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Numerical assessment of vibration control systems for multi-hazard design and mitigation of glass curtain walls

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ABSTRACT

Glass systems and facades are widely used in building structures, due to a multitude of aspects. Beside these motivations, from a pure structural point of view, glazing envelopes represent one of the most critical components for multi-storey buildings under the action of exceptional loads as impacts, explosions, seismic events or hazards in general. Such systems represent in fact the first line of defense from outside. Given the current lack of specific design regulations for the mitigation and enhancement of glass curtains under extreme loads, as well as the typically brittle behaviour and limited tensile resistance of glass as material for constructions, the same facades require specific, *fail-safe* design concepts.

In this paper, the feasibility and potential of special mechanical connectors interposed at the interface between a multi-storey primary building structure and the enclosing glazing facade are investigated via accurate Finite-Element (FE) numerical models, under various impact scenarios. At the current stage of research, careful consideration is given both to the observed global performances as well as to local mechanisms, based on computationally efficient FE models inclusive of damage models to account for failure mechanisms in each system component. Compared to earlier research efforts, the attention is focused on the multi-hazard performance of a given case study building, subjected to extreme loadings such as seismic loads or blast events. As shown, even the typically different features of the examined loading conditions, when the proposed vibration control devices are properly designed and the curtain wall is considered as part of a full 3D building, the final result is an overall assembled structural system in which the glazing facade can work as a passive control system for the building system, in the form of a distributed Tuned-Mass Damper (TMD), with marked benefits in terms of protection level as well as design optimization.

1. Introduction

Glazing facades are widely used in building structures, due to a series of aesthetic, thermal, lightening aspects. In most of the cases, wide transparent surfaces are created in commercial, residential as well as strategic buildings, including airports, museums, offices, etc.

From a structural point of view, however, under the action of exceptional loads as impacts or hazards in general, glazing envelopes represent a critical component for multi-storey buildings, due to the typically brittle behaviour and limited tensile resistance of glass panes, as well as to connection detailing etc., hence requiring specific, fail-safe design concepts [1,2]. In this regard, the appropriate estimation of the vulnerability of glazing systems under extreme loads, as well as the prediction of their actual dynamic behaviour under exceptional loads (including the interaction between a given envelope and the substructure/primary building), or the implementation and development of advanced retrofitting and enhancing techniques, consequently, are currently open topics still attracting the attention of several studies. Analytical, experimental and/or Finite Element (FE) numerical investigations can be found in the literature for glazing envelopes under seismic events (i.e. [3,4]), blast, explosions and accidental impacts (i.e. [5–10]), fire (i.e. [11,12]), hurricanes and climatic loads (i.e. [13–16]). Beside these efforts, the same issues still require further extended studies.

In this paper, taking advantage of major outcomes of an ongoing research investigation, careful consideration is paid for the multi-hazard performance of glass curtain walls, as well as to maximum effects mitigation due to seismic loads and blast events, being representative of emergency situations for protection of people. In doing so, the effects of special mechanical connectors interposed at the interface between a given multi-storey primary building structure and the enclosing glazing curtain wall are preliminary investigated via efficient FE numerical models, under various extreme loads scenarios. Such a special connectors are intended to act at the curtain-to-building interface, given a

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reference curtain wall and building. The major outcomes of the current paper, in addition, are representative of preliminary studies aimed to assess the feasibility and potential of the explored design concept. In this sense, extended experimental studies (both at the small-scale/connector level as well as at the full-scale/prototype level) would later on represent a key aspect for the validation and full optimization process.

Differing from existing studies related to retrofit and enhancement of glazing facades under extreme loads, being mainly related to single-hazard analyses only, as well as to single facades components only (i.e. [3,17]), as a main goal of the current project, the benefits or critical issues of such devices are hence preliminary assessed as potential tools for multi-hazard mitigation of traditional curtain walls, by taking into account both local and global structural aspects. As shown, when properly designed, the proposed connectors can in fact markedly improve the overall dynamic performance of a given glazing system. At the same time, part of these benefits are implicitly transferred also to the main building the facade belongs, both in terms of global building dynamic response and local performance of the curtain wall components. The final result, consequently, consists in a fully assembled structural system in which the glazing facade can work as a passive control system for the primary structure, in the form of distributed Tuned-Mass Dampers (TMDs).

2. Glass facades under exceptional loads: summary of current design philosophy and regulations

Design and mitigation of buildings under multi-hazards, including natural events (i.e. earthquakes, windstorms and hurricanes, floods) as well as accidental or human-induced exceptional events which may occur during their lifespan, represents the optimal goal of design, as well as the result of multidisciplinary issues and competences [1]. Such a design must in fact accommodate pure structural requirements aimed to enhance the response of a given building under an assigned loading configuration, but also thermal, economic, social, technological decisions. In doing so, it is clear that new techniques and methodologies aimed to assess the vulnerability of structures, control their dynamic performance or reduce their demand - together with reliability evaluations and risk analysis/management - have a crucial role, see [18].

In the specific case of structural glass facades subjected to exceptional loads, being representative of a part of often complex mechanical systems and buildings, but also of the first line of defense from outside, their optimal structural design is strictly dependent on the actual performance (i.e. stiffness, resistance, redundancy, etc.) of single structural materials and components, as well as their reciprocal interaction under the assigned combination of loads. As such, careful consideration should be paid not only for the glass panels composing the enclosure, but also for anchoring systems, supports, framing members, etc (see for example [19,20]).

In the case of unitized glass curtain walls (UGCW, in the following), pre-assembled modular units, typically consisting of insulating glass elements, are sealed to aluminum or steel framing members and fixed to the main building via rigid brackets, see Fig. 1. In terms of structural performance, these systems are traditionally designed to resist ordinary loads only, i.e. vertical loads due to self-weight and standard wind pressures acting in the direction orthogonal to glass panes surface, while enhanced-resistant UGCWs are properly designed, when required, for special buildings only. In both the cases, such a structural requirements must accomplish with other design issues, most of them related to the thermal performance of curtains, including also air infiltration, water penetration, condensation, glass surface distortion, etc (i.e. [21–23]). Thermal and structural aspects, in most of the cases, are strictly related (i.e. [24–26]) and should be jointly optimized at the preliminary design stage.

Generally speaking, the structural design of glass panels and framing components is then conventionally carried out by taking into account single facade components only, i.e. by assuming ideal restraints at the glazing module restraints as well as equivalent simplified formulations for the description of design loads and for local verifications, rather than exploring and optimizing the structural performance of curtain walls and related buildings in the form of 3D full assemblies.

In general terms, due to the relatively weaker tensile strength and brittle behaviour of glass as material for constructions [1,2], as compared to concrete, steel or timber elements of traditional use, glass windows and facades are typically fragile, and therefore highly vulnerable to extreme loads, shocks and impacts in general. Glass fragments represent in fact a critical issue for people, hence cracking of panes should be generally prevented (i.e. Fig. 2). As a result, specific *fail safe* design rules (still required for glass systems in general under ordinary loads, see [2] are needed especially when exceptional loading configurations are expected to occur. Appropriate design methods as well as mitigation tools should be in fact considered, aiming to enhance the security level, hence minimizing possible injures and optimizing the structural performance/cost of the system itself. In the specific case of structural systems composed of glass, major uncertainties are also represented by high scatter in the material tensile resistance, being this value highly susceptible to geometrical features, thermal and edge treatments, loading conditions, presence and position of holes, etc. (i.e. [27–31]). From a practical point of view, these aspects are conventionally accomplished by assuming a linear elastic behaviour for glass, and properly limiting maximums stresses and deflections under the assigned combination of loads [1,2].

As also in accordance with available design standards and regulations for buildings under exceptional loads such as seismic loads or blast events, a key role in design assumptions and performance limitations is given by the role assigned to glazing systems acting as a part of a whole building. As far as the given glass system to verify can be considered as a *secondary* component, compared to the primary structure, partial damage is in fact generally accepted by currently available design regulations. This is not the case of structural glass assemblies of *primary* importance within a given structural system, where the glass elements or facade components should in fact able to properly resist to the incoming impulse, as well as to accommodate the overall deformations of the building as a full three-dimensional assembly, including both out-of-plane and in-plane displacements. In the latter case, it is hence clear that special joints, mechanical connectors and fasteners are mandatory, together with connections detailing, in order to satisfy design standard limitations and avoid severe damage. In doing so, however, no specific rules are available for glass curtain walls designers.

Regarding the seismic design and verification, for example, general European standards for buildings can be applied also to glass curtain walls, without specifications (see for example [32]). In that document, secondary components only are in fact considered, and no specific regulations are available to account for the importance or typology the curtain wall belongs, as well as for detailing, anchoring systems, materials, etc. As a general rule, the building as a whole is only required to do not exceed specific inter-storey drift values. The mentioned EU regulations are in line with other standards for seismic design of buildings, see for example the New Zealand NZS 1170.5 [33] document. More detailed provisions are included in US FEMA 450 [34], even for so called "*secondary non-structural cladding systems*" only. Compared to the European or Australian scenarios, specific drift limit values are required for glazed curtain walls, storefronts and partitions, and hence should be satisfied to avoid glass fallout. Drift limitations are also given, as a reference design criteria, by the Japan Standards [35].

Actually, given a traditionally framed glass unit like Fig. 1, for example, no specific considerations are given by most of existing standards to its real performance under seismic loads. Research efforts and case studies observations highlighted, over the last decades, that sealant joints proving glass-to-frame bonding could have a key role in preventing glass failure. It was also observed, however, that most of the gaskets in use for such facades are not able to accommodate the

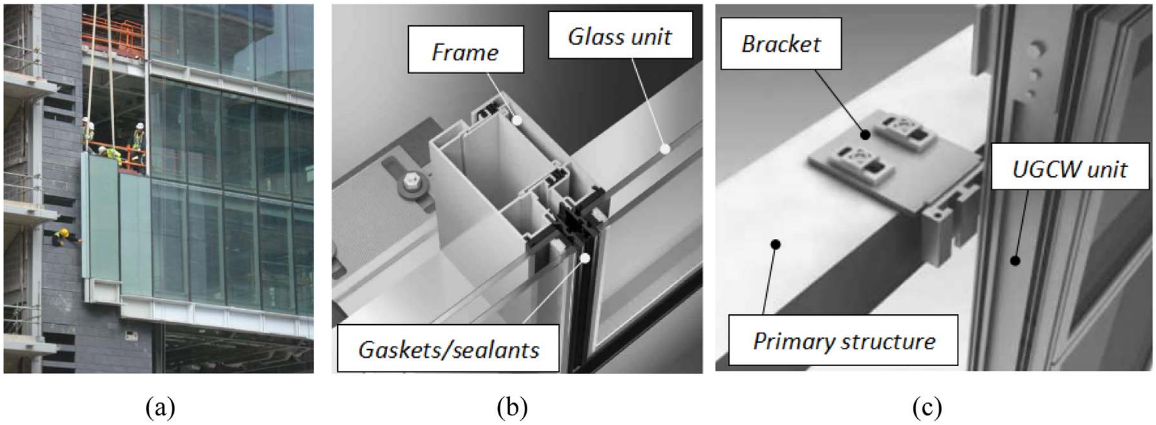

Fig. 1. Examples of UGCWs: (a) installation process, with typical detail of (b) glass-to-frame connection and (c) supporting rigid bracket.

