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ORIGINAL ARTICLE

EUROSTRUCT

A proposal for low cost condition assessment method for existing RC bridges

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Abstract

Aging infrastructures require huge budgets to preserve their functionality and the lack of effective maintenance leads to increasing deterioration and therefore higher repair costs. For these reasons, assessing the condition of infrastructures becomes mandatory, with particular attention to the ones still in service even when their life limit has been exceeded. In particular, this problem is really important in Countries, like Italy, in which there are several operating bridges at the end of their service life. In this framework, many companies responsible for the management of road networks have turned their interest towards Bridge Management Systems (BMS). This paper presents a fast and low-cost method for condition rating of reinforced concrete bridges. The proposal is based on visual inspection and non-destructive testing. A specific parameter is designed to take into account the mechanical degradation of materials and the damage location at the structural sub-component level. Some benchmark case studies have been discussed in order to compare the proposed method with other approaches available in literature.

Keywords

Existing bridges, BMS, concrete, RC bridges.

1 Introduction

The issue of aging infrastructure is a pressing concern today, particularly in countries such as Italy where major highways were constructed more than 50 years ago. The functionality of these structures is highly dependent on proper inspection and maintenance, and neglecting these activities leads to deterioration and higher repair costs. As a result, evaluating the reliability of these infrastructures is crucial, especially for structures still in use despite exceeding their lifespan: [1] and [2].

Recently, investments in the development of a Bridge Management System (BMS) have increased. BMS is a set of processes that include inspection, investigation, maintenance and repair of bridges or viaducts, organized by priority, using computer databases and algorithms. The conservation of a structure is usually evaluated through qualitative judgments and bridge rating or scoring is used to prioritize maintenance investments. The BMS takes into account the serviceability and importance of the bridge in the road network to prioritize maintenance activities.

An early approach to this problem was proposed by Znidaric and Perus [3], who analyzed condition rating techniques for Reinforced Concrete (RC) bridges. They suggested that the evaluation method should not be based on simple scoring, but on a numerical assessment of damage types revealed during inspection, which can have a significant impact on the safety and durability of the structure.

Gattulli and Chiaramonte [4] further expanded on this approach, including the evaluation of steel and masonry bridges and assessing the condition of each subcomponent of the overall structure. Kano and Morikawa [5] applied this system to RC structures damaged by chloride induced deterioration and introduced a parameter to represent inspection uncertainties.

In a different approach [6], the spatial time-dependant reliability analysis was combined with visual inspection to predict the likelihood of RC corrosion-induced cracking. Kušar and Šelih [7] studied a large number of bridges and found that climate and exposure to water have the greatest impact on bridge condition.

Other methods to prioritize bridges and suggest maintenance strategies at a network level have been proposed, such as the Integrated Bridge Index IBI [8] which considers the vulnerability risk and strategic importance of each component, and a ranking strategy based on a multi-attribute utility theory [9]. Fuzzy logic has also been used to obtain interesting results: [10] and [11].

There has been extensive research on using multiple sources of information for infrastructure condition ranking

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[12] , including information from laser scanners [13], Non-Destructive Techniques (NDT)[14] - [15] and visual inspection [16].

Materials degradation is a crucial issue for BMS [17] and should be taken into consideration in any approach. This element become critical especially in the case of extreme weather events producing an increase in environmental action [18] and threating the bridges, especially the ones of reduced spans [19] that should be tackled also at a network level [20].

In this paper, the authors present an improved version of the early method described by Znidaric and Perus [3] that takes into account mechanical degradation of materials and damage location, allowing for a fast and low-cost condition rating of the RC bridge network that can be easily integrated into BMS.

The paper is organized as follows: Section 2 presents the outline of the proposed method. Section 3 describe its application to significative case studies Finally, perspectives and conclusions are drawn in Section 4.

2 Outline of the method

The proposed evaluation method in this paper consists of three main phases. Firstly, a comprehensive visual inspection is conducted to identify any harm to the structure. Secondly, a series of non-invasive tests are conducted to assess the mechanical properties of the materials. Lastly, the outcomes of the initial two phases are combined and evaluated to obtain a Condition Rating Number (CRN), which is a non-dimensional number that reflects the extent of damage in the analyzed structure. The CRN is defined as follows:

$$CRN = \gamma \frac{\sum_{m=1}^{k} G_{Dm}}{\sum_{m=1}^{k} G_{D,ref\,m}} 100 \qquad (1)$$

where γ is an adjustable scale constant $G_{D,ref\,m}$ is the corresponding maximum value, while the definition of G_{Dm} the condition rating number for the m-th structural component is represented by equation (2):

