About a New Kind of Prestressed Glass-Steel Beams

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Summary

The present paper concerns the development of a new kind of glass beams made by a system of glass elements and prestressed steel cables. The conceptual design idea is to anticipate the cracks subdividing the beam into small modular elements and connecting them together by means of prestress, so that glass is mainly submitted to compressive stresses while tension stresses are adsorbed by the tensioned steel cables.

Until now two prototypes have been constructed and tested at the University of Pisa under dynamics loads and finally brought to collapse under static loads.

Numerical predictions, performed by appropriate FEM modeling, resulted to well approximate experimental results.

Keywords: Structural Ductility; Structural Glass; Prestressed Glass Structures; Chemical Tempering; Fracture Mechanics.

1. Introduction

Ductility is very frequently associated to metallic materials able to develop large plastic deformations while on the contrary fragility is normally associated to glass materials or ceramics.

Nevertheless, some famous and pioneer glass structures have been built even in highly seismic areas where fragile failures must absolutely be avoided (Haus Pavillon in Rheinbach by Ludwig & Weiler, Yurakucho canopy in Tokio by MacFarlane, glass stairs of the Apple Stores in San Francisco by Eckersley O'Callaghan).

Indeed, although glass is fragile and weakly tension resistant, it has a very high compressive strength (about 1000MPa) and may be associated to other ductile materials, eventually also by means of prestress, to form composite structures of high mechanical performances including also ductility.

It is well known that the stress concentrations that occur at the apex of microscopic surface cracks, always present even in untouched specimens, are responsible of the intrinsic fragility of glass [1] and of its relative low tensile strength.

Thermal or chemical tempering processes induce compression stresses at the surface that inhibit cracks initiation and propagation, thus attributing the material apparent higher tensile strength but exerting no influence on fragility [2].

On the contrary, the technique to glue two or more glass panes together by means of plastic sheets (lamination) can give the sandwich structure a post breakage stiffness and bearing capacity that is similar, to some extent, to ductility [3].

Fragility may thus be overcome in glass structures by organizing the components in a hierarchic order and by applying of redundancy principles [4].

2. The structural conception of a prestressed composite glass-steel beam

When a glass beam is submitted to flexural tests, it cracks at a certain load stage developing a characteristic crack pattern. To avoid the crack initiation and propagation process, it was thought to anticipate it by subdividing the glass surface in many equilateral triangle panels and connecting them by means of prestressed steel cables. Each panel is composed by two 5mm thick glass panes chemically tempered [5] and laminated together by means of a 1.52 mm thick PVB interlayer.

The panels are disposed in series as in a reticulated Warren beam (Figure 1) and jointed together at their vertexes by means of stainless steel knots (Figure 2). The vertexes of the glass panels have been rounded and a aluminium sheet has been interposed between steel and glass in order to attenuate contact stresses.

