Insights and Innovations in Structural Engineering, Mechanics and Computation – Zingoni (Ed.)
 ISBN 978-1-138-02927-9

Historical reinforced concrete arch bridges: Dynamic identification and seismic vulnerability assessment

M. Acito & C. Chesi
Department of Architecture, Built Environment and Construction Engineering, Politecnico di Milano, Milan, Italy

C. Lazzarin & E. Richermo
Politecnico di Milano, Milan, Italy

ABSTRACT: Safety assessment performed with reference to a relevant case of historical reinforced concrete arch bridge is here discussed. The case study corresponds to the Italian bridge located at Brivio, on the Adda River, completed in 1916. Experimental testing has been performed on the bridge, providing all the required information for the identification of the dynamic behaviour. The seismic vulnerability has been evaluated with both response spectrum modal analyses and non linear static (push-over) analyses. The structural analyses have allowed to interpret the dynamic structural behaviour of the bridge.

1 INTRODUCTION

The problem of structural safety is addressed, in this paper, with reference to the case of the bridge over the Adda river at Brivio (Italy), which was completed in 1916 (Santarella & Miozzi 1924), and constitutes a relevant case of historical reinforced concrete arch bridge (Figure 1).

The new technology based on the use of reinforced concrete had, with this work, one of the first applications to important structures, thus giving special historical significance to this bridge.

From the point of view of the structural scheme, the Brivio bridge spans over three bays, 51 m each, with the deck suspended to three couples of arches extending over the road level to a maximum height of 8.07 m (Figure 2). The total length is approximately 160 m. Each couple of parallel arches presents, in the central part, transversal connecting elements with a T cross section; a minimum free height of 6,6 m over the road surface is always available (Figure 3). For each of the three bays, fixed support conditions are present at one end, while sliding over steel rollers is allowed at the opposite end, so that thermal deformations are not restrained.

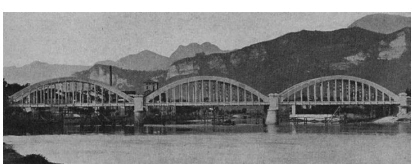

Figure 1. The Brivio bridge.

The double lane road running over the bridge is presently in use (Figure 3) as part of a main traffic flow direction, open to both light and heavy vehicles.

From the point of view of preservation, the structural complex is basically in good conditions; despite previous restoration interventions, classical forms of damage are present in the concrete exter-

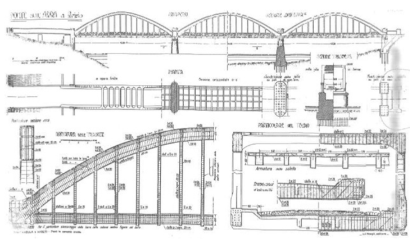

Figure 2. Brivio bridge: Original design drawing.

Figure 3. The Brivio bridge in the present situation.

nal layer. In some cases, reinforcement corrosion can be observed.

Numerical analyses have been developed under the assumption that material restoration has been performed, so that reference can be done to the original element cross sections. The overall structural response has therefore been reproduced, and the specific role of single structural elements within the global dynamic response has been interpreted.

On the basis of both the original design documentation and of survey activities performed during the bridge life time as well, numerical structural models have been set up and calibrated with reference to frequencies and mode shapes, as obtained from dynamic experimental testing. Due to the lack of information on the mechanical properties of the material, a sensitivity analysis has been done, to the purpose of highlighting which parameters mainly affect the structural dynamic response; on such a basis, an optimization procedure has been developed through manual tuning.

The refined numerical models have then enabled the estimation of the stress levels in different situations. Through simple static analyses, the effect of variable loads (i.e., traffic and snow) have been evaluated, while the response to the design seismic action for the specific site has been evaluated for some structural elements at the ultimate limit state through a response spectrum modal analysis. Finally, non linear static analyses have been performed for the evaluation of the post elastic resources, in view of the seismic vulnerability assessment.

The work done has allowed to point out both the conceptual design and the performance of the bridge, mainly with reference to the dynamic behaviour. The identification of the system properties through experimental and numerical testing, moreover, has provided the basis for the implementation of reliable computational models for structural analysis.

2 DYNAMIC TESTING

The experimental dynamic structural identification has been based on ambient vibrations due to vehicular traffic (Allemang & Brown 1982, Gentile 2006, De Stefano et al. 2013). In this way, the bridge did not have to be closed to traffic end, moreover, recordings correspond to normal use conditions.