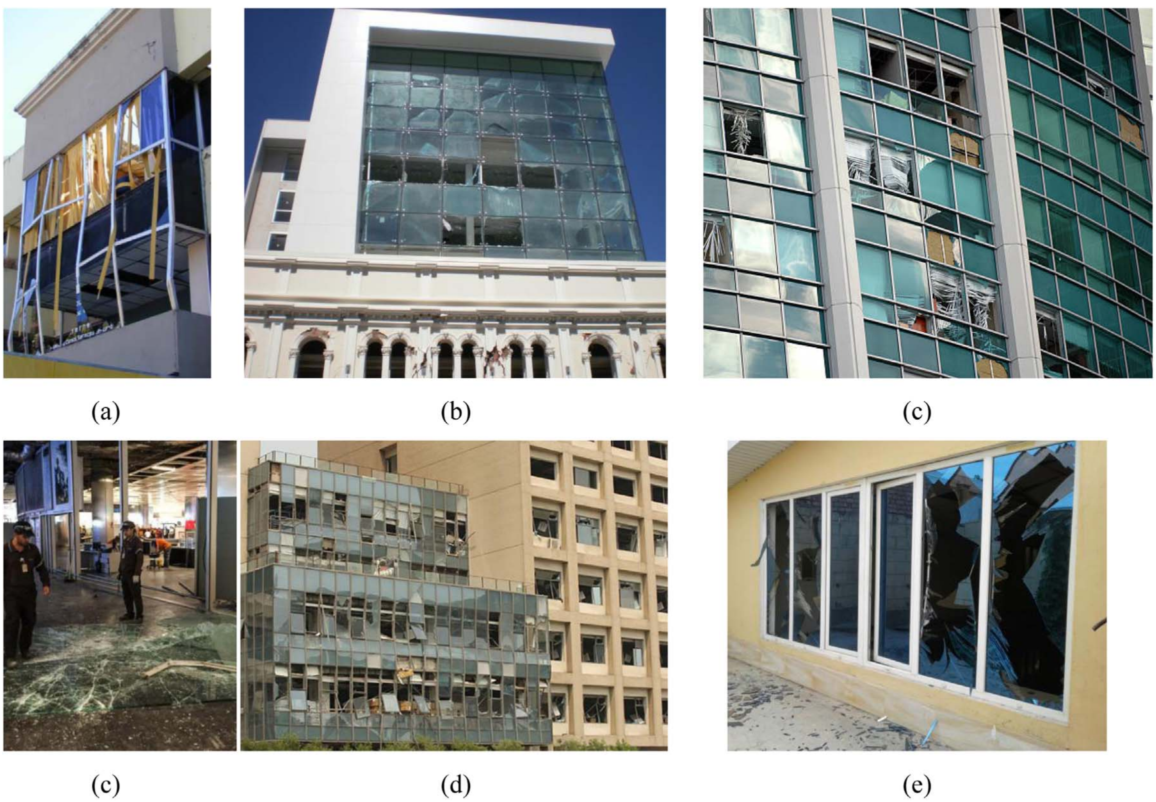

Fig. 2. Examples of glass systems under hazards: (a)–(b) seismic events, (c) hurricanes and (d)–(e) explosions.

required relative deformations, see [36]. The total deformation of a framed glazing unit under in-plane lateral loads (see Fig. 3(a)), moreover, was proved to be typically associated to two combined effects, as schematized in Fig. 3(b) and (c). A prototype of 'Earthquake-Isolated Curtain Wall System' was hence proposed in [37], being obtained by introducing special seismic decoupler joints to act as local seismic isolators and reduce possible damage due by seismic-induced inter-storey drifts. Design challenges further increase as far as glazing facades are frameless or point-fixed, etc.

Totally different performance levels and requirements are indeed considered in terms of design of blast resistant windows and facades, compared to seismic events. EU as well as US standards are available for testing laminated glass windows (see for example [38–42]), and hence to define the corresponding hazard level. Most of the available design codes assume similar glass fragment assessment criteria, that is classify glass fragments threat based on their projection distances into a given occupied area (Fig. 4). As a general rule for these codes, glass windows that do not break or break but retain fragments within frame members are rated as "no threat". When glass fragments are supposed to fail within 1 m distance from the openings, "very low" threat is considered. If the fragments are expected to fly longer, the hazard level is rated as "high". Beside these similarities in the vulnerability evaluation of blast-loaded glass windows, nevertheless, some key aspects such as glass fragments velocity, size, shape, etc., are not considered in defining the threat level for the mentioned standards. Most of the existing regulations and testing recommendations, moreover, are intended for glass windows with specific features and dimensions only (see for example [43,44]).

In this paper, the dynamic, multi-hazard performance of traditional UGCWs is assessed and enhanced by means of special mechanical connectors, proposed to replace rigid brackets in use (i.e. Fig. 1) and enabling the full UGCW to act as a passive control system for the building it belongs, in the form of a distributed TMD.

In doing so, following design standards regulations, a key role is

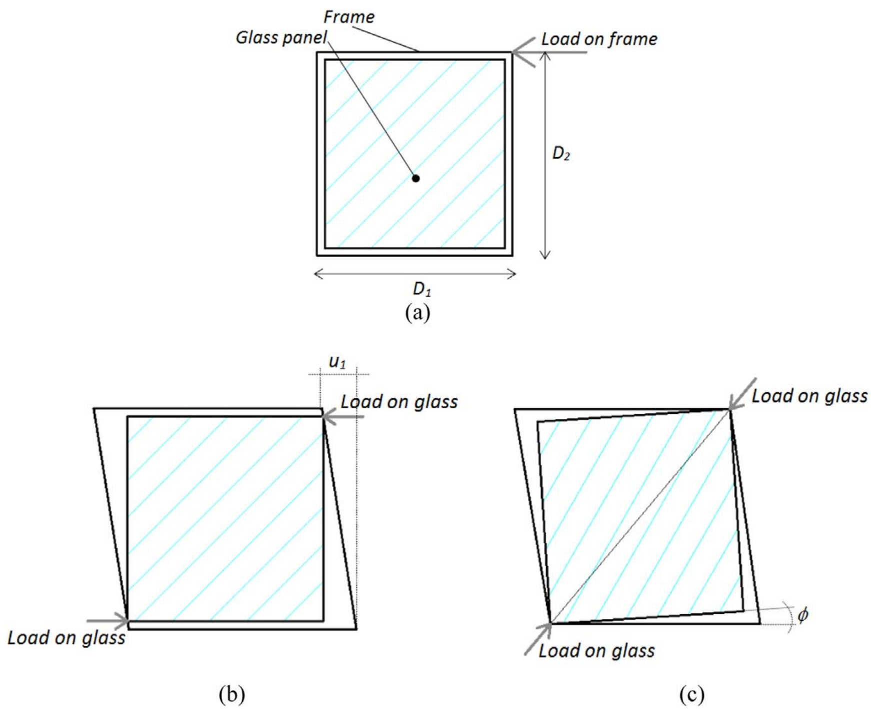

Fig. 3. Typical behaviour of glass framed units under in-plane lateral loads, in accordance with [36]. (a) Undeformed panel; (b) horizontal translation of the glass panel within the frame; (c) rotation of the glass panel within the frame, with evidence of reaction forces arising in glass.

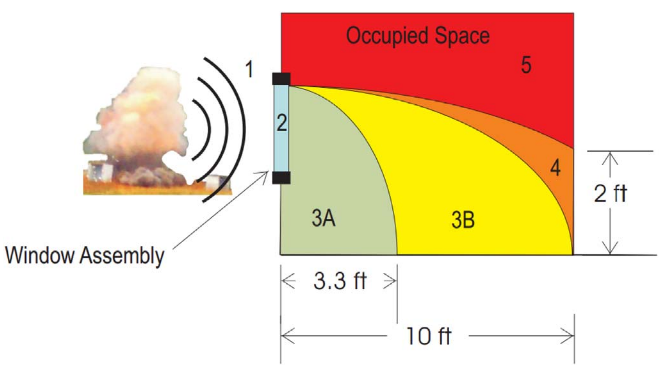

Fig. 4. Glass fragment threat approach, in accordance with [39].

assigned to the mechanical properties of the proposed devices, being responsible of maximum deformations and stresses achieved in the UGCW components, as well as in the building itself, under the action of extreme loads. The performance of such systems is separately assessed towards two different loading scenarios, including seismic events or explosions respectively. The distributed TMD concept, in particular, is first optimized towards high level seismic events, by taking into account a reference case study building located in a earthquake prone region of Italy. Major benefits, as shown, are given by minimization of stress peaks in the UGCW components, as well as by a marked decrease of seismic-induced maximum deformations on the full 3D assembly. Totally different intrinsic features characterize the typical blast loading configuration, compared to seismic loads, and hence the expected dynamic performance for the examined structural system. In any case, as shown, an optimal dynamic behaviour can be achieved also for the same TMD-equipped building under explosive events.

3. Unitized glass curtain walls acting as distributed TMDs

The design concept presented and preliminary assessed in this paper follows earlier research efforts aimed to mitigate glazing systems under single extreme loads, as well as is in line with mitigation techniques already in use for buildings and infrastructural systems. The novel aspect lies in the involvement of UGCWs as key components for distributed passive control systems, as well as in the multi-hazard optimization and analysis of related benefits and effects.

3.1. State-of-the-art on passive control systems

The general concept of glazing facades belonging as primary and fully structural components in buildings is relatively recent. As such, only recent studies have been focused on the enhancement of their structural performance or on the implementation and proposal of special fasteners.

Despite a non-effective and expensive over-dimensioning of glazing components to make them hazard-resistant, a valid technological and rather innovative solution can be represented by special connectors able to reduce the effects of the incoming design loads.

Passive control and vibration monitoring of structural systems under exceptional or high-rise design loads actually represents, both for buildings and infrastructures, a key topic for researchers and designers (i.e. [45–49]). Within the possible passive technological solutions currently available or under investigation for the mitigation of multi-storey buildings, tuned-mass-dampers (TMD) are widely used in structural engineering to reduce translational displacements and accelerations due to wind and seismic loads in bridges [50–53] and buildings or assemblies [54–58]. Den Hartog [59] first derived analytical expressions to determine the optimal values of mass, frequency and damping ratios of the TMD as a function of the dynamic properties of the structure. Several studies focused on the optimal design of such devices can be found in literature [60–63].

