, this interaction is considered to be satisfactory if the construction vibration under in-plane seismic actions results in:

$$u_{max,in} \leq u_{lim,in} = f(H, building) \quad (2)$$

but also if glass deflections under out-of-plane seismic action do not

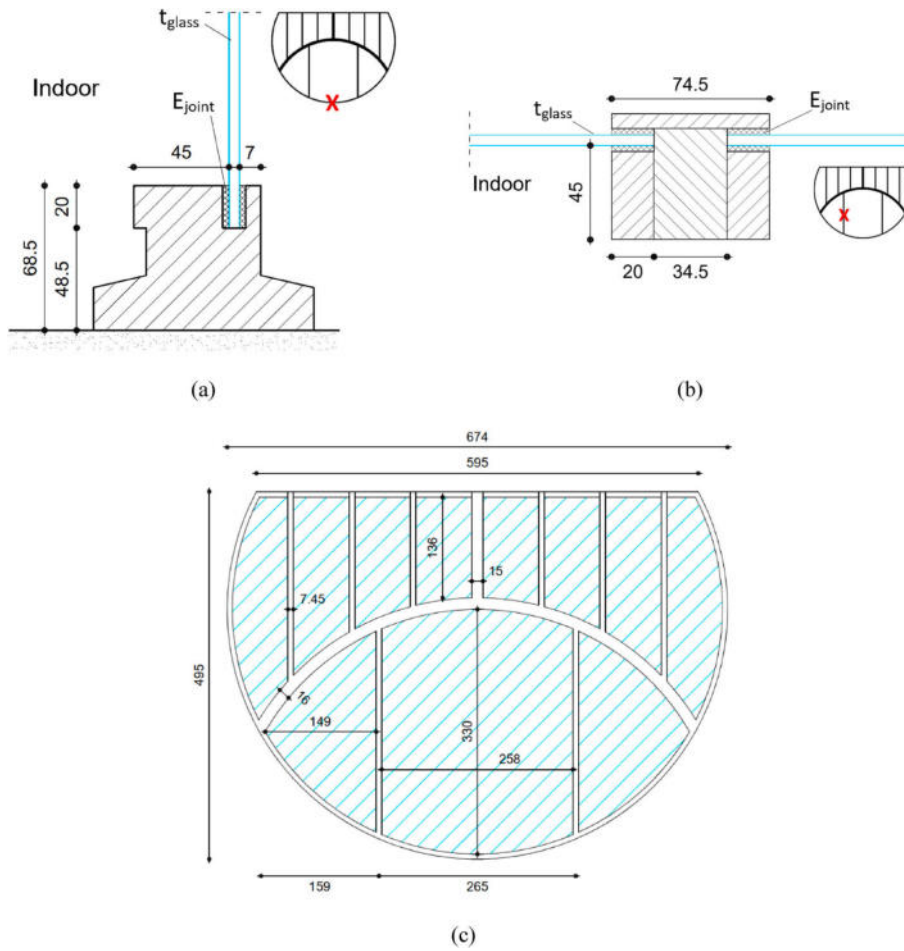


Fig. 5. Cross-sectional details of (a) arched transom and (b) mullion, as derived from the geometrical survey (dimensions in mm); (c) front view (dimensions in cm).

Table 1

Summary of glass components for CID museum envelope (mass values calculated after OMA testing and dynamic identification of the structure).

Panel	A [m ²]	M [kg]	M/M _{TOT}	Span [m]
1	8.070	100.47	0.043	3.30
2	2.449	30.49	0.013	2.71
3	1.53	19.10	0.008	2.71
4	1.808	22.51	0.010	2.64
5	1.385	17.24	0.007	1.92
6	1.193	14.86	0.006	1.56

exceed certain limits:

$$u_{max,out} \leq u_{lim,out} = f(\text{glass size, boundaries}) \quad (3)$$

To satisfy stress and deformation checks, local seismic analyses can be carried out in place of global analyses, including both in-plane and out-of-plane seismic action effects [5,20].

Most importantly, from Eq. (1) and (R1), the design tensile strength of non-prestressed glass under seismic action should be calculated as [5]:

$$f_{g,d} = \frac{k_{mod} k_{ed} k_{sf} \lambda_{gA} \lambda_{gl} f_{g,k}}{R_M \gamma_M} \quad (4)$$

with $f_{g,k} = 45$ MPa; $k_{mod} \leq 1$ depending on the loading time (1 for instantaneous loads); $R_M = 1$ for second-class annealed glass; $\gamma_M = 2.5$

for annealed glass. Among various coefficients, a critical role in Eq. (4) is assigned to k_{mod} , which is intended to account for static fatigue effects [2] and can be empirically calculated as in Ref. [5], based on time loading. Minimum variations can be obtained from existing formulations [24]. In present study, the value $k_{mod} = 0.78$ for transient seismic loads (30 s the corresponding time loading) is taken into account [7]. When the resistance check is carried out at a distance d from glass edges which is at least equal (or higher) than $5s$ (with s the thickness of glass), it is assumed that $k_{ed} = 1$ and $\lambda_{gl} = 1$. Furthermore, $k_{sf} = 1$ for float annealed glass. Finally, λ_{gA} is comprised between 0.75 and 1, to account for the area of glass under maximum stress and the statistically higher probability of the presence of defects in glass. Worth to note that for the verification check in the center, i.e., $d < 5s$, Eq. (1), suggests $k_{ed} \rightarrow 0.7$ (for annealed glass); $\lambda_{gA} \rightarrow 0.75$; $\lambda_{gl} < 1$ (depending on size of glass and

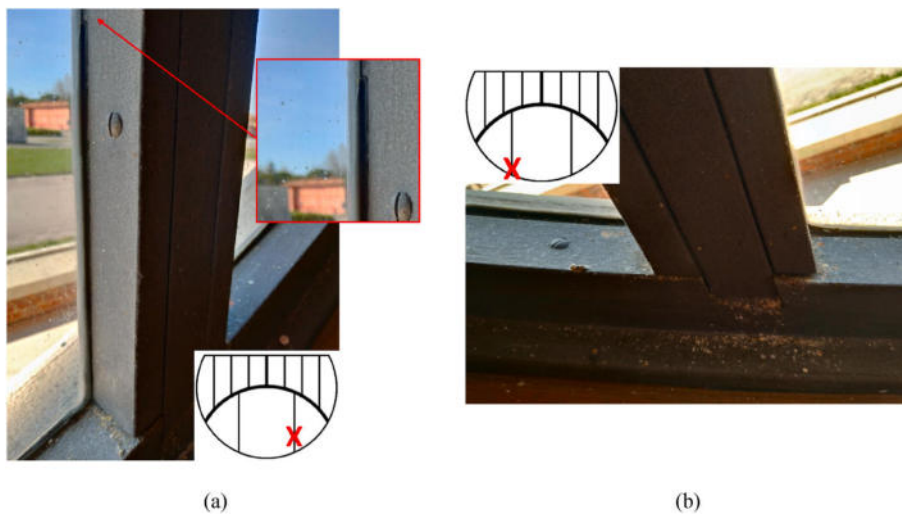


Fig. 6. Structural details from visual inspection (Spring 2022).