Accelerometers have been located nearby the columns of the right side bay, initially on the uphill walkway and, subsequently, on the downhill one, according to the configuration indicated in Figure 4. The green arrows refer to the roving sensors, while blue arrows correspond to reference stations. An accelerometer located at one

a)

b)

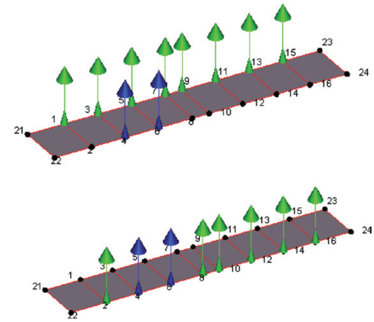

Figure 4. Instrumented locations.

Figure 5. Recording equipment.

of the suspending columns is shown in Figure 5. A metal square plate can be seen, attached to the walkway, for the correct placing of the accelerometer; this last is connected to an amplifier, which transmits the amplified measured signal to a Data Acquisitions unit (DAQ) for signal recording and processing.

3 EXPERIMENTAL RESULTS

Modal parameters have been identified through the *Frequency Domain Decomposition* technique (*FDD*, Brincker et al. 2001), which is based on the estimation of auto-spectral *Gpp(f)* and cross-spectral *Gpq(f)* density functions (Bendat & Piersol 1993) for all the recorded signals, and subsequent singular value decomposition of the spectral matrix. Auto-spectra and cross-spectral have been estimated through the classical approach based on the Welch's modified periodogram method (Welch 1967). The *FDD* procedure is a further development of the classical spectral analysis in relation to both the evaluation of mode shapes (which comes out to be more accurate and faster) and the possibility of identifying modes characterized by close frequencies by means of singular values following the first one.

Data processing has allowed the definition of autospectra; from the relative peak values the system main frequencies have been identified, Figure 6.

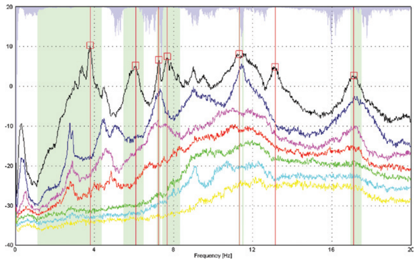

Figure 6. Singular value lines and identified modes.

Figure 7. The bridge deck.

Table 1. Dynamic properties.

Mode	Frequency [Hz]	Mode shape
I	3,78	Anti-symmetric flexural mode
II	6,08	Symmetric flexural mode
III	7,25	Torsional mode
IV	7,68	Symmetric flexural mode
V	11,32	Torsional mode
VI	13,78	Anti-symmetric flexural mode
VII	17,1	Torsional mode

Within the analyzed frequency range, seven modes have been recognized, see Table 1.

4 NUMERICAL MODELLING AND VALIDATION

4.1 *Sensitivity analysis*

A sensitivity analysis has been performed to the purpose of identifying those parameters which mainly influence the structure dynamic behaviour (Saltelli et al. 2000, Brownjohn et al. 2001, Daniel & Macdonald 2007); in view of accomplishing the best correlation between the real structure and the numerical model, several parameters have been selected and analyzed. Specifically, material properties and restraint conditions have been chosen for the sensitivity analysis; the adopted error function has been defined as:

$$\varepsilon(x) = \sum_{i=1}^{m}\left[\alpha_i\left(\frac{f_{ai}-f_{ei}}{f_{ei}}\right)+\beta_i\left(\frac{1-\sqrt{MAC_i}}{MAC_i}\right)\right] \quad (1)$$

where:

- f_{ai} indicates the i-th analytical frequency,
- f_{ei} indicates the i-th experimental frequency,
- α_i is the weight factor for modal frequencies; a value of 0.8 is used,
- β_i is the weight factor for mode shapes; a value of 0.2 is used,
- MAC_i is the Modal Assurance Criterion, computed with reference to the i-th identified mode and the corresponding analytical mode.

The sensitivity analysis (Daniell & Macdonald 2007) is based on the evaluation of the local sensitivity index, defined as:

$$\xi_i = \frac{\ln(y)-\ln(y_b)}{\ln(x_i)-\ln(x_{ib})} \quad (2)$$

where x_i and y are the values for the i-th parameter under study and the corresponding error function, $\varepsilon(x)$, respectively. Specifically, the b index indicates the reference values, while x_i corresponds to a 5% variation of the input parameter.

The graphical form shown in Figure 9 has been used to represent the different values obtained for the sensitivity indexes and their influence on the structure behaviour. Based on these results, a manual tuning procedure has been applied to all the parameters for which no experimental value was available.

