Seismic design of frameless glass structures – Requirements and practice

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ABSTRACT
Glass is largely used in buildings, both for novel or existing constructions. On one side, there is evidence of an high aesthetic impact and versatility of glass. On the other hand, the typical brittleness and vulnerability of glass represent critical aspects for structural design purposes. This is especially the case of seismic prone regions, where rarely specific calculation methods and design specifications are provided by existing standards or guideline documents, to perform their seismic verification. Even more attention is required for glass systems in which bracing and supporting members (i.e., metal frames) are reduced to a minimum. In this paper, a discussion on current design requirements for the seismic performance assessment of glass systems is presented, with careful consideration for the Italian scenario. The attention is focused on frameless glass assemblies, in which the use of restraints is minimized in favor of steel point connections (i.e., bolts and mechanical fixings, spiders, friction clamps, etc.).

1 INTRODUCTION

Glass is increasingly used in buildings and constructed facilities. Typical applications can be found in the form of curtain walls, innovative "adaptive" facades and even load-bearing members (beams, columns), or complex 3D systems and shear walls intended to contribute to the overall structural performance of the building they belong (Figure 1). Given the intrinsic features and applications of glass (also in combination with other materials), dedicated design methods are required under ordinary design loads, and even more under extreme events like earthquakes (Bedon et al., 2018).

As known, ordinary structural assemblies composed of conventional constructional materials and located in seismic regions are commonly required to offer adequate safety and serviceability performance capacities, namely represented by:

- a limited probability of collapse at the Ultimate Limit State (ULS). Although yielding and extensive structural damages are accepted, collapse should not occur;
- an appropriate capacity to accommodate the displacement demand, at the Service Limit State (SLS).

(a)

(b)

Figure 1. Glass in buildings: (a) point-supported façade and (b) pavilion (photos reproduced from (Santarsiero et al. 2019)).

In such a scenario, the brittleness of glass represent a key issue, and the need of robustness, redundancy, ductility is further enforced. The current challenge for glass designers is even more complex because no detailed attention is given by standards to assess (or improve) the performance of glass systems in seismic regions (EN 1998-1:2004). As a general rule, most of the seismic requirements are in fact related to "*non-structural*" components, hence focused on providing adequate *clearance* gaps, to accommodate the relative displacements of primary buildings, etc.

In this paper, special attention is given to frameless glass systems, in which metal components are reduced to a minimum. Design requirements from current standards and guideline documents are first commented, with reference to the Italian scenario. A case-study glass system (i.e., a frameless partition assembly realized in 2018, in the framework of a historical building in Italy) is then presented, with a focus for some of the aspects that should be properly taken into account.

2 RESEARCH TRENDS

While the current research efforts are aimed at implementing and optimizing appropriate design rules for glass structures – covering a wide multitude of loading / boundary conditions – limited investigations have been focused on the seismic performance of glass systems.

Most of the existing studies, in addition, are related to curtain walls in which glass components are fully braced by continuous framing members, and merely intended as infill components, see (Sucuoglu and Vallabhan, 1997; Wensheng and Baofeng, 2008; Sivanerupan et al. 2011). Only few investigations are available for point-supported facades (Martins and Delgado, 2012; Sivanerupan et al. 2014; etc.). The dissipation capacity of ordinary curtain walls under seismic events was partly assessed by (Casagrande et al. 2019), while (Krstevska et al. 2013) proposed a dissipative timber-glass wall for earthquake resistant building. Bedon and Amadio (2018) first demonstrated that curtain walls can be efficiently involved in the dynamic response of multi-storey buildings under seismic events. Based on special connectors, the feasibility of a "distributed-TMD" concept was numerically explored, giving evidence of potential benefits. Santarsiero et al. (2019) recently assessed the seismic performance of glass portal frames, presenting some preliminary estimates for their *q*-behaviour factor.

3 SEISMIC DESIGN OF GLASS SYSTEMS BASED ON THE CNR-DT 210 GUIDE

3.1 Basis of design

Section §4.4 of CNR-DT 210/2013 focuses on seismic design actions and general rules for safe design purposes. The CNR guide includes a pre-standard study on the performance of glass structure and components, and is not prescriptive. However, it actually represents one of the most detailed guides in support of glass designers, given the lack of specific regulations in the Italian Technical Standards for Constructions (NTC2018). In addition, several sections are supporting the drafting of the in-progress Eurocode 10 for glass structures, see also Figure 2 and (Feldmann et al. 2014).

3.2 Consequences classes and "secondary" structural components

For seismic purposes, the CNR document detects different levels of analysis and design for structural glass systems / elements that can have a certain role as constructions. These are implicitly related to the class of use and consequences classes (CC) of the system / part to verify.

The guide, as also in line with Eurocodes, excludes from analysis all the glass elements that do not have any kind of structural role, and basically fall in the consequences class zero (CC0, see also §3.2.1 and EN1990:2002-Annex B1).

Accordingly, a given glass system / element under ordinary design loads is expected to belong to classes CC1 to CC3, and more precisely:

- CC1= when glass failure has *limited* consequences in terms of loss of human life and *small or negligible* consequences in economic, social or environmental terms. CC1 includes glass structures / elements in buildings with people present only occasionally. The probability of failure is $P_{f,50}= 5.83\times10^{-4}$ and $P_{f,1}= 1.335\times10^{-5}$;
- CC2= failure has *medium* consequences for human life, but *considerable* consequences in economic, social and environmental terms. Typical examples are glass structures / elements for residential / office buildings. In this case, $P_{f,50}= 6.2353\times10^{-5}$ and $P_{f,1}= 1.3\times10^{-6}$;
- CC3= failure has *high* consequences in terms of human life and *very great* consequences in economic, social terms.

