VULNERABILITY ASSESSMENT AND DYNAMIC CHARACTERISATION OF A GLASS FOOTBRIDGE: ON-SITE VIBRATION TESTS AND FE NUMERICAL MODELLING

C. Bedon
University of Trieste, Department of Engineering and Architecture, Trieste, Italy

Introduction. In the last decades, the use of glass as a load-bearing material showed an exponential increase, both on the side of projects and research studies. Although it represents a relatively new construction material, requiring appropriate design methods and knowledge, glass is largely used for facades, roofs, footbridges. Given a series of intrinsic features, special care should be spent at the design stage, to ensure appropriate *fail-safe* requirements, but also in the life-time of these structures (Haldimann *et al*. 2008). The brittle behaviour and limited tensile resistance of glass, as well as the typical high flexibility of glazing assemblies, represent the major issues. Further critical aspects may derive from time and ambient effects, due to the sensitivity of glass-related materials and components to long-term loads, humidity, fatigue, etc., or extreme loads. The vulnerability assessment of glazing systems under exceptional loads (seismic events, fire, etc.) is hence an open topic, still requiring huge efforts. In this paper, the preliminary dynamic characterisation of an existing glass suspension footbridge is presented. As a case-study, the walkway of the Basilica of Aquileia (UD) is taken into account. On-site vibration experiments are discussed, to estimate the fundamental parameters of the structure. A Finite Element (FE) numerical study is then carried out, to further assess and explore the walkway performance.

Case-study. The Basilica of Aquileia, listed by UNESCO as a World Heritage Site, includes the largest and one of the best preserved early Christian mosaics, and attracts over than 300,000 visitors every year. The suspension walkway object of study was realised in the early 2000, as a key strategy to protect the mosaics and allow for their visibility. The original design concept was aimed at maximising the transparency of the footbridge, hence minimising its impact on the Basilica context. A total of 118 glass panels was used (79 in the central nave and 39 in the crypt). In structural terms, the footbridge was assembled via a glass roof, a set of steel frame members and suspension tendons, and steel-glass handrails (Fig.1(a)). More in detail, each roof panel consists of a triple laminated glass (LG) section, obtained by bonding fully tempered (FT) layers (3×12mm the thickness) via flexible Polyvinyl Butyral films (PVB®, 0.76mm thick). An additional protective layer, composed of annealed (AN) glass (6mm) was also accounted on the LG panels, so as to preserve their integrity under antropic loads. No mechanical connection was considered between the LG-AN layers, however, with contact interactions only. A small,