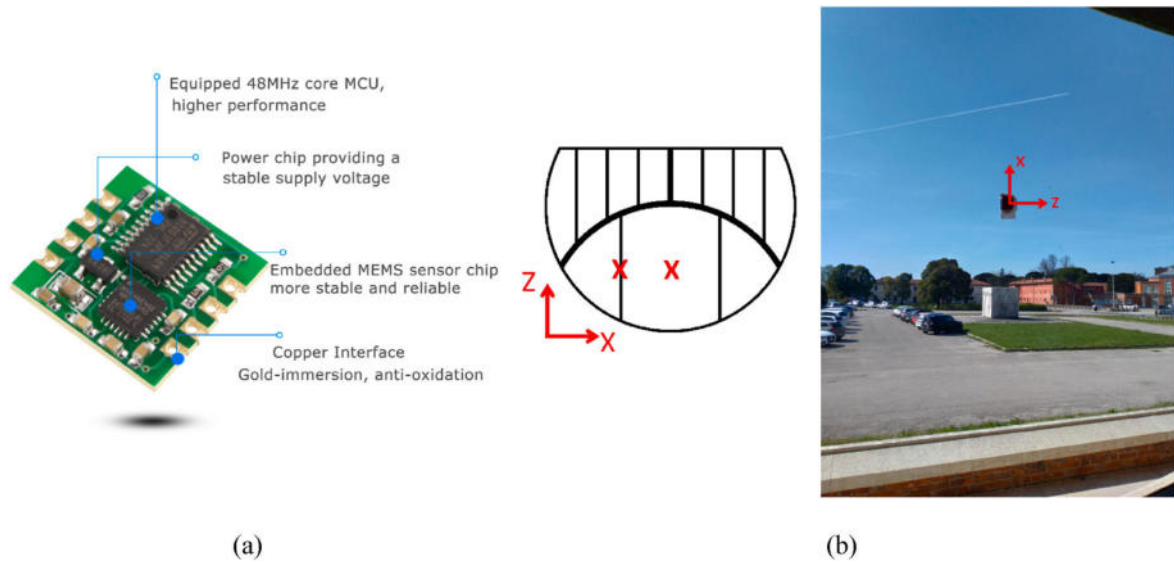


Fig. 7. OMA investigation: (a) detail of MEMS sensors and (b) example of test setup for one of the planned configurations (Spring 2022).

load pattern), thus reducing even down to $\approx -50\%$ the reference strength value for seismic verification.

In terms of deformation limit (R2) and Eqs. (2) and (3), maximum amplitudes are defined respectively in Ref. [20] for inter-story drift of primary constructions (based on story height H and building typology) and in Ref. [5] for glass deflections (based on glass size and boundaries). For example, the limit out-of-plane deflection for monolithic glass with linear supports at the edges should not exceed the value $u_{lim,out} = L_{min}/60$ (and in any case $u_{lim,out} < 30$ mm), with L_{min} the minimum size of glass panel).

2.3. Existing glass systems and operational diagnostic methodology

The load-bearing performance and seismic vulnerability assessment of an existing system, in general terms, is a rather complex issue that is affected by several uncertainties, but can be addressed in different methods [25–29]. For composite glass systems, it can be expected that such a kind of procedural uncertainties could have direct consequences on structural capacity estimates [30]. Uncertainties may derive from geometrical details but also from possible degradation of mechanical components, first of all sealants and gaskets exposed to long-term

climatic phenomena or unfavourable conditions [18,31–33]. In-service traditional system may suffer for poor load-bearing capacity against accidental loads, and even impact [34]. Practical difficulties may also arise when carrying out a forensic analysis of glass failure [35], or technical limitations could require the availability of robust and rapid in-field assessment and monitoring tools for fractured in-service glass pedestrian structures [36,37].

In terms of computational analysis and analytical methods that could be taken into account to precisely assess the seismic demands of structures [38], the nonlinear analysis is usually required to be used, and can be generally into two groups, such as Non-Linear Time History Analysis (NLTHA) and Non-Linear Static or Pushover Analysis (NLSA/POA). Robust feedback and design support can be offered by fragility curves of glass components and assemblies [39,40], based also on extensive Cloud Analysis [41]. In all these cases, engineering knowledge of structural details, materials and components is known to represent a critical step.

In case of existing/historic glass systems like envelopes, roofs, etc., even additional uncertainties may arise. First, it is important to note that, in most of cases, no technical drawings and details are available to describe geometrical features. Secondly, materials that are typically used in combination with glass (i.e., gaskets, spacers, etc.) may result

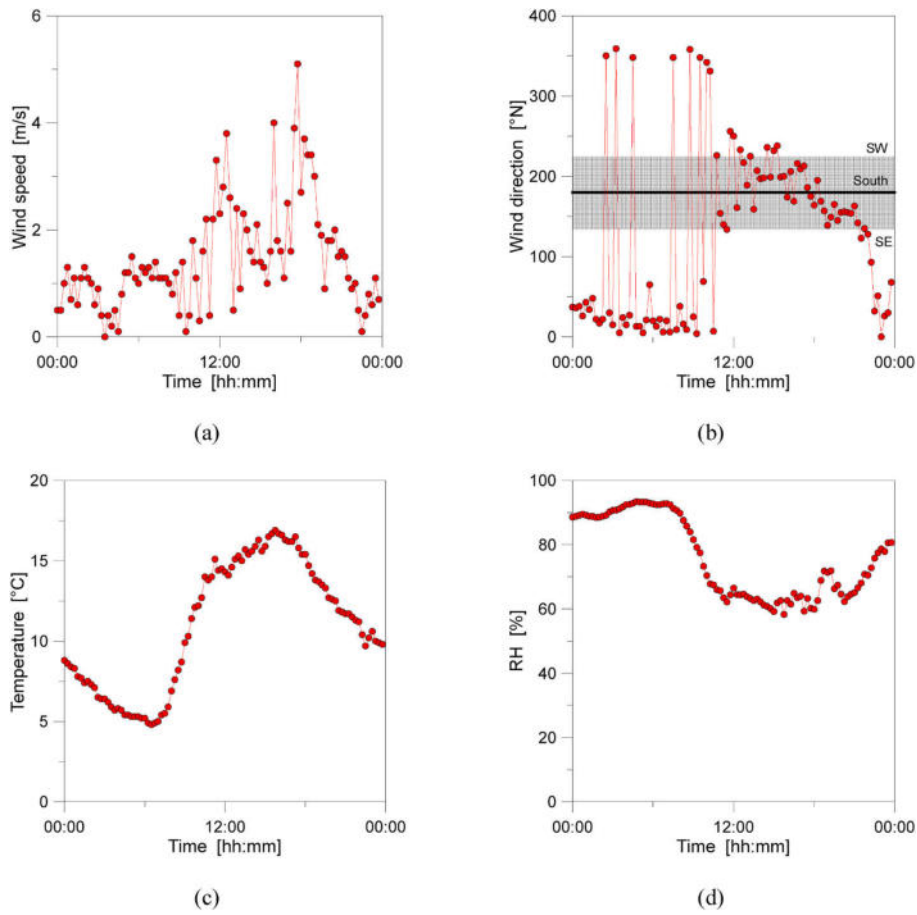


Fig. 8. Example of ambient parameters at the time of experiments. (a)–(b) Wind conditions in the day of experimental records: (a) maximum speed and (b) direction (measured at a distance of 10 m from ground); (c) temperature and (d) relative humidity. Records derived from <https://monitor.protezionecivile.fvg.it/>.

affected by long-term and operational conditions [18,36,42]. This means that their mechanical characteristics are unknown and also possibly degenerated compared to design stage. Finally, geometrical and mechanical details could have been conceived in a way that unfavourable mechanical effects are induced in glass panels under external loads. According to Fig. 3, it is thus important to have (Step 1) a clear and detailed picture of geometrical details, as well as possible defects or damage phenomena. Successively, (Step 2) mechanical properties at the time of survey should be also assessed, and this can be carried out by means of in-field OMA measurements and output interpretation. The number and setup of instruments should be planned to optimize the experimental stage and capture any kind of complex dynamic behaviour, especially for composite systems and/or unconventional geometries. Multiple sensors and optimized arrangements (i.e., [37]) could be also required especially under variable ambient conditions (i.e., where temperature and humidity modifications could have further relevant effects on material properties). In the interpretation of OMA output, (Step 3) a reliable and properly calibrated FE structural model will support the mechanical characterization of components, as well as (Step 4) the analysis of various loading conditions and design combinations of technical interest to assess/quantify the current capacity and plan the required retrofit interventions.