The use of vibration control systems, with specific application for glazing facades, aiming to enhance the tall buildings performance under wind and seismic loads, has been first theoretically explored in [64], with careful attention for double skin (DS) facades. There, DS facades with special connectors proved to reduce the seismic effects on tall buildings (up to − 35%). Further applications of special connectors have been proposed for cable supported facades under explosions [65–67], while special viscoelastic (VE) or ADAS brackets for GCWs have been numerically investigated in [17], giving evidence of their potential. In this regard, research studies aimed to assess the multi-hazard response of similar systems are currently not available in the

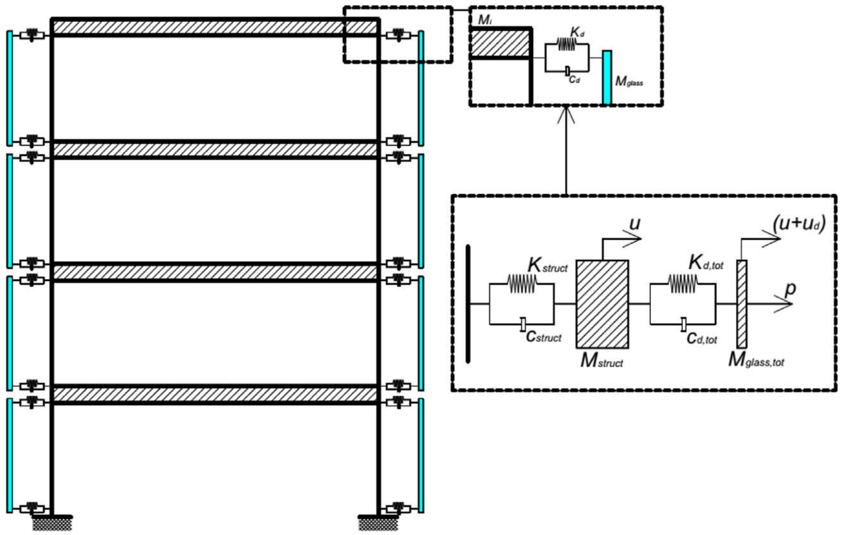

Fig. 5. Reference mechanical model for the analysis of a building with dissipative GCW.

literature. In addition, most of the past FE investigations (i.e. [17,63–65]) have been focused on single facade modules only, rather than full 3D systems inclusive of the actual loading and boundary conditions for curtain walls modules intended as a part of complex architectural systems.

3.2. Reference TMD theoretical formulation

The dynamic performance of a building with UGCWs and vibration control devices can be generally described as shown in Fig. 5. The primary structure (total mass M_{struct}, stiffness K_{struct}, damping c_{struct}) interacts with the UGCW – namely representative of additional mass $M_{glass,tot}$ but limited stiffness, for the primary structure – via special connectors, replacing the usually rigid brackets, characterized by $K_{d,tot}$ and $c_{d,tot}$ total stiffness and damping. The dynamic behaviour of such a SDOF-TMD under a given design load, in particular, is described by [68]:

$$M_{struct}\ddot{u}(t) + C_{struct}\dot{u}(t) + K_{struct}u(t) = P(t) + C_{d,tot}\dot{u}_d(t) + K_{d,tot}u_d(t) \tag{1}$$

$$M_{glass,tot}\ddot{u}_d(t) + C_{d,tot}(\dot{u}_d(t) - \dot{u}(t)) + K_{d,tot}(u_d(t) - u(t)) = p(t) \tag{2}$$

where $u(t)$ is the deflection in time of the primary structure; $u_d(t)$ denotes the GCW relative displacement; $P(t)$ and $p(t)$ represent respectively the force acting on the building mass or on the TMD mass.

Following Eqs. (1) and (2), as far as the facade mechanical and geometrical features modify and the curtain panels are stiff (i.e. due to the presence of thick layers and frame components, or different materials), the proposed solution can still offer benefits, since the vibration control systems would theoretically act as flexible supports for facade modules with mostly rigid body deformations, compared to the primary building. An appropriate fine tuning of connectors features, in this regard, is strictly required at a preliminary design stage, being dependent on the loading condition as well as on the primary building and facade features.

3.3. Connector prototype, preliminary considerations and design strategy

For preliminary design assessment purposes, the typical control system can be composed by a VE solid damper, consisting of metal plates and a middle rubber layer (h_d its thickness and A_d the surface area), see Fig. 6(a). The base plate of the device is directly attached to the structural backup (e.g. inter-storey floor of the primary structure) by means of anchoring bolts, whereas the UGCW framing members are rigidly connected to a further sliding steel plate, enabling possible crushing and rotations of the VE layer when the curtain wall is subjected to external pressures. To this aim, at the interface between the VE compound and the sliding steel bracket, additional gaskets are also expected to be interposed (see details in Fig. 6(a)), so to avoid direct contact and infinitely rigid restraint (hence possible crushing mechanisms or local peaks of reaction forces) for the sliding VE layer. The prototype of Fig. 6 takes inspiration from classical viscous dampers in use for the seismic mitigation of tall buildings, see for example [69].

Assuming that the single UGCW unit is then connected to the adjacent primary structural system by means of four special connectors (i.e. one device at each panel corner), the preliminary estimation of $K_{d,tot}$ and $c_{d,tot}$ can be carried out based on Fig. 6(b). There, M_{glass} is the total mass of a single UGCW panel (inclusive of the glazing elements, plus the metal supporting frame, while non-structural gaskets and sealants are neglected), while K_d and c_d denote the stiffness and damping terms for a single device, being defined as a function of the rubber compound features. Under well-defined loading conditions (e.g. operating frequency ω and temperature T), in particular, the damping ratio c_d can be estimated as [69]:

$$c_d = c_d(\omega) = \frac{K''}{\omega} = \eta\frac{K'}{\omega}, \tag{3}$$

with η the loss factor of the VE layer, while the corresponding storage and loss stiffnesses are given by:

$$K' = \frac{G'(\omega)A_d}{h_d}, \tag{4}$$

$$K'' = \frac{G''(\omega)A_d}{h_d}, \tag{5}$$

with $G'(\omega)$ and $G''(\omega)$ the shear moduli.

For the current research study, a high damping rubber compound was taken into account for the VE layer (ξ = 20%), with $G' = 0.35$ MPa [70,71]. In terms of small gaskets at the interface between the steel bracket and the VE layer, conversely, an hard compound was considered ($G' = 1.2$ MPa). Since such gaskets are expected to activate under certain loading conditions only (i.e. to stop the sliding devices), their possible stiffness as well as damping contributions can be rationally neglected for the elastic mechanical characterization and preliminary design of the proposed control systems.

At a first stage of the design process, given a traditional building with fundamental period T_1, the estimation of K_d can be carried out by equaling the vibration period $T_{1,glass}$ of the UGCW unit and the primary structure one:

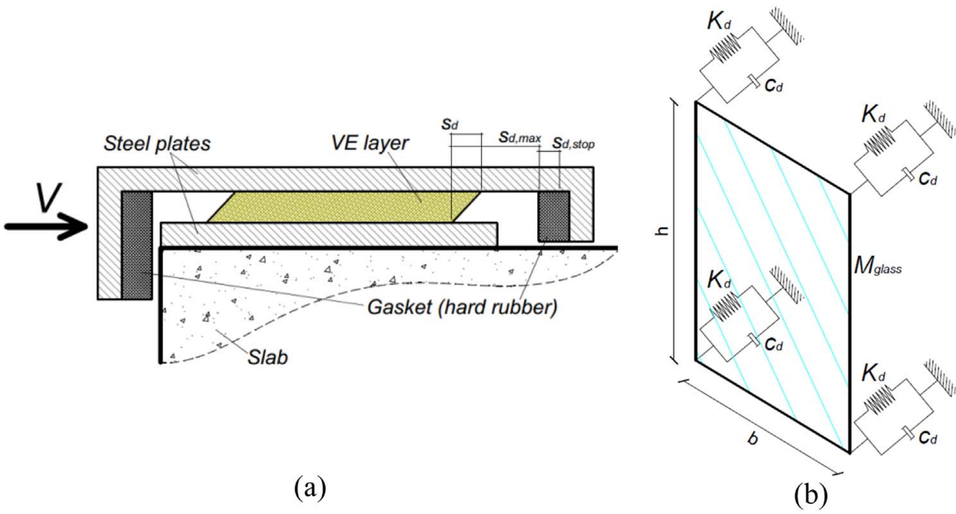

Fig. 6. UGCW with vibration control devices. (a) Example of a typical VE solid damper, with evidence of its working mechanism under shear loads (cross-section); (b) reference mechanical model for the UGCW with vibration control systems.

$$K_d = \frac{1}{4}K_{d,glass} = \frac{M_{glass}\pi^2}{T_{1,glass}^2} \tag{6}$$

As far as the single UGCW module is expected to offer a certain dynamic performance as a part of a full 3D building it belongs, it is interesting to notice that in Eq. (6) – as a reference assumption for preliminary design considerations – the fundamental vibration period T_1 of the primary system is considered, rather than the UGCW unit vibration period $T_{1,glass}$.

Given K_d from Eq. (6), the corresponding damping term c_d is hence given by [72]:

$$c_d = \frac{1}{4}c_{d,glass} = \frac{1}{4}(\xi\cdot c_{cr}) = \frac{1}{4}(\xi\cdot 2M_{glass}\omega_{1,glass}), \tag{7}$$

with $\omega_{1,glass}$ the operating frequency, ξ the damping coefficient of the VE compound and c_{cr} its critical value.

Finally, see Fig. 6(a), possible failure mechanisms in the same VE layer (i.e. tearing) under a given design load should be properly prevented, by limiting its maximum shear deformations s_d:

$$s_d \leq s_{d,\max} = \min(2h_d;\ 30) \qquad \text{in [mm]}. \tag{8}$$

4. Finite Element numerical investigation

The feasibility and potential of the proposed VE connectors is assessed in this paper via FE numerical simulations carried out in ABAQUS [73]. Compared to earlier research efforts, careful consideration is given to a given UGCW module under the effects of various types of high-rise loads, including seismic events as well as blast pressures. To this aim, simulations are carried out at the level of a full 3D building representative of a case study of technical interest, as well as in terms of single UGCW components, so to explore both global and local effects and give evidence of benefits or critical aspects.

4.1. Reference building system, solving approach and FE modelling assumptions

Beside the source and features of the input load, the same general FE modelling approach was taken into account to investigate the seismic or blast-induced response of the reference case study.

As an example, a glazing curtain with modular units spanning from floor-to-floor was taken into account. In accordance with the schematic drawings and labels provided in Fig. 7, a total dimension of h = 2.90 m × b = 1.6 m was considered for each façade unit. The glass panels, supposed to consist in insulated glass units, were assumed as composed by a monolithic, 8 mm in thickness, annealed glass ply (outdoor side), plus a laminated glass panel obtained by assembling two 6 mm thick, heat strengthened glass plies with 1.52 mm thick PVB foil (indoor). Such glass panels were then considered to be continuously supported by an aluminum frame, and hence rigidly connected to the main structure by means of fully rigid steel brackets (see for example Fig. 1).