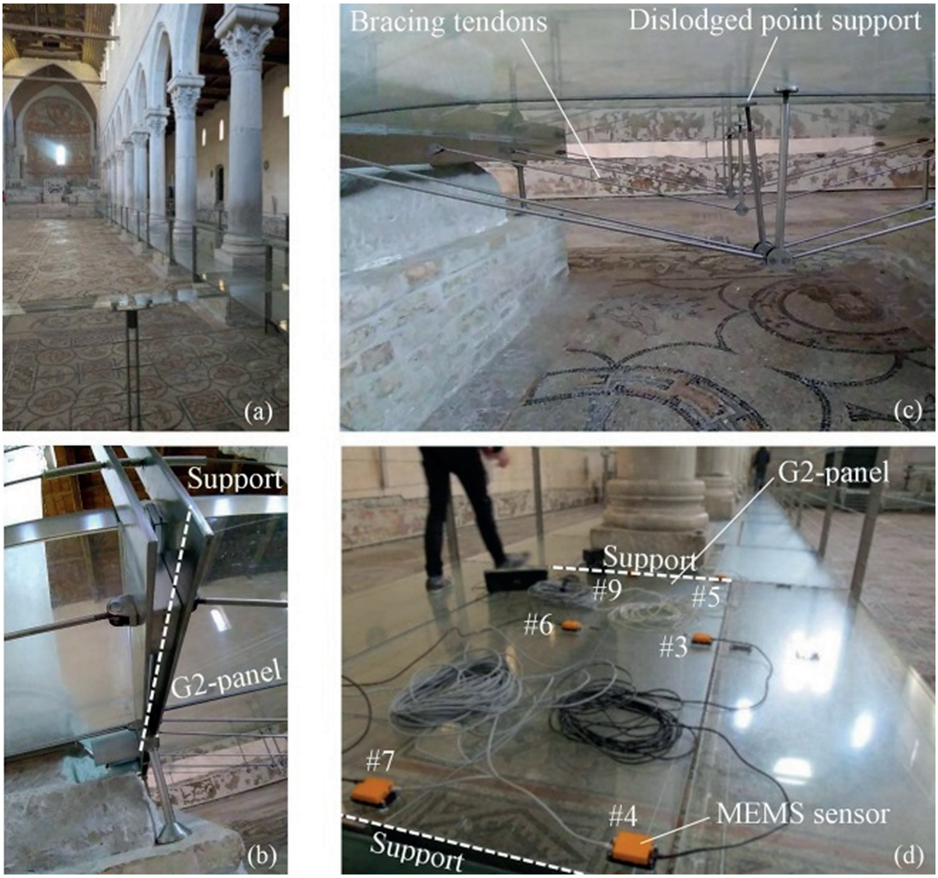

Fig. 1. - Aquileia glass walkway: (a) general view, with (b) edge support and (c) bracing tendons for G2-type panels; (d) test setup.

non-structural metal frame was hence designed along the edges of each glass panel, to keep the AN layers in position. Within the full walkway of the central nave, the LG panels have variable dimensions (depending on their location), spanning from 1.35×1.35m (G1-type), up to 1.45×2.65m (G2-type). While the G1-panels are point supported via mechanical restraints, major structural uncertainties are related to the bending performance of G2-panels, being supported along the short edges and spanning over ≈2.65m (Fig.1(b)). To limit the deformations of the roof, a bracing system was originally planned, with pre-stressed steel tendons (10mm the diameter). The tendons were designed to ideally half the span of G2-panels, but contact mechanical interactions were only provided between the mid-span restraints and the upper glass panels, hence resulting in temporary supports (Fig.1(c)). After the realisation of the walkway, in fact, long-term phenomena partly minimised the benefits of initial pre-stressing loads. Actually, the walkway suffers also for other degradation effects, being related to a combination of time, unfavourable ambient conditions (i.e., high relative humidity) and fatigue phenomena (i.e, continuous dynamic loads due to visitors). A non-destructive experimental campaign was hence planned in late 2017 (Fig.1(d)), so as to assess the effective dynamic characteristics and vulnerability of the structure.

Structural design requirements. Generally speaking, glass roofs must be checked - under service loads - towards maximum deformations and stresses due to permanent and human induced live loads (including snow and wind effects, for outdoor structures), see (CNR-DT 210/2013). In service conditions, vibrations of glass roofs should be also properly limited, being characterised by high flexibility and small thickness-to-size ratios. Even more restrictive design requirements are given by standards for seismic actions, in terms of deformation capacity of the glazing components with respect to the other load-bearing elements. Such a limitation requires careful detailing of connections (that should allow movement accommodation but preserve the redundancy of the system) and a reliable estimation of the dynamic characteristics for the glass assembly to verify (CNR-DT 210/2013).

Experiments. The Experimental Modal Analysis of the Aquileia footbridge was carried out in November 2017 (T=4°, RH=9%). The full set of measurements was performed using the MEMS accelerometers prototyped in (Bedon *et al*. 2018). Given the average size of glass panels, six sensors were used and optimally located on the roof (i.e., #*n* sensors in Fig.1(d)). Output-only tests data were recorded, based on human induced vibrations (Fig.2(a)). More in detail, the experiments were first performed on the walkway panels characterised by