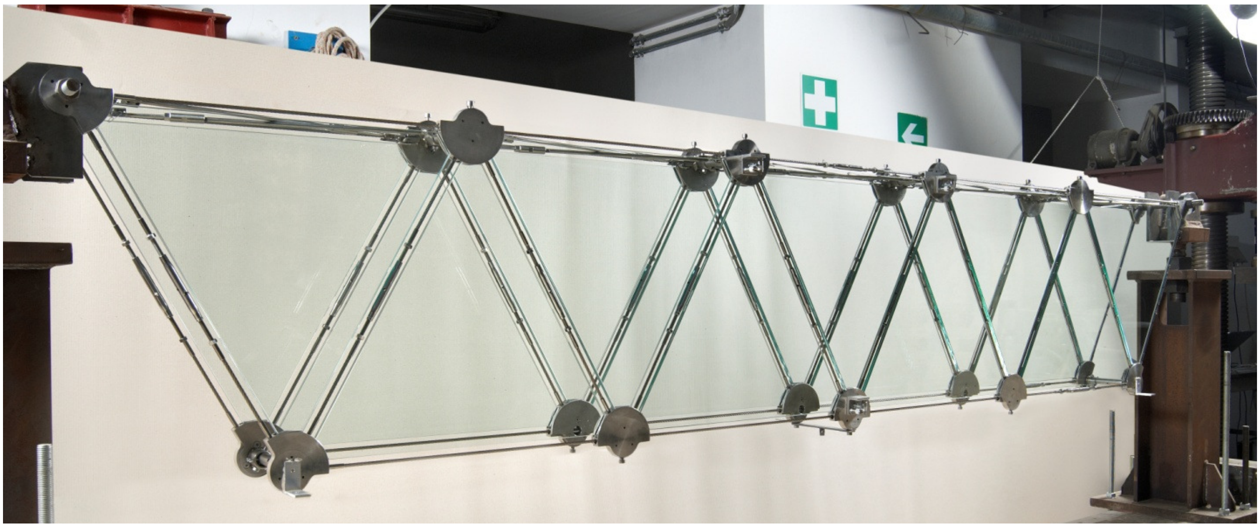

Fig.1 The prototipe beam TVTβ

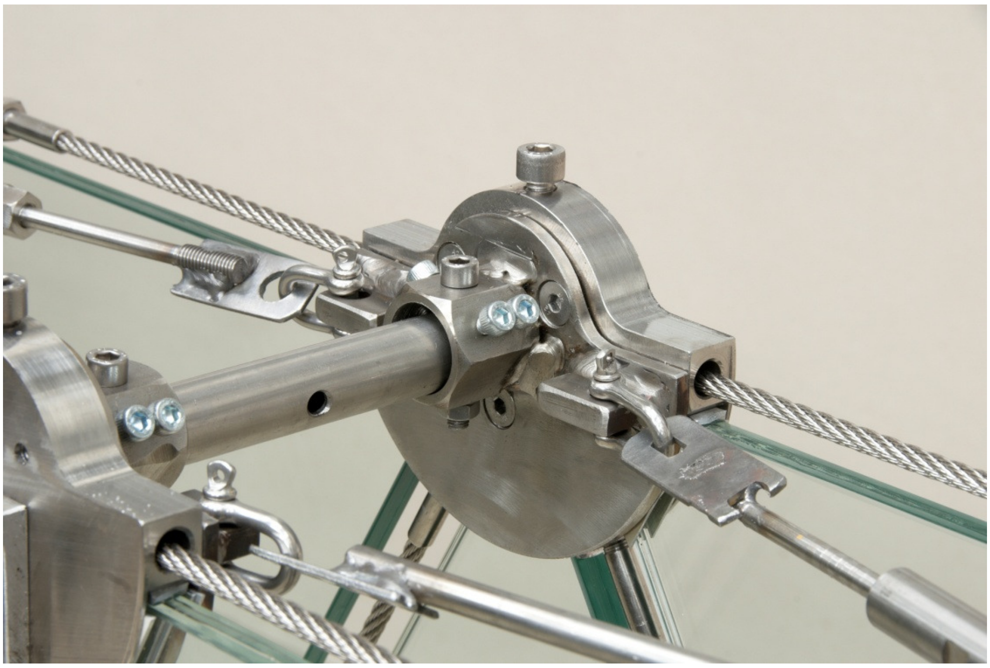

Fig.2 Steel knot

The steel knots are mutually connected by means of prestress steel cables equipped by a screw tightner so that prestress forces can be developed and a quasi isotropic distribution of compression stresses can be induced in the glass panels (Phase “0”, Figure 3).

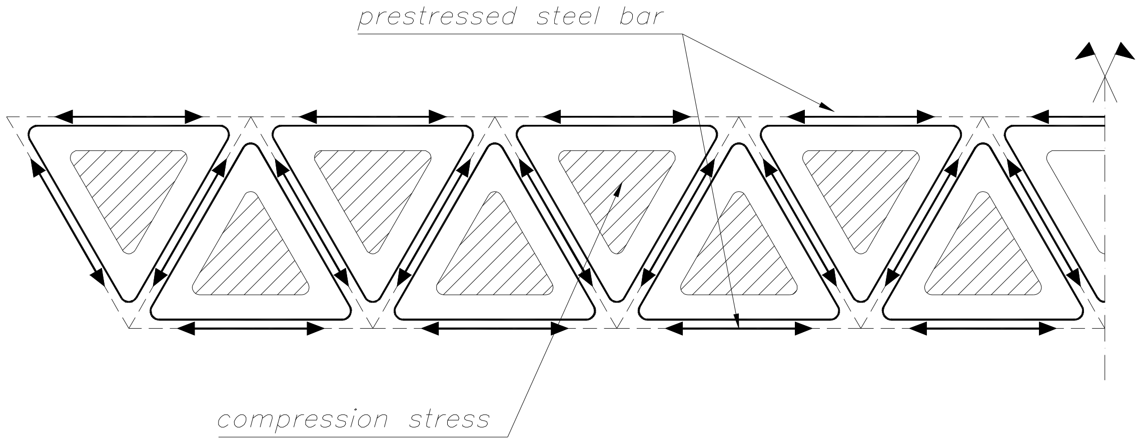

Fig.3 Phase “0”

The first prototype, named TVTα, had a length of 2970 mm and a height of 285 mm. It was realized with two parallel orders of glass panels put at a mutual distance of 5 mm. The connecting rods were metallic bars with a diameter of 6 mm which resulted in phase of assemblage difficult to be prestressed. The experimental results evidenced a low flexional and torsional stiffness of the beam.

Therefore a new prototype beam, TVTβ, was prepared having a length of 3300 mm, and a height of 572 mm. In order to satisfy the redundancy principle, the system of glass panels and steel cables have been doubled, at distance of 174 mm, so that the entire beam consists in a parallel arrangement of two identical systems connected in the upper part by a braced horizontal truss. Aim of this system was to increase the torsional stiffness of the beam.

3. Qualitative structural behaviour

The structural behaviour of TVT beams is analogous to that of a segmental prestressed concrete beam. After phase “0” of pure prestressing, under the flexural action of dead loads and service loads, tension stresses are adsorbed by the steel bars while the compression stresses in the lower parts of the glass panel gradually diminish until a limit state of decompression is reached in the central part of the beam. By increasing the external loads the decompression propagates towards the two supports. This stage has been denoted as Phase “1” - Decompression phase.

Since the steel knots exert just a contact, unilateral restraint, the decompressed vertexes of the glass panels cannot develop tension stresses and simply move a little quantity from their supports. The static scheme of the beam changes thus into that sketched in Figure 4 where flexural and shear tension forces are taken respectively by the lower steel bars and one order of the diagonal steel bars. Compression fluxes are on the contrary adsorbed by the upper parts of the glass panels and by the diagonal parts of the other order. Compressed steel cables are gradually de-tensioned until the prestress load is compensated and, further, they buckle. After buckling, each cable cannot react and belongs no more to the static scheme. This limit state has been denoted as “Phase 2” – Buckling phase.

After Phase “2” has been reached, a further load increasing traduces into a augmentation of stress compressions in the glass parts and tension stresses in the steel rods. The dimensioning of the component parts of the beam can be performed so that the final Phase”3” – Collapse takes place due to the yielding of the steel cables and not because of glass rupture in compression, thus attaining a ductile collapse accompanied by large displacements.

Depending on the beam height, and therefore on the slenderness of the steel cables, Phase 2 may occur after phase 3, as indeed happened in the prototype beam, and illustrated hereafter.

