EXISTING BRIDGES IN ITALY: A REINFORCED CONCRETE MID-CENTURY CASE STUDY




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Abstract

In the framework of an ongoing research project which involved the Politecnico di Milano and the Lombardy region, the paper presents the preliminary investigations conducted on a reinforced concrete post-World War II bridge. The discussed infrastructure was selected, among others, as a pilot case for the development of novel guidelines aimed at: i) defining a first-level methodology for the priority-setting of maintenance operations on existing bridges belonging to the regional road network; and ii) establishing harmonized rules for the structural monitoring of bridge infrastructures. The results presented in the paper confirm the importance of historical investigations and highlight the conflict between economically-constrained diagnostic campaigns and the requirement to maximize the knowledge level of a structure that, after 70 years of operation and a minor maintenance, still carries increasing volumes of traffic loads.

Keywords: Bridge inspection; Bridge maintenance; Structural safety; Reinforced concrete structure

1 Introduction

Ensuring adequate performance of the infrastructures over time is nowadays a basic concept, the intention of which is providing good manufacturing quality, the careful maintenance of structures over time and a continuous survey of bridges behaviour under service condition.
A proactive infrastructure management system is needed to develop a sustainable plan of inspection, maintenance and rehabilitation. It is necessary to focus on improving a risk assessment method that can be efficiently implemented by infrastructure managers [1].
Within this context, the Politecnico di Milano is currently involved in a research project aimed at developing a rational approach to identify the intervention strategies and priorities targeting the broad infrastructural heritage of the most industrialized region in Italy (Lombardy), which counts one-sixth of the Italian population and about 10,000 bridges.

 ISBN 978-80-907611-3-1

The first purpose of the research activity was to systematically collect the documentation needed to build a technical and historical information database regarding about 400 selected bridges, carefully identified based on their location, intersecting object, material and structural scheme. At present, jointly with the 12 provinces involved in the work, the first database, consisting of 92 main parameters, has been filled for 289 of the reference bridges. The data collection campaign, which occupied a large portion of the project timeline, highlights the actual knowledge of the infrastructural heritage on our territory. Since many structures of interest resulted in lack of historical and technical documents, a clustering based on the amount of reliable information about the bridges was initially considered. Subsequently, a step-by-step analysis of parameters led to a classification of the whole sample of bridges. Comparative datasets were assumed by combining information included in the database and the knowledge regarding the economic impact concerning the network served by each infrastructure [2].
The comparative analyses finally defined four main priority classes and four relative subclasses that could be used by infrastructure managers in choosing the actions to be implemented for the maintenance of the structures.
After a final clustering, a selection of eight bridges was considered for further investigation and as representative case studies for the definition of general criteria and guidelines for the management and maintenance. Specifically, structural diagnostics, load tests and structural monitoring campaigns have been designed and implemented.
One of the eight pilot bridges is hereunder presented. The case study is framed within a mid-high class of priority, because of the adequate overall bridge condition; additionally, collected documents, consisting of calculation reports and original drawings, allowed to reconstruct the history and the design choices.

2 Case study: a mid-century reinforced concrete bridge

2.1 Historical overview and preliminary structural assessment

Over the past decades, the capacity and service condition of bridges have worsened due to rapid traffic growth and often insufficient maintenance funds. The current demands in terms of load magnitude and frequency are much higher than the ones that were initially considered in the design, because of the increased freight transportation. Therefore, when the performance of existing bridges calculated according to current design codes is considered, many infrastructures inadequately respond to the Ultimate Limit State (ULS) design loading. The paper explores the bearing capacity of a 70-year-old bridge, based on the results obtained from diagnostic tests and structural monitoring, still ongoing. The purpose of the study is to verify the actual conditions of the structure and to define how its service life can be furtherly extended.
The case study presented in this paper focuses on a reinforced concrete grillage bridge in the North of Italy. Because the infrastructure serves an important industrial area and connects a densely populated municipality to the highway network, significant traffic volumes insist on the bridge. For this reason, it is considered strategic in the road network.
Gaining knowledge about the structure and its service history is crucial for the development of diagnostic tests and the analysis of monitoring results. The bridge, constructed in the 1950s, consists of three-span continuous girders, which cross a river; the riverbed intersects the structure obliquely (about 33°). The superstructure of the bridge is composed of

ISBN 978-80-907611-3-1

longitudinal and transverse beams, with variable depth throughout the length; the overall shape results in a sequence of three arches, as depicted in **Fig. 1**.
The full length of the bridge is about 43 meters; the deck is supported by two reinforced concrete piers placed on the riverbed at a distance of 22 meters (i.e. the length of the middle span). Two lateral cantilever spans cover a length of 10.4 meters. The supports consist of a layering of lead sheets, aimed at allowing relative sliding modes.
The caisson-type foundations interact with the soil through a system of circular reinforced concrete piles. The deck has a width of 17.5 meters and includes four lanes (two for each opposite directions of the traffic flow) and two cantilever sidewalks.