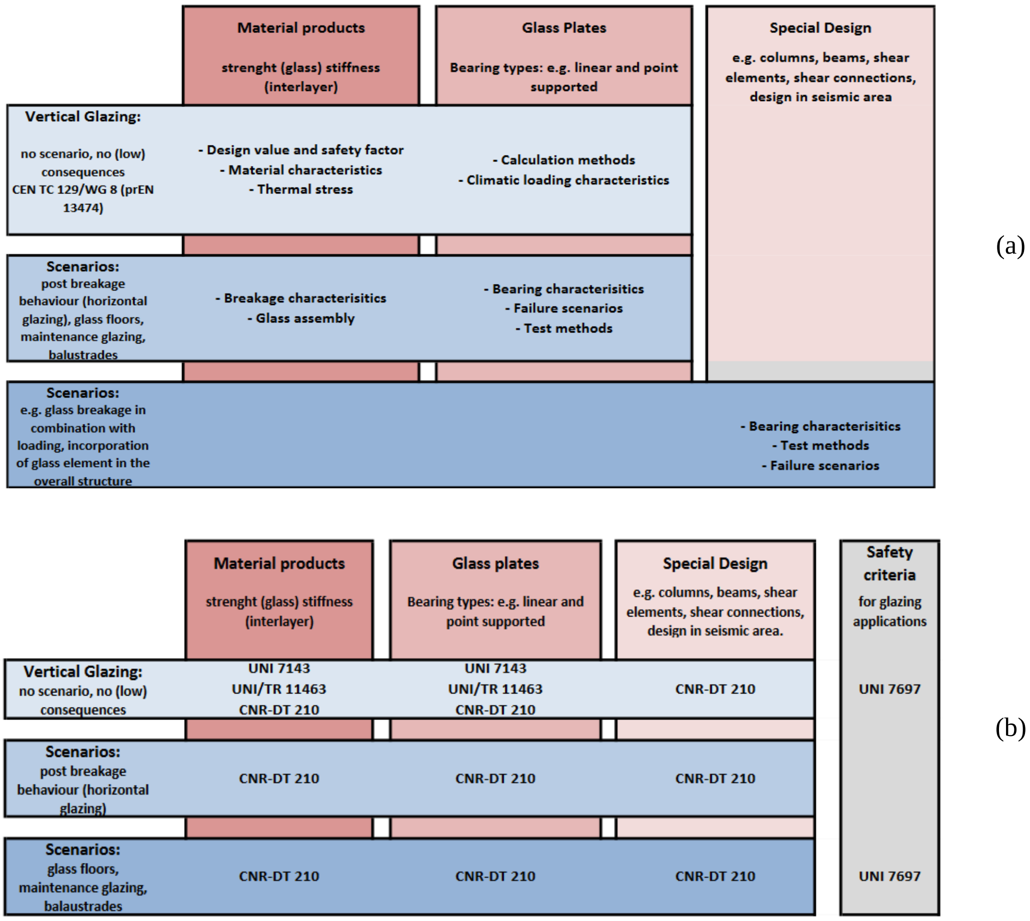


Figure 2. Structural design of glass (reproduced from (Feldmann et al. 2014)): (a) possible scenarios and (b) reference documents for Italy.

CC3 includes public buildings and places susceptible to overcrowding ($P_{f,50}$= 8.54×10^{-6}, $P_{f,1}$= 9.96×10^{-8}).

Compared to other constructional materials, the appropriate verification of glass elements can be implicitly related to possible uncertainties, when designers are asked to associate a given assembly to a certain CC (Table 1). Special care is hence required for such a delicate aspect, in favour of appropriate and safe verifications.

Table 1. Classification of common glass elements (CC0= secondary, non-structural elements). n.a.= no performance assessment is required; F= failure

Element	CC_n (pre-F)	CC_n (post-F)
Vertical (linear restraints)	1	1/n.a.
Vertical (point fixings)	2/1	1/n.a.
Roofs	2	2/1
Fins	2	2/1
Railings (fall danger)	2	2/1
Floors, beams	2	2
Pillars	3	2 (pre-F loads)

At the same time, a key role is assigned not only to the design of glass itself, but especially to the detailing of connectors and restraints. A minimum gap (i.e., *clearance*) is in fact mandatory, so as to ensure that relative deformations of a given glass element, with respect to the bracing system, could not manifest in premature fracture, under ordinary loads and even more in case of earthquakes.

For seismic design, the distinction of the CNR guide is between (a) "*secondary*" structural elements or (b) glass elements that have a relevant structural role under seismic events (§4.4.1).

Definition (a) – see also NTC2018, §7.2.3 – is reliable as far as the stiffness and resistance can be neglected (that is, less than 15% the full system, based on NTC2018). These elements can be disregarded in the global seismic analysis of the primary system, because they are expected to do offer a negligible contribution towards the design horizontal forces. However, *secondary* structural parts are in any case required to withstand vertical loads and accommodate the main deformation of the primary system, under the most unfavourable seismic combination of loads (Collapse Limit

State (SLC, section 3.4)). In other words, both in-plane and out-of-plane deformations in glass, as well as in joints and restraints, must be properly verified in seismic conditions, where their load-bearing capacity must be preserved. Even the assumption of the NTC design requirements for "*non-structural constructive elements*" – i.e., with high contribution for safety levels and potential risk for people – would reflect in a detailed seismic design and verification of glass members and restraints. Inter-storey drifts under the Limit States of interest should be hence ensured.

In case (b), the CNR document includes glass systems and components that have a relevant stiffness / resistance contribution, or consist of stand-alone / special glass structures. All these solutions fall in the CC3, and even minor damage must be necessarily avoided under seismic events. In this latter case, dedicated experiments can be also required, in support of design (for glass and /or joints and restraints).