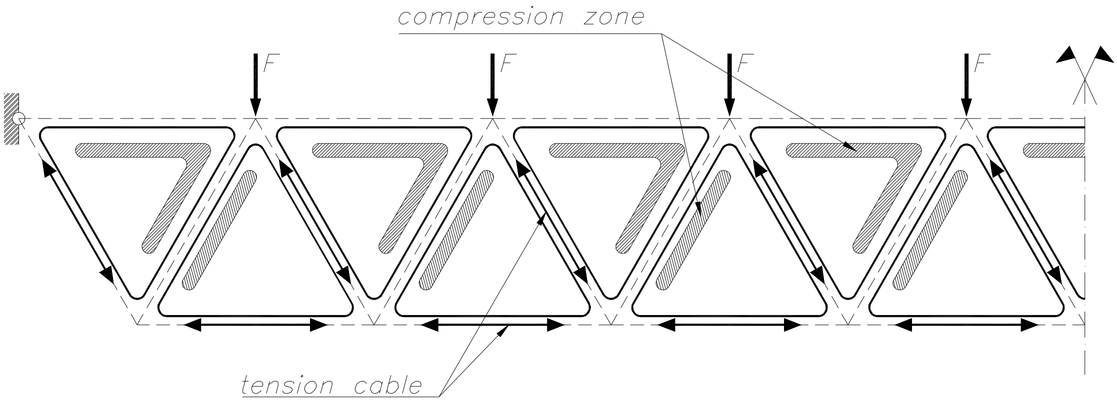

Fig. 4 Phase “2”

4. Numerical modelling

Many FEM (Fig. 5) models have been developed to predict the numerous aspects of the structural behaviour of the beam and to adjust its preliminary dimensioning.

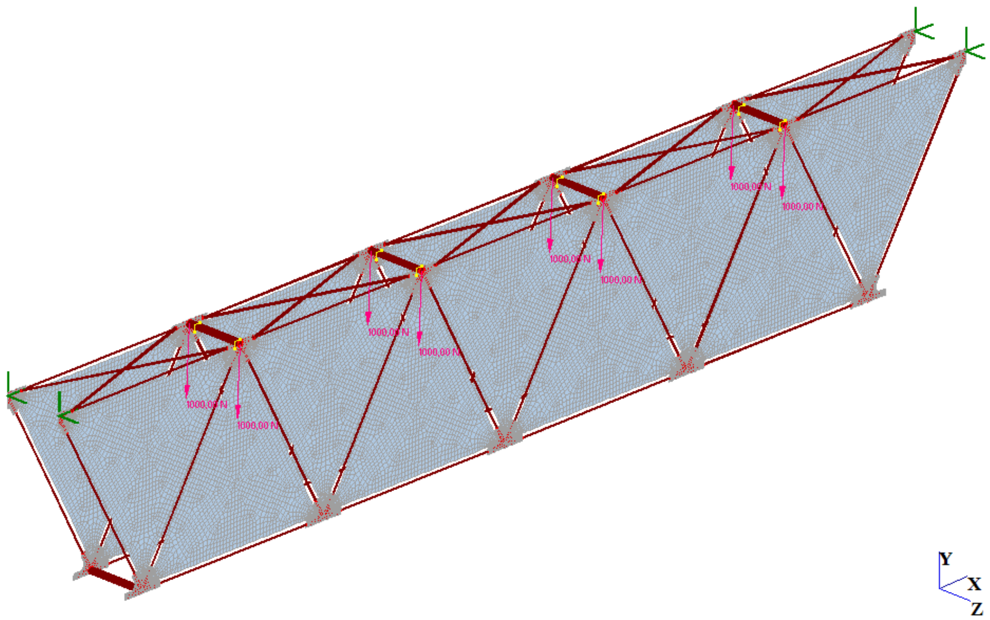

Fig. 5 Global model of TVTβ

The glass panels have been modelled by shell elements and for the steel cables truss elements able to react only in tension stresses. The implemented constitutive laws of the two materials have been deduced by available literature data [6]. Glass has been schematized has a linear elastic material and since stainless steel has been used for the cables, a linear hardening branch has been introduced.

Particular care was devoted to the modelling of the contact parts between glass panels and steel knots by using a set of *Point Contact* elements able to react just to compression stresses. The transversal stiffness of the joint has been investigate by a three dimensional local model with solid elements (figure 6).

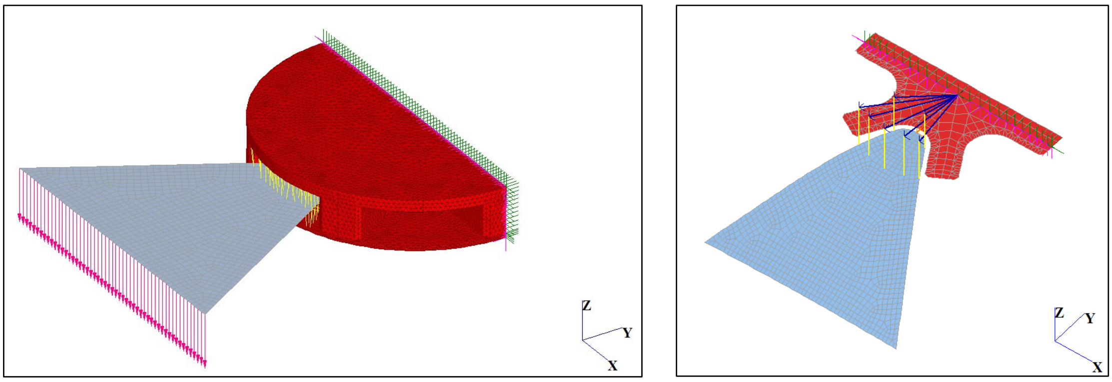

Fig. 6 Joint model

The kind of performed analysis are the followings:

- non linear geometrical analysis (2D model) to evaluate the effect of prestress steel cables on the flexural stiffness of the beam;
- local buckling analysis to evaluate the effect of prestress steel cables on the single glass pane;
- local non linear geometrical analysis to evaluate the transversal stiffness of the joint;
- non linear geometrical analysis (3D model) to evaluate the torsion stiffness of the beam;

The numerical analyses confirm the qualitative structural behaviour since although a decompression phase (*phase* 1) and a yielding phase (*phase* 3) can be clearly recognized, the buckling phase of steel cable (phase 2) doesn't influence in meaningful way the flexural stiffness of the beam. In figure 7 it is possible to see the compression stresses in the upper sides of the glass panels induced by bending moments and the compression stress in the inclined sides induced by shear loads.

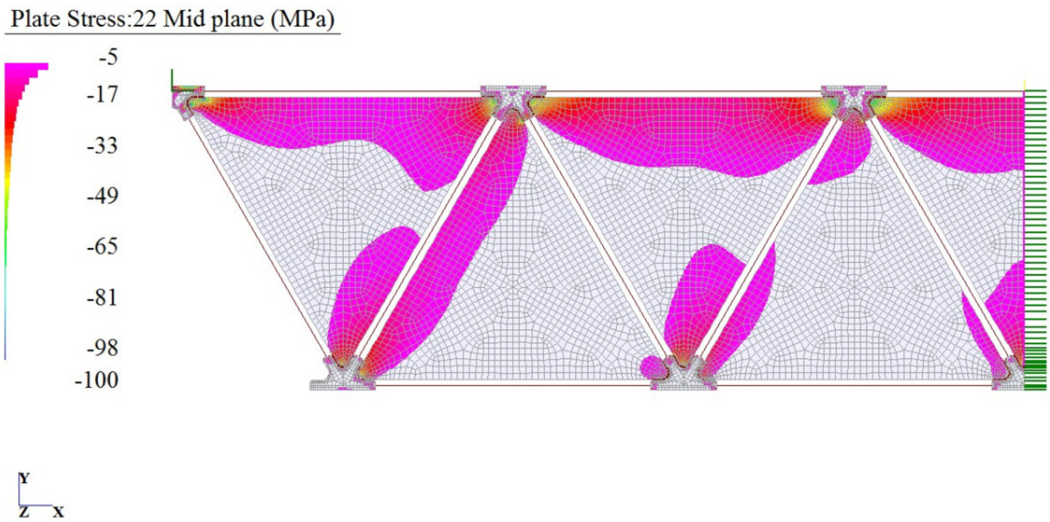

Fig. 7 Principal stress (compression)

Figure 8 shows the load factor versus displacement of middle point of the beam for different levels (from 2 kN to 12 kN) of prestress load in the steel cables N_p. A first stiffness reduction associated to the decompression of the lower part of the beam (*phase 1*). By increasing the prestress level the intensity of the external load corresponding to the decompression phase increases. The second step of stiffness decay is associated to the yielding of the lower steel cable but of course the Ultimate Limit Load results independent on N_p. These results confirm a very well established principle that applies in every prestressed structure, that is, the prestress level controls only Serviceability Limit States but non Ultimate Limit States.

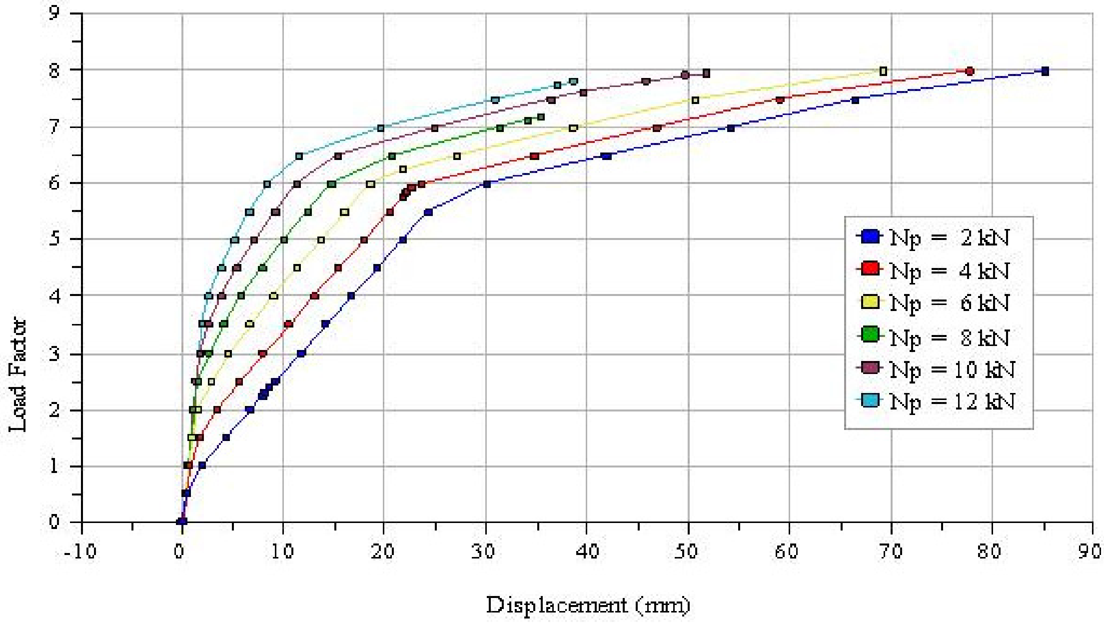

Fig. 8 Load Factor Vs vertical displacement

Figure 9 shows the influence of N_p on the axial force in the lower cable of the beam, and how the Ultimate Load is independent on the value of prestress.