The so defined UGCW belongs to a 4-storey, continuous steel frame building located in an earthquake-prone region of Italy, see Fig. 7(b). As such, the glazing envelope is connected to the perimeter steelwork (see the detail of Fig. 7(b)).

For the primary steel building, assumed to have a residential destination (category of use “A”, in accordance with the Eurocode 1 provisions [74]), base dimensions of 10 m × 20 m were taken into account, with 12 m the total height, 3 and 2 bays in the longitudinal and transversal directions respectively. Inter-storey floors composed of steel-concrete composite slabs were then taken into account, hence providing in-plane rigid diaphragms to the building. All the steel members, S275 grade with f_y = 275 MPa and f_u = 360 MPa the yielding and collapse stresses respectively, were preliminary designed in accordance with Eurocode 3 [75], by taking into account the effect of permanent loads and accidental loads under ultimate (ULS) and service (SLS) limit state conditions, see Table 1.

A FE model representative of a single UGCW unit was hence first described in ABAQUS [73]. In order to maximize the computational efficiency but preserve the accuracy, the reference UGCW unit was numerically described in the form of *shell* elements, beam elements and mechanical joints, see Fig. 8. A total number of 6000 elements was used to represent the glass pane and the supporting frame, with 35,000 DOFs. For the glass panel, 4-node and 3-node monolithic shell elements (S43, S3R) were defined, with a total thickness of 24 mm [17].

At the current stage of the research study, in accordance with [17] and Fig. 7, the total thickness of glass was taken into account for shell elements, being well representative of the actual inertial and stiffening contribution of a full IGU panel. Based on a free meshing technique, the size of shell elements was then refined in the central region of the pane (i.e. where cracks are expected) and optimized towards the edges, see Fig. 8(b), with average length comprised between 0.004 m and 0.1 m. The metal frame was described via 1D beam elements (B31 type), with box cross-sectional shape well representative of a typical framing system for UGCWs (see [17] and Fig. 1). In the latter case, the mesh size was set to 0.1 m, so to have an optimal correspondence between mesh nodes lying on the glass panel edges and along the frame members.

In terms of materials, the *brittle cracking* damage model was used for glass, so to take into account possible tensile cracking of the panel. Input parameters were calibrated in accordance with [17,76], with 85 MPa the reference tensile resistance of annealed float glass under impulsive loads [1,2] and E_g = 70 GPa the modulus of elasticity. An

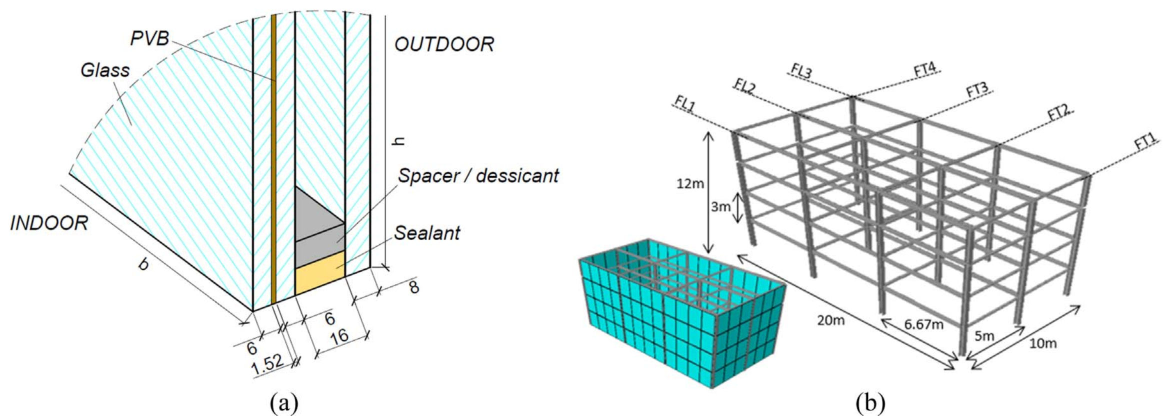

Fig. 7. Reference case study. (a) Typical insulated glass modular unit (cross-sectional detail, with nominal dimensions in millimeters) and (b) full steel building with UGCWs.

Table 1
Steel member sections for the 4-storey seismic resistant building object of investigation.

	Storey #1–4		
Bay	Columns	Primary beams	Secondary beams
Internal	HEB 340	IPE 360	IPE 270
External	HEB300	IPE 330	IPE 240

elasto-plastic constitutive law was indeed then defined for the metal frame, with $E_a = 70$ GPa, $\sigma_{y,a} = 200$ MPa, $\sigma_{u,a} = 280$ MPa the nominal reference values for Young modulus and yielding/ultimate stresses of aluminum [17].

A key role was then assigned to joints, being representative of the actual mechanical interaction between the glass panel and the frame (where sealant joints and gaskets are usually interposed between them), as well as of the structural behaviour of proposed VE fasteners. In the first case, a combined *"cartesian + rotation"* joint was assigned to all the glass panel/frame nodes. Aiming to take into account metal gaskets and frame detailing, relative rotations were restrained between the involved nodes, while a brittle elastic behaviour was defined to represent the in-plane stiffness of sealant joints [17].

To reproduce the mechanical response of VE connectors, nonlinear joints (*"cylindrical"* type of ABAQUS library [73]) were indeed linked to the frame corner nodes, see Fig. 8(b). Stiffness and damping properties for the VE compound were estimated by taking into account Eqs. (6) and (7). The presence of hard rubber gaskets was also properly considered.

As a key aspect due to the assigned mechanical properties and constitutive laws for materials, damage models and joints behaviour, FE simulations were then carried out by taking into account potential failure mechanisms in the main UGCW components, such as tensile cracking in glass, yielding and collapse in the frame members, as well as possible propagation of damage in the sealant joints providing glass-to-frame bonding. In terms of VE connectors, in the same way, the local performance of the proposed devices was assessed in terms of local behaviour under the assigned loading configurations, giving evidence of possible failure mechanisms due to improper design of their mechanical features.

4.2. Expected global dynamic effects of dissipative UGCWs

At a preliminary stage of the FE parametric investigation, the overall dynamic effects due to UGCWs with VE fasteners were first considered, by taking into account a full 3D system, being representative of the examined steel framed building, see Fig. 7(b).

To this aim, the typical UGCW unit shown in Fig. 8 was in fact reproduced in series, so to describe a full facade, and properly restrained to the steel frame, in accordance with Section 4.1, so to reproduce the effect of fully rigid brackets or VE devices with assigned input mechanical features. The steel framed building itself was properly described in ABAQUS, in the form of B31 type beam elements with nominal cross-sectional features, in accordance with Table 1. Steel for structural members was defined in the form of ideal elasto-plastic material, with nominal mechanical properties given in EC3 [75] and previously recalled in Section 4.1.

Following Section 3, K_d was hence estimated as a function of the building period T_1, as numerically calculated in ABAQUS for the steel frame with rigid brackets (see also Fig. 9(b)). A non-dimensional magnifying coefficient R_K was then also defined, so that the actual

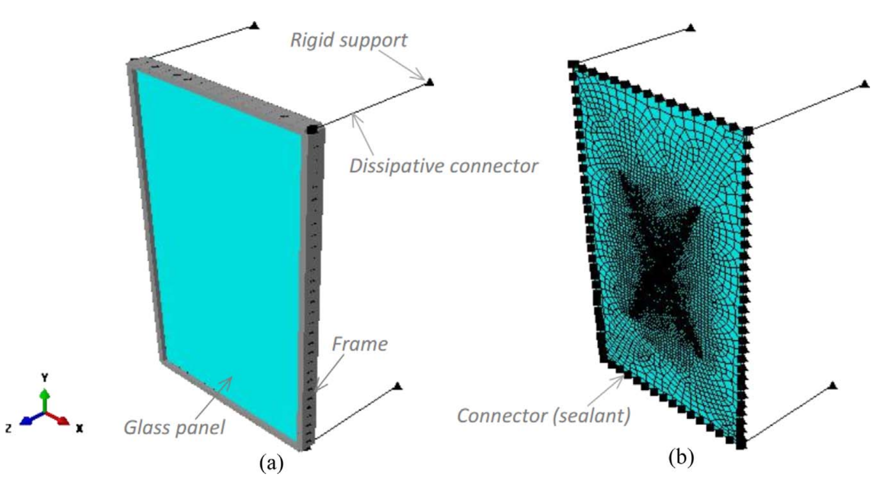

Fig. 8. FE model assembly and features. (a) Extruded view and (b) overview of mesh patterns and mechanical connectors (ABAQUS [73]).

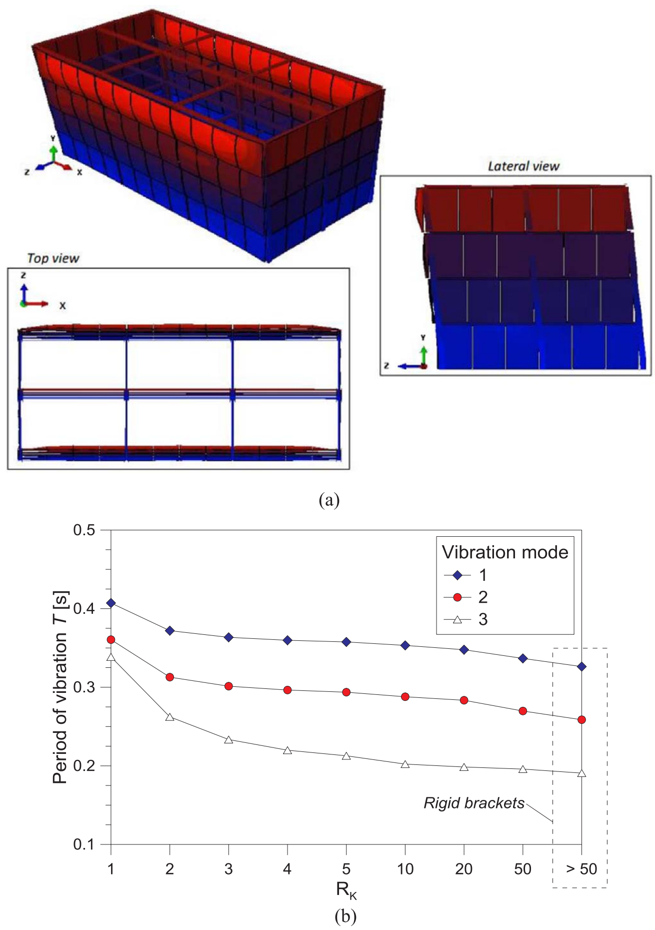

Fig. 9. Dynamic performance the 3D steel building with dissipative UGCWs (ABAQUS [73]). (a) Fundamental vibration shape of the reference system with fully rigid brackets and (b) variation of the fundamental periods of vibration, as a function of the VE devices' stiffness.

stiffness of a single VE device could be calculated – through the FE parametric study – as (with $1 \leq R_K \leq 50$, in this study):

$$K_d = \left(\frac{M_{glass}\pi^2}{T_1^2}\right)\cdot R_K \tag{9}$$

Eigenvalue simulations were carried out on the so assembled 3D building model, to estimate the first vibration periods and modal shapes of the examined structural system, by changing the R_K reference value.