3.3 Nominal design life and reference life

As for other constructional materials, the seismic design action is related to a series of parameters. The nominal life V_N of a glass system, as usual, defines the period over which it is assumed that it can be safely used for the intended purposes (with scheduled maintenance). Commonly, V_N= 50 years, but other conditions may occur (Table 2).

Table 2. Definition of design life V_N (pre-failure [1]) for glass structures / elements

V_N (years)	Examples
10	Temporary structures [2]
10-25	Replaceable parts
15-30	Agricultural structures
50	Buildings, common structures
100	Monumental buildings, bridges, other

[1] Post-failure V_N= 10 years for CC1 and CC2; to derive from specific studies for CC3

[2] Excluded structures / parts that can be dismantled / reused

According to Table 2, it is possible to perceive a more detailed distinction of V_N intervals, in the range from 10 to 30 years, and in particular a specific distinction of "*temporary*" and "*replaceable*" components. Given the typical applications of glass in buildings, such a definition fills the gap of NTC2018, where recommended V_N values are (minimum) 10, 50 and 100 years for construction types from 1 to 3. The first one includes "*temporary*" and "*provisional*" systems, while types 2 and 3 are for constructions with "*ordinary*" and "*exceptional*" performances. From the NTC2018 classification, moreover, structures that can be dismantled and reused cannot fall within type 1.

With reference to the consequences of interruption of service or ultimate failure, structural glass systems must satisfy specific demands that are strictly related to the *importance class* of the assembly / building they belong. Basically, these classes agree with NTC definitions, where:

- Class I= for occasional presence of people or agricultural buildings
- Class II= normal crowd levels or factories, without essential public / social functions
- Class III= significant crowd levels, and
- Class IV= important public or construction with strategic functions

Table 3. C_U factor as a function of the importance class

	Importance class			
	I	II	III	IV
C_U	0.7	1.0	1.5	2.0

The design seismic action hence depends on the reference life V_R, given by V_N and the class of use (C_U, see Table 3), that is:

$$V_R = V_N \times C_U \tag{1}$$

All the other relevant parameters for the definition of the seismic action – including the return-period T_R – are in line with NTC provisions, and can be found in §4.4.2.

3.4 Performance levels

Given the intrinsic vulnerability and brittleness of glass, compared to other traditional materials, special care is required for the evaluation of the capacity of these systems to accommodate the earthquakes demands. This allows to limit possible risk from partial damage or failure of structural glass elements (including all the possible types of restraints).

According to the conventional definition of Limit States (LS – i.e., Operational (SLO), Damage (SLD), Safeguard of human life (SLV) and Collapse prevention (SLC)), the expected performances for glass systems under seismic loads are reported in Tables 4 and 5.

Table 4. Required performances for structural glass systems under seismic loads (see also Table 5). Subscript= T_R

	Importance class			
LS	I	II	III	IV
SLO	-	-	ND_{45}	ND_{60}
SLD	SD_{35}	SD_{50}	SD_{75}	SD_{100}
SLV	HD_{333}	HD_{475}	HD_{713}	HD_{950}
SLC	-	-	F_{1463}	F_{1950}

Table 5. Definition of performance levels

Performance level		Description
ND	No damage	No damage in glass; no replacement; watertightness preserved
SD	Slight damage	Partial loss of functionality; usable building; no risk for users
HD	Heavy damage	High degree (and cost) of functionality loss; still no risk for users
F	Failure	Severe damage; evidence of failure; risk for users

3.5 Design seismic force and q-behaviour factor

When more detailed methods of analysis are not available, the local seismic verification of a given glass element (part of an assembly / building) can be carried out by taking into account an horizontal force acting in its centre of mass (§4.4.3):

$$F_a = \frac{S_a W_a}{q_a} \quad (2)$$

The CNR guide requires that the out-of-plane response of an individual plate, based on Eq.(2), is performed with an equivalent uniform pressure. In Eq.(2):

W_a is the weight of the element,

q_a the behaviour factor,

S_a the peak acceleration.

The latter is given by:

$$S_a = \frac{a_g}{g} \cdot S \cdot R_a \quad (3)$$

with the magnification factor equal to:

$$R_a = max \begin{cases} \frac{3(1+Z/H)}{1+(1-T_a/T_1)^2} - 0.5 \\ 1 \end{cases} \quad (4)$$

and:

- a_g the peak ground acceleration (rock soil) for the LS of interest;
- g the acceleration of gravity;
- S accounts for soil category and topographical conditions;
- Z the height of centre of gravity of the glass element (from the foundation);
- H the height of the assembly / building (from the foundation);
- T_a the fundamental period of glass;
- T_1 the fundamental period of the full assembly / building, in the direction of interest.

For non-structural constructive elements, NTC2018 provisions agree with Eq.(2), with the exception that the appropriate estimation of S_a and q_a is demanded to specific technical documents.

Actually, the behaviour factor assessment is one of the critical aspects of glass structures in seismic regions (see also (Santarsiero et al. 2019)). Besides the large use of glass in buildings, no recommendations or suggested values are given for q_a. Worth of interest, in this regard, that NTC2018 removed the earlier reference behaviour factor values for non-structural constructive elements. In the earlier version of NTC standards, up to q_a= 2 was in fact suggested for components that could be of interest for glass, that is falling in the group of "*facades*". Such a lack of recommended values unavoidably turns out in verifications for glass that are generally carried out with q_a= 1. On one side, wide safety levels can be preserved for glass, whose damage could certainly have relevant risk for people. On the other hand, the system itself could be overdesigned, even in presence of joins and restraints with relevant dissipation capacity (Santarsiero et al. 2019).

