EXPERIMENTAL AND THEORETICAL INVESTIGATION FOR SAFETY EVALUATION OF AN EXISTING R.C. BRIDGE

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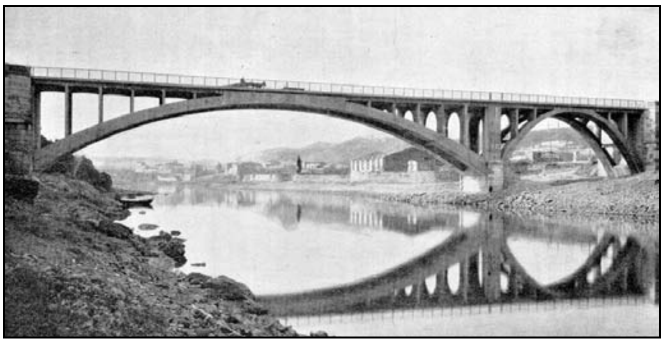
INTRODUCTION

A significant number of existing r.c. bridges in Italy was designed and constructed soon after the end of World War II, during the late forties and fifties.
In that period, i.e. prior to the extensive application of prestressing technology, arched structures were used extensively for medium span bridges.
The problems encountered in maintaining and, when necessary, upgrading and strengthening this type of structures are very specific. They are related to the working conditions of the principal components, which proceed from the spatial geometrical configuration of the structure, to the construction technology and materials utilized and the design criteria adopted.
It is worth noting, for example, that the states of stress are characterised by relatively low values of both compressive and tensile components. However, such bridges were frequently constructed in emergency conditions, when adequate materials and equipment were scarce. Consequently, the quality of concrete may be rather poor while the concrete and steel sections were often deliberately over-sized.
Calculation methods were generally rather simplified and this occasionally led to an inadequate distribution of steel reinforcement.
The properties of the materials, the construction characteristics and structural behaviour of this bridge have been investigated by means of laboratory and on-site tests, and using a numerical model which has been calibrated on the basis of the experimental data. Interesting information has been obtained from the original technical documentation, i.e. calculations and drawings.
In order to evaluate the real safety margin of the bridge a probabilistic approach has been used. The measure used for structural safety is the reliability index (β).



[fig.1] Photograph of the bridge after construction

DESCRIPTION OF THE STRUCTURE

The structure consists of two spans of 60 and 25 meters respectively, each consisting of two adjoining arches with ends fixed in the foundations. The two-column bents supporting the slab rest on the arches at regular intervals (which differ in the two spans). The slab has a section with an arch-shaped lower profile. The portals connecting the arches to the slab are very rigid and strong across the axis of the bridge, whereas longitudinally they take the form of tie bars hinged at the ends.

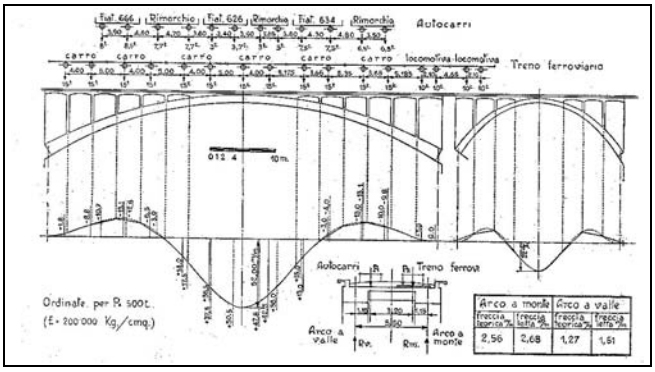
Useful information for an assessment of the present safety of the bridge has been gathered from the technical documentation prepared at the time of construction.
The design documentation shows the geometrical and reinforcement characteristics of the arches and the results of the proof-loading tests, but data on the foundations is scarce, probably owing to the fact that part of the foundations of the previous steel bridge were used.
The description of the construction of the bridge in a book on Italian reinforced concrete bridges (Santarella L., Miozzi E.), published in 1948, provides some useful information regarding the dimension loads of the bridge, the materials utilized and the foundations built to support existing ones. The information lacking in the design documentation is therefore partially available.

The structure was built to carry a road on the downstream side and a railway upstream.
Therefore, the two arches that make up the bridge have different dimensions. The downstream arch was designed to bear loads as required by contemporary Italian legislation for road construction (D.M.n.8 of 15/09/1933), which provided for the transit of two road rollers weighing 18 tons, in addition to an unspecified column of 12-ton trucks. The upstream arch was sized to bear the burden of a train with 40-ton engine and an unspecified number of 30-ton carriages.

The concrete casting of the arches of the bridge required the construction of extensive wooden centering. The arches were built by casting the concrete at two different times.