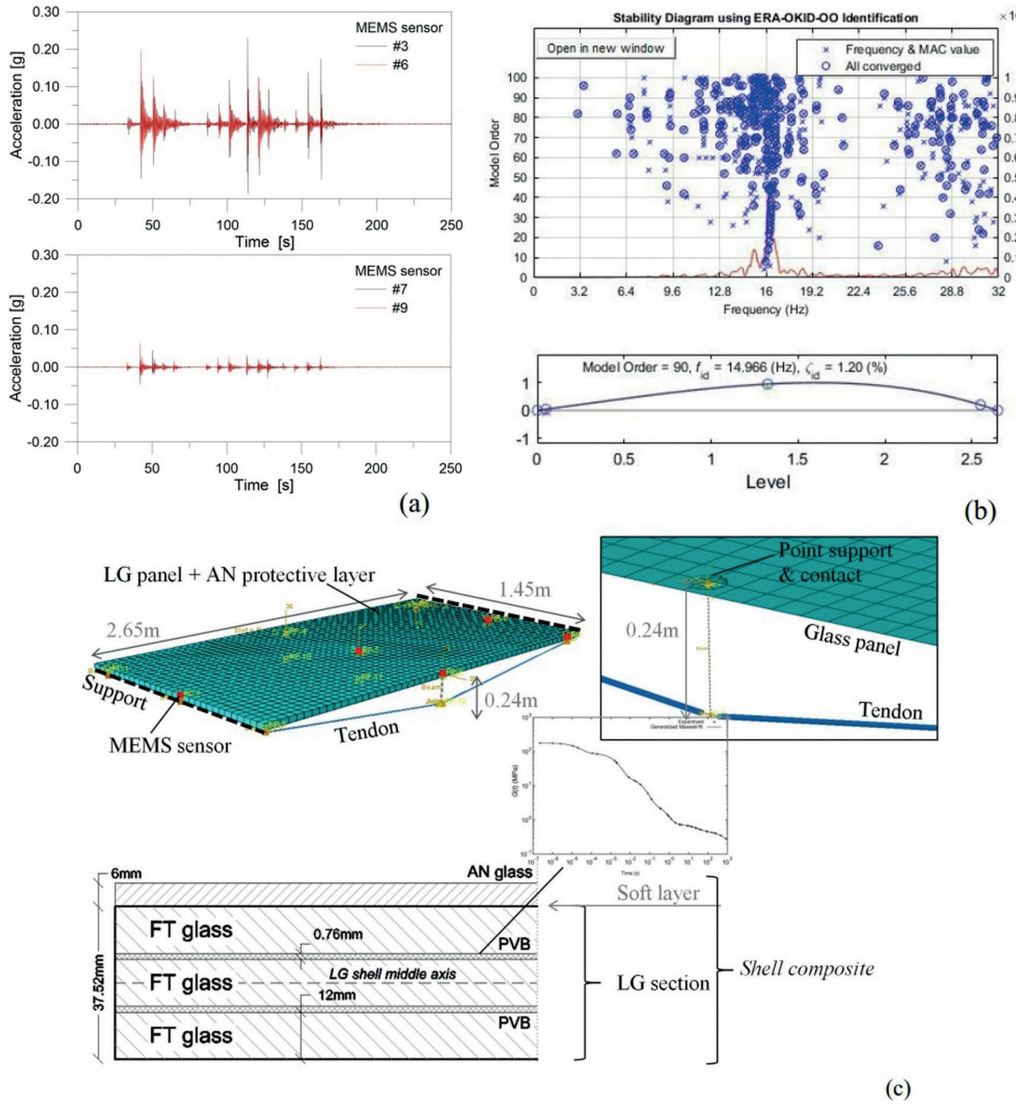

Fig. 2. - Experimental and numerical analysis of the walkway (G2-panel). (a) Vibration test measurements, with (b) fundamental vibration mode of the roof panel (SMIT). (c) FE model of the G2-panel (ABAQUS), axonometry and details.

maximum dimensions, and later on thes panels with severe/visible damage. Preliminary on-site observations carried out during Summer, to qualitatively assess the state-of-the-art of the structure, gave in fact evidence - for some of the roof panels - of (d_1) surface abrasion and minor glass cracks; (d_2) condensation and debonding phenomena (especially at the LG-to-AN interface); and (d_3) dislodgement and pre-stressing loss, for the steel tendons and supports (see Fig.1(c)). All the test measurements were analysed (SMIT), to detect the fundamental mode of the roof (Fig.2(b)). Careful consideration is spent, in this paper, for the 1.45×2.65m G2-panel of the central nave, see Fig.1(d). According to the test setup of Fig.1(d), through the post-processing phase, #4-sensor measurements were disregarded, due to data corruption. The examined glass panel showed a beam-like bending response (Fig.2(b)), with f_{TEST}=14.97Hz the fundamental frequency and ξ_{TEST}=1.20% the estimated damping.