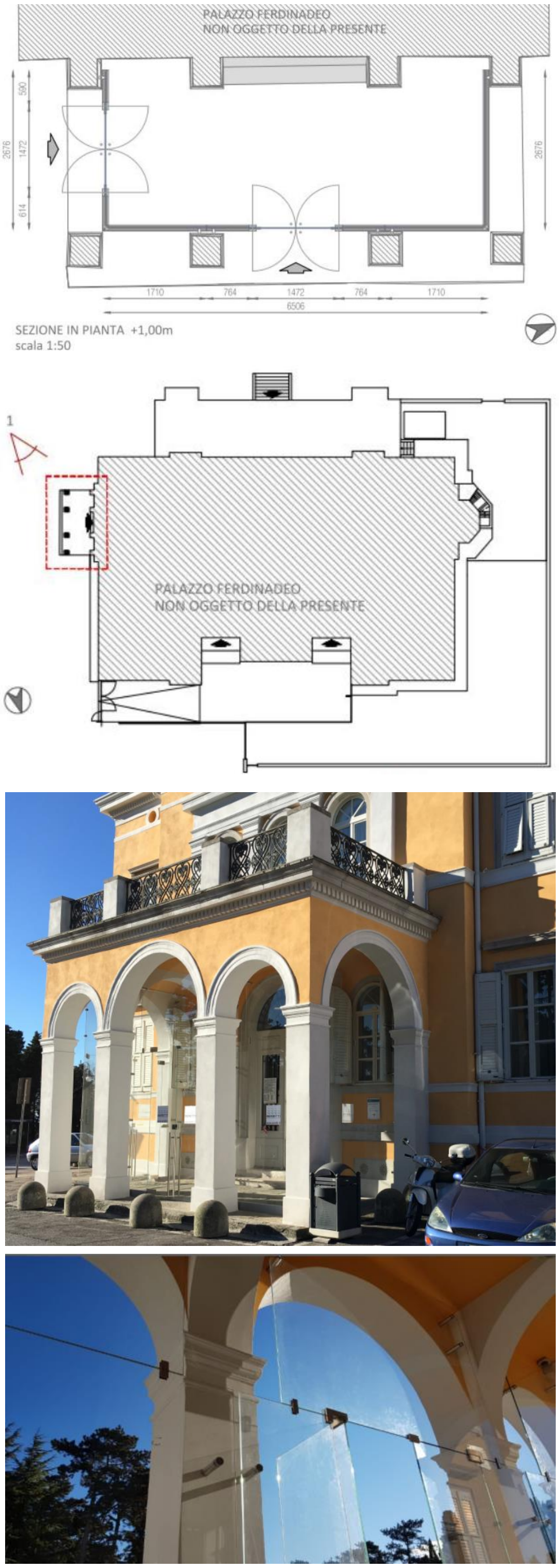
However, it is also important to remind that in some cases, seismic design loads can involve performance demands and effects on glass structures that are relatively low (compared for example to crowd or wind pressures). The choice of the appropriate q_a directly reflects on the displacement demand of glass components and related joints / restraints (§4.4.4).

4 CASE-STUDY EXAMPLE: GLASS PARTITION ASSEMBLY

4.1 Description of the system

The system object of analysis consists of a glass partition assembly composed of laminated glass (LG) walls with metal point fixings. The partition walls were designed in 2018, to take place in an existing building, Italy, so as to protect one of the entrances of the construction from wind and rain (Figure 3). Compared to the primary building, such a glass assembly represents a secondary part and does not modify the global resistance and stiffness of the construction (see Figures 3(a) and (b)). Otherwise, the glazed partition requires careful consideration for design, given the typical glass vulnerability of glass, the class of use of the main construction, the location (and corresponding design loads) and some other constructional requirements.

The partition walls are part of the Ferdinandeo Palace in Trieste. The construction is one of the most prestigious historic buildings of the city (erected in 1858 in honour of the Emperor Ferdinand I of Habsburg), and since 1999 hosts the MIB School of Management.



(a)

(b)

(c)

(d)

Figure 3. Glass partition assembly: (a) plan view of the partition (dimensions in meters) and (b) the building (with evidence of the region of intervention), with (c)-(d) final result (drawings and photo by A. Danelutti).

4.2 *Reference design parameters, major requirements and loads*

Given the value of the building, the Palace and all the related components / interventions are under the supervision of the Italian Ministry for Cultural Heritage (MIBAC). For the case-study assembly, the strict request was to minimize the use of bracings for glass panels, as well as to realize a stand-alone, transparent, 3D partition system with a minimum of metal restraints between glass members and the Palace. The same 3D assembly was hence required to be rigidly reversible, without any kind of permanent damage for the primary structure. Such a series of considerations was the basic condition for all the structural demands to satisfy.

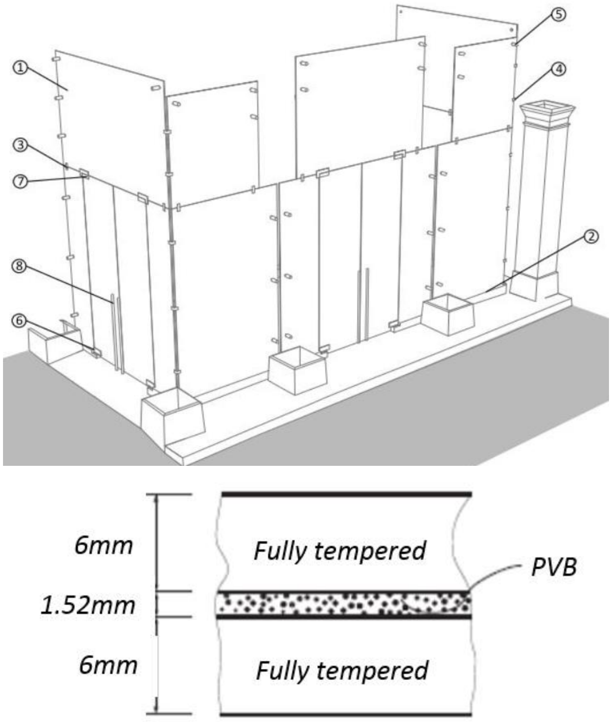
From a structural point of view, the glass partition system is in fact part of an historical building falling in Class III, with V_N= 50 years. Based on the design assumptions described in section 4.1.3, dead loads are only partly sustained by the metal fixings in elevation for the 3D assembly of Figures 3(c)-(d), while a large amount is directly transferred to the base foundation system (see also Figure 5). The main advantage of such a design assumption was that stress peaks in the region of elevation fixings can be minimized, and metal joints were mainly required to brace the 3D glazed system against horizontal loads.