On December 21st, 1946, the bridge proof-loading tests were conducted. The loading conditions implemented were seven in all, measuring the deformations at 20 points. It has therefore been possible to reconstruct the arch deformation curve [fig.2] and to compare the data with the theoretical values.

The structure presents widespread phenomena of deterioration typical of reinforced steel structures and partly due to the choices in building and modes of execution adopted during construction.
The arches present poor transverse reinforcement for the containment of the main reinforcement; moreover, there is discontinuity in the concrete casting of the arches lengthways. The discontinuity of the longer arch is justified by the construction technique adopted (Santarella L., Miozzi E., 1948), which entailed the segmental-method construction of two successive layers of cast-in-place concrete.
The inadequate wearing course waterproofing system has allowed water to infiltrate through the expansion joints, with consequent concrete washout phenomena and corrosion of the reinforcements.



[fig. 2] Proof-loading test.

MATERIALS AND STRUCTURAL SURVEY

An accurate visual inspection of the bridge has revealed the presence of typical reinforced concrete deterioration phenomena: inadequate waterproofing of the slab, poor concrete cover of the reinforcement bars, discontinuous concrete casting.

In order to assess the safety of the structure it is necessary to mechanically characterize the construction materials utilized.
It is relatively simple to define the mechanical characteristics of the steel by referring to information on industrial production at the time the bridge was constructed, or by consulting the tests made on specimens drawn from contemporary structures. The initial characteristics may be reduced as a result of corrosion, and the problem then might be to evaluate the presence and extent of corrosion phenomena.

The definition of the mechanical characteristics of the concrete is more difficult since it varies from structure to structure depending on the components, and is uneven within the same structure depending on the laying conditions and subsequent exposure to weather.

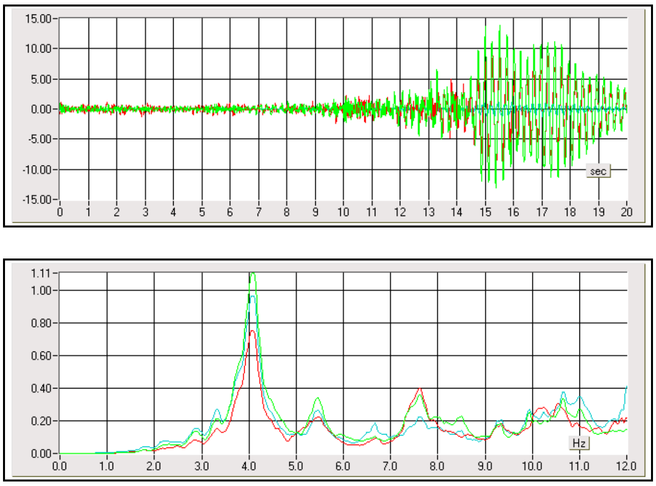
To define the mechanical characteristics of the concrete used in the structure, non-destructive tests have been conducted, and three samples of the material have been taken using the core boring technique.
The testing techniques adopted were: rebound hammer, pull-out and ultrasonic pulse velocity; the on-site values obtained were compared with those obtained from the cores later subjected to a compression test.
The tests for the definition of the characteristics of the materials in sections subjected to the greater stress have been executed on the support zone of the arches and on the bents.
The results of the different tests, in terms of compressive strength, have been compared in table 1.

In-situ tests					In laboratory tests
Rebound Hammer		Pull-Out		Pulse Velocity	Coreboring
Index	MPa	kN	MPa	m/s	MPa
50.4	65.5	31.8	39.0	3040	33.0

[Tab.1] Mean compressive strength value

<u>Carbonation depth in the concrete cover, measured on the core, was typically found to 48 mm, and this one could be the reason of high rebound hammer index.</u>

In order to define the overall behaviour of the bridge structure a dynamic identification method has been used, which consists in measuring the vibrations of the structure induced by environmental excitation factors such as traffic and wind [fig.3].
The frequency and time-frequency analysis (Short Time Fourier Transform) of the signals has enabled us to measure the natural frequencies and modal shapes of the structure.

