Static and dynamic behaviour of a historical stone bridge investigated by means of a 3D numerical model

R. Roccati\textsuperscript{1} & S. Petruzzi\textsuperscript{2}
\textsuperscript{1}Department of Structural Engineering, Politecnico di Torino
\textsuperscript{2}Civil Engineer, Province of Turin, Italy

Abstract

The investigation method developed in order to validate the bearing capacity of a three-arch 18\textsuperscript{th} century stone bridge for the passage of 2\textsuperscript{nd} class loads lends itself to a generalised use for a type of structure that is very common over the entire Italian territory. The exact 3D geometry of the arcades of the bridge was determined in situ, by taking into due account irregularities due to the construction process. The actual consistence of the masonry of the piers, arches, tympanums and buttresses was determined on samples and cores. The dynamic behaviour of the structure was evaluated in both environmental and random conditions (under heightened traffic levels) with the aid of accelerometers, spectrum analysers and f.f.t. processing tool for the identification of the natural oscillation frequencies and the damping coefficient necessary for linear-elastic analysis. Based on these dynamic characteristics, a solid element F.E. numerical model was produced with the aid of the COSMOS/M computation code.

1 Introduction

The purpose of a convention stipulated with the Province of Vercelli, owner of the bridge on the Strona torrent, is the definition of the feasibility lines for an intervention to preserve and upgrade the functional and physical characteristics of this ancient bridge. The aim of the intervention is to adapt the bridge to present-day performance
requirements without altering its identity as a work of considerable cultural importance on account of its age, while preserving the consistence of the structure, in terms of both its original layout and the configuration it assumed later on with the addition of a new deck in the early 20th century. In greater detail, in view of the new functional requirements, it was deemed necessary to ascertain the compatibility of the ancient structure with the present-day static and dynamic load-effects imposed by vehicle traffic.

The bridge across the Strona torrent, in the territory of the commune of Postua, is believed to date back to the first half of the 15th century, and to have been erected by the Lords of Masserano (fig. 1). It is made up of three arcades of different sizes, decreasing from the right to the left bank, for reasons to do with the need to drive the foundation piles into the solid rocks emerging from the river bed, as was the common practice adopted, whenever possible, in the construction of the Alpine valley traditional bridges, since the middle ages.

Figure 1: View from valley floor of 3-arcade stone bridge on Strona torrent

Stone bridges in split stone and pebbles, constructed with big, roughly regular stones for the facing elements ("armille") and smaller sized elements for the rest of the structure, using considerable quantities of embedding mortar and providing buttresses and filling, as well as tympanums and masonry head walls of stone fragments and pebbles bonded with the same lime mortar, have been typical of the Alpine valleys since the middle ages. In their present configuration, however, the vaults are believed to date back to the late 17th century, on account of the regular presence of forged iron braces passing through the vaults and anchored to iron rods running parallel to the tangent to the intrados of the arcade.
The bridge was originally designed to withstand loads carried on the back of mules, as borne out by its net width of only 1.80 m between the two 40 cm thick masonry parapets, which raise the total width of the structure to 2.50 m. The three arcades, with 12, 11, and 6 m long spans, and the two 2.40 and 1.80 m piers interposed add up to a total length of 38.20 m. The inclination of the deck followed the arch tops, and declined slightly towards the abutments and the piles, the accesses being at a lower level than today's.

In the early 20th century, the bridge deck was aligned with the horizontal tangent to the extrados of the main arch by installing 42 I-beams sized 190x50; sidewalks superelevated by a dozen cm were added on either side, consisting of angle irons and small vaults of good quality facing bricks with a concave surface.