3. Analysis of a case-study system

3.1. Geometry and location

The structural system object of study consists of a glass-steel envelope built in 1962, as a composition of glass panes and a grid of steel

mullions and arched transoms (Fig. 4). The facade belongs to the so-called Information and Documentation Centre (CID) museum, which is located in Torviscosa (Udine, Italy) and has been designed by Cesare Pea in 60s, see Fig. 4(a) and (b), and [2]. The particular aspect of this system is presented by its shape (circle-based facade concept on the Southern side of building) and by its size (overall included in a rectangular shape of approximately 7 m in width and 5 m in height), compared to the thickness and load-bearing capacity of single components (Fig. 4(c)). Glass elements are in fact shaped to follow the overall arched path and are characterized by simple monolithic (and relatively small, but unknown) total thickness, compared to the covered surface. Moreover, steel members are reduced to a minimum, to preserve the transparency of the facade.

3.2. Experimental characterization

An in-field experimental investigation was carried out in Spring 2022 to achieve some important geometrical and mechanical characteristics of the envelope, and thus to obtain a realistic state-of-art description of the composite system, composed of (i) thin monolithic glass panels, (ii) steel transom and mullions and (iii) flexible/degenerated joints the interface.

3.2.1. Inspection

A preliminary visual inspection was performed to verify the conditions of facade elements, namely the glass panels, the steel members and the bonding layers between glass and steel components. The preliminary inspection was further optimized to measure the actual size of the bracing system.

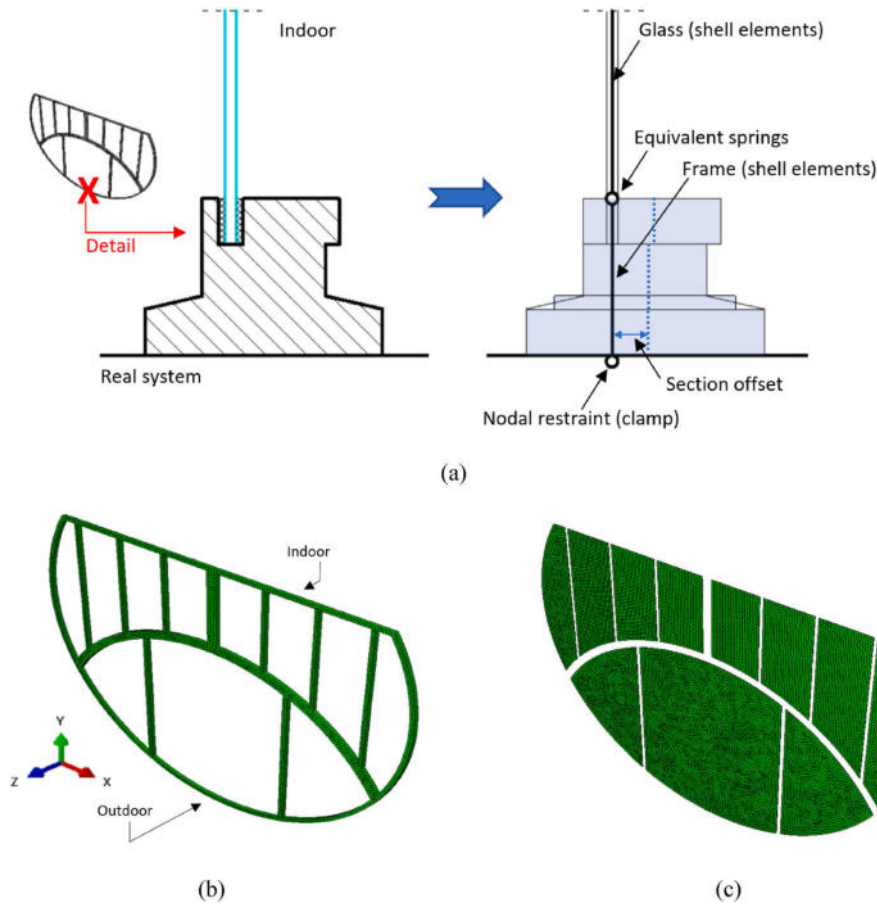


Fig. 9. Reference FE structural model (ABAQUS/Standard): (a) assembly detail (cross-section); frame (extruded axonometric view) and (c) glass panels.

From the visual analysis, see Fig. 5, it was noted for example a rather linearly clamped boundary condition for all the glass plates. The clamp condition is achieved by means of solid steel members that are screwed together. The central member of each mullion or arched transoms is rigidly connected to compose the frame grid. At the same time, two additional solid steel members are connected by screws to the central element, to create a gap for positioning and restraining glass edges. Finally, a silicon infill bonds the adjacent glass and steel elements.

The geometrical relief of facade components was summarized as in Fig. 5(c). The maximum size of central glass panel, more precisely, was measured in 3.3 m meters in height by 2.58 m in width. The total mass of steel and glass members (calculated only after OMA testing and dynamic identification of the structure) was measured in approximately $M_{TOT} = 2358$ kg, where the total sum of glass elements was calculated in $\approx 0.13 M_{TOT}$. A list of geometrical and inertial properties for all the glass components is also proposed in Table 1.

The experimental analysis and interpretation of OMA results and structural FE model updating in Section 3.2.3 represented a key step for the characterization of structural components. More precisely, dynamic identification techniques as in Section 3.2.3 allowed to confirm that monolithic glass panels are characterized by $t_{glass} = 5$ mm of total thickness, which is not in line with current technical requirements as in Section 2. Worth to note, in this regard, that the thickness of glass was not explicitly measured during the in-field inspection (due to continuous bonding along the edges of glass panels), but preliminary calculated by subtraction of thicknesses as in Fig. 5(a) and (b), and successively confirmed by FE numerical analyses.

Regarding the interposed linear connection, a total thickness of ≈ 3 mm was measured. Besides, several local manifestations of degradation were noticed in the sealant layer, in the form of fragments and

interruptions in the continuous bond (and thus interruption of linear clamped boundaries for glass panels). An example can be seen in Fig. 6 (a). Apart from that, no further degradation was noticed, and only some superficial deposits were observed for the metal components, as reported in Fig. 6(b).

3.2.2. In-field vibration experiments

The on-site experimental investigation was carried out in the form of OMA-based measurements.

Two different MEMS sensors were used simultaneously for the envelope system under random excitation. To minimize the influence of instruments and setup on the vibration performance of such slender glass components, mini devices were used (detail in Fig. 7(a)), based on IMU AHRS MPU6050 chip board, wireless, three-axes accelerometer sensor (± 16 g its range, 0.005 g the resolution, 0.2–200 Hz the available sampling rate). Each device was characterized by integrated battery and embedded microcontroller, ARM® Cortex®-M0 single core type (32 bit, clock frequency up to 48 MHz). The size of each sensor was measured in 36×36 mm, with 15 mm the thickness and 20 g the weight.

