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Experimental characterization and numerical investigation on the Azzone Visconti bridge in Lecco (Italy)

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Abstract

The paper presents the results of a numerical investigation on the Azzone Visconti Bridge in Lecco (Italy). Starting from the historical data and from an extensive mechanical characterization of both the soil constituting the riverbed and of the masonry constituting the piers, the aim of the analyses is to predict the behaviour of the structure under the testing loading scheme prescribed by the current Italian Code. A finite element structural model has been conceived, and three different models describing the mechanical behaviour of the foundation have been implemented. Limited differences are observed in terms of absolute vertical settlement of the bridge, but important effects are highlighted in terms of stress redistribution within the piers.

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Keywords: Historical heritage; arch masonry bridge; soil-structure interaction; macroelement model; numerical analyses

1. Introduction

Within the framework of an institutional collaboration between Politecnico di Milano (Polo Territoriale di Lecco) and the Municipality of Lecco (Italy), an extensive research activity involving several disciplines and research groups has been initiated in 2015 with the aim of characterizing the mechanical behavior of the Azzone Visconti Bridge in Lecco. The present work shows some preliminary numerical results concerning a finite element model of the structure, subjected to a loading scheme corresponding to that required by the current Italian Standards for a bridge of the first category. The bridge (also known as “Ponte Vecchio”) is a historical masonry arch bridge built in

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the XIV century consisting of ten piers, eleven arches and two bridge abutments, one on the Lecco side and one on the Milano side of the Adda river. During its life, the bridge experienced several modifications of length and use, hosting even (for a certain period) a military fortress with a drawbridge (Fig. 1a).