FE numerical study. An extended numerical investigation of the glass walkway was then carried out in ABAQUS, to further assess the on-site measurements (see Fig.2(c)). Major uncertainties on the interpretation of the vibration test data derived from the actual supports and restraints of the glass panels, as well as on the effective material properties (especially PVB). The typical FE numerical model herein discussed was implemented to reproduce the G2-panel of Fig.1(d). 2D *shell composite* elements were used to describe the nominal LG cross-section. Similarly, for the top AN layer, a shear flexible bond was defined on the top surface of the LG sandwich (E_{SOFT}=1MPa its stiffness), to account for a contact mechanical interaction. 1D *beam* elements were used for the steel tendons, accounting for their nominal circular section (10mm the diameter). MEMS sensors were then considered via lumped masses (0.15Kg/each, see Fig.2(c)), while a key role was assigned to the lateral restraints (linear supports for the roof panel) and to the intermediate restraints (for the tendons). In the latter case, see Fig.2(c), an unilateral contact interaction was properly defined, being able to react in presence of compressive loads only, hence allowing free relative displacements and rotations at the steel support-to-LG interface. Pre-stressing effects of tendons were fully disregarded, based on on-site observations. A final uncertainty was represented by the input material properties. While nominal elastic features were reasonably considered for glass (E_g=70GPa, ν_g=0.23, ϱ_g= 2500Kg/m^3) and steel (E_s=160GPa, ν_s=0.3, ϱ_s=7850Kg/m^3, with 1600MPa the resistance at rupture, according to technical drawings of the structure), a tentative value was initially used for the shear stiffness of PVB (G_{PVB}=8MPa and E_{PVB}=24MPa, with ν_{PVB}=0.49, ϱ_{PVB}=1100Kg/m^3). Such a stiffness value is in fact suitable for short-term loads (3s) and room temperature (20°) only, while a certain material degradation was expected for the Aquileia walkway (see for example Fig.2(c) and (CNR-DT 210/2013)).

Results. The dynamic performance of the glass structure was extensively investigated. Parametric FE estimations, in particular, were compared with experimental measurements and preliminary analytical calculations, where (k=1 for the first vibration mode):

$$f_k = \omega_k / 2\pi = \left[(k\pi / L)^2 \sqrt{EI/m} \right] / 2\pi \tag{1}$$

In Eq.(1), m is the linear density of the beam-like element, L the span, $E=E_g$, I the flexural inertia. For the analytical calculations, a 3×12=36mm thick monolithic glass section was roughly considered, in place of the LG+AN sandwich. Within the full set of parametric FE models (Tab.1), the attention was spent for several geometrical and mechanical aspects. The numerical predictions resulted in a fundamental beam-like shape well agreeing with the test observations (Figs.2(b) and 3(a)). Marked variations, otherwise, were observed for vibration frequency, see Fig.3(b). There, the percentage scatter of the calculated frequencies f_x is defined as:

$$\Delta_f = 100 \times (f_x - f_{TEST}) / f_{TEST} \tag{2}$$

Worth of interest in Tab. 1 and Fig. 3(b) is that Eq.(1) does not allow to capture the actual vibration of the roof (i.e., poor description of the LG+AN section, lack of tendons, etc., with Δ_f