4.2 *Numerical model and sensitivity analysis*

The bridge deck is constituted by beams forming a grid (with transversal elements in correspondence of suspension columns) and by a slab with a thickness of 15 cm, covered by a 30 cm thick asphalt layer (see Figure 2 and Figure 7); this layer has been included in the model, in order to represent the contribution to the deck stiffness. Lateral sidewalks, resting on transversal beams, also have a thickness of 15 cm.

In the numerical model (Figure 8-a), reasonable values have been initially used for the mechanical properties of the different elements, i.e., for the elastic modulus, the Poisson ratio and the concrete density; in the absence of specific experimental data, reference has been done to other bridges, built in the same period (see Table 2).

Arches, columns and beams have been modelled as frame elements, while plate elements have

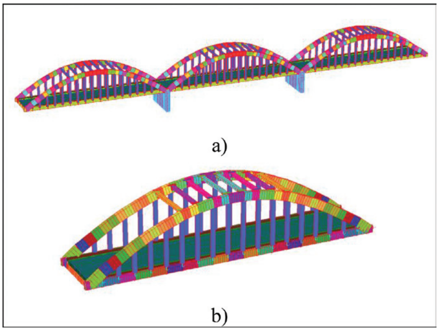

Figure 8. Numerical models.

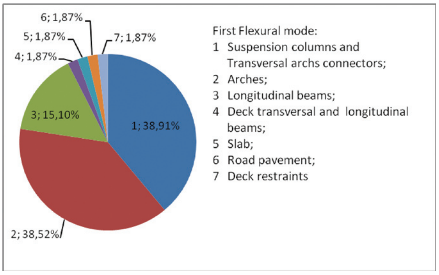

Figure 9. Sensitivity analysis for the first flexural mode.

been used for the slab. Brick elements have been employed only for the modelling of piers. A single bay model, corresponding to the first bay on the bridge right side, has been used for the comparison with dynamic experimental data (Figure 8-b).

As already mentioned, a sensitivity analysis has been performed to the purpose of highlighting which elements mainly influence the structure dynamic behaviour; consequently, a manual tuning procedure has been applied in order to reach a minimum error in the difference between experimental and numerical frequencies.

The sensitivity analysis has been performed separately for the first three translational modes and the first two torsional modes, in order to point out the conditioning parameters for each mode group. The analyzed parameters include both the elastic modulus for the different element groups and the stiffness of the elements simulating the deck end support.

From the graphical results produced by the sensitivity analysis (see Figure 9), it has been possible to determine the relative importance of each single parameter on the global structural behaviour; specifically, the following considerations can be developed for the different mode typologies.

The first flexural mode is influenced by the suspension columns, the transversal arch connectors, the arches and the external longitudinal beams; the second one is mainly influenced by the arches, while

Table 2. Element mechanical properties.

Structural element	E [MPa]	ν	Density [kg/m^3]	Finite element
Suspension columns/ Beams	25600	0.2	2500	Beam
Deck	29300	0.2	2500	Plate/ Brick
Arches	36000	0.2	2500	Beam
Central pier	25600	0.2	2500	Brick

the other elements have a more reduced effect; the third one is affected almost entirely by the arches. Globally, the arch effect is dominant over this set of modes (97%).

In the first torsional mode, important effects are due to the suspension columns, the transversal arch connectors, the arches and the longitudinal external beams; in addition to these elements, the slab also has influence on the second mode.

Globally, the torsional modes are under the influence of a higher number of parameters compared to the flexural modes: suspension columns and arch transversal connectors (45%), longitudinal external beams (23%), arches (20%) and slab (12%).

As a conclusion, it can be stated that the arches play a fundamental role in relation to the structure flexural behaviour, while suspension columns and transversal arch connectors mainly influence the torsional behaviour.

To the purpose of the model validation, experimental and numerical frequencies have been compared, also considering the mode shapes; specifically, the seven main modes coming from the experimental testing have been considered, which are reproduced in the same order by the numerical model. See Table 3 for numerical values and errors.

For each mode, the MAC and the normalized modal difference (NMD) indexes have been computed, with reference to the measured locations, see Table 4.

As it can be seen from both the MAC and NMD indexes and from the mode shapes, correlation is very good for all the considered modes. Therefore, it can be concluded that the numerical model provides a good representation of both the flexural and torsional behaviour. Examples of mode shapes are shown in Figure 10 and Figure 11, in terms of both numerical and experimental results.

5 SAFETY CHECKS

5.1 *Linear analyses*

Safety checks have been based on the knowledge levels and the corresponding confidence factors, as

Table 3. Data for the final numerical model.