From the entrance in evidence in Figure 3(b), officers (up to 5-6 units) can access the Palace (i.e., CAT. B1 destination from NTC2018). In addition, the building entrance acts as one of the emergency exits. Such a detail can be accounted in the form of an accidental pressure $q_{k,crowd}$= 2kN/m^2. Regarding other accidental loads, the reference design wind velocity at sea level is certainly of interest, and for Trieste Province is in the order of 30m/s. This turns out in a characteristic pressure on glass walls up to $q_{k,wind}$= 1kN/m^2.

Given such a series of conditions, the seismic analysis of glass walls and metal connectors can be carried out according with Eq.(2) – see section 4.1.4. Additional relevant structural verifications (omitted in this paper) are then related to the buckling resistance of the glass assembly, and should be also separately assessed.

4.3 *Design concept*

In order to overcome the input requirements from section 4.1.2, the final choice of the design process resulted in a series of laminated glass (LG) panels with metal point fixings.



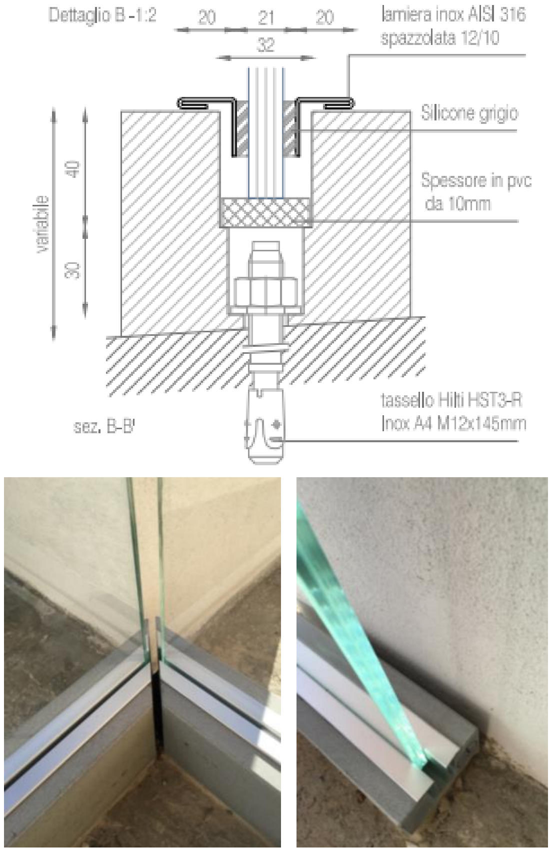
(a)

(b)

Figure 4. Glass partition. (a) Axonometric view, with: 1= LG panels; 2= foundation system; 3-4= friction clamps; 6-7-8= door fixings (drawing by A. Danelutti). (b) LG section.

In Figure 4(a), a schematic view of the assembly is shown, with evidence of point connectors. Globally, the constructed system involves five partition walls, with nominal span in the range from 1.68m to 2.65m. For these walls, the top height (in the range 4.3m-4.65m) is obtained by means of two glass panels in the elevation. Two movable glass doors are also part of the 3D assembly. In total, ≈50 square meters are covered by glass. In order to withstand the reference design loads, the resisting sandwich section of each LG panel was composed of two, 6mm thick fully tempered layers and a 1.52mm thick PVB® foil (Figure 4(b)). The base support was designed in the form of a linear clamp restraint. Bespoke stone rails were used, to ensure a stable foundation system for each glass wall, and at the same time minimize the impact of the 3D assembly, with respect to the Ferdinandeo Palace (see Figure 5).

These rails were properly designed with different global height, so as to accommodate the geometry of the Palace foundation (i.e., irregular slopes and other geometrical defects). In this manner, all the glass plates were allocated in 40mm deep slots (with appropriate sealant joints and gaskets, to avoid stress peaks). Hilti® connectors provided then a rigid link for the foundation system and the building.



(a)

(b)

Figure 5. Foundation system. (a) Cross-sectional drawing and (b) detail views (drawing and photos by A. Danelutti).

An additional set of metal connectors was used to erect the partition walls and provide an appropriate restraint to the glass panels, in their elevation. Point fixings composed of AISI 304 (EN 1.4301) and AISI 316 (EN 1.4436) steel types were used.

Some of them (see Figure 6(a)) were realized in the form of bespoke joints agreeing with DP-44-100 devices from Metalglas®, so as to connect the glass panels and the columns of the Palace. These joints consisted of a central M12 bolt, and a resisting steel solid section (with 45mm the nominal diameter) for the main body and the head of the devices. The total length of these devices (up to 180mm) was properly defined for each one of them, in order to accommodate the actual distance between the glass walls and the adjacent columns. Holes with a diameter of 22mm were also accounted in the LG panels, to ensure a certain gap with the M12 bolt and prevent stress peaks due to design loads.

In some other cases – i.e., in the region of corners (see Figure 6(b)) – steel friction clamps were used to link together two adjacent panels, without any connection with the building (V-083-90N type, Metalglas®).