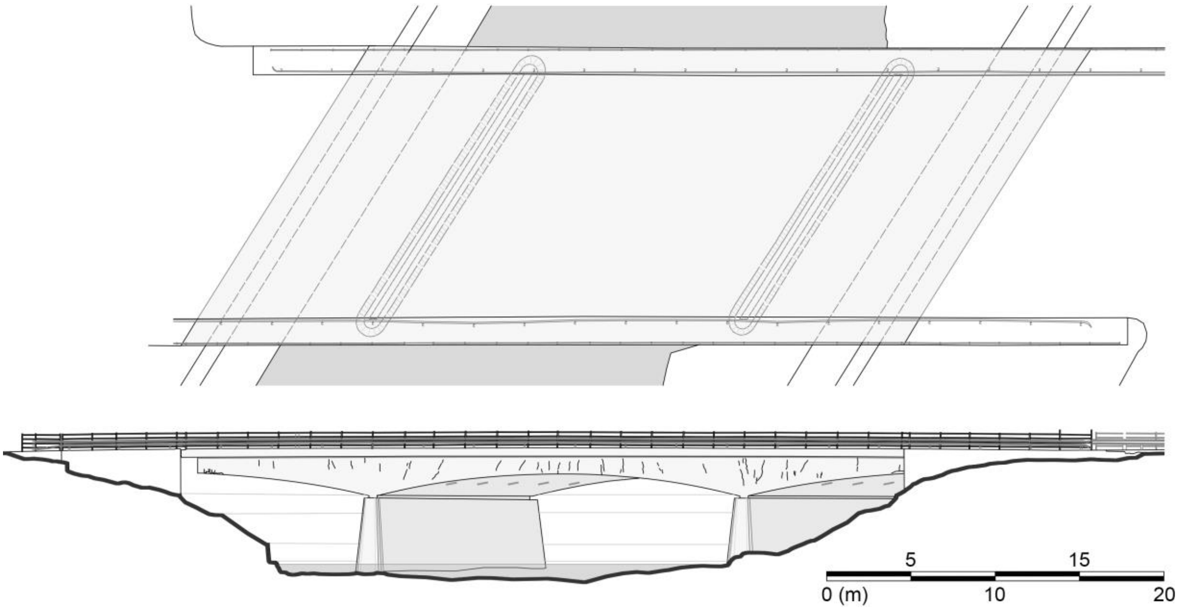


Fig. 1 Top and downstream views of the bridge.