Exp. freq. [Hz]	Num. freq. [Hz]	Mode	Error	Mode shape
3.78	4.11	3	8.73%	Flexural, anti-symmetric
6.08	5.92	4	2.63%	Flexural, symmetric
7.25	6.16	5	15.03%	Torsional
7.68	7.57	7	1.43%	Flexural, symmetric
11.32	10.4	13	8.13%	Torsional
13.13	11.13	14	15.23%	Flexural, anti-symmetric
17.1	15.03	16	12.11%	Torsional

Table 4. Values for the MAC and NMD indexes.

Exp. freq. [Hz]	Num. freq. [Hz]	Mode	MAC	NMD	Mode shape
3.78	4.11	3	0.99	0.08	Flexural, anti-symmetric
6.08	5.92	4	0.97	0.16	Flexural, symmetric
7.25	6.16	5	0.96	0.21	Torsional
7.68	7.57	7	0.97	0.16	Flexural, symmetric
11.32	10.4	13	0.88	0.36	Torsional
13.13	11.13	14	0.98	0.15	Flexural, anti-symmetric
17.1	15.03	16	0.95	0.24	Torsional

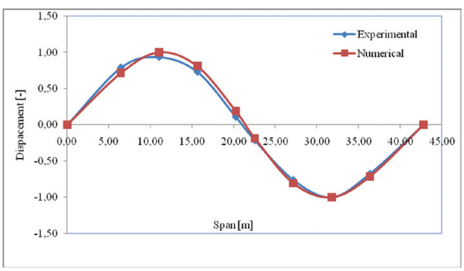

Figure 10. 3rd mode (first non-symmetrical flexural mode).

indicated in the Italian Building Code. Specifically, the limited knowledge level LV1 has been assumed for both concrete and steel.

As to the ultimate limit state verifications under static conditions, reference has been done to permanent and variable actions as prescribed by the Building Code, considering flexure plus axial load effects in the most unfavourable conditions. All checks are satisfied, with the exception of the arch segment at the key-location where, however, the assumptions made on the material quality and the action values might be unrealistic.

As to the seismic safety requirements, a response spectrum modal analysis has been performed; the design spectrum for the collapse limit state has

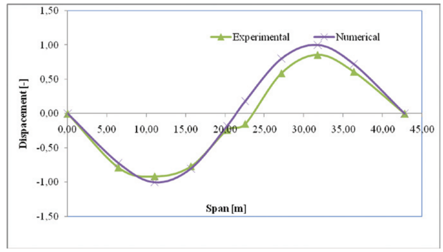

Figure 11. 5th mode V (first torsional mode).

been considered, corresponding, in relation to the bridge usage class, to a 1462 years return period.

A high number of modes had to be considered; all verifications were satisfied.

5.2 *Push-over analysis: The procedure*

For the push-over analyses the N2 method has been used (Fajfar & Gašperšič 1996); the system capacity curve has been computed without consideration of the plastic resources in the deck, limiting the plastic behaviour to beam elements.

As prescribed, two different profiles have been considered for loads acting in the transversal direction, proportional to masses only and to masses plus first mode displacements, respectively.

Diffused plasticity has been considered, yet limiting non linear moment curvature diagrams to elements for which the plastic behaviour could reasonably be expected to take place.

According to the adopted procedure, an equivalent SDOF system is considered, for which the capacity curve is represented in a displacement-acceleration diagram; in this way, a final meaningful comparison with the Acceleration-Displacement-Response-Spectrum is possible.

5.3 *Push-over analysis: Application*

For the determination of the capacity curve, the control point has been selected at the arch key point on the down stream right span arch.

All the obtained capacity curves have been converted into the equivalent SDOF curves, following both the procedure indicated by the Italian Building Code and the one given by Eurocode 8 as well; the corresponding most unfavourable situations have been selected, i.e., those associated with the load profile proportional to a mode shape.

The seismic demand for the SDOF system can then be easily represented graphically and the final verification easily performed (Figure 12 and Figure 13). In both the considered cases, the capacity curve intersects the demand curve in the linear range, i.e., ductility resources are not needed to respond to the design earthquake.

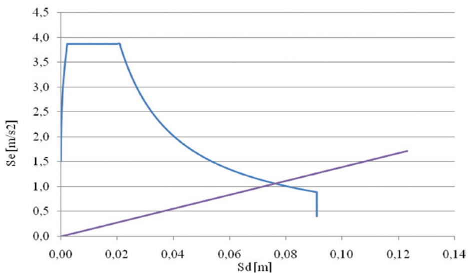

Figure 12. Capacity and demand curves (Italian Code).