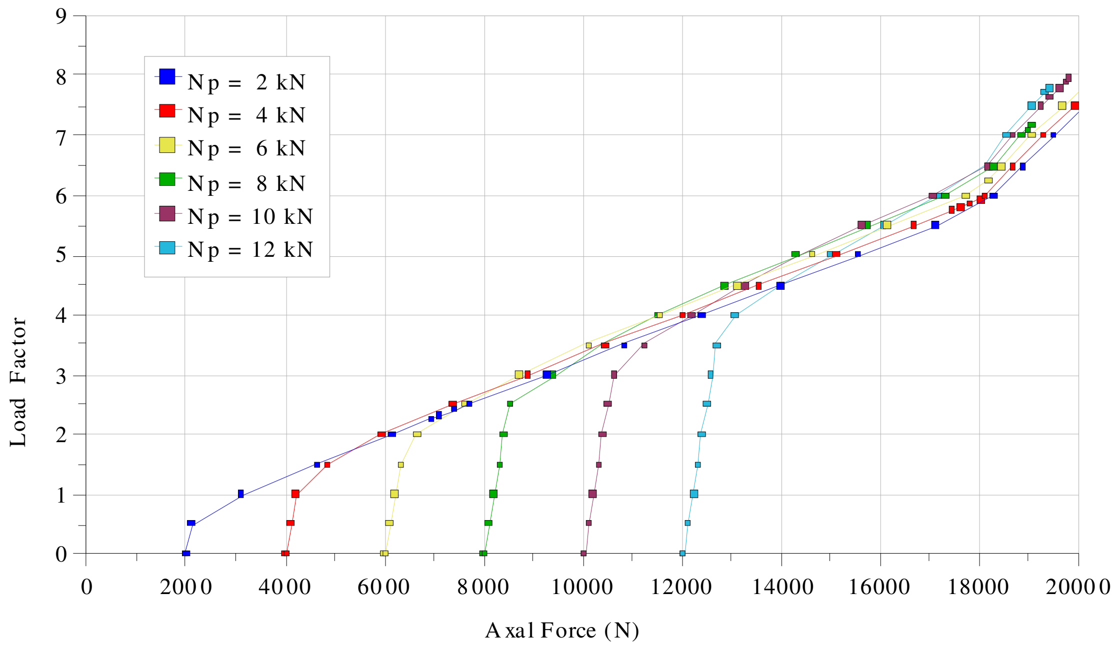

Fig. 9 Load Factor Vs axial force

The results of the numerical model confirmed the expected mechanical behaviour (Figure 7): the static scheme evolves gradually as long as the external loads increase and in correspondence of the different phases a sudden decrease in the overall stiffness of the beam can be observed.

The hardening branch allowed to prosecute the analysis beyond the yielding initiation of the bars till the buckling load was reached in the upper bars.

It can be observed that the theoretical mechanical response of the beam is substantially bi-linear until yielding of the steel bars occur while the buckling of the upper cables seems not to have a relevant influence on the overall residual stiffness.

The Ultimate Load is independent from prestress of steel cable, that control only the decompression phase and the relative service load.

5. Experimental analysis

The prototype TVTβ has been submitted to static and dynamic laboratory tests with the purpose to completely characterize the structural behavior of the beam. The dynamic tests have been conducted inducing some perturbations of impulsive nature and consequently analyzing the data give by the accelerometers applied in representative points of the beam. The study of vertical and horizontal accelerations, has allowed to evaluate the eigen vibration periods and to control the attitude of the structure to damp free oscillations.

The static analyses have been conducted applying different cycles of load and finally the specimen was loaded up to breakage. Local specific deformations and the displacements of some representative points of the beam have been measured.

5.1 Dynamic test

The dynamic test has been conducted imposing an initial displacement of the middle point of the beam in horizontal direction and, subsequently, in vertical direction in analogy to the most representative modal forms in the plan and out of the plan of the structures.

The displacement has been imposed applying a weight of around 34 daN to the beam with a steel road, which was suddenly cut thus inducing damp free oscillations in the beam. The test-equipment before the test is illustrated in figure 10.