Fig. 9(a) and (b) present the typical fundamental vibration mode, as well as the variation of fundamental periods as a function of R_K, as obtained from these preliminary parametric simulations.

Basically, the first vibration mode was found to be translational along the transversal (z) direction, with $T_1 = 0.326$ s the corresponding period for the UGCW with rigid connectors (i.e. traditional brackets). As far as different R_K values were considered, no tangible effects were generally found for the vibration shapes of fundamental periods, for most of the examined configurations. A coefficient $R_K \geq 50$ in Eq. (9), in particular, proved to coincide – for the examined case study – with almost fully rigid connectors, hence suggesting null benefits due to devices themselves (i.e. traditional brackets). R_K values lower than 2, conversely, gave evidence of qualitatively high relative deformations of the glazing façade, compared to the steel frame, hence suggesting to further investigate the R_K range of values in the order of ≈ 2–30. In this respect, critical observation of FE predicted modes also emphasized the occurrence of additional local vibration modes for the glass panes, in the case $R_K < 2$. These modes, however, proved to not affect the fundamental vibration shapes of the fully assembled structural system, being associated to typically higher vibration frequencies.

Beside the rather stable vibration shapes for the fundamental period of the building, interesting variations were also generally observed in terms of vibration periods for the examined configurations.

As far as the devices stiffness K_d was decreased from the fully rigid case (i.e. traditional rigid brackets), in fact, the corresponding periods of vibration of the building typically increased, see Fig. 9(b). An overall nonlinear dependency was found between these period variations and the added flexibility due to passive VE devices. At the same, time, it should be also noticed that a strong limit of the eigenvalue results proposed in Fig. 9 is given by the total lack of any information related to the effects deriving from the additional damping contribution provided by the same VE devices. The structural performance of the 3D FE models with vibration control systems was thus properly extended by means of nonlinear dynamic analyses.

4.3. Assessing the performance of dissipative UGCWs under seismic events

Following Section 4.2, nonlinear dynamic simulations were carried out on the same full 3D systems, by imposing a set of 7 seismic records at the building base, see Fig. 10(a). All the records, consisting of two-component acceleration data, were derived to be consistent with an EC8 [32] acceleration spectrum associated to type A soil (rock soil), T1

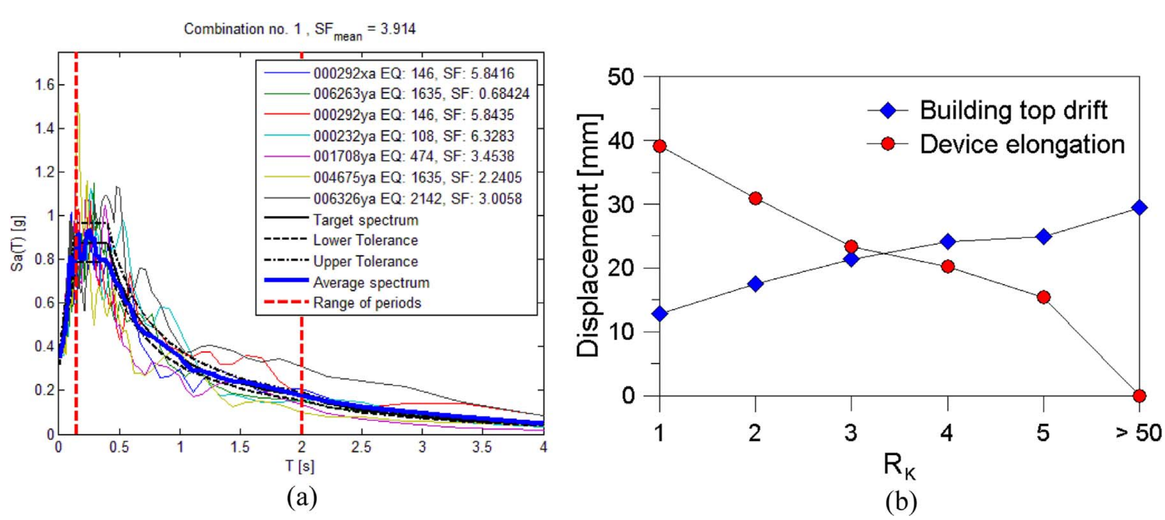

Fig. 10. Seismic dynamic analyses on the 3D steel building with UCGWs. (a) Set of natural seismic spectra, as derived from the REXEL software [77]; (b) effect of vibration control devices on the examined building, with evidence of top drift and relative device sliding, as a function of R_K (ABAQUS [73]).

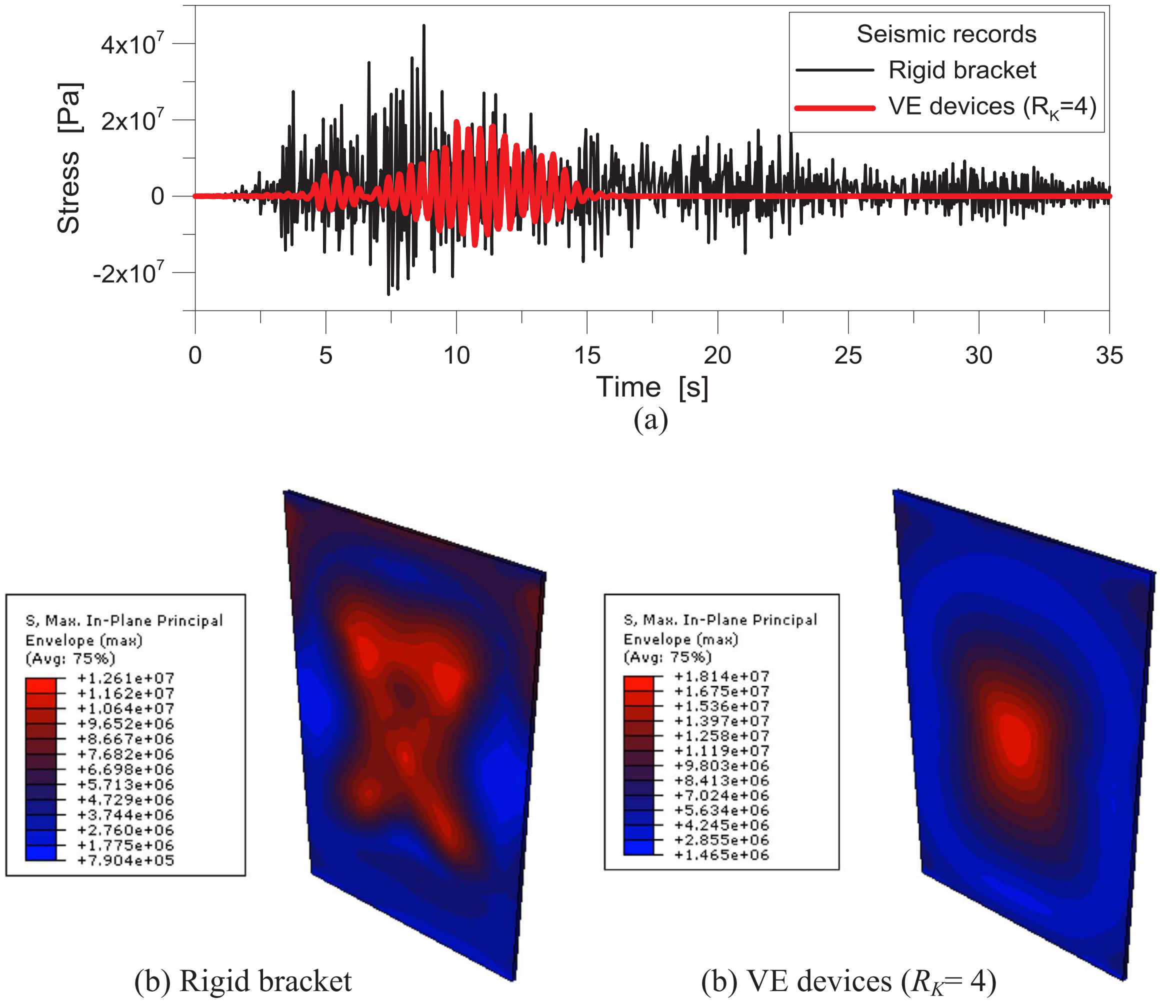

Fig. 11. Tensile stresses in glass for the UGCW unit under seismic events (ABAQUS [73]). Maximum envelope (panel center) and (b)–(c) contour plot (t = 10 s, with legend in Pa).

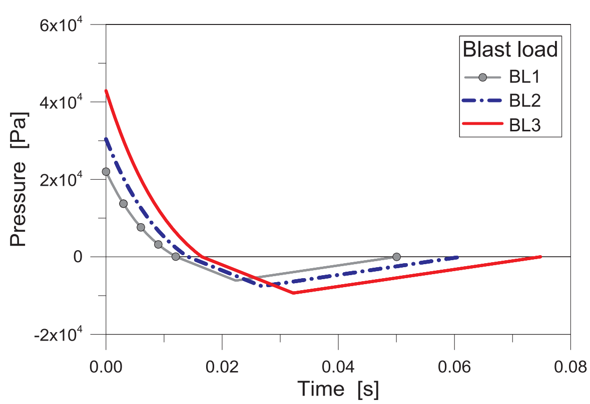

Fig. 12. Reference time-pressure histories for blast loading.

topographic category, 0.35 g Peak Ground Acceleration and 50 years of nominal life (Ultimate Limit State).

Fig. 10(b) presents a comparison of the main results derived from the full seismic investigation. In the figure, both the maximum top drift and the maximum deformation in the VE devices are proposed, as a function of R_K. Each dot of Fig. 10(b), in particular, was calculated through the seismic parametric study as the average value deriving from the maximum envelope value (in absolute terms) of deformations due to the assigned set of seismic records.

Worth of interest is the overall effect of dissipative UGCWs, both in global and local terms. As far as the device stiffness increases (i.e. R_K in Eq. (9)), the damping capacity of VE devices further increases, in accordance with Eqs. (6) and (7). On the other hand, limited relative displacements only are attained in the VE connectors, with corresponding minimum damping contributions and overall benefits for the UGCW components as well as for the full 3D steel frame members. As such, as also in accordance with Fig. 10(b), an optimal balance of local and global performances for the examined case study was found to lie in a range of $R_K \approx$ 3–4 values. In terms of dynamic response of the single UGCW module and VE devices effects on the unit components, further comparative FE results are proposed in Fig. 11, with evidence of tensile stresses amount and distribution in glass under the assigned set of ground accelerations. Also in the latter case it is possible to perceive that – due to combined flexibility and damping properties of VE devices (when properly designed) – part of the incoming seismic input energy is preliminary dissipated. As a result, the single UGCW unit as well as the bare steel frame supporting the facade are potentially subjected to a reduced impulse, compared to the same 3D building system with fully rigid brackets.