(a)

(b)

(c)

Figure 6. Elevation restraints. (a) Glass-to-building connectors and (b) glass-to-glass friction clamps for corners or (c) overlapping panels (photos by A. Danelutti and F. Trevisan).

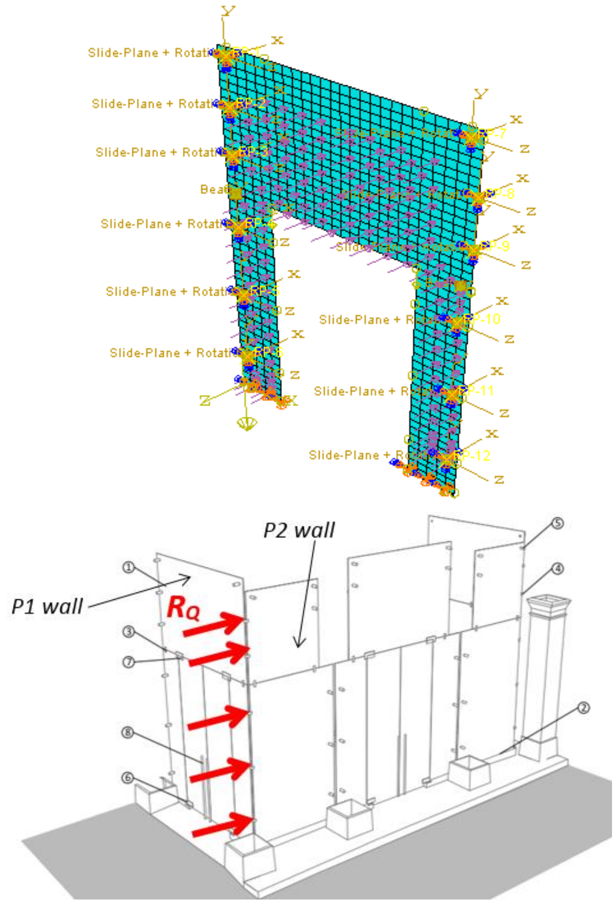
Finally, overlapping panels were connected via V-083-180N joints (Metalglas®, Figure 6(c)), that is planar friction clamps kept together by two M10 bolts.

4.4 Seismic analysis

The local analysis of each panel and the overall 3D glazed system was carried out assuming that q_a= 1 and S= 1.2 in Eqs.(2)-(3), and accounting that W_a≈ 250Kg for the glass panel of Figure 4 with maximum dimensions. Given the peak accelerations summarized in Table 6 for each LS of interest (ground type B, category of soil T2), a maximum seismic force F_a was hence expected for local analyses. Such a series of F_A values resulted in equivalent uniform pressures Q_a according with Table 6.

Table 6. Definition of the design seismic force (horizontal component – example values for the glass wall with W_a≈ 250Kg and A_a= 7.8m^2)

LS	$a_{g,max}$ [g]	S_a [g]	F_a [kN]	Q_a [kN/m^2]
SLO	0.128	0.154	0.385	0.049
SLD	0.167	0.201	0.503	0.064
SLV	0.442	0.531	1.328	0.170
SLC	0.546	0.655	1.638	0.210



(a)

(b)

Figure 7. Numerical modelling. (a) Example of the P1 partition wall (ABAQUS/Standard) and (b) reaction forces that the P1 wall transfers to the orthogonal members (for the in-plane analysis).

In order to properly estimate the stress and displacement demand of the design seismic loads on the glass partition walls, Finite Element (FE) numerical simulations were carried out in ABAQUS/Standard (Simulia 2019). In doing so, glass and PVB were described in the form of linear elastic materials, with nominal mechanical properties. The nominal geometry of each partition component was reproduced in ABAQUS based on the available technical drawings.

The exception was represented by glass doors, that were taken into account via equivalent nodal loads for the adjacent glass panels (depending on the joints in use). Special care was spent for the mechanical description of metal connectors. Based on Figure 6, kinematic constraints and connector sections were properly defined, so as to reproduce the actual behaviour of the steel joints in use, and the restraint effect for the involved portions of glass (including also the effect of gaps). A set of static analyses was hence performed for separate walls, where the out-of-plane seismic effects were assessed based on the equivalent pressure from Eq.(2), i.e. Figure 7(a). Demands from in-plane seismic forces were also verified with the support of FE simulations, by accounting for the total effect due to in-plane forces according to Table 6 and transferred through the point fixing connections from the orthogonal walls (see for example Figure 7(b)).

4.5 Seismic verification

The goal of the seismic analysis of the partition walls is to assess that maximum stresses in glass do not exceed the material resistance at the LSs of interest, and that the system is able to accommodate the required displacements. Special care should be spent in the region of holes, due to the occurrence of potential stress peaks.

For example, assuming that the glass walls could be subjected to out-of-plane seismic loads only, major stress concentrations close to the metal restraints should be compared to the design resistance of tempered glass, where (CNR §7.4):