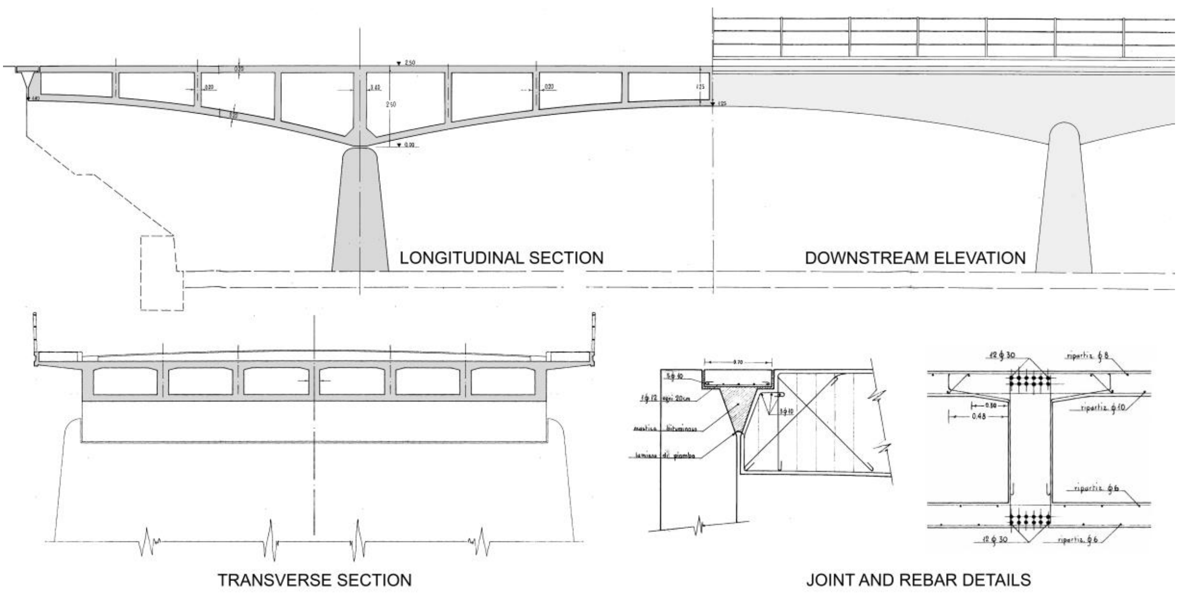


Fig. 2 Representative technical drawings of the original design.

 ISBN 978-80-907611-3-1

The bridge structure is fully described by the original documents, which define the geometry of the structural elements (Errore. L'origine riferimento non è stata trovata.), the assumed design loads, the structural scheme, the static calculation of internal forces and stresses, the joints detail and the arrangement of the reinforcing bars (Errore. L'origine riferimento non è stata trovata.). The calculation report shows the main checks carried out at the most stressed sections of the deck, the foundation system and the piles.
With reference to the material properties, the design documents state that a high strength cement (680 kg/cm^2 type) was used, while, concerning the reinforcing bars, a "*RUMI L.U. 4000*" type was adopted. These bars are of high adhesion type and, according to historical information, are characterized by a yielding strength ranging between 4000−4500 kg/cm^2 and a tensile strength ranging between 5700−6500 kg/cm^2.

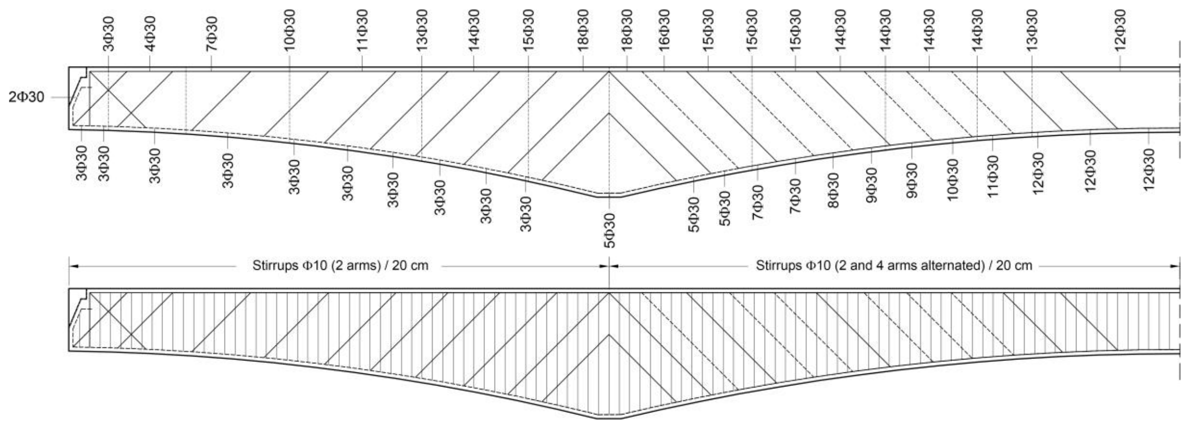


Fig. 3 Longitudinal beams rebar arrangement (left half of the bridge span), as reconstructed from the original design report.

Analysing the available technical reports, it also emerged that no significant maintenance nor rehabilitation interventions were executed during the working life of the bridge. Nevertheless, a few capacity assessments have been carried out over time, concomitantly to the passage of heavy trucks exceeding the national road code limit (more than 108 t).
A review of the project documentation indicated that the bridge was designed imposing the action of heavy military vehicles, as prescribed by the military Codes mandatory in the 1950s. With reference to the variable loads that were considered in the original design, the deck was proportioned imposing the transit of different military vehicles (global weight up to 74.5 t) and a road roller with an overall weight of 18 t. Two examples of the different load configurations assumed in the longitudinal and transverse calculations are shown in **Fig. 4**.
The original design resulted in a massive and robust structure, characterized by a significant incidence of the dead load. In addition, the heavy loads prescribed by the guidelines n. 19096 of 1953 **Errore. L'origine riferimento non è stata trovata.** and the design approach adopted during the 1950s, led to a structural capacity that, according to ongoing preliminary analyses, could guarantee an adequate safety factor also with respect to the current design Codes.
A recent photogrammetric inspection, carried out in February 2020, allowed to identify the cracking pattern on the external longitudinal beams (**Fig. 1)**, particularly dense in the support regions and at the midspan of the bridge (maximum negative and bending moments, respectively). However, according to preliminary monitoring results, none of those cracks -