In this sense, as a primary effect of the proposed design solution, maximum peaks of stresses in glass were found to be highly mitigated by VE devices (see Fig. 11(a), with evidence of the maximum envelope of tensile stresses in a UGCW unit, as obtained from the building with fully rigid brackets or VE devices (R_K = 4)). Even the imposed seismic records were found to do not lead glass panes to failure (i.e. maximum peaks of stresses in Fig. 11(a) being significantly lower than the reference characteristic tensile resistance of glass (see Section 3)), it is possible to notice that the R_K = 4 configuration is associated to a

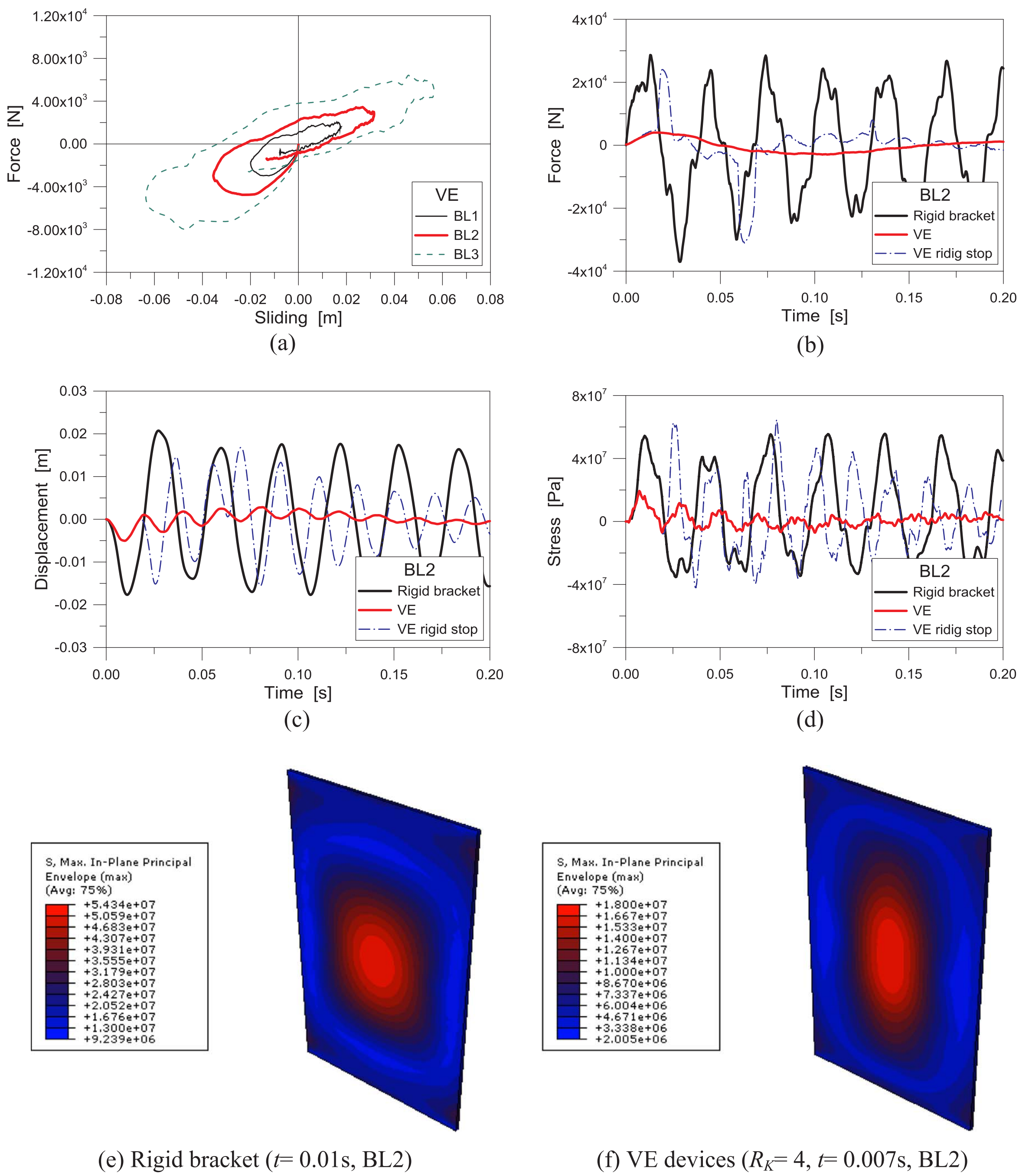

Fig. 13. Response of the GCW unit under blast loads (ABAQUS [73]), with FE results proposed for the $R_K = 4$ configuration. (a) VE connector behaviour; (b) reaction forces at the supports, as a function of time; (c) relative displacements and (d) maximum stresses at the center of glass. (e)–(f) Blue-to-red contour plot of tensile stresses in glass (legend in Pa).

reduction up to ≈ 50% the maximum stresses recorded for the traditionally restrained UGCW.

At the same time, the additional flexibility of GCW supports also proved to beneficially affect the overall distribution of maximum stresses in glass panels. As shown in Fig. 11(b) and (c), a secondary effect of the same VE devices lies in fact in an enhanced distribution of stresses in glass elements. Local peaks of stresses close to the glass supports and edges of glass can be in fact generally avoided, due to properly designed VE fasteners, with obvious benefits for the overall dynamic performance of the full structural system.

4.4. Assessing the performance of dissipative UGCWs under blast loads

By taking into account the same UGCW unit of Fig. 8 and Section 4.1, its dynamic response under blast pressures was then also investigated, in order to further assess the potential of proposed VE devices.

Based on Section 4.3, in particular, the reference configurations with (i) fully rigid brackets and (ii) VE devices with $R_K = 4$ were first taken into account, being the latter well representative of a rather optimal balance of hazard mitigation & VE device performances for the case study building under seismic events.

In doing so, several input blast waves were also taken into account, by considering a certain amount of equivalent TNT charge and a given stand-off distance of 30 m from the facade surface. Both the positive and negative phases were considered, in accordance with [17]. The so defined blast pressure waves were hence applied on the glass surface in the form of uniform, time-varying pressures.

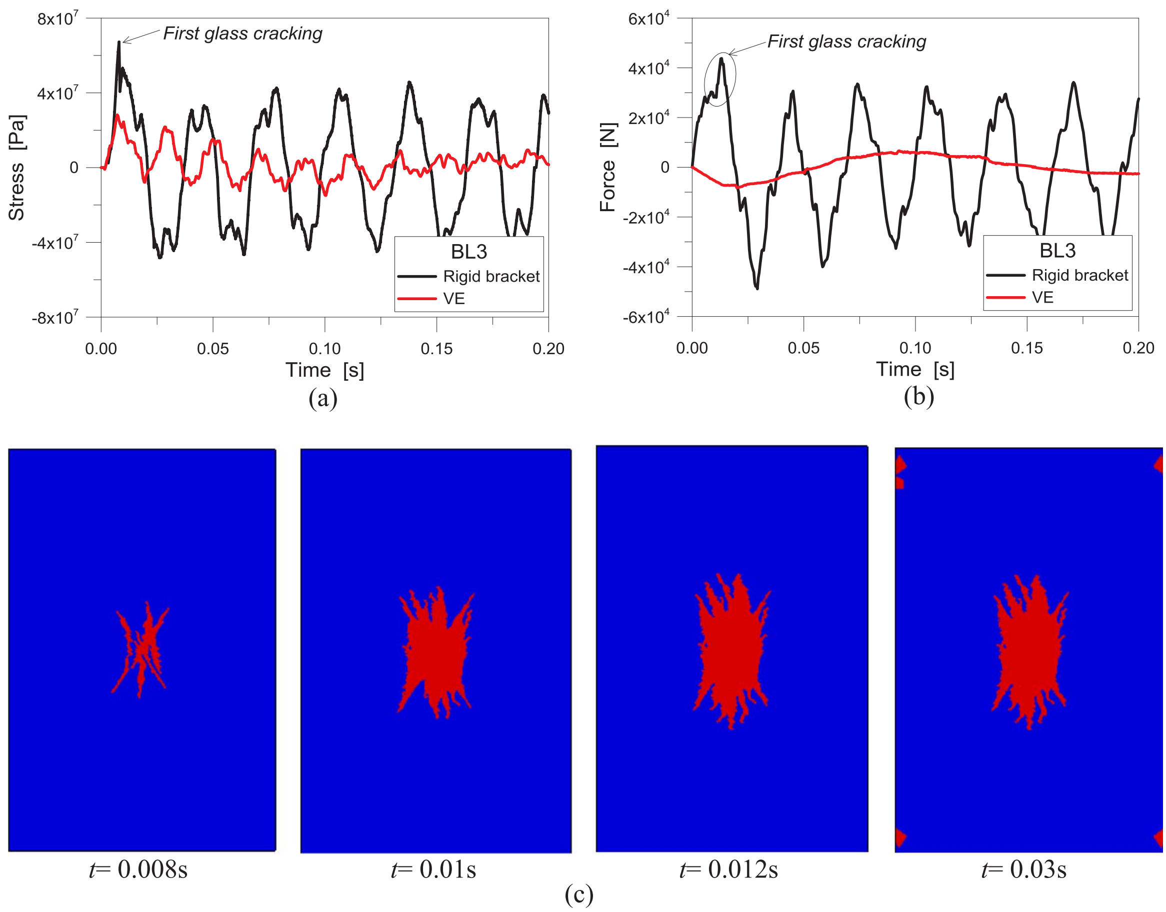

Fig. 14. Response of the UGCW system under blast loads (ABAQUS [73]), with FE results proposed for the R_K = 4 configuration. (a) Maximum stresses at the center of glass and (b) reaction force at the support, as a function of time. (c) Glass damage propagation for the system with rigid brackets (blue = elastic; red = cracked; front view).

In Fig. 12, major outcomes of selected FE simulations are proposed. There, the loading configurations denoted as BL1, BL2 and BL3 correspond respectively to a charge of 12.5 kg, 25 kg and 50 kg of equivalent TNT explosive, with a 30 m stand-off distance from the facade surface, representative of blast loads type B (low level), C (medium) and D (high), as defined by existing design standards [39], see Fig. 12.

As shown in Fig. 13(a), rather stable behaviour was generally observed for the VE devices, even under impulsive blast waves typically characterized by abrupt release of energy and limited duration, in the order of thousand of milliseconds. Under the BL2 loading configuration (i.e. medium blast pressure), in particular, it can be seen in Fig. 13(b)–(d) that the actual effects of the incoming blast pressure are again markedly reduced, compared to the traditional UGCW restrained by means of rigid brackets. In accordance with Fig. 13(b), moreover, maximum reaction forces monitored at the UGCW corners are in the order of ≈ 5 kN for the unit with VE fasteners, compared to ≈ 38 kN for the module with rigid brackets, with marked benefits for the primary structure.