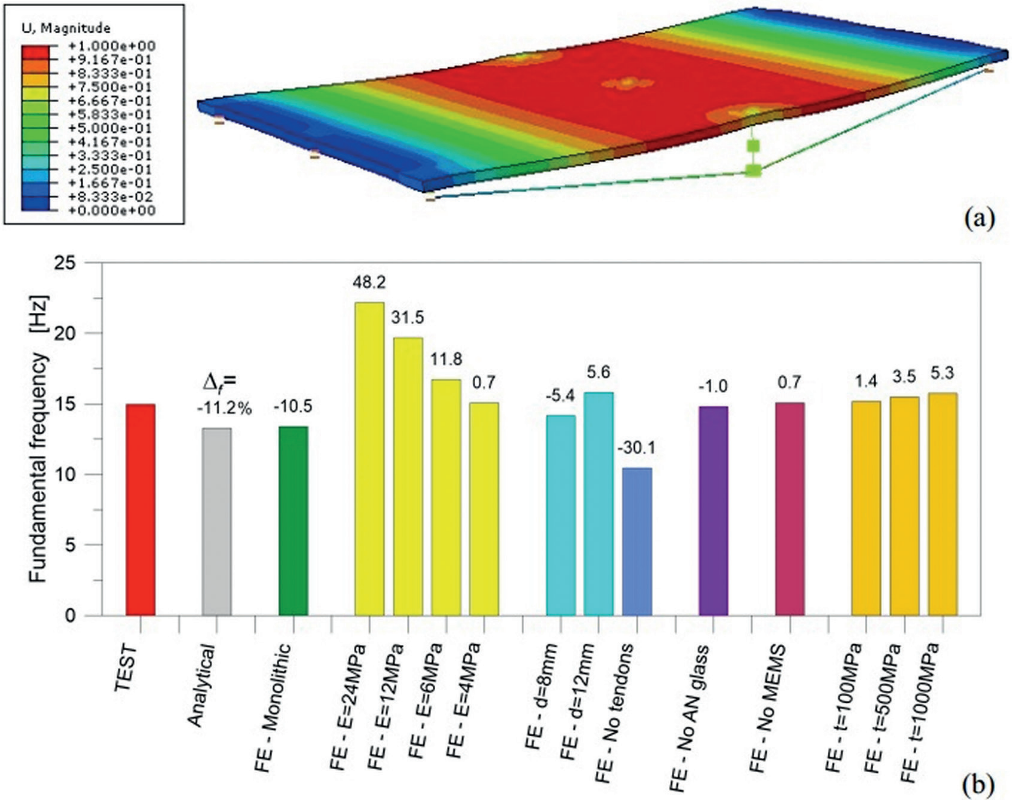

Fig. 3 - Numerical results for the G2-panel (ABAQUS). (a) Normalised fundamental mode (undeformed shape) and (b) calculated frequency (input parameters according to Tab. 1).

Tab. 1 - Reference input features for the experimental, analytical and FE numerical prediction of the fundamental mode of the glass walkway. Key: Y= yes; X= no; ??= not known.

		Glass panel			**MEMS**	**Steel tendons**		
		LG section	E_{PVB} [MPa]	AN layer	Lumped mass [Kg]	Tendons	Diameter d [mm]	Pre-stress t [MPa]
	TEST	Y	??	Y	0.15	Y	10	??
	Analytical (Eq.(1))	X (36mm)	X	X	X	X	X	X
FE model #	Monolithic	X (36mm)	X	X	X	X	X	X
	E=24MPa	Y	24	Y	0.15	Y	10	X
	E=12MPa	Y	12	Y	0.15	Y	10	X
	E=6MPa	Y	6	Y	0.15	Y	10	X
	E=4MPa	**Y**	**4**	**Y**	**0.15**	**Y**	**10**	**X**
	d=8mm	Y	4	Y	0.15	Y	8	X
	d=12mm	Y	4	Y	0.15	Y	12	X
	No tendons	Y	4	Y	0.15	X	X	X
	No AN glass	Y	4	X	0.15	Y	10	X
	No MEMS	Y	4	Y	X	Y	10	X
	t=100MPa	Y	4	Y	0.15	Y	10	100
	t=500MPa	Y	4	Y	0.15	Y	10	500
	t=1000MPa	Y	4	Y	0.15	Y	10	1000

=-11.2%). Conversely, the FE models can describe its actual bending response. Major frequency variations resulted from the PVB stiffness. Compared to the original design configuration (E_{PVB}=24MPa, Δ_f=+48.2%), the optimal rigidity of PVB foils was found in E_{PVB}=4MPa (Δ_f =+0.7%), hence suggesting a weak mechanical connection of the LG layers, and confirming the presence of local debonding/severe material degradation, to properly address. The top AN layer - with limited stiffness and mass - resulted in minimum frequency variations for the roof (Δ_f=-1%, when disregarded). Modifications of tendons size and pre-stress level were indeed associated to ±5% frequency variations. The full removal of bracing tendons from the FE assembly, otherwise, largely underestimated the first frequency of the unrestrained LG+AN roof panel, hence remarking the actual role of all the structural components.

Conclusions. In this paper, a preliminary dynamic characterisation of the Basilica of Aquileia glass walkway was presented, including on on-site vibration tests and refined FE investigations. Based on test measurements, on-site observations and accurate FE description of several key parameters, it was shown that a combination of multiple aspects can markedly affect the modal estimations of the structure, hence requiring careful consideration towards *fail-safe* design performances and vulnerability assessment purposes. The post-processed data, in particular, confirmed the importance of non-destructive diagnostic investigations for retrofitting.

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