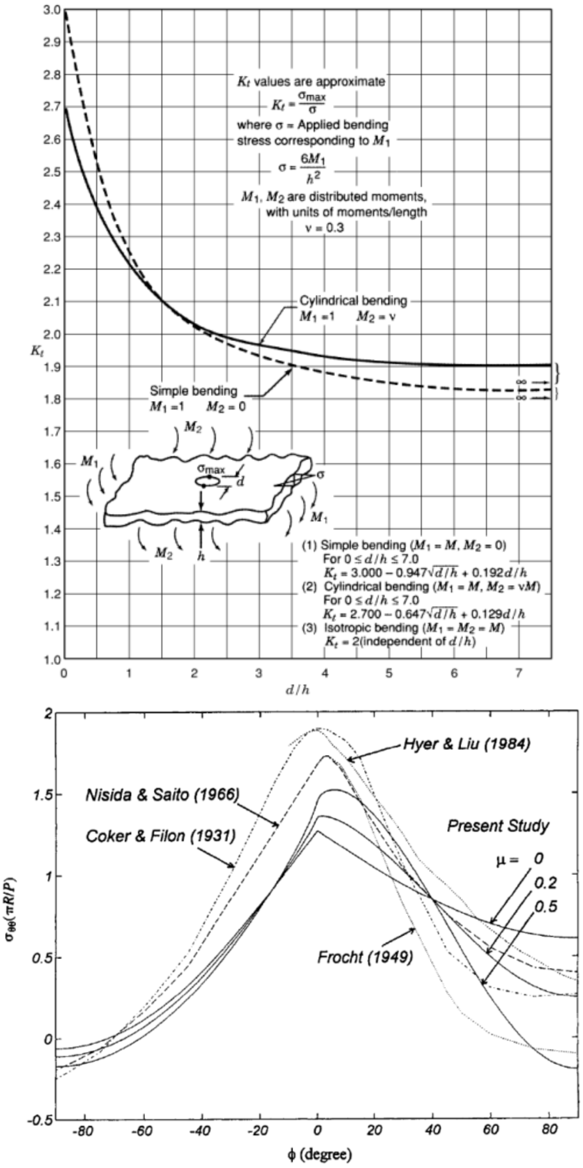
$$f_{g;d} = \frac{k_{mod} k_{ed} k_{sf} \lambda_{gA} \lambda_{gl} f_{g;k}}{R_M \gamma_M} + \frac{k'_{ed} k_v \left(f_{b:k} - f_{g;k}\right)}{R_{M;v} \gamma_{M;v}} \tag{5}$$

and the limit condition for the region of holes is:

$$\sigma_{max} = \sigma_{FE} K \leq f_{g;d} \tag{6}$$

The coefficients of Eq.(5) are better described in the CNR guide (§7.4), but for the examined fully tempered panels give a "near holes" design resistance for short term loads (k_{mod}) of ≈75MPa. The so calculated value must be assessed in Eq.(6), where σ_{FE} is the actual stress estimate (i.e., Figure 8) and K the stress concentration factor.

For glass plates in bending, K follows Figure 8(a), and for the examined system ($d/h \approx$ 1.83) gives $K \approx$ 2.05. Similar considerations (with stress amplifications in the order of ≈1.9-2 times the FE value) must be taken into account for glass panels under in-plane seismic loads (Figure 8(b), ϕ=0°).



(a)
(b)

Figure 8. Stress peaks in the region of glass holes, with evolution of (a) concentration factor for plates in bending (as a function of the hole diameter (d) and glass thickness h), and (b) normalised tensile stress evolution near holes, as a function of the force inclination ϕ (charts reproduced from CNR-DT 210).

The seismic combination of loads, however, requires that the system is analyzed under the effects of dead (G), seismic (E) and accidental loads (Q_{kj}), that is:

$$F_{d,E} = G + E + \sum_j \Psi_{2j} Q_{kj} \tag{7}$$

with Ψ_{2j}= 0 for wind and Ψ_{2j}= 0.3 for CAT. B1 offices. In this study, Eq.(7) turns out in seismic forces that are sensitively lower than the combined crowd load (0.3×2= 0.6kN/m^2).

In the latter case, it is still convenient to estimate maximum stresses (and displacements) in glass based on separate FE analyses and amplify them (for holes) in Eq.(6). In Figure 9(a), a typical stress distribution is proposed for the P1 wall under out-of-plane seismic pressure.

Given the simultaneous presence of multiple actions with a specific characteristic duration, in particular, the combined stress effect must then satisfy the condition:

$$\sum_{i=1}^{n} \frac{\sigma_{max}^{i}}{f_{g;d}^{i}} \leq 1 \qquad (8)$$

where $f_{g;d}$ for the i-th action is estimated with the corresponding k_{mod} reduction coefficient (Eq.(5)). In this study, based on the typology and destination of the glass partition, it is verified that glass does not crack during SLC seismic events (Table 6).