As in the case of seismic loading, see Fig. 13(d), maximum stresses in the glass panes due to blast were found to do not exceed the reference resistance for glass, even in presence of fully rigid brackets. Beside the full elastic performance of glass panels, for the proposed plots, however, the clear benefit due to VE devices can be still appreciated, with decrease in peaks of stresses up to ≈ 70% compared to the traditional system. Such potential benefits derive from the optimal behaviour of VE connectors. In this sense, Fig. 13(b)–(d) give also evidence of the contribution of hard rubber gaskets interposed between the VE compound and the steel bracket (i.e. cross-section of Fig. 6(a)).

As far as an infinitely rigid stop is considered only for the deformed VE layer, in fact, an abrupt increase of measured reaction forces at the supports, as well as maximum deflections and stresses in glass can be observed. The same fully rigid stops, in particular, would involve an additional impulse for the sliding UGCW unit. As a result, the potential benefits of the same VE fasteners could fully vanish, with a dynamic performance of the UGCW panels rather in close correlation with the traditional modules. In Fig. 13(e)–(f) the distribution of maximum tensile stresses is finally also compared, for the UGCW panel with rigid supports or VE fasteners. Compared to Fig. 11, due to the assigned input load as well as to the main membrane behaviour of glass panes, no obvious variations were noticed in terms of distribution of tensile stresses in glass, due to the presence and/or assigned mechanical features of VE devices.

Further FE comparative results are also proposed in Fig. 14, for the same UGCW system under BL3 blast pressures. In the case of high pressure waves, in particular, the proposed VE connectors still proved to highly enhance the overall performance of the examined system. The traditional UGCW panel with fully rigid brackets, in fact, resulted unavoidably subjected to maximum tensile stresses leading the glass panel to breakage (see Fig. 14(a)), with maximum reaction forces at the system supports in the order of 40–50 kN (Fig. 14(b)). Tensile cracks were found to occur first at the glass panel center, being subjected to maximum bending deformations, as well as close to the UGCW supports, due to the presence of fully rigid brackets leading to peaks of stresses (Fig. 14(c)). For the UGCW system with VE connectors, limited tensile stresses in glass were only recorder, i.e. in the range of 30 MPa, with obvious benefits for the overall structural dynamic response of the full assembly.

As far as the UGCW is affected by low dynamic pressures only (i.e. the BL1 wave considered in this study), rather close correlation between BL2 and BL3 results can be expected, with the difference that both the fully rigidly supported and the VE connected systems behave fully elastically.

5. Summary and conclusions

In this paper, the feasibility and potential of special mechanical joints interposed at the interface between a given multi-storey primary building and a traditional unitized glazing curtain wall (UGCW) have been investigated via accurate Finite-Element (FE) numerical models, under various impact scenarios.

The main goal of the ongoing research study partly summarized in the paper is in fact represented by the development and optimization of a novel, structurally smart design concept able to enhance the facade and building dynamic performance.

To this aim, careful consideration has been paid for seismic events as well as blast loads, being representative of exceptional and high rise loading conditions for facades and buildings. On one side, glass systems are largely used in buildings due to a series of motivations. At the same time, however, such systems are typically vulnerable to extreme loads, and current design standards do not provide specific regulations for their enhancement as a structural part of full 3D assemblies. As a result, specific design assumptions and special tools are often required to ensure appropriate protection levels for the occupants, as well as performances for the building itself. This is true especially when multi-hazard mitigation is required.

As shown in the paper, even seismic and accidental or blast-induced loads are characterized by typically different input features and expected behaviour for a given structural system, rather well promising effects were generally observed by using the proposed VE connectors. Worth of notice is that the current structural outcomes should be properly validated by extensive small-scale and full-scale experiments, as well as properly combined – towards the full finalization and optimization of the design concept – with other key issues in the design of glass curtains, such as thermal and insulation requirements.

Based on properly designed fasteners able to introduce additional flexibility and damping capacities in the traditional building, in particular, the maximum effects and benefits of such connectors have been emphasized via a case study of technical interest, both in terms of global performances as well as local and component behaviour for the building object of investigation, giving evidence of the potential of UGCWs acting as distributed Tuned Mass Dampers (TMDs) for multi-storey buildings under hazards.

For the case study investigated in the paper, for example, being located in an earthquake prone region in Italy, the design optimization has been carried out with respect to the dynamic performance of the full 3D system under high-level seismic loads. As shown, once the input mechanical parameters of the dissipative devices are fully optimized, overall structural benefits can be achieved both at the component level (i.e. maximum stresses in the UGCW elements, or reaction forces transferred to the primary building) as well as at the global level (i.e. maximum inter-storey drifts under the assigned seismic loads). Despite the huge differences in the basic features of seismic loads rather than explosive events, a very good dynamic performance has then been observed for the same 3D assembly under high-rate blast pressures, hence suggesting the strong multi-hazard potential of the examined design concept. It is hence expected, based on current research outcomes, partly emphasized in the paper, that the same concept could be further calibrated, as well as that related design criteria could be fully implemented towards the definition of practical tools for designers. In this sense, small-scale (i.e. at the connector level) and full-scale (i.e. on facade prototypes) experiments will represent a key step for the validation and optimization of the theoretical study.

References

[1] M. Haldimann, A. Luible, M. Overend, Structural Use of Glass, IABSE, Zurich, Switzerland, 2008 (ISBN 978-3-85748-119-2).

[2] M. Feldmann, R. Kasper, B. Abeln, P. Cruz, J. Belis, J. Beyer, et al., Guidance for European Structural Design of Glass Components – Support to the Implementation, Harmonization and Further Development of the Eurocodes (Report EUR 26439), Joint Research Centre–Institute for the Protection and Security of the Citizen, Pinto Dimova, Denton Feldmann, 2014, http://dx.doi.org/10.2788/5523.

[3] R.A. Behr, A. Belarbi, A.T. Brown, Seismic performance of architectural glass in a storefront wall system, Earthq. Spectra 11 (3) (1995) 367–391.

[4] C.P. Pantelides, K.Z. Truman, R.A. Behr, A. Belarbi, Development of a loading history for seismic testing of architectural glass in a shop-front wall system, Eng. Struct. 18 (12) (1996) 917–935.

[5] D. Smith, Glazing for injury alleviation under blast loading – United Kingdom practice, in: Proceedings of Glass Processing Days, 2001, pp. 335–340.

[6] P.A. Hooper, R.A.M. Sukhram, B.R.K. Blackman, J.P. Dear, On the blast resistance of laminated glass, Int. J. Solids Struct. 49 (2012) 899–918.

[7] X. Zhang, H. Hao, G. Ma, Parametric study of laminated glass window response to blast loads, Eng. Struct. 56 (2013) 1707–1717.

[8] P. Del Linz, X. Liang, P.A. Hooper, L.Z. Wang, J.P. Dear, An analytical solution for pre-crack behaviour of laminated glass under blast loading, Compos. Struct. 144 (2016) 156–164.

[9] C. Bedon, R. Kalamar, M. Eliášová, Low velocity impact performance investigation on square hollow glass columns via full-scale experiments and Finite Element analyses, Compos. Struct. 182 (2017) 311–325.

[10] R. Kalamar, C. Bedon, M. Eliášová, Assessing the structural behaviour of square hollow glass columns subjected to combined compressive and impact loads via full-scale experiments, Eng. Struct. 143 (2017) 127–140.

[11] K. Machalická, M. Charvátová, M. Eliášová, P. Kuklík, The behaviour of fire resistant glass under fire, in: Cruz (Ed.), Structures and Architecture – Beyond their Limits, 2016, pp. 991–997 (ISBN 978-1-138-02651-3).

[12] S.L. Manzello, R.G. Gann, S.R. Kuckak, K.R. Prasad, W.W. Jones, An experimental determination of a real fire performance of a non-load bearing glass wall assembly, Fire Technol. 43 (2007) 77–89.

[13] R.A. Behr, Architectural Glass to Resist Seismic and Extreme Climatic Events, Woodhead Publishing, 2009 (ISBN 9781845693695).

[14] X. Zhang, H. Hao, G. Ma, Laboratory test and numerical simulation of laminated glass window vulnerability to debris impact, Int. J. Impact Eng. 55 (2013) 49–62.

[15] F.J. Masters, K.R. Gurley, N. Shah, G. Fernandez, The vulnerability of residential window glass to lightweight windborne debris, Eng. Struct. 32 (2010) 911–921.

[16] M.S. Shetty, J. Wei, L.R. Dharani, D.S. Stutts, Analysis of damage in laminated architectural glazing subjected to wind loading and windborne debris impact, Buildings 3 (2013) 422–441.

[17] C. Bedon, C. Amadio, Passive control systems for the blast enhancement of glazing curtain walls under explosive loads, Open Civ. Eng. J. 11 (Suppl-1) (2017) S396–S419 (M8).

[18] P. Gardoni, J.M. LaFave, Multi-hazard approaches to civil infrastructure engineering: mitigating risks and promoting resilience, in: P. Gardoni, J.M. LaFave (Eds.), Multi-hazard Approaches to Civil Infrastructure Engineering, Springer, Cham, 2016.

[19] R.M. Sanders, C.A. Hargrove, Preventing and treating failure in glazed curtain wall systems, J. Arch. Technol. 29 (2) (2012) 1–8 ⟨http://www.hoffarch.com/assets/journal1171.pdf⟩.

[20] Centre for Window and Cladding Technology-CWCT, Curtain Wall Installation Handbook, (2017) (ISBN 1 874003 96 3).

[21] R.S. Matthews, M.R.C. Bury, D. Redfearn, Investigation of dynamic water penetration tests for curtain walling, J. Wind Eng. Ind. Aerodyn. 60 (1996) 1–16.

[22] E. Ilter, A. Tavil, O.C. Celik, Full-scale performance testing and evaluation of unitized curtain walls, J. Facade Des. Eng. 3 (2015) 39–47.

[23] R. Hinojosa, Designing, specifying, and testing windows for water penetration resistance: the effects of changes in the AAMA 502 test standard, Interface 4 (2010) 1–6 ⟨www.rci-online.org⟩.

[24] H. Mitani, H. Ogawa, T. Mizukami, Y. Kojo, Thermal performance of glass curtain walls with thermal break framing system, AIJ J. Technol. Des. 19 (43) (2013) 831–836.

[25] M.J. Bae, J.H. Oh, S.S. Kim, The effects of the frame ratio and glass on the thermal performance of a curtain wall system, Energy Procedia 78 (2015) 2488–2493.

[26] C. Bedon, F. Favoino, C. Pascual, A.L. Navarro, M. Overend, Thermal and structural assessment of novel GFRP-glass sandwich facade components via numerical simulations, in: Proceedings of COST TU1403 "Adaptive Facades Network" Mid-Term Conference, Munich, Germany, 2017, ⟨http://thescipub.com/abstract/10.3844/ajeassp.2016.713.723⟩.

[27] M. Lindqvist, Structural Glass Strength Prediction Based on Edge Flaw Characterization (Ph.D. Dissertation, Thesis n. 5627), EPFL, Lausanne, CH, 2013 (free download at (Accessed October 2017): ⟨https://infoscience.epfl.ch/record/183860/files/EPFL_TH5627.pdf ⟩).