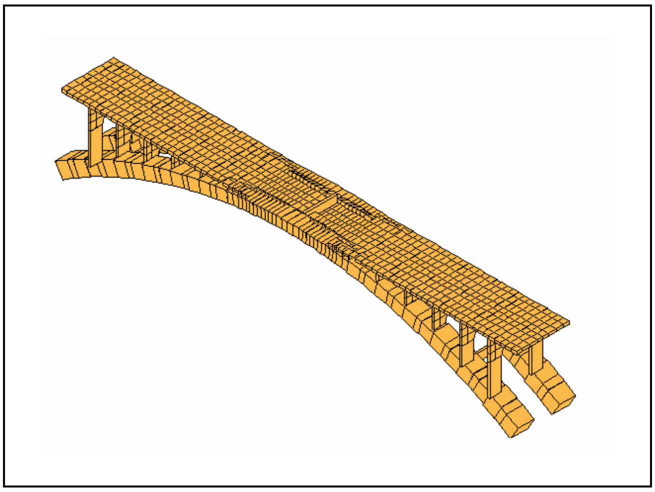


[fig.3] Example of recorded accelerogram and Fourier Transform.

ESTIMATE OF CURRENT SAFETY LEVEL

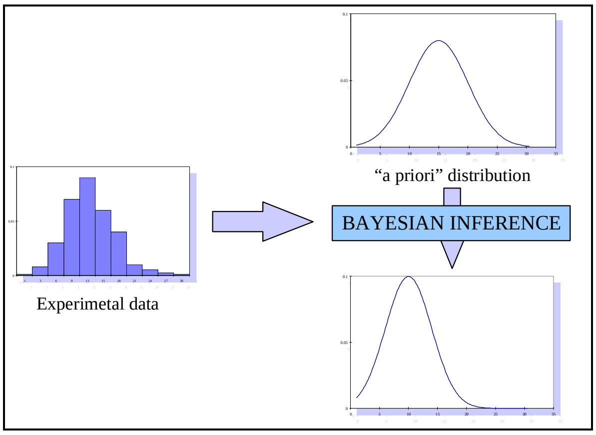
The bridge is currently being used for road traffic only; the railway line no longer exists. The structure

was designed to bear different loads from those provided for by current legislation concerning road bridges.
A refined mathematical model [fig.4] for the analysis of stress distribution in the structure allows us to conduct more accurate structural checks as compared with the necessarily simplified ones utilized in the design (Modena C., Sonda D., 1994).



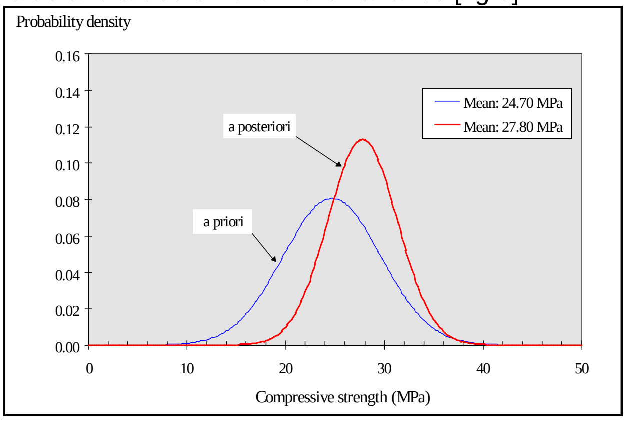
[fig.4] Numerical model of the bridge.

At the design stage the arches of the bridge were designed with dimensions that varied according to the different loads of the railway line and road traffic. Given the load bearing capacity currently required for roads the upstream arch is oversized; if the overall spatial behaviour of the structure is considered the entire bridge proves to be adequate for present loads.
The finite element model utilized for the structural analysis [fig.4] has been calibrated by means of a comparison between the natural frequencies of vibration measured and those of the numerical model, as well as by reproducing in the model the proof-loading tests and by comparing the deformations with the measured ones.
The use of numerical modelling has allowed us to determine with a lesser degree of uncertainty than at the design stage, the state of stress of the sections. Stress distribution has therefore been updated thus reducing the margin of variability.
In order to update the values of the mechanical characteristics of the materials utilizing the test values a Bayesian inference technique has been used, with a probabilistic approach to the data (Madsen H.O., Krenk S., Lind N.C., 1986).
If the distribution law is Gaussian-type, the Bayes theorem assumes a simple expression, and allows us to calculate the conditional probability (Ciampoli M., Napoli P., Radogna E.M.,1990).



[fig.5] Bayesian inference procedure

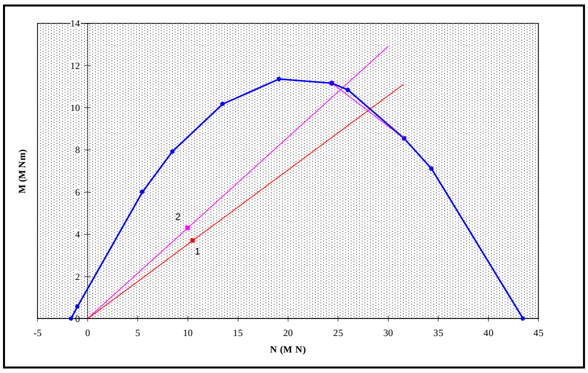
The procedure has been applied to concrete, defining its characteristics "a priori" on the basis of indications provided in the design documentation and of considerations concerning the characteristics of the materials utilized at the time.
The values concerning concrete strength supplied by the tests have been utilized to update the "a priori" distribution by means of a procedure based on conditional probability to obtain a definition of the distribution "a posteriori".
The concrete compressive strength, defined "a posteriori", shows an increment in the maximum value and a decrement in the variance [fig.6].