2 State of preservation

The stone used for the construction of the bridge is Sienite quarried in the area, showing no appreciable signs of deterioration. The embedding mortar, produced in a nearby furnace, was examined with an X-ray machine and it turned out to consist of non-hydraulic lime made from limestone including a certain quantity of dolomite (as reflected by the B peaks - Brucite Mg(OH)₂ - in the spectrogram). The 18th century metal chains are in very poor conditions, instead, with their end parts and iron rods badly corroded. Corrosion and exfoliation phenomena are also observed in the I-beams of the deck constructed in 1907, especially at the ends, where hammer shearing caused the deformation of the section. The small brick arches built atop the I-beams to support the cantilevered portions of the deck suffered from the passage of heavy vehicle wheels, which caused strain cycles resulting in the opening of microcracks in the mortar joints.

3 Dynamic tests

The natural and excited vibrations of the bridge were measured for diagnostic purposes (i.e. to determine the natural frequencies of the structure) by exploiting the vehicle traffic going across the bridge. The measuring chain included four instruments:

- a piezoelectric accelerometer with max. sensitivity of 5000 mV/g covering a 0 to 1 kHz frequency range; the instrument is made integral with the road surface with a bicomponent cyano-acrylic adhesive;
- a pre-amplifier to amplify the signal from the accelerometer and enable the frequencies to be processed;
Figure 2: CAD representation of topographic survey of heights of bridge
- a magnetic vibration recorder for in situ use;
- an analyser for the stresses recorded in situ, with signal processing equipment.

![The deck constructed in 1907, consisting of I-beams and brick arches](image)

Figure 3: The deck constructed in 1907, consisting of I-beams and brick arches

The table below lists the main natural frequencies of the system, as determined in situ (see fig. 4):

<table>
<thead>
<tr>
<th>Frequency [Hz]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st span</td>
</tr>
<tr>
<td>2nd span</td>
</tr>
<tr>
<td>2.6 4.5 5.7 10.2 15.0 17.0 18.2 21.5 27.3</td>
</tr>
<tr>
<td>3.1 7.6 8.0 8.7 13.7 16.0 27.3</td>
</tr>
</tbody>
</table>

4 The finite element numerical model

The model was constructed with solid elements in order to simulate the stiffening effect of the tympanums and the buttresses through the variations in the elastic moduli as a function of the mechanical properties of the various kinds of masonry.

The geometric construction of the model was based on the topographic survey performed in situ, and the model was produced directly in DXF format by the survey station. After that, the 3D model was processed with the COSMOS/M computation code, to be used as the geometric base for the construction of the F.E. model; the latter was
obtained by modelling first the old bridge and then the deck added in the early 20th century (the yellow area in fig. 5).

It was decided to resort to an elastic linear analysis of the structure on the basis of the experience acquired by the Department of Structural and Geotechnical Engineering on monumental masonry buildings.

As a first approximation, the limited responsiveness of a linear elastic model to the behaviour of masonry structures was obviated by drastically reducing the elastic modulus of the zones that initially seemed to be in tension. Through the reiteration of the analysis it proved possible to construct a model in which the stresses are redistributed as a function of progressively recalculated stiffness values. As for the restraints, the presence of rock in the river bottom justified the adoption of restraints at the bases of the piles and the abutments. The mechanical properties of the materials, as used in the calculation process, are listed in the table below:

<table>
<thead>
<tr>
<th>Material</th>
<th>E [N/mm²]</th>
<th>ν</th>
<th>G [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>&quot;armille&quot; (facing stone)</td>
<td>10000</td>
<td>0.15</td>
<td>4348</td>
</tr>
<tr>
<td>vaults ad piles</td>
<td>5000</td>
<td>0.15</td>
<td>2174</td>
</tr>
<tr>
<td>buttresses</td>
<td>4000</td>
<td>0.15</td>
<td>1739</td>
</tr>
<tr>
<td>roadbed</td>
<td>1000</td>
<td>0.15</td>
<td>434.8</td>
</tr>
<tr>
<td>reinforced concrete</td>
<td>30000</td>
<td>0.15</td>
<td>13043</td>
</tr>
<tr>
<td>metal chains</td>
<td>200000</td>
<td>0.20</td>
<td>8696</td>
</tr>
</tbody>
</table>