[28] M. Zaccaria, M. Overend, Thermal healing of realistic flaws in glass, J. Mater. Civ. Eng. 28 (2) (2016), http://dx.doi.org/10.1061/(ASCE)MT.1943-5533.0001421.

[29] M. Overend, S. De Gaetano, M. Haldimann, Diagnostic interpretation of glass failure, Struct. Eng. Int. 17 (2) (2007) 151–158.

[30] M. Haldimann, Fracture Strength of Structural Glass Elements – Analytical and Numerical Modelling, Testing and Design (Ph.D. Dissertation, Thesis n. 3671), EPFL, Lausanne, CH, 2006 (free download at (Accessed October 2017): ⟨https://infoscience.epfl.ch/record/89658/files/EPFL_TH3671.pdf ⟩).

[31] M. Santarsiero, M. Froli, A contribution to the theoretical prediction of life-time in glass structures, J. Int. Assoc. Shell Spat. Struct. 52 (4) (2011) 225–231.

[32] EN 1998-1-1, Eurocode 8 – Design of Structures for Earthquake Resistance – Part 1: General Rules, Seismic Actions and Rules for Buildings, CEN, Brussels, Belgium,

2004.
[33] NZS 1170.5, Structural Design Actions – Part 5, ZEALAND, STANDARDS NEW, Wellington, NZ, 2004.
[34] FEMA 450, Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, (2003).
[35] JASS 14, Japanese Architectural Standard Specification Curtain Wall, AIJ, Architectural Institute of Japan, 1996.
[36] H. Sucuoglu, C.V.G. Vallabhan, Behaviour of window glass panels during earthquakes, Eng. Struct. 19 (8) (1997) 685–694.
[37] J.L. Brueggeman, R.A. Behr, H. Wulfert, A.M. Memari, P.A. Kremer, Dynamic racking performance of an earthquake-isolated curtain wall system, Earthq. Spectra 16 (4) (2000) 735–756.
[38] ISO 16933:2007/Cor1, Glass in building – Explosion-resistant security glazing – Test and Classification for Arena Air-blast Loading, (2008).
[39] GSA-TS01, Standard Test Method for Glazing and Window Systems Subject to Dynamic Overpressure Loadings, US General Services Administration, 2003.
[40] ASTM F1642, Standard Test Method for Glazing and Glazing Systems Subject to Air Blast Loadings, West Conshohocken, PA, USA, 2004.
[41] Home Office Scientific Development Branch - HOSDB, British Glazing Hazard Guide. Special Services Group, Explosion Protection. Glazing Hazard Guide-Charts, Security Facilities Executive (Report SSG/EP/3/97), Cabinet Office, London, UK, 1997.
[42] S. Meyer, L. Little, E. Conrath, Injury Based Glass Hazard Assessment. DTIC Document, freely available online at (Accessed July 2017): 〈http://www.dtic.mil/dtic/tr/fulltext/u2/a548169.pdf〉.
[43] M. Arrigoni, C. Bedon, A. van Doormal, C. Haberacker, G. Hüsken, O. Millon, A. Saarenheimo, G. Solomos, A. Stolz, L. Thamie, G. Valsamos, A. Williams, Alexander Stolz (Ed.), Suggestions for Adaptations of Existing European Norms for Testing the Resistance of Windows and Glazed Façades to Explosive Effects, European Union Publications Office, Technical Report, 2017, 〈https://www.scientific.net/KEM.755.121〉.
[44] M. Larcher, M. Arrigoni, C. Bedon, A. van Doormaal, C. Haberacker, G. Hüsken, O. Millon, A. Saarenheimo, G. Solomos, L. Thamie, G. Valsamos, A. Williams, A. Stolz, Design of blast-loaded glazing windows and facades: a review of essential requirements towards standardization, Adv. Civ. Eng. 2016 (2016), http://dx.doi.org/10.1155/2016/2604232 (Article ID 2604232, 14 pages).
[45] G. Nestovito, A. Occhiuzzi, Implementation of smart-passive dampers combined with double concave friction pendulum devices to retrofit an existing highway viaduct exploiting the seismic early warning information, Eng. Struct. 120 (2016) 58–74.
[46] R. Greco, G.C. Marano, Multi-objective optimization of a dissipative connection for seismic protection of wall-frame structures, Soil Dyn. Earthq. Eng. 87 (2016) 151–163.
[47] R.M. Oinam, D.R. Sahoo, Seismic rehabilitation of damaged reinforced concrete frames using combined metallic yielding passive devices (Published online), Struct. Infrastruct. Eng.: Maint. Manag. Life-Cycle Des. Perform. (2016), http://dx.doi.org/10.1080/15732479.2016.1190766.
[48] F. Fathi, O. Bahar, Hybrid coupled building control for similar adjacent buildings (Published online), KSCE J. Civ. Eng. (2016), http://dx.doi.org/10.1007/s12205-016-0708-x.
[49] C.-H. Lee, J. Kim, D.-H. Kim, J. Ryu, Y.K. Yu, Numerical and experimental analysis of combined behavior of shear-type friction damper and non-uniform strip damper for multi-level seismic protection, Eng. Struct. 114 (2016) 75–92.
[50] K. Lievens, G. Lombaert, G. De Roek, O. Van den Broeck, Robust design of a TMD for the vibration serviceability of a footbridge, Eng. Struct. 123 (2016) 408–418.
[51] Q. Wen, X. Hua, Z. Chen, Y. Yang, H. Niu, Control of human-induced vibrations of a curved cable-stayed bridge: design, implementation, and field validation (Published online), J. Bridge Eng. (2016), http://dx.doi.org/10.1061/(ASCE)BE.1943-5592.0000887.
[52] A.Y. Pisal, R.S. Jangid, Vibration control of bridge subjected to multi-axle vehicle using multiple tuned mass friction dampers, Int. J. Adv. Struct. Eng. 8 (2) (2016) 213–227.
[53] J. Wang, C. Lin, B. Chen, Vibration suppression for high-speed railway bridges using tuned mass dampers, Int. J. Solids Struct. 40 (2003) 465–491.
[54] G.H. Poh'sie, C. Chisari, G. Rinaldin, C. Amadio, M. Fragiacomo, Optimal design of tuned mass dampers for a multi-storey cross laminated timber building against seismic loads (Published online), Earthq. Eng. Struct. Dyn. (2016), http://dx.doi.org/10.1002/eqe.2736.
[55] C.M.R. Gaspar, J.G. Santos da Silva, L.F. Costa-Neves, Multimode vibration control of building steel-concrete composite floors submitted to human rhythmic activities, Comput. Struct. 165 (2016) 107–122.
[56] G. Song, P. Zhang, L. Li, M. Singla, D. Patil, H. Li, Y. Mo, Vibration control of a pipeline structure using pounding tuned mass damper (Published online), J. Eng. Mech. (2016), http://dx.doi.org/10.1061/(ASCE)EM.1943-7889.0001078.
[57] X. Zhao, L. Wang, Y. Zheng, Combined tuned damper-based wind-induced vibration control for super tall buildings, J. Tongji Univ. 44 (4) (2016) 550–558.
[58] X. Tong, X. Zhao, Z. Zhao, Passive structural vibration control of a monopole wind turbine tower, in: Proceedings of the IEEE Conference on Decision and Control, 2016, pp. 1352–1357.
[59] J. Den Hartog, Mechanical Vibrations, Mc-Graw Hill, New York, 1956.
[60] C.-L. Lee, Y.-T. Chen, L.-L. Chung, Y.-P. Wang, Optimal design theories and applications of tuned mass dampers, Eng. Struct. 28 (2006) 43–53.
[61] G. Warburton, Optimum absorber parameters for minimizing vibration response, Earthq. Eng. Struct. Dyn. 9 (1981) 251–262.
[62] N. Hoang, Y. Fujino, P. Warnitchai, Optimal tuned mass damper for seismic applications and practical design formulas, Eng. Struct. 30 (3) (2008) 707–715.
[63] M. Mohebbi, A. Joghataie, Designing optimal tuned mass dampers for nonlinear frames by distributed genetic algorithms, Struct. Des. Tall Spec. Build. 21 (1) (2012) 57–76.
[64] K.S. Moon, Integrated damping system for tall buildings: tuned mass damper/double skin façade damping interaction system, Struct. Des. Tall Spec. Build. 25 (5) (2016) 232–244.
[65] C. Bedon, C. Amadio, Exploratory numerical analysis of two-way straight cable-net facades subjected to air blast loads, Eng. Struct. 79 (2014) 276–289.
[66] C. Amadio, C. Bedon, Viscoelastic spider connectors for the mitigation of cable-supported facades subjected to air blast loading, Eng. Struct. 42 (2012) 190–200.
[67] C. Amadio, C. Bedon, Elastoplastic dissipative devices for the mitigation of blast resisting cable-supported glazing facades, Eng. Struct. 39 (2012) 103–115.
[68] K.S. Moon, Vertically distributed multiple tuned mass dampers in tall buildings: performance analysis and preliminary design, Struct. Des. Tall Spec. Build. 19 (2010) 347–366.
[69] S.W. Park, Analytical modeling of viscoelastic dampers for structural and vibration control, Int. J. Solids Struct. 38 (2000) 8065–8092.
[70] S. Fujita, S. Morikawa, Y. Suizu, T. Fujita, T. Teramoto, H. Kitamura, O. Furuya, Development of high damping rubber damper for vibration attenuation of high-rise buildings, in: Proceedings of the 10th WCEE – World Conference on Earthquake Engineering, vol. 4(7), 1992, pp. 2097–2101.
[71] M. Nakao, N. Yamaguchi, T. Furuta, Development of a passive damper device with high damping rubber for wooden houses, in: Proceedings of the 8th International Conference on Structural Dynamics – EURODYN 2011, Leuven, Belgium, paper code: 116680, 2011.
[72] A.K. Chopra, Dynamics of Structures – Theory and Applications to Earthquake Engineering, 4th ed, Prentice Hall, 2011.
[73] ABAQUS, Dassault Systèmes. ABAQUS v. 6.14, Providence, RI, 2016.
[74] EN 1991-1-1, Eurocode 1 – Actions on Structures – Part 1-1: General Actions, CEN, Brussels, Belgium, 2004.
[75] EN 1993-1-1, Eurocode 3 – Design of Steel Structures – Part 1-1: General Rules and Rules for Buildings, CEN, Brussels, Belgium, 2004.
[76] C. Bedon, C. Amadio, A. Sinico, Numerical and analytical investigation on the dynamic buckling behavior of glass columns under blast, Eng. Struct. 79 (2014) 322–340.
[77] I. Iervolino, C. Galasso, E. Cosenza, REXEL: computer aided record selection for code-based seismic structural analysis, Bull. Earthq. Eng. 8 (2) (2010) 339–362, http://dx.doi.org/10.1007/s10518-009-9146-1.