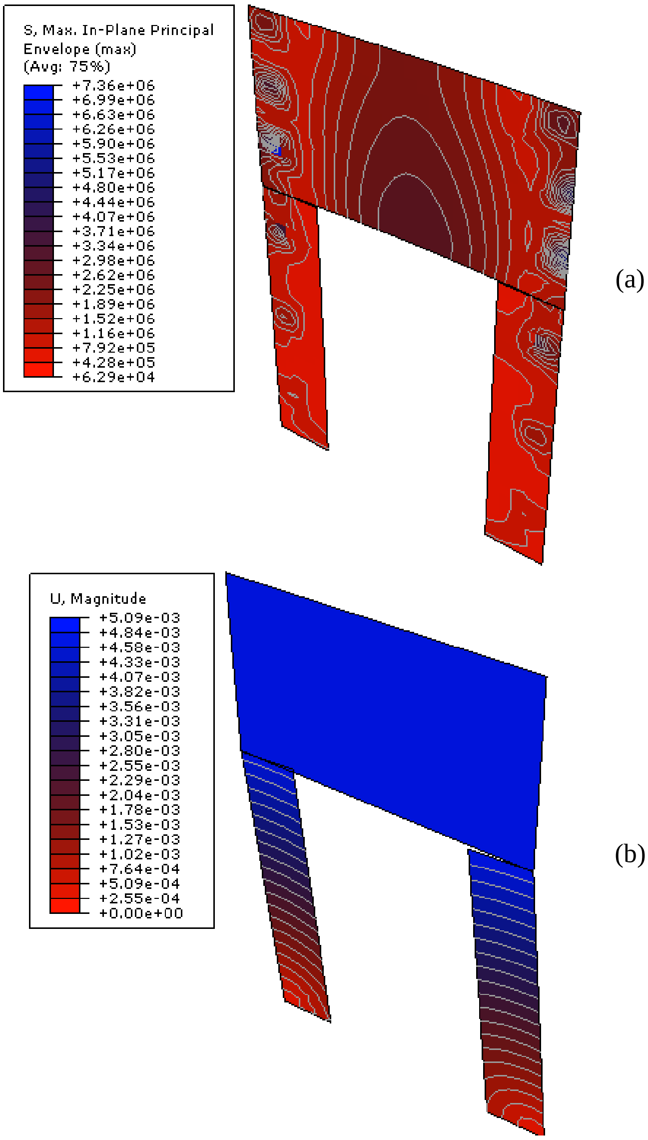


Figure 9. P1 wall (ABAQUS/Standard): examples of expected (a) tensile stresses due to out-of-plane seismic pressure (SLC) and (b) displacements (×50) deriving from in-plane SLD seismic forces (legend values in Pa and m).

Maximum deformations demands – given by the sum of displacement contributions due to the i-th action – must be also in line with the capacity of the system (see for example Figure 9(b)).

Such a displacement demand must be checked towards the limit inter-storey drift for the Ferdinandeo Palace. Given the lack of detailed data and more detailed analyses, a reference SLD drift equal to $0.002H$ = 9.3mm can be taken into account (with 0.002 for ordinary masonry buildings and H= 4.65m the maximum height of the examined glass partition walls). At the SLC, the drift of the wall is still governed by steel connectors in use. Thanks to the gap between the M12 bolts (Figure 6(a)) and the panel holes, a certain accommodation of deformations can be ensured to the system, before the contact activation of steel joints could involve stress peaks in glass.

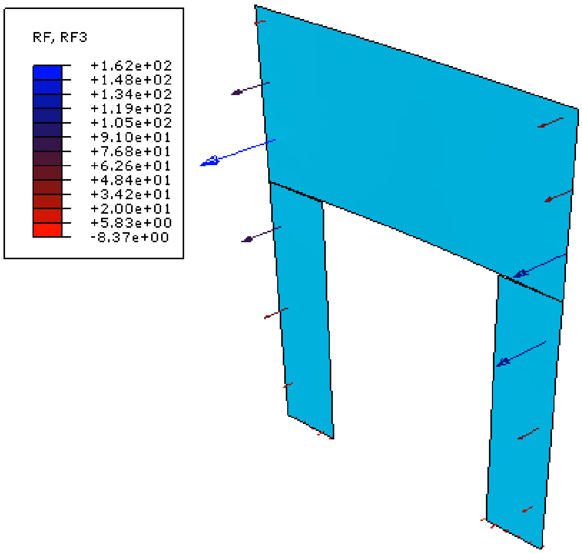


Figure 10. Example of reaction forces in the connectors of P1 wall under out-of-plane SLC seismic pressure (ABAQUS/Standard, legend values in N).

The metal connectors in use, finally, must be verified and assessed towards the maximum forces that they are expected to sustain. For the examined system, for example, the maximum SLC shear forces that the steel devices must sustain are mostly negligible, due to the limited amount of the in-plane seismic forces. Special care is finally required for the effects of out-of-plane design loads. In Figure 10, for example, reaction forces that the steel joints of Figure 6 (wall P1) are required to withstand under the SLC out-of-plane seismic pressure. The potential fracture of both glass (contact regions) and steel joints must be properly verified, and this can be carried out with the support of experimental tests.

An example is shown in Figure 11, where the tensile fracture mechanism is proposed for the steel point fixings with passing M12 bolts (glass-to-column connections). The test example is part of a series of experiments carried out at the University of Trieste, Department of Engineering and Architecture, in support of design. While the presence of a M12 bolt section would reflect in a relatively high tensile resistance of the net surface, the experiments gave evidence of a potential failure of joints in the region of head-to-bolt connection. Also in the latter case, the test failure was observed for a relatively high tensile load (≈54kN), compared to the design stresses for the glass partition assembly. Otherwise, the robustness and redundancy of the overall system should be properly verified at different levels of analysis.

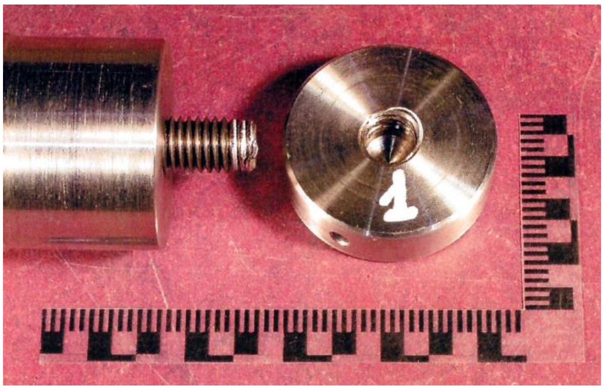


Figure 11. Tensile fracture (uniaxial testing, with collapse at 54.2kN) of a steel point fixing according with Figure 6(a) – photo by F. Trevisan.

5 CONCLUSIONS

In this paper, the design strategies for glass systems under seismic loads were discussed, by taking into account existing standards and guideline documents for earthquake resistant structures.

A special focus was given to the Italian scenario for glass designers (i.e., NTC2018 and CNR-DT 210/2013), as well as the typology of frameless glass systems. There, safe performances must be ensured for glazed assemblies in which the metal connectors and restraints are reduced to a minimum, and the potential risks for a typically fragile and vulnerable material are further enforced. This is the case of ordinary design loads and even more extreme events like earthquakes.

A case study system was then presented, giving evidence of the actual approaches and methods for the analysis and verification of these vulnerable structures.

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