For the experimental measurements, one of these MEMS sensors was fixed to the mullion as in Fig. 7(b). At the same time, the second MEMS sensor was fixed to glass, like at the center of maximum size panel, or in other relevant positions. The sampling rate was set on 200 Hz and several records were collected under ambient excitations (i.e., trucks and cars). Further extensive records were collected for the system subjected to random soft-body impact. In this regard, due to major uncertainties on the actual load-bearing capacity of tested glass panels, random finger hits were used to impose low-amplitude vibrations and derive the dynamic parameters of the system, while preserving its integrity. During all these experiments, each sensor was oriented as in

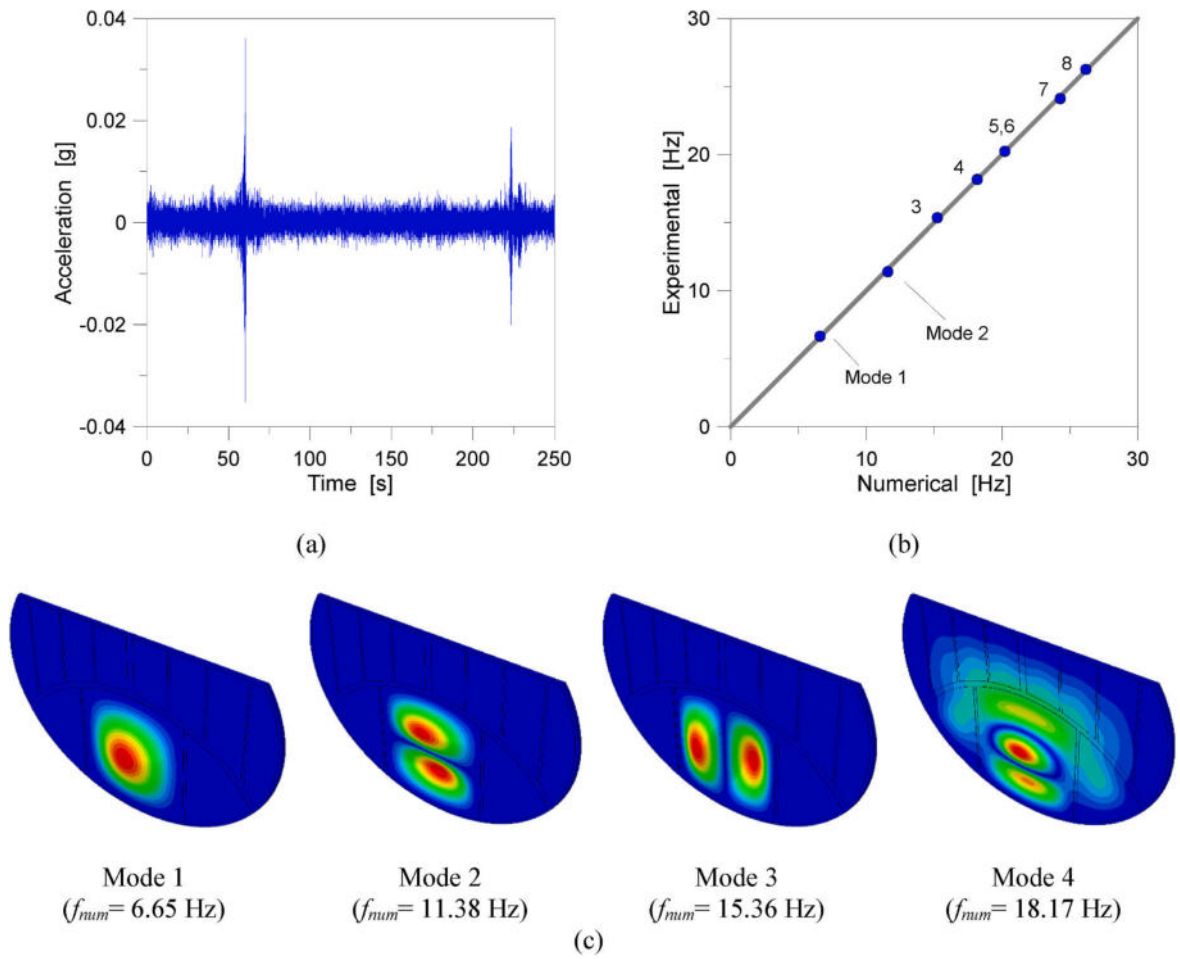


Fig. 10. Dynamic identification of the historic glass-steel envelope: (a) example of OMA acceleration records (ambient excitations); (b) optimized comparison of OMA experimental and FE numerical vibration frequencies (modes 1 to 8); (c) corresponding selection of FE numerical vibration shapes (ABAQUS/Standard).

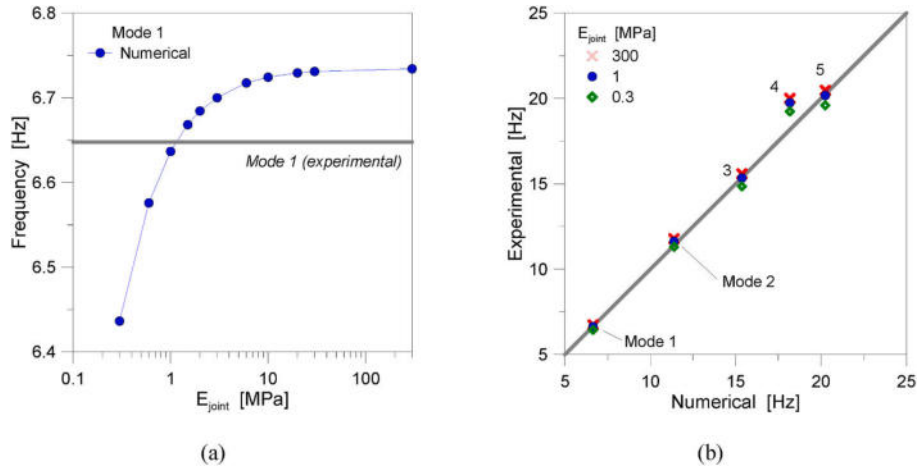


Fig. 11. Vibration frequency variation for the historic glass-steel envelope with parametric modification of stiffness for bonding joint (E_{joint}): (a) evidence of vibration frequency for mode 1 (with $0.3 \text{ MPa} \leq E_{joint} \leq 300 \text{ MPa}$) and (b) modes 1 to 5 (with $E_{joint} = 0.3, 1, 300 \text{ MPa}$).

Fig. 7(b), with the local x-axis coinciding with the global Z-axis of the building. Furthermore, the sensors were installed on the interior side of glass facade, to avoid disturbance of ambient conditions (especially direct wind exposure).