probably developed by shrinkage and thermal effects - appears able to compromise the overall stability of the structure.
Following the in-depth investigation campaign promoted by the mentioned research project, some structural and maintenance problems have emerged. Firstly, a crucial issue concerns the discrepancy among the original documents and the as-built structure. Because of this, past capacity assessments were based on the erroneous documentation, exposing the structure to potentially dangerous situations. The most significant and macroscopic difference is about the number of longitudinal beams. As a matter of fact, the original drawings and the calculation report present 7 longitudinal beams distributed over the transverse section (see Errore. L'origine riferimento non è stata trovata.), while only 6 beams were identified by the topographical survey and the radar investigations (**Fig. 5**). Moreover, minor discrepancies of the beams width and the slabs thickness were observed.

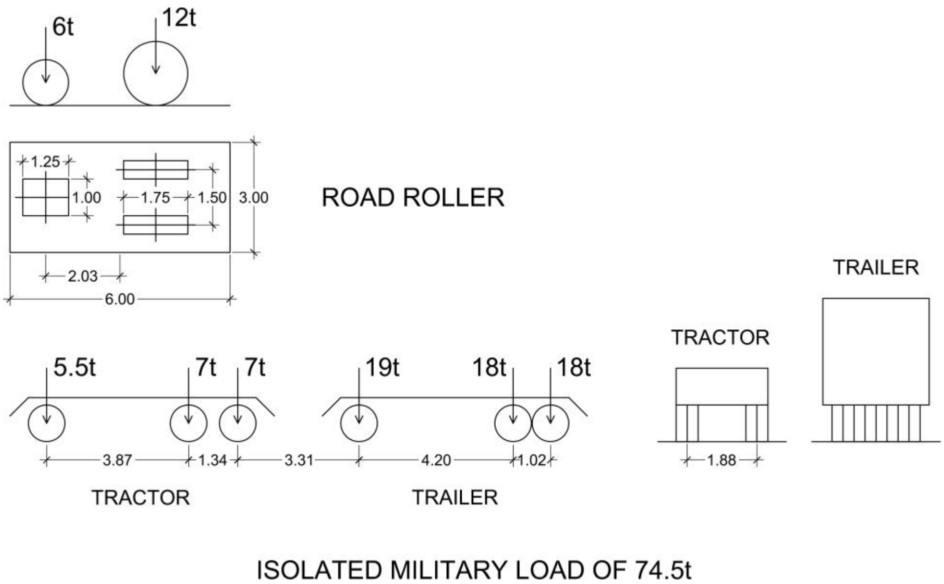


Fig. 4 Examples of the loading diagrams contained in historical documentation, with indication of the load applied by each axle.

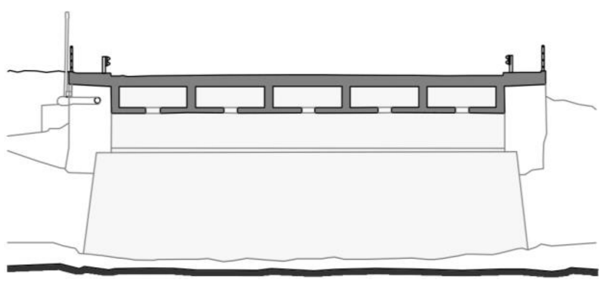


Fig. 5 Transverse section of the bridge, as reconstructed by the topographical survey.

Regarding the maintenance issues, abutments and piers exhibit local damage due to insufficient concrete cover, which is the result of a faulty construction. Moreover, cracks in the concrete, discontinuities in the waterproof barrier and damaged structural joints become the main entry point of water (e.g., rain, snow, ice, salts) and this could lead to hazardous failures associated to the corrosion of internal reinforcing bars. Furthermore, also the

 ISBN 978-80-907611-3-1

structural bearings seem to be affected by ageing and this may result in prevented longitudinal thermal elongations.