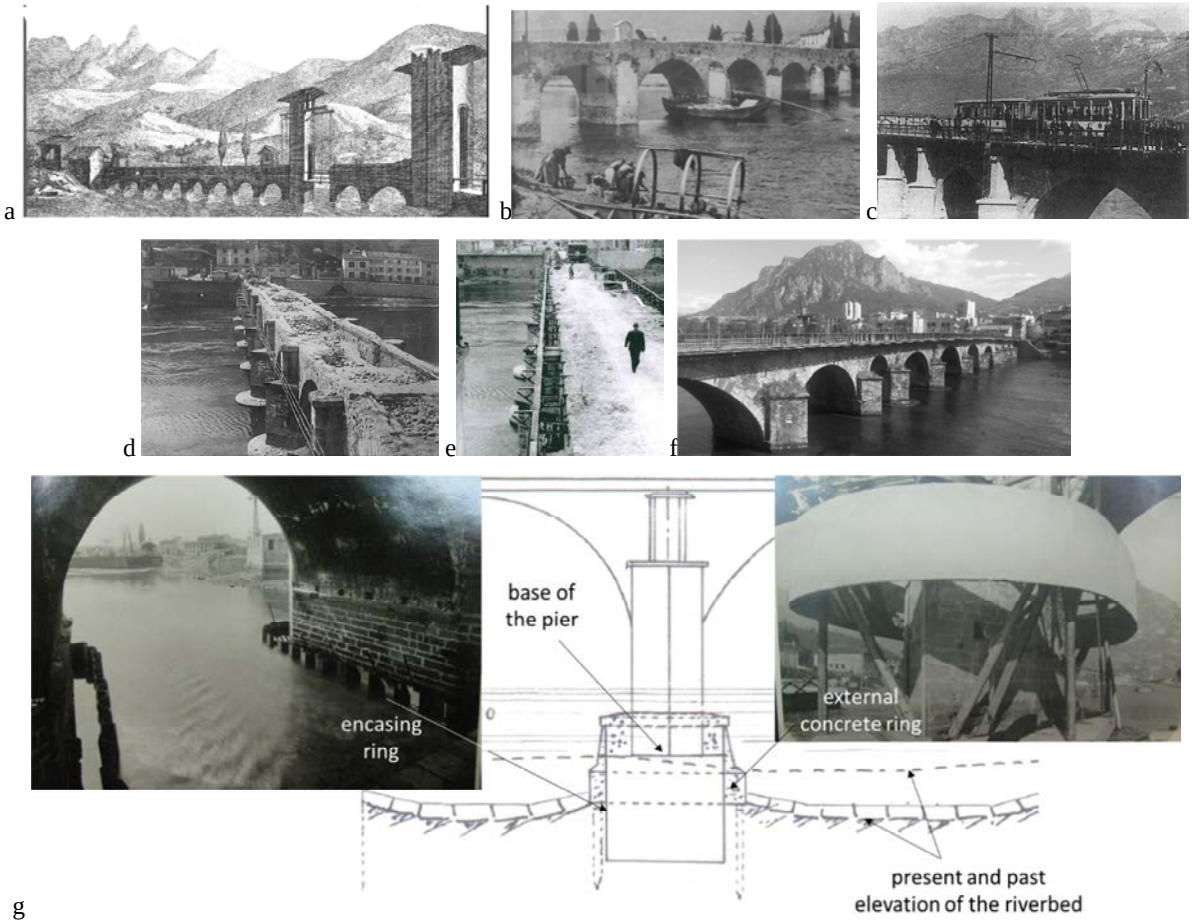

Fig. 1. (a) Plot of the bridge in the XVI century; (b) view of the bridge in 1900; (c) tramway transit in 1910; (d) view of the works in 1960; (e) detail of the concrete caisson and of the external footbridges; (f) view of the bridge in 2014; (g) placement of the steel encasing ring and of the reinforcing concrete ring in 1960. Courtesy of Lecco Municipality and of Consorzio Fiume Adda.

At the beginning of the XX century, the existing structure (Fig. 1b) was reinforced to host a tramway (Fig. 1c). In 1960 important strengthening interventions were realized: the deck was completely removed (Fig. 1d) and a continuous concrete caisson (Fig. 1e) was inserted between the north and south masonry spandrel walls and filled with coarse granular material (mix of pebbles and mortar); external pedestrian footbridges were also added (Fig. 1f). The foundations of the piers have been reinforced by introducing a large diameter steel encasing ring (Fig. 1g) around each pier. The ring, about five meters depth, was finally constrained by and an additional external reinforcing concrete ring (also shown in Fig. 1g). During the works, the natural elevation of the riverbed was reduced of about two meters, in order to facilitate the outflow of the waters in the river. No further interventions were carried out in the last fifty years, with only a limited maintenance activity.

2. Geotechnical and structural characterization

2.1. Geotechnical characterization of the riverbed

Five vertical cores (external diameter equal to 127 mm; diameter of the sample equal to 101 mm) have been drilled into the piers, one per each pier on the western half of the bridge, i.e. on the Milano side (Fig. 2a). The aim was to investigate the internal filling of the piers and, only for cores G10, G08 and G06, of the riverbed. In order to derive the grain size distributions, several (disturbed) soil samples were extracted from the riverbed. Within these three piers, several dynamic penetrometric tests (SPT) have been run at different depths. For cores G09 and G07 only, below the base of pier, SCPT tests (DPSH hammer) were performed up to a depth of about 26 meters from the deck, i.e. about 13 meters below the encasing steel ring (Fig. 2b).