The experimental records were in the mid-morning (≈ 10 – 12 a.m.) of few working days (March and April 2022). For all these measurements,

the operational conditions were characterized by average temperature and relative humidity (in the order of $\approx 10^\circ \text{C}$ and $\approx 65\%$ respectively), as well as by a minimum influence of wind. This can be noticed also in the example of monitored ambient data of Fig. 8, where the maximum wind speed is reported for one of days of experimental analysis (in the order of ≈ 1 – 1.5 m/s for the period of analysis). Furthermore, data in Fig. 8(b)

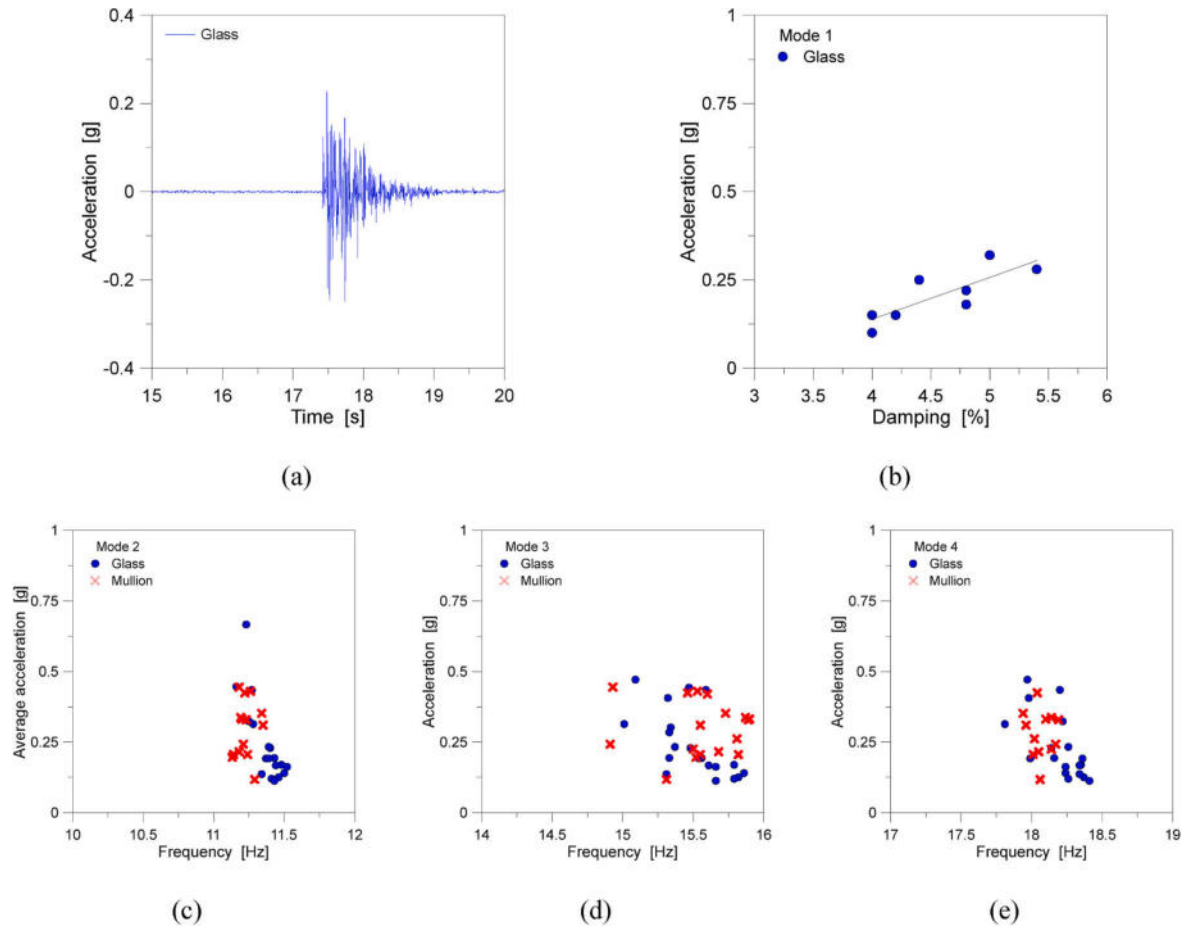


Fig. 12. OMA vibration test under soft-body impact (random finger hits): (a) example of out-of-plane acceleration records, with (b) calculated damping for mode 1 and (c)-(d)-(e) calculated vibration frequency for modes 2, 3 and 4 respectively.

show the wind direction (expressed in °N), which confirms the lack of Southern wind pressure and direct exposure of the facade system.

3.2.3. Dynamic identification and mechanical characterization of the historic glass-steel system

The interpretation of OMA output was maximized by the use of a structural FE model carried out in ABAQUS/Standard [43] to accurately represent the geometrical and mechanical properties of the whole envelope (metal frame, glass panels, joints), based on in-field survey and FE model updating for uncertain parameters.

For preliminary linear modal analysis and vibration mode extraction, more in detail, the reference FE model was assembled in the form of 4-node monolithic shell elements able to reproduce the geometrical features schematized in Fig. 5, see Fig. 9. Monolithic shell elements were used for glass panels (with $t_{glass} = 5$ mm their thickness), as well as for the frame members (equivalent rectangular sections according to Fig. 9 (a)). Nodal boundaries were finally assigned to the external edges of frame members, to reproduce a rigid mechanical fixing of the arched metal structure to the concrete background. The edge size of mesh elements was described in the form of free variable pattern, with 0.03 m the average, 0.01 m the minimum and 0.06 m the maximum. The overall geometry of steel frame and glass panels consisted in total of $\approx 21,000$ shell elements and $\approx 136,000$ DOFs.

In terms of characterization of linear joints at the interface of glass and steel components, special care was given to equivalent springs as in Fig. 9(b), which were calibrated based on the nominal joint geometry (from Fig. 5), the adopted mesh size/pattern and the assumption of a linear elastic material for bonding layers (with E_{joint} the tentative Young's modulus). Through the interpretation of OMA results, model

updating included variations in E_{joint} and the analysis of major effects on vibration frequencies and modal shapes for the examined historic glass-steel system (with $0.3 \text{ MPa} \leq E_{joint} \leq 300 \text{ MPa}$ for numerical simulations).

In doing so, regarding the characterization of materials, simplified constitutive laws were used. Input parameters for glass were set in $E_{glass} = 70 \text{ GPa}$, $\nu_{glass} = 0.23$ and $\rho_{glass} = 2490 \text{ kg/m}^3$. For steel in frame components, input features were specified in $E_{steel} = 210 \text{ GPa}$ for Young's modulus, $\nu_{steel} = 0.3$ for the Poisson's ratio, $\rho_{steel} = 7800 \text{ kg/m}^3$ for its density, with yielding and failure at $\sigma_{steel} = 235 \text{ MPa}$. For the purpose of present study and the simulation approach in use, damping was disregarded for all the envelope components [44,45].

OMA results according to Fig. 10(a) for the envelope under ambient excitation were first post-processed to extract vibration modes of the system. The comparison of OMA experimental and numerical estimates of vibration frequency can be seen in Fig. 10(b) for the optimized structural FE model. Post-processing analysis suggested the presence of rather weak bonding joints at the interface of glass edges and steel members, with $E_{joint} \approx 0.8\text{--}1 \text{ MPa}$ which is inclusive of material degradation effects for the historic envelope, but also of ambient conditions (temperature, humidity) at the time of in-field OMA experiments (i.e., Fig. 8). In this regard, further testing configurations could be performed to assess more in detail the effects of climatic conditions.