5 Dynamic analysis

The numerical model was fine-tuned by gradually changing the values of the elastic moduli around the levels inferred from the literature through tests and modifying the degrees of restraint at the abutments; by reducing the elastic moduli and the degrees of restraint, model frequencies were lowered down to values approaching those obtained in situ. The 1st and 2nd modal vibration forms, corresponding to frequencies of 3.20 Hz and 3.34 Hz, are characterised by the oscillation of the central and right arches, in the plane of the deck, with an appreciable rotation of the piers around their bases in the crosswise direction (figs. 6a and 6b). The 3rd and 5th forms (4.57 Hz and 5.67 Hz) are characterised by the oscillation of the bigger arches in their own plane, in phase and counter-phase respectively, with marked rotations of the relative piers around their bases in the longitudinal direction (figs. 6c and 6e). In the 4th mode (4.92 Hz) the small arch (at the left end of the bridge) moves in resonance in the plane of the deck with an appreciable rotation of the corresponding pier which is forced to follow the displacement of the arch (fig. 6d). In the 6th mode (5.76 Hz) the left arch oscillates in the deck plane, and the right arch deforms in its plane with the relative pier (fig. 6f).
Figure 4: Natural frequencies of the central arch

Figure 5: Solid element numerical model of the bridge for FEM analysis

6 Static analysis

Static analysis supplied the stress and strain data relative to the load-effects produced by the permanent loads and the variable loads (overloads originated by vehicle traffic).

Of special interest, for 3D structures, is the graphic representation of the principal compressive stresses in the individual elements ($\sigma_b$); in addition to indicating the stress rate of the various elements, in fact, it shows the compressive stress lines in the structure.
The highest compressive stress, obtained at the impost of the arches, came to 1.4 N/mm², a value that should be rated as high in view of the considerable thickness of the mortar joints in the inner part of the vault. The analysis showed that the chains did no fulfil any primary static function and were only meant to keep the arch elements together during the construction process.

The variable loads applied, in keeping with D.M. 4.5.90, consisted of a conventional 600 kN three-axled vehicle and an evenly distributed load of 30 kN/m. However, the overloads are negligible compared to the dead weight of the structure, as borne out by the dead weight + permanent loads/dead weight ratio (efficiency factor).

The state of stress was also determined with reference to the original conditions of the bridge. It should be noted that in its original configuration the structure was in a better position, being subjected to smaller, more favourably distributed loads, giving rise to a thrust curve that ran closer to the geometric axis of the arches (see fig. 8).

7 Conclusions

Dynamic analysis has shown that the addition of a new deck in 1907 resulted in an increase in the masses in play in the event of an earthquake without offering an appreciable improvement in stiffness, while static analysis has shown that it caused an increase in the state of stress due to dead weight alone from 7.0 daN/cm² to 12.0 daN/cm².

The state of stress also due to 1st class variable loads would reach a peak of 35.0 daN/cm², which is clearly unacceptable. Accordingly, the need to control the load levels makes it necessary to reclassify the bridge as a 2nd class structure, while safety-related considerations entail the need to separate vehicle traffic from pedestrian traffic.

A possible way to improve the response of the bridge to seismic actions might be to insert in the 1907 deck a reinforced concrete slab firmly anchored to the piers and the abutments by means of micropiles. This stiffening slab would perform a twofold function: it would serve as a horizontal bracing anchored to the piers and the abutments to resist seismic actions, and as the cross member of a frame with two central props to bear the variable loads. The micropiles added to strengthen the stone piers might extend down into the foundation rocks: their presence, combined with the effective strengthening of the bottom parts of the piers might make it possible to remove the collar-type RC plinths applied in the '60s, which restrict the free section of water outflow and make the bridge look bulkier.
Figure 6: The first 6 modal vibration forms with the relative frequencies
Figure 7: Principal stresses, $\sigma_3$, due to dead weight and variable loads for 1st class bridges

Figure 8: State of stress in the bridge before the new deck, showing a thrust curve that ran closer to the geometric axis of the arches