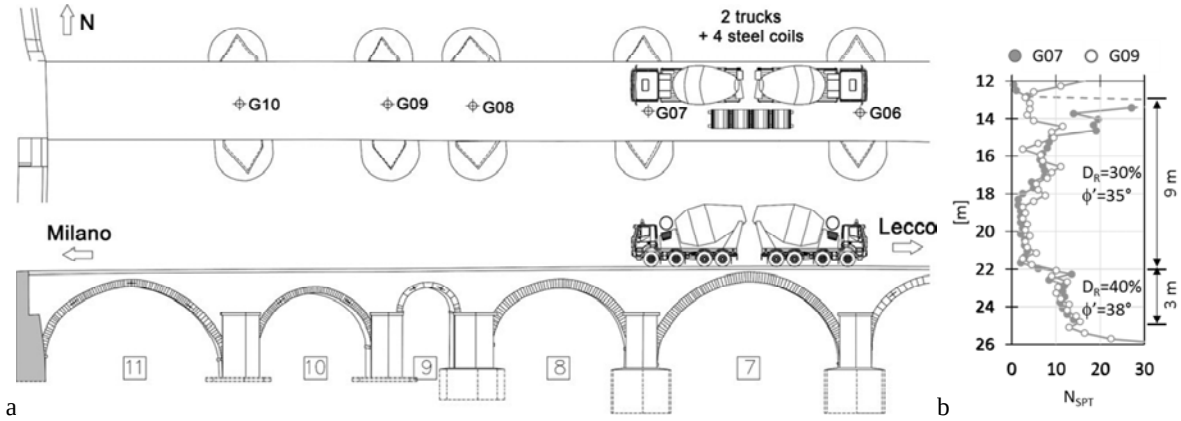

Fig. 2. (a) Plan and front views of the bridge: vertical cores and loading scheme; (b) SCPT-DPSH profiles and preliminary characterization.

Several layers of soils have been recognized by direct inspection of the extracted cores, ranging from gravel, to silty sand, to silty clay (the corresponding grain size curves have not been reported for the sake of brevity). Despite this large variability, all the foundations of the piers are stably positioned into a layer of a rather uniform gravelly sand (or fine gravel), approximately 9 meters thick. The whole set of penetrometric data were then interpreted by means of the well-known abaci proposed by [1,2], in order to estimate the relative density of the granular deposits. The correlations proposed by [3] have finally allowed to derive representative profiles of the values of the friction angle along the depth (Fig. 2b). For the sake of simplicity (and always being on the safe side) a homogeneous soil characterization has been assumed ($D_R \approx 30\%$; $\phi' = 35°$), corresponding with a mid loose granular material.

2.2. Mechanical characterization of the piers

From the cores G06 and G09 it has been possible to select 13 cylindrical specimens (diameter D equal to 80 mm) at depths ranging between 1.5 m to 10.6 m from the deck. Six specimens are constituted by homogeneous rock, whilst seven specimens are made by mixed pebbles and mortar. An average density of to 2420 kg/m^3 was estimated for the samples. All the specimens were analyzed by means of ultrasonic direct test [4], and an average ultrasonic speed of to 3.95 km/s has been measured. Seven cylindrical specimens with height H of 80 mm (H/D=1) were tested under uniaxial compression [5], whilst the remaining six with height H of about 40 mm (H/D=0.5) were tested under indirect tension (Brazilian test; [6]). Compression tests on homogeneous material provided values in the range of 50-150 MPa, while specimens composed by mixed pebbles and mortar provide values around 20 MPa; Brazilian tests gave an average tensile strength equal to 3.5 MPa.

3. Definition of the numerical model

3.1. FE model of the bridge

A bi-dimensional finite element (FE) model of the whole bridge was created by means of the software Abaqus/Standard [7]. The main geometrical characteristics of the structure (i.e. arches and piers) were derived from a topographical survey. The FE model combines several types of plane elements for different parts of the bridge as schematically indicated in Figure 3a. The piers and the filling material above the piers are discretized by employing plane stress elements, whilst the vaults are modelled with beam element. The top reinforced concrete (RC) caisson was modelled as an equivalent RC beam element connected to the vaults by means of equivalent springs (see Fig. 3a). These latter have been calibrated in order to reproduce the axial deformability of the caisson and of the spandrel walls. The vaults are rigidly connected to the piers (see point 6 of Fig. 3a). A linear elastic mechanical behaviour for all the aforementioned components have been assumed, since the loading scheme is not expected to induce significant non-linear effects in the structure.