2.2 Structural diagnostics

In order to assess its structural condition, and to estimate the mechanical properties of concrete, the bridge was subjected to multiple destructive and non-destructive testing methods. The introduced visual inspection of the structure, combined with the topographic survey, allowed to select the points of interest, thus the optimum location of the tests, the number of which was constrained by budget restrictions.
Regarding destructive testing, concrete cores were drilled out from the left bank pile and from the downstream deck beam, while carbon-tests were performed on the right bank pile. It is important to mention that, since the aim of the diagnostic campaign was to acquire only a preliminary and representative knowledge on the mechanical properties of the concrete, the number of drilled cores was limited to four specimens. On the other hand, less costly non-destructive tests were distributed throughout the accessible regions of the structure. In particular, pulse velocity tests were conducted on the deck downstream beam and on the left bank pile, while rebound hammer tests were carried out in correspondence to the drilled cores.

Concrete core drilling test

The drilling of the concrete cores was carried out following the procedure established by the UNI EN 12504-1. Two specimens were drilled out from the left bank pile (B1-downstream- and B2-upstream) and two from the deck beam facing the downstream side of the bridge (B3 and R1, left and right side respectively). All specimens presented a diameter of 94 mm and were subjected to ultrasonic and carbonation tests before being tested under uniaxial compression (UNI EN 12390-1, UNI EN 12390-3). In order to obtain a reliable estimation of the on-site concrete strength, results from the compression tests of the specimens ($f_{c,i}$) were corrected (see Eq. 1) based on the geometrical characteristics of the specimens ($C_{h/D}$, C_{dia}), the presence of embedded rebars (C_r), and the possible alteration on specimen integrity produced by the drilling process (C_d) [3]:

$$f_{c,i\_corrected} = (C_{h/D} \cdot C_{dia} \cdot C_r \cdot C_d) f_{c,i} \tag{1}$$

Tab. 1 summarizes the results of the ultrasonic, carbonation, and compression tests performed on the specimens. It is worth to mention that before the compression testing, longer specimens B1 and B2 were cut in half, thus resulting in 4 cylindrical specimens (B1A, B1B, B2A, and B2B in **Tab.1**).

Tab. 1 Results of concrete core drilling tests.

Specimen	**Height** (mm)	**Ultrasonic Pulse test**	**Carbonation test**	**Compression test**		**Corrected compressive strength** $f_{c\_corrected}$ (MPa)
		Mean velocity (m/s)	**Depth of carbonation** (mm)	**Max. compressive load** (kN)	**Compressive strength** f_c (MPa)	
B1A	189	4088.0	60	149	21.47	23.65
B1B	188	4162.4		112	16.14	19.37
B2A	193	3683.2	40	113	16.28	19.66

B2B	98	4323.6		177	25.51	22.82
B3	173	4455.0	35	234	33.72	36.31
R1	83	4724.9	-	314	45.25	37.82

Carbon tests

The test method consists of drilling the concrete structure with a percussion drill equipped with a tip for masonry. The powders produced during the drilling are collected in a test tube and are immediately wetted with an alcoholic solution of phenolphthalein. Because of the chemical reaction, the concrete not yet affected by carbonation adopts a magenta red colour, while the colour of the carbonated material remains unvaried. For the case study, three equidistant tests were performed on the right bank pile of the bridge (see results in **Tab. 2**).

Tab. 2 Results of carbon tests performed on the right bank pile

Location of the test	**Drilling depth** (cm)	**Powder height** (cm)	**Carbonation depth** (cm)
Downstream	7	13.5	6
Middle	6	15.5	6
Upstream	6.5	15	2.5

Sonic and Ultrasonic Pulse Velocity tests

Among the available NDT for the structural assessment of reinforced concrete structures, Sonic and Ultrasonic Pulse Velocity (SPV, UPV) tests are well-known as promising methods to examine the homogeneity and quality of the material. The test consists of measuring the time taken by sonic or ultrasonic pulses to pass through a solid between two transducers located at a given distance. The propagation velocity of the elastic wave is calculated by dividing the distance between the generation and acquisition points by the travel time of the wave. A reduction in the propagation velocity is observed when lower density areas are present (e.g., presence of voids or cavities).