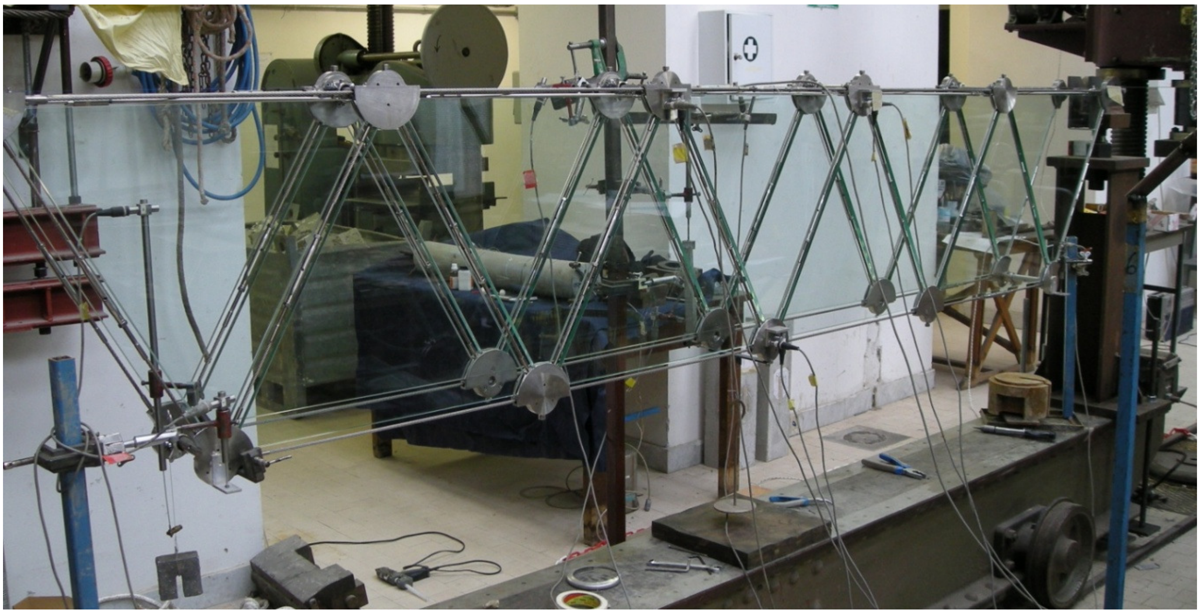

Fig.10 Specimen in dynamic analysis

The assessment of the structure frequencies has been performed analyzing the trend of the accelerations, appraising the distance among two consecutive maxima. Table 1 compares the theoretical and the experimental results, while in figures 11 and 12 are plotted the vertical and horizontal accelerations of the middle point of the beam.

Table 1 resulted dynamics tests

Test n.	Form	Frequency (Hz) measured	Frequency (Hz) calculated
1	In plane	19.1	76.9
2	In plane	16.9	76.9
1	Out of plane	13.4	12.1
2	Out of plane	15.0	12.1

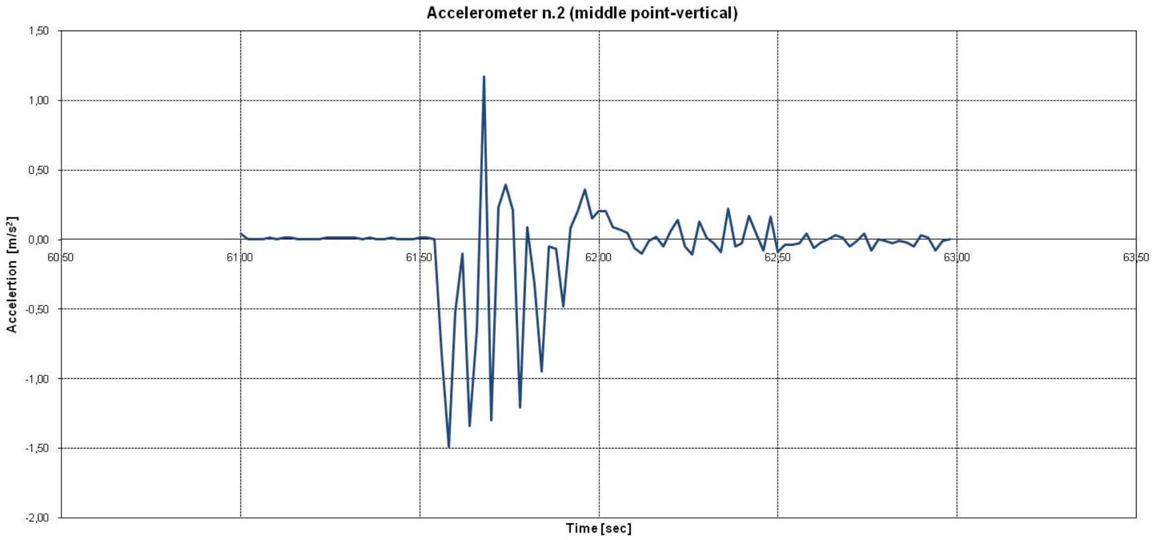

Fig.11 Vertical acceleration of middle point

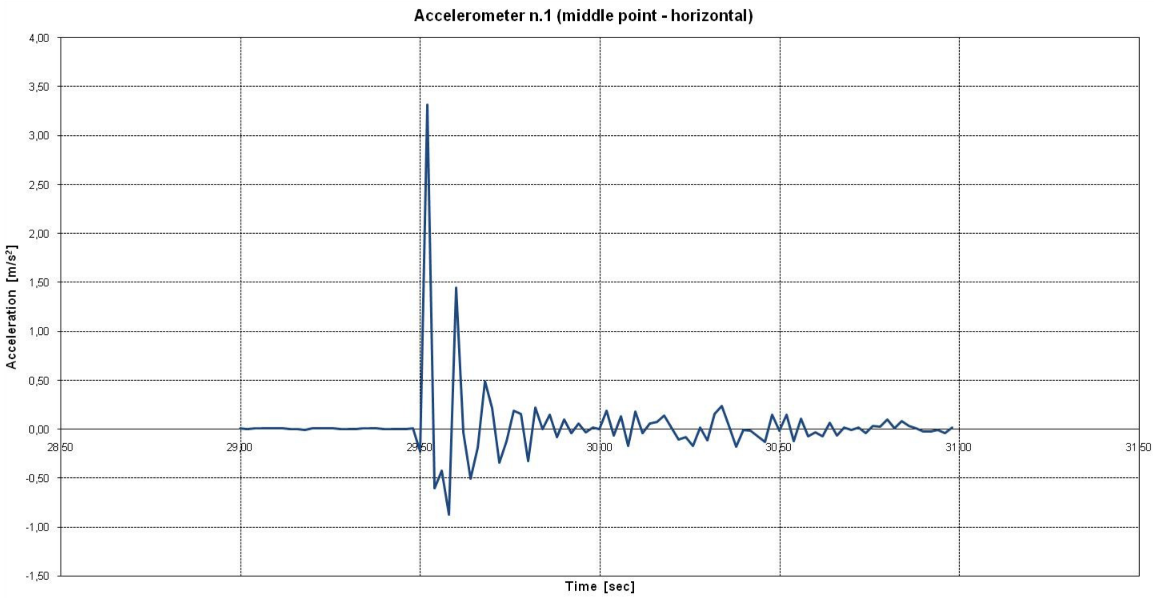

Fig.12 Horizontal acceleration of middle point

The results of table 1 show that measured and calculated out of plane frequencies, are comparable. The much greater flexural stiffness of the beam in the vertical plane would have needed a greater displacement perturbation so to induce larger deflections and accelerations. Indeed, vertical acceleration measurements could not be performed with the necessary accuracy. That explains the greater differences between calculated and experimental vertical oscillation frequencies.