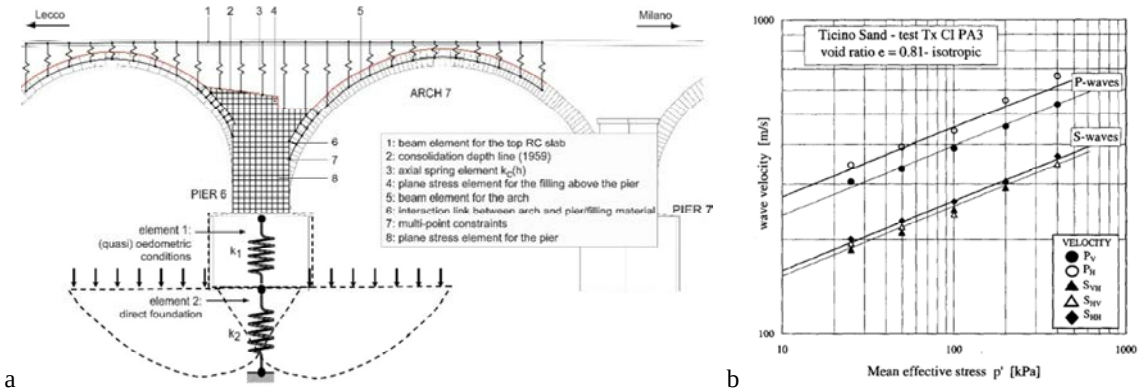

Fig. 3. (a) Main components of FE model for the Azzone Visconti bridge; (b) P- and S-waves velocity in loose sample of Ticino sand [8].

3.2. Mechanical behaviour of the foundations

An average self-weight Q_0=8000 kN was estimated for each pier and, according with the historical data, the estimated maximum traffic overload ever supported by each pier is ΔQ_{max}=440 kN, corresponding with the transit of a very heavy truck in early Sixties. The piers are then expected to behave as slightly preloaded systems. The maximum expected final overload ΔQ_{fin} acting on each pier and corresponding to the prescribed loading test (see §3.3 for details), is about 600 kN, significantly exceeding the values ΔQ_{max}. This will theoretically push the foundation into a virgin loading phase. To correctly model such transition, the pier foundation was described as a series of two 1D rheological elements (Fig. 3a), subjected to purely static vertical loads. Element 1 corresponds to the soil within the encasing ring, thought as a linear elastic spring of constant stiffness K_1, modelling a "quasi" oedometric condition. All the non-linearities have instead been concentrated in Element 2, corresponding to the soil below the encasing ring, thought as an equivalent rectangular shallow foundation.

3.2.1. Characterization of Element 1

Because of technical (and budget!) limitations, an indirect strategy has then been adopted. Given the confinement provided by the encasing ring, and owing to the small amplitude of the applied load with respect to the static load ($\Delta Q_{fin}/Q_0$<10%), the soil within Element 1 can be thought to behave in a small strain regime. With reference to the data of [8] regarding the shear wave velocity in loose Ticino sands (Fig. 3b), and considering an average value of the mean effective pressure p'≈100 kPa (corresponding to the ratio between the self-weight of the pier and the area of its foundation), a representative value of the S-wave velocity of 230 m/s can be estimated. By assuming a Poisson ratio of 0.25 and a soil unit weight of approximately 18 kN/m^3, the corresponding value of the soil Young modulus

E=240 MPa is derived, and, then, the value of the generalized stiffness K_1 (listed in Table 1).

3.2.2. Characterization of Element 2

The ultimate bearing capacity V_m of Element 2 (listed in Table 1, as is computed by means of the usual Brinch-Hansen formula) is remarkably larger than the applied load ($Q_0+\Delta Q_{fin}$=8.6 MN), and no global failure condition can be expected for the foundations. It is however fundamental to adopt a suitable modelling approach for describing the non-linear evolution of settlements. Three models have been employed:

(i) model 1: Burland & Burbidge [9]
A compressibility index I_C, depending on SPT test results and of the width of the foundation, is introduced. The model considers in a simplified way the transition to the virgin phase by reducing to 1/3 the value of I_C.

(ii) model 2: Berardi & Lancellotta [10]
A modulus number k_E, depending on the relative density D_R of the soil and on the expected settlement amplitude, is defined. This model does not take explicitly into account the transition to the virgin loading phase, but, by considering the evolution of k_E with the settlements, it captures the non-linear global response.

(iii) model 3: Nova & Montrasio [11]
A rigid-plastic modelling approach with strain hardening and non-associated flow rule, capable of reproducing the response of a shallow foundation to complex loading systems (i.e. vertical and horizontal forces, overturning moments) is considered. For the considered cases, however, only monotonic vertical loads are present, and the response is governed only by the parameter R_0 (function of D_R and representing the initial stiffness of the load-settlement curve) and the vertical bearing capacity V_m.