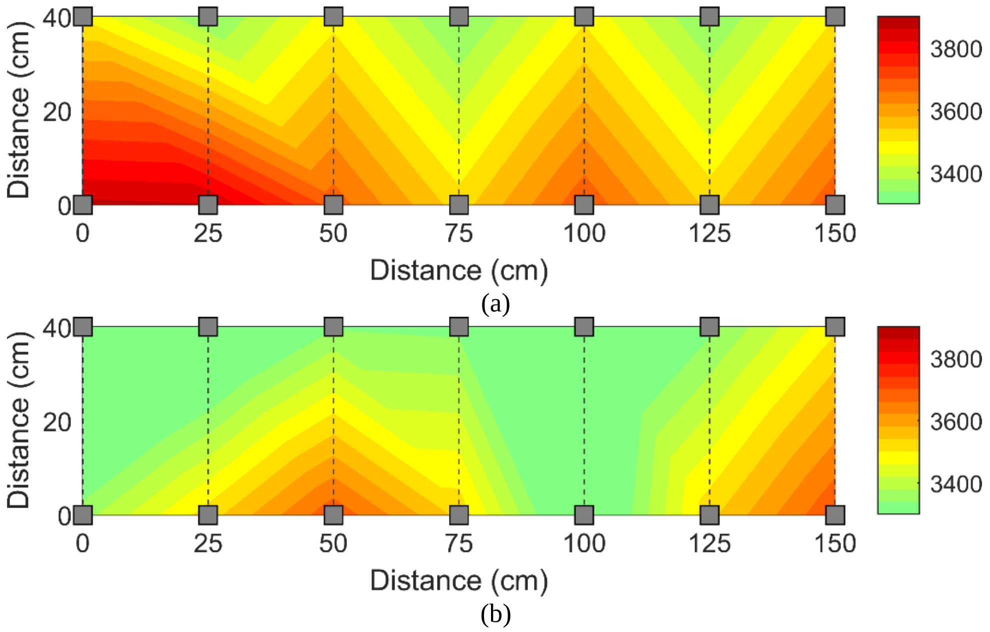


Fig. 6 Results of SPV tests conducted on left pile (a) upstream, (b) downstream (UP velocities expressed in m/s).

 ISBN 978-80-907611-3-1

For the case study, two direct SPV transparency tests were conducted, according to the current regulation (UNI EN 12504-4) by using an impulse hammer and a receiver transducer on the left bank pile of the bridge. The distance between the generation and acquisition points corresponds to the transverse thickness of the pile. In **Fig. 6**, the results of the SPV tests are illustrated in terms of rate of wave propagation (m/s) throughout the section of the pile.
In addition to the direct SPV tests, some measurements of rate of wave propagation were carried out using the UK1401 ultrasonic pulse velocity tester. The device allows to obtain measurements by performing surface scanning with a fixed base. Mean values and standard deviation of the results are reported in **Tab. 3**.

Tab. 3 Results of UPV tests.

Test device	**Test location**	**Mean SP/UP velocity** (km/s)	**Standard deviation** σ (km/s)
Direct test	Pile (upstream)	3.56	0.178
	Pile (downstream)	3.39	0.193
UK1401 tester	Pile	3.01	0.821
	Deck (left side)	4.48	0.119
	Deck (right side)	4.43	0.201
	Deck beam (downstream)	3.98	0.158

Rebound hammer test

In order to estimate the compressive strength of the concrete surface layer, rebound hammer tests were performed in correspondence to the points where destructive tests were carried out, in particular at the location of specimens B1, B2 and B3. Tests were conducted according to the procedure described by UNI EN 12504-2, with a direction of the concrete test hammer perpendicular to the test surface (0° inclination). The output of the test is the Rebound Number (RN), namely the bounce height measured after projecting an elastic mass within the device against the test surface, at a certain measuring point.
The grid of measurements was set equal to 20×20 cm for each test. Results are reported in **Tab. 4**. According to the UNI EN 12504-2, a minimum of 9 measurements must be performed, and, when calculating the mean value of the Rebound Number, if one of the results exceeds by 6 or more units the mean value, it must be excluded, and a new mean of the Rebound Number should be calculated.

Tab. 4 Results of Rebound Hammer tests.

Test Notation	**Inclination angle**	**Number of measurements**	**Mean Rebound Number** (RN)	**Standard deviation** σ	**Number of excluded measurements** (RN ≥ MRN + 6)
B1	0	25	35	4.91	5
B2	0	25	32	3.09	1
B3	0	13	45	2.40	0

Sonreb method

When estimating the mechanical properties of the concrete, factors as the ageing of the structure or the uncertainties in the concrete matrix may play an important role, especially when these estimations are obtained from non-destructive techniques. In fact, considering that the superficial strength of concrete increases with the ageing, higher values of rebound number may be obtained from aged concretes, while ultrasonic wave propagation decreases.

The Sonreb (Sonic + Rebound) method takes into consideration these effects and allows to estimate the compressive strength of concrete by combining the results from ultrasonic testing (*V*) with those of rebound hammer testing (*S*). The general expression of the method (Eq. 2) introduces some correction parameters (*a*, *b* and *c*) which may adopt slightly different values depending on the literature.