In terms of detected vibration frequencies for modes 1 to 8, comparisons in Fig. 10(b) confirm the accuracy of reference FE model, with rather good correlation with most of experimental frequencies. The fundamental vibration mode was experimentally detected in $f_{exp} = 6.58 \text{ Hz}$ ($f_{num} = 6.65 \text{ Hz}$ from numerical analysis). The average scatter of FE numerical predictions was calculated in $\approx 0.67\%$ for modes 1 to 8

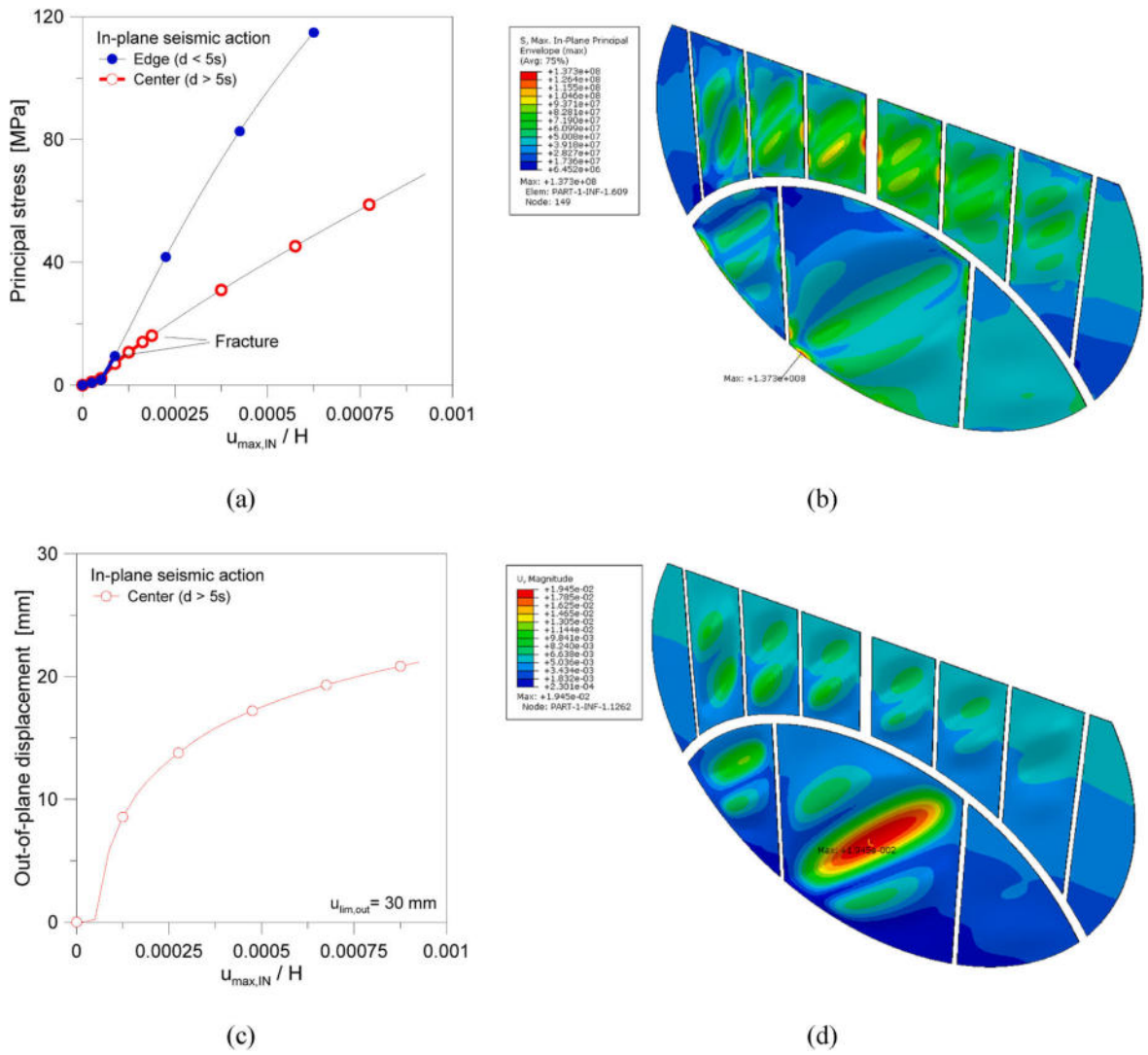


Fig. 13. Local seismic analysis under in-plane action (ABAQUS/Standard): (a) stress evolution at the center or at the edges of glass, with (b) corresponding contour plot (selection, legend values in Pa); (c) out-of-plane deflection at the center of glass, with (d) contour plot (selection, legend values in m). Frame components hidden from FE model view.

(+0.92% the scatter for mode 1).

In terms of mechanical analysis of the composite system, Fig. 10(b) shows that the low vibration modes are well separated, thus prevailing structural behaviours under design loads can be predicted without uncertainties.

Fig. 10(c), in this regard, shows selected vibration shapes for the reference FE structural model, where it is possible to note that the dynamic response of the historic glass-steel system is governed by bending of central glass panel, and rather null deformations for frame elements and other glass components. Higher vibration modes only are able to involve also the frame members and other glass components in more complex shapes, like mode 4 in Fig. 10(c). Such a finding could suggest the use of simplified structural FE models for the stress and deflection analysis under relevant design actions, like for example a computationally efficient structural FE model representative of the central glass panel only, with ideal mechanical restraints. Besides, see also Section 4.3, geometrical and mechanical details and their effects on individual structural components (and mechanical interactions in composite systems) should be in general carefully addressed (see also Section 4).

In terms of actual stiffness for bonding joints ($0.3 \text{ MPa} \leq E_{joint} \leq 300 \text{ MPa}$) along the edges of glass panels, for example, the parametric modal analysis of the structural FE model in Fig. 9 provided relevant feedback

for mechanical analysis. Selected parametric results are shown in Fig. 11, in the form of vibration frequency sensitivity as a function of E_{joint} , with evidence of joint effects on the fundamental vibration frequency (Fig. 11(a)) and on low vibration frequencies (Fig. 11(b)) respectively. While modal shapes resulted negligibly affected by stiffness modification in joints, and mostly agreeing with Fig. 10(c), some major sensitivity was noticed both for low and high vibration frequencies of the composite system.

Experimental soft-body impact configurations based on random finger hits were also taken into account during the post-processing stage of in-field records. As in case of ambient excitations, major support was taken from two MEMS sensors in different locations to capture complex vibration modes (Fig. 7). Typical experimental results and outcomes can be seen in Fig. 12. More in detail, Fig. 12(b) shows the average damping of the system as a function of the imposed acceleration peak (out-of-plane acceleration for the glass-steel envelope), while Fig. 12(c)–(e) present the trend of derived vibration frequency as a function of imposed acceleration peak (out-of-plane) for modes 2, 3 and 4 respectively. In evidence, there are experimental outcomes from sensors placed on glass or on steel members.