The expected load-settlement curves for the three models (only with reference to pier 7) are plotted in the Figure 4a (continuous lines). Models 1 and 3 capture the reduction in stiffness when the vertical load exceeds the value of ΔQ_{max} =440 kN. Model 2 shows instead a smooth progressive reduction in the global stiffness, with no sharp changes.

Table 1. Equivalent stiffness K_1 of Element 1 and bearing capacity V_m of Element 2 of the foundation of the piers.

	pier 6	pier 7	pier 8	pier 9	pier 10
K_1 [MN/m]	7232.6	5860.9	4458.5	2972.7	2536.4
V_m [MN]	123.5	116.7	151.5	179.3	149.3

3.3. Loading schemes

The load tests are performed in accordance with the provisions of the Italian Technical Regulations for Construction [12], for a first category bridge. The most severe loading procedure is realized by step-wise positioning two sets of steel coils and two four-axes truck mixers on the arch between piers 6 and 7. Four loading steps are considered: (i) first set of coils, 320 kN; (ii) second set of coils, 460 kN; (iii) first truck, 830 kN and (iv) second truck, 1200 kN.

4. Results and discussion

The obtained vertical settlement profile of the top deck along the bridge axis is plotted in Figure 6b. For model 3 the complete step-wise evolution is reported, whilst for models 1 and 2, only the results corresponding to step (iv) are shown. A difference in the maximum settlement of the midpoint of the arch is evident (ranging from 2 to 3 mm approximately), as well as in the area of influence of the applied load (for models 2 and 3, piers 5 and 8 does not suffer any settlement). More important differences are instead observed in terms of base reaction in the piers, as evidenced in Figure 6a, where the evolution of the base reaction in pier 7 is also plotted. In case of model 3, the final load exceeds the maximum value ever experienced by the foundation and the final loading condition (500 kN)

corresponds with a virgin phase. In case of model 2 the final value of the pier reaction is very similar, but a slightly larger settlement value is expected (for the sake of computational efficiency, model 2 has been here implemented by adopting an equivalent secant stiffness approach; as a consequence, only the final loading condition of step (iv) lies on the expected load-settlement curve). In case of model 1, on the contrary, a remarkably lower value of the final pier reaction is obtained (about 300 kN), and the behaviour of the system does not reach a virgin loading condition.

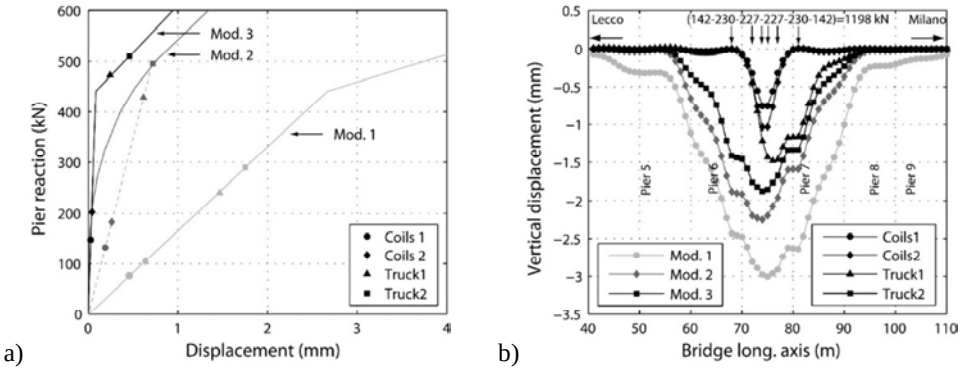

Fig. 4. (a) Load-settlement curves for pier 7 (markers indicate the loading steps); (b) vertical settlement of the deck for the three adopted models.

5. Conclusions

A finite element model of the Azzone Visconti bridge in Lecco has been implemented, by collecting a number of data deriving from historical information, topographical survey and mechanical characterization of the materials. Particular attention was devoted to the definition of the model representing the mechanical behaviour of the foundation of the piers. For the tested loading scheme, relatively limited influence of the foundation model has been observed on the amplitude of the final settlement profile of the bridge deck, whilst much larger influence is evident in the redistribution of loads within the piers. The choice of the mechanical model of the foundation is then a fundamental issue for a valuable description of the mechanical behavior of the whole structure. The numerical data shown in the paper will be soon validated on real loading tests performed on the bridge.

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