[fig.6] "A priori" and "a posteriori" probability distributions of concrete compressive strength.

The use of Bayesian inference techniques, coupled with a method of estimating failure probability based on FORM-type procedures (First-Order Reliability Method), allows for direct updating of failure probability as compared with the initial one.
Utilizing level II probabilistic procedures the measurement of structural safety is carried out by means of the safety index β, which is directly linked to the failure probability through the relation: $P_f = \Phi(-\beta)$, and is the measure of the distance of the limit state surface from the origin.
The support cross sections of the arches are stressed by combined compression and bending, and therefore the problem has arisen of defining an adequate limit state function.
A preliminary analysis was carried out to locate in the M-N interaction diagram the area where the representative points of the stress state were

situated, and in this area a linearization of the limit curve was made [fig.7].



[fig.7] M-N interaction diagram in the support cross section of the main arch.

By expressing the line as based on the problem random variables, the limit state function has been defined as the distance between the representative point of the stress state and the line with which the failure M-N interaction diagram has been linearized.
Utilizing the results of the core boring and pull-out tests for the arches we have proceeded to update the probability of exceeding the limit state. In order to update the probability of reaching the bents limit state the results of the core boring, pull-out and rebound hammer tests have been utilized.
The results of the different failure probability values for the main span arches have been compared in table 2.

L = 60.00 m		" *A priori* " *values*		" *A posteriori* " *values*	
Section	*Load Com.*	Safety index β	Failure probability P_f	Safety index β	Failure probability P_f
Springing section (downstream arch)	1	3.97	$3.66 \cdot 10^{-5}$	6.43	$6.54 \cdot 10^{-11}$
	2	3.78	$7.93 \cdot 10^{-5}$	6.16	$3.64 \cdot 10^{-10}$
Springing section (upstream arch)	1	4.15	$1.66 \cdot 10^{-5}$	6.68	$1.17 \cdot 10^{-11}$
	2	3.96	$3.81 \cdot 10^{-5}$	6.46	$5.10 \cdot 10^{-11}$
Section at 45° (downstream arch)	1	4.43	$4.80 \cdot 10^{-6}$	5.99	$1.07 \cdot 10^{-9}$
	2	4.14	$1.76 \cdot 10^{-5}$	5.80	$3.27 \cdot 10^{-9}$
Section at 45° (upstream arch)	1	4.44	$4.41 \cdot 10^{-6}$	6.23	$2.38 \cdot 10^{-10}$
	2	4.32	$7.86 \cdot 10^{-6}$	6.07	$6.39 \cdot 10^{-10}$

[Tab.2] "A priory" and "a posteriori" failure probability values for the main span arches.
The β safety index values can be estimated by referring to the ones indicated in Eurocode 1: the safety index for the ultimate limit state is β=3.8.

CONCLUSIONS

In the paper the application of experimental and theoretical procedures for updating the safety level evaluation of an existing r.c. bridge and thus deciding its possible upgrading is presented.

Numerical models capable of reproducing the spatial behaviour of the structure, which have been calibrated with dynamic identification techniques and with the results of proof-loading conducted at the test stage, have been used for reproducing in an adequately reliable manner the actual behaviour of the structure also under load conditions differing from those of the calibration.

Bayesian inference techniques allowed for a consistent use of the values of the tests conducted on the materials in the frame of the failure probability updating methods applied to the sections of the structure subjected to the more severe effects of the applied loads as provided for by current regulations.

The verifications that have been conducted have demonstrated that the loads effects are lower then the resistance capacity of the structure. It has therefore been possible to hypothesise a widening of the bridge roadway in order to adapt it to present-day traffic conditions, with interventions limited to the elimination of the effects of deterioration, but without having to introduce any structural reinforcements on the existing main structural components (the arches).

The adaptation of the section involved the widening of the roadway from the present 8 meters to 12.30 meters through the construction of two overhangs along the sides of the slab. The new overhanging structure provides for reinforced concrete cantilever beams anchored to the existing structure by means of dywidag-type prestress bars.

The verification of the structure through a numerical model which reproduces the widening for the roadway, and the consequent application of the extra loads required by law, has demonstrated the adequacy of the structure.

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