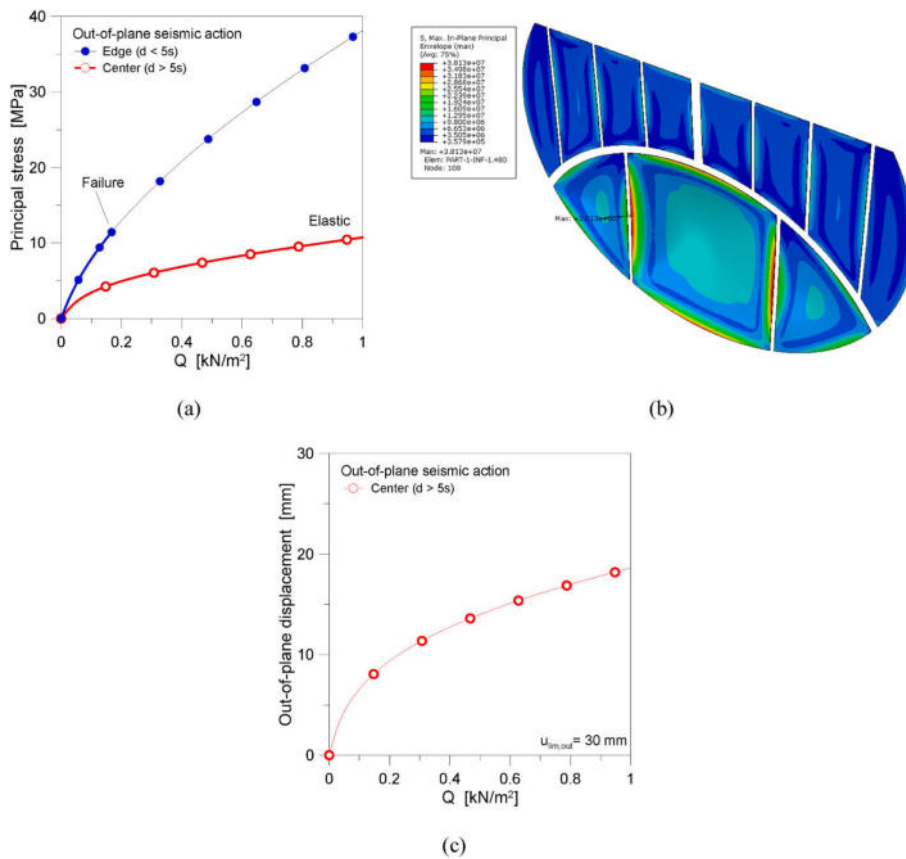


Fig. 14. Local seismic analysis under out-of-plane action (ABAQUS/Standard): (a) stress evolution at the center or at the edges of glass, with (b) corresponding contour plot (selection, legend values in Pa); and (c) out-of-plane deflection at the center of glass. Frame components hidden from FE model view.

4. Local seismic capacity analysis and discussion of results

4.1. Seismic performance limits

In terms of seismic analysis, the primary construction (CID museum, reinforced concrete frame structure) falls in a Class of use III (i.e., relevant buildings with possible overcrowding), with $V_N \geq 50$ years (Section 2). Regarding glass components, the reference value for the resistance check to take into account for the present case-study system should be assumed equal to ≈ 14 MPa for verification at the center of panels ($d > 5s$, with $s = 5$ mm) and ≈ 10 MPa for stress peaks at the edges of glass ($d < 5s$).

Further, the limit out-of-plane deflection for SLD (as required for monolithic glass with linear supports at the edges) should be fixed in $u_{lim,out} = 30$ mm (given that $L_{min}/60 \approx 40$ mm, with $L_{min} = 2.58$ m the minimum size of central glass panel). Regarding the in-plane displacement capacity (inter-story drift accommodation), finally, this was set in $u_{lim,in} = 0.005H = 45$ mm (with $H = 9$ m the total height of storey/building).

4.2. Numerical analysis of the composite glass-steel system

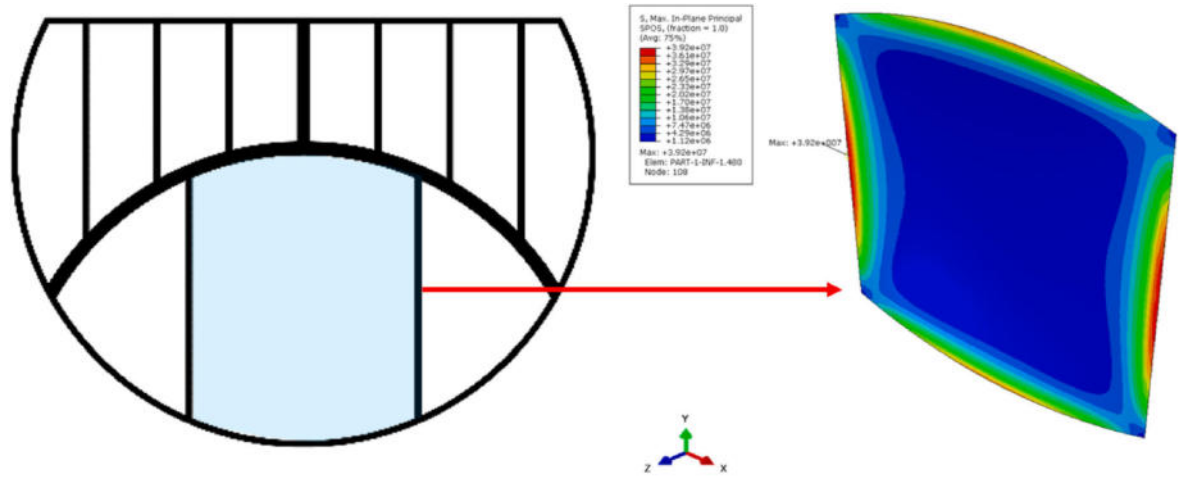
The seismic performance and capacity assessment of the examined glass-steel envelope was numerically investigated by means of monotonic, linear application of an in-plane lateral force at the top of the envelope frame (in-plane seismic action), as well as of an equivalent uniformly distributed pressure (out-of-plane seismic action). This is in line with [5], where local verification is allowed for secondary glass elements belonging to primary buildings. The structural FE model from Section 3 was adapted with modification in step features (NLSA/POA type) and corresponding boundaries. Gravity loads of facade

components were included in the analysis. Overall, the attention was focused on the analysis of stress evolution and peaks in glass components, as well as on the deformation trends under seismic action.

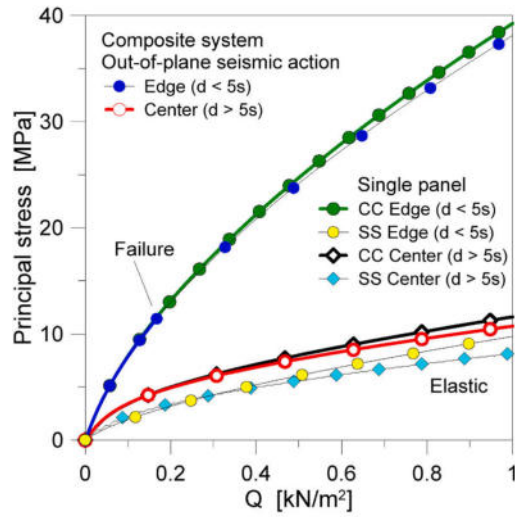
In Figs. 13 and 14, major outcomes of its structural response under in-plane or out-of-plane seismic action are collected, with selected evidences of stress or deflection distributions.

Fig. 13(a) and (b), in particular, represent the principal stress in glass and the out-of-plane glass deflection under the imposed in-plane seismic loads. These are monotonically increased up to the achievement of inter-story drift in the order of $0.001H$. The corresponding out-of-plane deformations are reported in Fig. 13(c) and (d). More in detail, it can be seen the critical role of restraints and stress peaks at glass edges. Black plots are used to denote the trend of stress peaks once the reference design glass strength is exceeded for the examined system. The first exceedance of limit design strength in glass can be associated to cracking/fracture of glass components, and thus potential risk for occupants. According to previously defined performance indicators, as well as numerical output in Fig. 13, it can be seen that the present envelope would be able to accommodate a maximum in-plane displacement up to $\approx 0.0001H$ only (due to premature glass cracking), which is rather small compared to current requirements for the building context it belongs. Conversely, no failure risk can be perceived in terms of out-of-plane deformation (Fig. 13(c) and (d)).