5.2 Static test

The static test of **TVT β** prototype has been performed in two phases: during the first one a cyclic loading condition was applied, in the second phase the load was increased in monothonic way up to the collapse of the beam.

Before applying imposed loads, vertical and horizontal displacements induced by self weight and prestress were measured. Such investigation allowed to conclude that tension and rigidity reductions that could occur due to the relaxation of the cables or to other viscosity phenomena are substantially negligible.

Fig.13 Displacement versus time –the colours lines are related to different instruments

The program of cyclic load have been performed to evaluate the effect of geometric imperfections or the real ability of the structure to dissipate energy. In figure 14 is reproduced in the same graph the experimental (cyclical) and analytical (monotonic) results related to the displacement of the middle point as a function of the applied load.

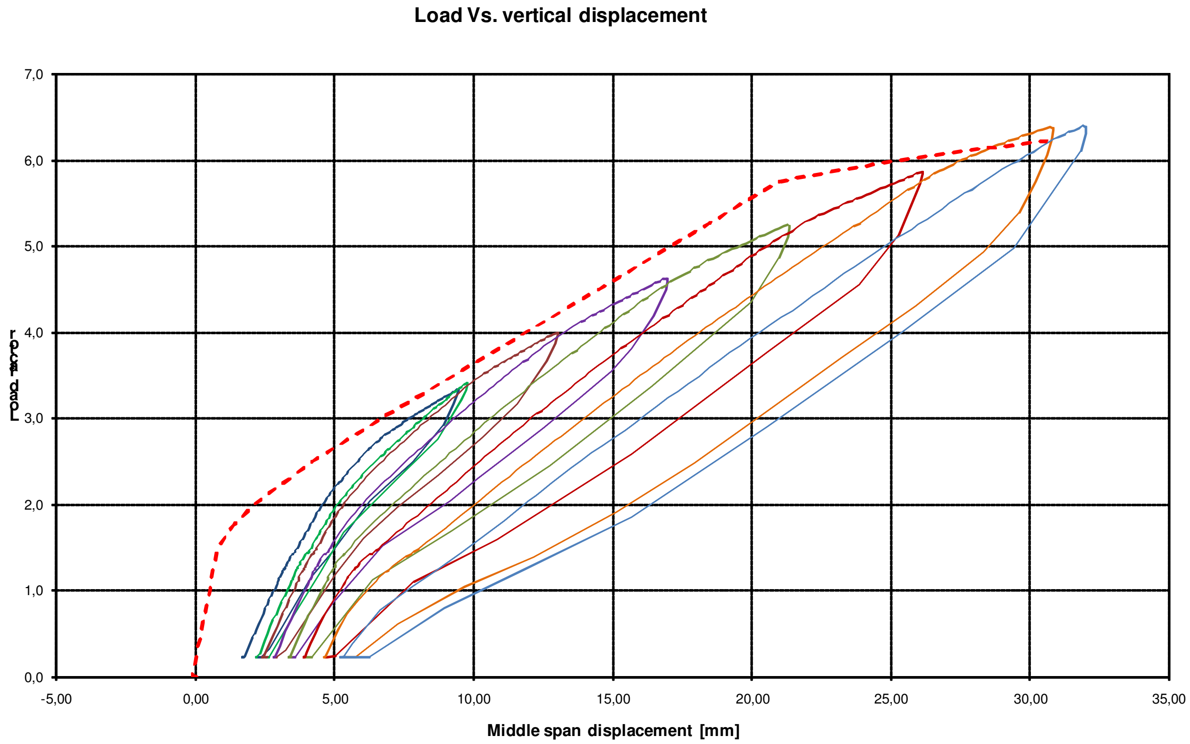

Fig.14 Load of middle point versus vertical displacement compared with FEM model (dot line)

From figure 14 is possible to conclude that the theoretical and experimental results are very similar to each other, and that the first knee related to the "decompression" of the lower part exists but is in reality very less marked. The load at which the decompression knee occurs decreases. Beside that the second knee corresponding to the yielding of the steel lower cable, is non visible, this result may be attributed to the conventional yielding load assigned to lower steel cable that has not been reached during the test.

The graph 14 shows also how the beam is even able to dissipate energy by means of the friction occurring between glass and steel knots. This property is extremely important in dynamic and seismic applications.

The very small transversal displacements present at each load cycle show how good the torsion stiffness of the beam is (Figure 15) with the progression in the cycles number.