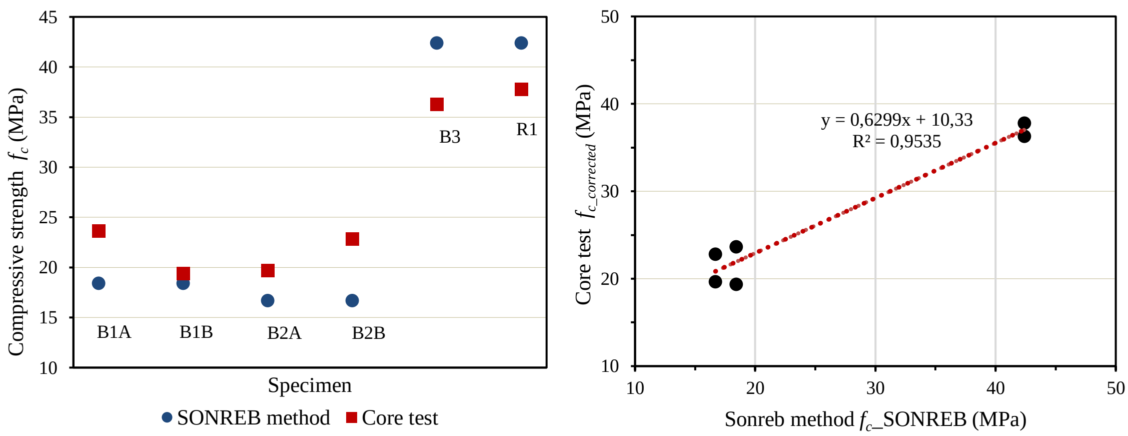
$$f_{c\_SONREB} = a\, V^b S^c \tag{2}$$

In **Tab. 5**, results from the application of the Sonreb method are reported with values of correction parameters *a*, *b* and *c* adopted from [4]. In order to apply the method with values obtained from same testing devices, only the results of mean ultrasonic pulse velocities measured with the UK1401 tester were considered.

Tab. 5 Results from the application of Sonreb method

Test location	**Mean UP velocity** *V* (km/s)	**Mean Rebound Number (RN)** *S*	**Compressive strength** f_{c_SONREB} (MPa)
Pile (upstream)	3.01	32.29	16.69
Pile (downstream)	3.01	34.95	18.42
Deck beam (dowstream)	3.98	45.08	42.39

A good correlation is observed when results estimated with the Sonreb method are compared with those obtained from uniaxial compression tests (see **Fig. 7**).



(a) (b)

Fig. 7 Comparison between Sonreb method and core testing results (a) compressive strength of specimens, (b) analysis of the results: linear regression.

Effects of time in the compressive strength of concrete

Considering that the case study bridge was built in the early 1950s, the effect of ageing of concrete was evaluated on strength values obtained from compression tests [5] (Eq. 3). Hence, an estimation of concrete strength at the time of construction was obtained by reversing the following equation:

$$f_{cm}(t) = \beta_{cc}(t) \cdot f_{cm,28d} \tag{3}$$

Proceedings
ISBN 978-80-907611-3-1

where $f_{cm}(t)$ is the mean compressive strength at (t) days, $f_{cm,28d}$ is the mean compressive strength at 28 days and $\beta_{cc}(t)$ is a function describing the strength development with time:

$$\beta_{cc}(t) = \exp\left\{s \cdot \left[1 - \left(\frac{28}{t}\right)^{1/2}\right]\right\} \tag{4}$$

where s is a coefficient which depends on the strength class of cement.

The results of cubic strength of concrete at time of construction $R_{ck,28d}$ (**Tab. 6**) are finally compared with the data contained in the database of concrete cubes tested at the Politecnico di Torino in 1955 [6] (see **Fig. 8**). As one should note, the obtained strengths correspond to the typical values measured in the 1950s on cast-in-place concretes.

Tab. 6 Time effects on concrete strength at t = 25.550 days (70 years).