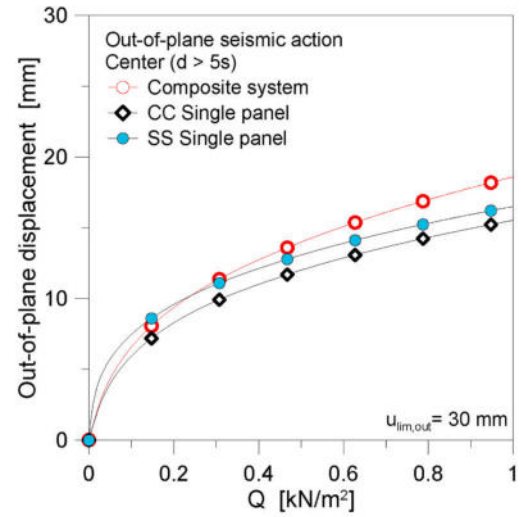
As far as the local seismic analysis is carried out in terms of out-of-plane equivalent pressure, see Fig. 14, maximum stress peaks are still achieved at the edges of central glass panel (Fig. 14(a)). The equivalent seismic pressure the system could resist is in this case in the order of $Q \sim 0.2$ kN/m², due to premature glass fracture at the edges. Worth to note in Fig. 14(a) and (b) is the limited amplitude of stress peaks at the center of glass, thus enforcing the critical role of restraints and boundaries. Also, Fig. 14(c) shows that the out-of-plane displacement of glass panel for a



(a)



(b)



(c)

Fig. 15. Local seismic analysis of single glass panel with ideal restraints under out-of-plane action (ABAQUS/Standard): (a) setup; (b) stress evolution at the center or at the edges of glass, with (c) out-of-plane deflection at the center of glass.

Table 2

Summary of stress and deflection verifications for the central glass panel under out-of-plane seismic action (ABAQUS/Standard).

FE model	Stress				Deflection	
	$d < 5s$		$d > 5s$		Failure	$u_{max}/u_{lim,out}$
	Failure	$\sigma_{Ed,max}/f_{gd}$	Failure	$\sigma_{Ed,max}/f_{gd}$		
Composite glass-steel envelope	Yes	1 for $Q = 0.148$ kN/m ²	No	0.755 for $Q = 1$ kN/m ²	No	0.619 for $Q = 1$ kN/m ²
Single glass panel (SS restraint)	No	0.965 for $Q = 1$ kN/m ²	No	0.586 for $Q = 1$ kN/m ²	No	0.550 for $Q = 1$ kN/m ²
Single glass panel (CC restraint)	Yes	1 for $Q = 0.130$ kN/m ²	No	0.837 for $Q = 1$ kN/m ²	No	0.510 for $Q = 1$ kN/m ²

seismic pressure up to 1 kN/m² was measured in around a maximum of ≈ 20 mm, which is still far away from the reference limit value (30 mm in the present study).

4.3. Numerical analysis of the central glass panel with ideal restraints

Some considerations can be drawn from the seismic performance assessment of the investigated glass-steel system, and taken into account for future extended studies and investigations. On one side, there are standards and technical guidelines for buildings and structures under seismic action which provide precise calculation methods for capacity and demand considerations, both in case of primary and secondary structural components. Besides, the attention for glass structures and components under seismic events is still limited to specific documents and recommendations (see for example [5]). In any case, existing recommendations are conceived for newly designed glass systems (and accessories), while there are no recommendations for capacity and performance assessment of existing/historic assemblies.

When local seismic analyses are used to support the seismic performance assessment of these elements/components, a major issue can be

represented by appropriate engineering knowledge of structural details, material properties, actual state of boundaries, etc. In this sense, the use of simplified mathematical models could result in computationally efficient predictions but also unreliable and uncertain estimates of structural phenomena.

For example, Fig. 15 presents additional comparative results for the examined historic system under out-of-plane seismic action. More in detail, the seismic response of the composite glass-steel envelope (based on the reference FE numerical model from Fig. 9) is compared with the expected stress and deflection performances derived from local numerical analysis of glass only. As far as the dynamic characterization in Fig. 10(c) confirmed that the fundamental vibration shape is governed by out-of-plane bending of the central glass panel, being representative of the most flexible component of the whole facade, a simplified FE numerical model was derived as in Fig. 15(a). In doing so, steel frame members were disregarded. Similarly, ideal restraint conditions of linear simple supports (SS in Fig. 15) or linear clamps (CC in Fig. 15) were considered along the edges of glass panel, while the local out-of-plane seismic action was monotonically increased on the surface of glass, in the form of uniform pressure Q . As expected, compared to Fig. 14, it is worth to note that both stress distributions and deflection trends are strongly affected by edge restraints for the selected central glass panel, thus enforcing the need of a more complex and detailed analysis of the historic composite system as a whole. More in detail, Fig. 15(b) shows the evolution of tensile stresses in glass as a function of the imposed uniform pressure. Stress estimates are compared as obtained from the composite system (Fig. 14) or from the simplified FE model in Fig. 15(a), both at the center ($d > 5s$) and at the edges of glass ($d < 5s$). Disregarding the effect of steel frame and interposed joints shows that the SS boundary condition for the single glass panel is associated to relatively low stress peaks at the edges and in the center of glass, thus resulting in a fully elastic response of glass (until $Q = 1 \text{ kN/m}^2$) and rather unsafe predictions for seismic mitigation purposes, for the presently investigated envelope. Besides, the presence of an ideal rotational restrain as for the CC single panel would reflect in premature failure at the edges of glass ($Q = 0.130 \text{ kN/m}^2$), which is more conservative than the composite FE model, as also summarized in Table 2 to enforce the key role of details.

5. Conclusions

The use of glass in building components is nowadays supported by standards and technical guidelines for structural design. Besides, safety issues still arise from existing/historical glass systems in old buildings. Often, these are not conceived to sustain mechanical loads, and represent potential risk for customers.

In this paper, open issues on methodological steps and performance indicators for existing glass systems under seismic events have been discussed. Considerations have been extrapolated for a case-study system, where the major issue was represented by the use of relatively thin glass components (monolithic elements with 5 mm in thickness) which were not specifically conceived to provide any kind of load-bearing capacity or seismic resistance and displacement accommodation. In this sense, a strong limitation affecting the possible performance of glass components was represented by steel-to-glass connection detail, which resulted in a rather stiff connection not able to facilitate any kind of local displacement, or even to prevent localized stress peaks in glass.

As shown, the study overall confirmed that existing/historic glass envelopes are highly vulnerable to seismic actions and – even in the form of secondary building components – should be preserved against large displacement or stress demands from the primary buildings in case of earthquakes. In such a kind of consideration, the use of in-field OMA techniques to integrate the lack of technical specifications and drawings for the selected glass envelope (designed in 1961 and built up in 1962) provided meaningful effort in the structural analysis of the state-of-art conditions and actual mechanical capacity of the facade. On one side,

the investigation proved that existing glass envelopes and components are critical elements for safety of building customers, since not able to respond to actual design performance demands. At the same time, the investigation suggested that there is urgent need of harmonized methodological procedures and standardized evaluation framework steps that could be used for the vulnerability and risk analysis of historic heritage, as well as for (when possible) retrofit proposals for existing glass components. As far as a general procedure inclusive of practical rules and recommendations in support of the vulnerability analysis of existing/historic glass envelopes is currently missing (for in-field non-destructive assessment and numerical analysis), future studies will be spent for the proposition of damage indexes or methodologies in support of designers.

CRedit authorship contribution statement

Chiara Bedon: Conceptualization, Methodology, Investigation, Software, Data curation, Writing – original draft, preparation, Writing – review & editing. **Maria Vittoria Santi:** Investigation, Visualization, Writing – original draft. **Marco Fasan:** Investigation, Visualization, Writing – original draft.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Data availability

Data will be made available on request.

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