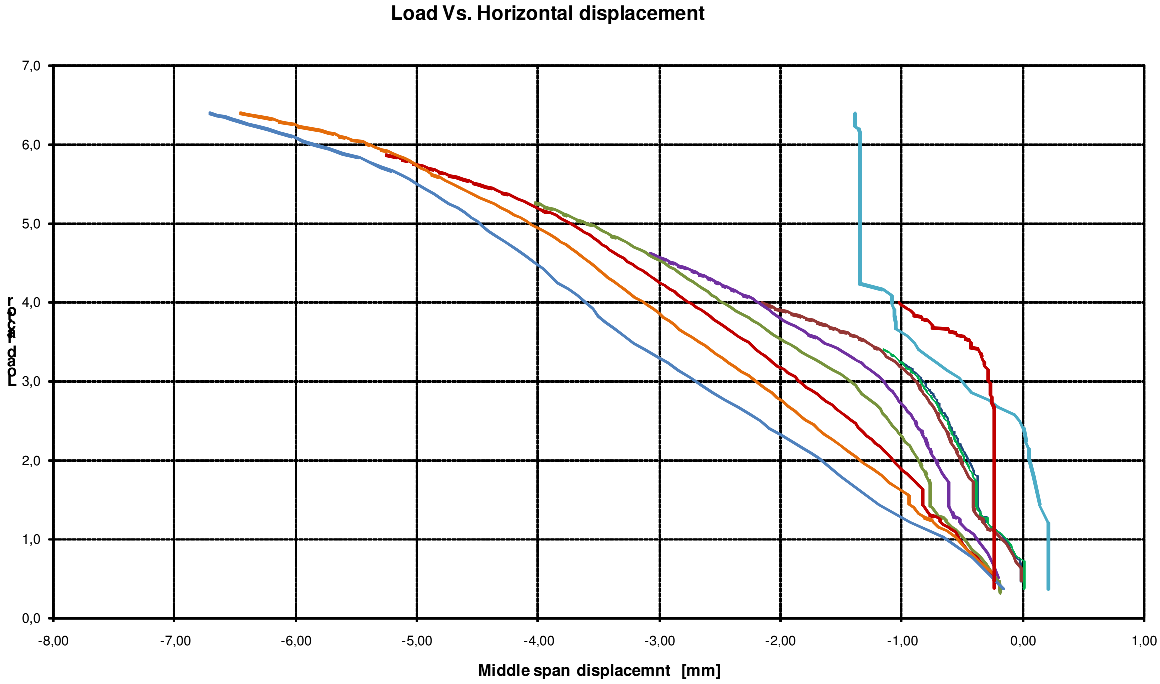

Fig.15 Transversal displacement of the middle point for load cycles

After the cyclical load program, the beam has been submitted to monotonously increasing load up to the attainment of the breakage. In the graph of figure 16 the load vs. displacement curve of this stage is compared with the theoretical one.

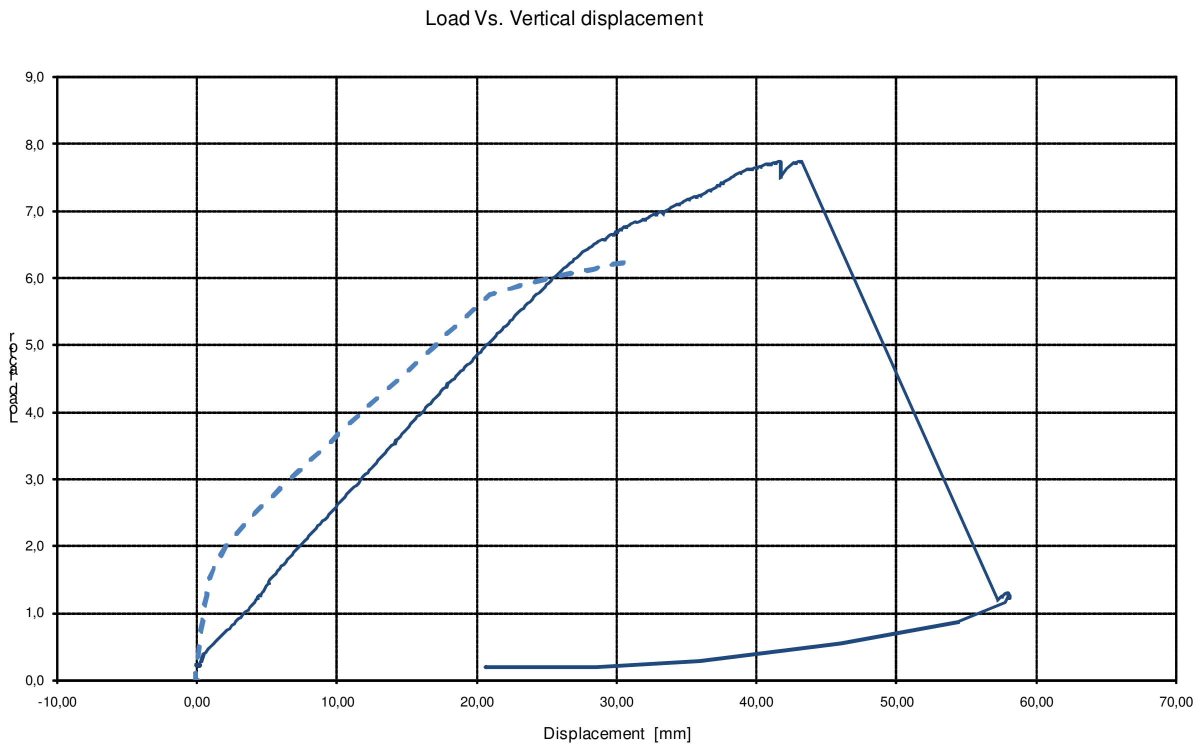

Fig.16 Vertical displacement of the middle point compared with FEM model (dot line) monotonic load increment

The breakup happened following the collapse for instability of the two middle panels, which resulted to be the only damaged glass parts at the end of the test (figure 18).

Fig.18 TVTβ at collapse

6. Conclusions

The numerical results and the test experiments on the prototype of the TVTβ prestressed composite glass-steel beam allow to conclude that the constructional principle is valid and merits further technological improvement and research work.

The experimental and theoretical results have underlined that the TVT composite glass – steel beam is able to develop a ductile breakup, and the serviceability limit state is governed by the level of prestress in the steel cables. The cyclic load programme evidenced also that even glass structures can be able to dissipate energy.

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