Reference structural element	f_{cm} **(t)** (MPa)	s	β_{cc} **(t)**	f_{cm} (MPa)	R_{ck} (MPa)
Pile	21.4	0.38	1.44	14.80	17
Deck beam	37.1	0.38	1.44	25.66	30

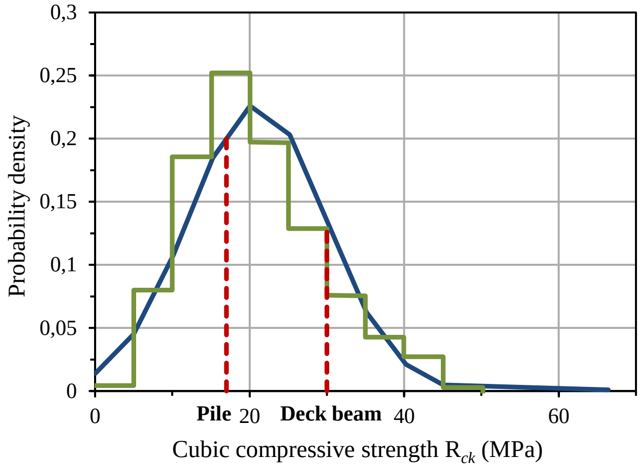


Fig. 8 Comparison between the obtained mechanical strengths at 28 days and the probability distribution of cubic compressive strengths in 1955, after [6].

Ground-penetrating radar investigations

Multiple sections of the bridge were scanned with a ground-penetrating radar (GPR), to investigate the internal structure of the bearing members, with the aim of confirming unmeasurable geometrical information, concrete covers and rebar positions. The method allowed to identify further discrepancies between the original design and the as-built structure, with particular reference to beams widths and slabs thicknesses.

However, it is fair to note that, referring to the rebar distributions displayed in Errore. L'origine riferimento non è stata trovata., only qualitative information was gathered, as the ability of radar techniques to fully reconstruct the reinforcement layout of an existing structure still appears limited. The results of a GPR scan on the external face of a downstream beam are shown in **Fig. 9**, where the horizontal axis measures the position along the beam length and the vertical one corresponds to the depth along the beam width. It can be noticed that the vertical stirrups are satisfactorily identified (at a depth of about 50 mm), while internal rebars as the 45° inclined shear reinforcement are located with lower accuracy and can be easily mistaken with concrete inclusions.

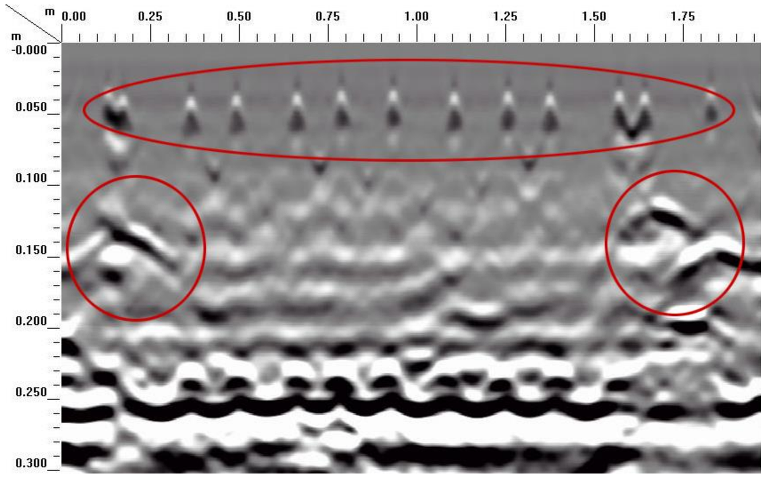


Fig. 9 GPR scan on the external face of the downstream beam (positions expressed in m).

3 Concluding remarks and possible retrofitting interventions

The paper presents the results of the historical and experimental investigations conducted on a 70-year-old reinforced concrete bridge, still operating on a heavy traffic network in northern Italy. It is possible to draw the following conclusions:

1) preliminary analyses suggest that, despite the age, the bridge can acceptably sustain the loads imposed by the actual operating conditions, thanks to the inherent structural robustness of the grillage deck and the concrete strength gained over time (about 40% greater than the 28 days one).
2) The critical review of historical documents and the check of geometrical and mechanical parameters through field operations are of utmost importance, since the lack of investigation may result in the continuous adoption over time of macroscopically wrong assumptions (e.g. the number of longitudinal beams).
3) After 70 years of service life, the bridge exhibits local damage due to rebars exposure and carbonation depths frequently greater than the concrete cover. The loss of alkalinity in the rebar regions might favour the build-up of corrosion products that can compromise the bearing capacity of the affected members and should be investigated with greater care.

Possible maintenance interventions may comprise the restore of concrete covers and the application of innovative fabric-reinforced cementitious mortars (FRCM), even though further research is needed to better clarify whether such technologies can effectively control the degradation of the embedded reinforcement.

4 Acknowledgements

The authors thank the Lombardy Region, the provincial technicians and the Politecnico di Milano team involved in the bridge assessment research project mentioned in this paper.

 ISBN 978-80-907611-3-1

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