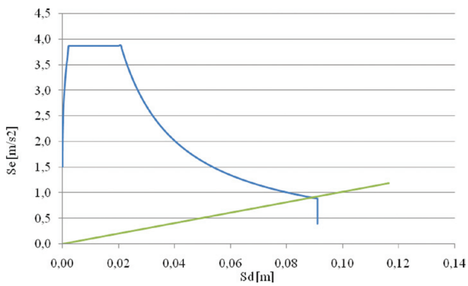

Figure 13. Capacity and demand curves (Eurocode 8).

The maximum displacement ($d_{1,\ max}$), with reference to the SDOF system, is 0.075 m for the Italian Code and 0.088 m for Eurocode 8. The corresponding displacement requirement for the original system, obtained multiplying by the participation factor, is 0.12 m for the Italian Code and 0.14 m for Eurocode 8. As required by the codes, the analysis has been performed again, evaluating the capacity curves up to maximum displacements as large as 150% the above values. Also in such cases, the structural response to the design earthquake remains in the elastic field.

6 CONCLUSIONS

The performed analyses have allowed to show that the reinforced concrete Brivio bridge, built in 1916, can provide a positive dynamic performance and is globally in line with the safety requirements corresponding to present codes.

Considerations developed on the structural safety, however, should not be considered as an official global check of the structural system, as verifications have been extended to a limited number of elements and the material decay, as appearing from a visual inspection, has not been considered.

With reference to the global dynamic response, analyses have shown that the flexural behaviour is mostly influenced by the arches (97%), whereas the torsional behaviour depends on a large number of elements: suspension columns, transversal arch connectors, longitudinal external beams.

The push-over analyses, due to the peculiarity of the structural system, do not allow for the specification of the real seismic safety margins. All the same, it is possible to state that the structural response to the design earthquake is in the linear field.

ACKNOWLEDGEMENTS

Prof. C. Gentile is kindly acknowledged for providing dynamic experimental data.

Prof. A. De Stefano is acknowledged for his kind contribution to the sensitivity analyses.

REFERENCES

Allemang, R.J., Brown, D.L. 1982. A correlation coefficient for modal vector analysis. *First International Modal Analysis Conference, SEM, Orlando, FL*: 110–116.

Bendat J.S. & Piersol A.G. 1993. Engineering applications of correlation and spectral analysis. Wiley Interscience.

Brincker R., Zhang L. & Andersen P. 2001. Modal identification of output-only systems using frequency domain decomposition. *Smart Materials and Structures* 10(3): 441–445.

Brownjohn, J.M.W., Xia, P.Q., Hao, H., Xia, Y. 2001. Civil structure condition assessment by FE model updating: methodology and case studies. *Finite Elements in Analysis and Design*, 37 (10): 761–775.

CEN—TC250, 2004. EN 1998-1. Design of structures for earthquake resistance Part 1: General rules, seismic actions and rules for buildings.

Daniell, W.E., Macdonald, J.H.G. 2007. Improved finite element modeling of a cable-stayed bridge through systematic manual tuning. *Engineering Structures*, 29 (3): 358–371.

De Stefano, A., Ceravolo, R., Matta, E., Quattrone, A., Zanotti Fragonara, L. 2013. Experimental dynamic identification of strategic building structures under real seismic loading. *In: Second Conference on Smart Monitoring, Assessment and Rehabilitation of Civil Structures, Istanbul, 9–11 September 2013.*

Fajfar, P., Gašperšič, P. 1996. The N2 method for the sismic damage analysis of RC buildings. *Earthquake Engineering & Structural Dynamics*, 25 (1): 31–46.

Gentile, C. 2006. Modal and structural identification of a R.C. arch bridge. *Structural Engineering and Mechanics*, 22 (1): 53–70.

Italian Ministry for Infrastructures, 2008. DM 14 gennaio 2008 - Nuove norme tecniche per le costruzioni. *Gazzetta Ufficiale n. 29 del 4 febbraio 2008 - Suppl. Ordinario n. 30.*

Saltelli, A., Chan, K., Scott, E.M., 2000. *Sensitivity Analysis.* Wiley, Chirchester (UK).

Santarella, L., Miozzi, E. 1924. *Ponti italiani in cemento armato*. Volume 2. Hoepli.

Welch P.D. 1967. The use of Fast Fourier Transform for the estimation of power spectra: a method based on time averaging over short modified periodograms, *IEEE Trans. Audio Electroacoust., AU,* 15:70–7.