$$G_{Dm} = \mathrm{K_m} \sum_{i=1}^{n} B_i K_{2i} K_{3i} L_i T_i \quad (2)$$

$\mathrm{K_m}$ is a factor denoting the significance of the specific component in the structure. Its values are listed in [3]. The potential impact on the safety of the structural element due to the i-th damage is represented by B_i It varies between 1 and 4, see [3] Table A1. K_{2i} represents the severity of the ith damage, divided into four classes, with values ranging from the lowest to the highest. Its values range between 0.5 and 2.0, the complete definition is reported in [3] Table A3. The extent of the damage along the structural element is represented by K_{3i} and has values ranging from 0 to 1, as specified in [3]. The symbol L_i represents the location of the i-th damage on the structural component, and it can have binary values of either 1 or 2. A value of 1 indicates that it is not a critical point, while a value of 2 means it is a critical point, as shown in Table 1. The term "critical point" refers to the portion of the single structural component that is considered to be crucial for the structural stability, such as areas with maximum stress levels or stress concentrations (such as beam midspans, support zones, and holes). Determining critical points requires knowledge of the boundary and loading conditions, and so a careful evaluation of this parameter must be performed for each individual case. The coefficient T_i represents the materials degradation. Its values are listed in Table 1 and depend on the relationship between the design strength of the material (f_{mk}^{d}) and the strength determined through experimental tests such as the rebound index, coring strength test, or ultrasound pulse test (f_{mk}^{exp}). If the design strength is not available, it is assumed to be equal to the minimum value required for the specified exposure class (as stated in [21-22]). It should be necessary to have at least 3 mechanical tests on each structural element in order to take the average value. If it is not possible to conduct any experimental tests on the structural element, T_i is set to 4 as a safety precaution. In the case of reinforced concrete structures, f_{mk} is initially assumed to represent the concrete compressive cylindrical strength. However, if additional information about the reinforcing elements is available, f_{mk} becomes the weighted average of the concrete and steel mechanical properties, as expressed in equation (3).

$$f_{mk} = \mathrm{f_{ck}} + f_{yk} \cdot \frac{E_c}{E_s} \quad (3)$$

In the equation (3), E_c and $\mathrm{f_{ck}}$ represent the Young's modulus and the characteristic strength of concrete, while E_s and f_{yk} represent the corresponding values for steel. The equation provides the values of f_{mk}^{exp}, when the mechanical properties determined through experimentation are taken into account, and f_{mk}^{d}, when the design values of the materials are used.

Table 1 Material characteristics degradation expressed as T_i values

Criterion	f_{mk}^{exp} / f_{mk}^{d}	**T_i**
High resistance	>1	1
Poor resistance	from 0.66 to 1	2
Low resistance	from 0.33 to 0.66	3
No resistance	from 0 to 0.33	4

To determine the characteristic values of G_{Dm} and $G_{D,ref\,m}$ (as shown in equation (2), each component of the structure must be thoroughly examined to identify any defects. Once this information is gathered, the overall condition of the structure can be quantified using the CRN value calculated by equation (1). Based on this value, the structure can be classified into one of the 4 damage categories listed in Table 2. A higher CRN value indicates a worse condition, while a lower value corresponds to a better condition. To have more details please see [23].

It is important to highlight that the proposed method is based on visual inspection and on a small number of tests to assess the mechanical properties of the materials. For this reason, it is considered a low-cost condition assessment method.

Table 2 Damage categories for the proposed method

Damage categories	CRN/γ
In service	0.00 -1.36
Little deterioration	1.36 -1.86
Severe deterioration	1.86 – 2.27
Urgent intervention	2.27 – 2.95
Out of service	>2.95

3 Case Studies

3.1 The Concorde Bridge case in Canada

The Concorde Bridge in Laval, Quebec (located in Montreal) spans Canadian Highway 19. It is made up of 20 pre-stressed concrete box girders, each measuring 28 meters in length, that support a 24-meter-wide deck slab. The deck was cast on the construction site and then covered with a waterproof membrane and asphalt. The overpass is supported by two abutments at either end, which consist of an inclined front wall and four longitudinal retaining walls. These abutments hold a thick cantilever slab that is connected to the deck slab through an expansion joint. Unfortunately, on September 30th, 2006, the Concorde Bridge suffered a collapse, resulting in five fatalities and six injuries (refer to [24]).

Table 3 G_{Dm} values for the structural components of Concorde bridge according to the different reports, the first two columns represent the Km parameter and the reference value $G_{D,ref\ m}$.

	Abutment East	Abutment West	Girder beam	Deck slab
K_m	0.4	0.4	0.6	0.4
$G_{D,ref\ m}$.	512	512	499	320
1984	0.0	0.0	0.0	0.0
1991	1.6	0.0	0.0	0.0
1992	24.0	14.4	4.8	0.0
1997	67.2	14.4	4.8	7.2
2004	89.6	19.2	9.6	9.6
2005	89.6	19.2	19.2	9.6

This bridge was built in 1970 and underwent multiple inspections throughout its service life, as documented in [24]. The first recorded inspections from 1977-1978 showed no major abnormalities. However, by 1980 a leaky expansion joint was identified. Inspections from 1985-1991 showed small signs of deterioration but the overpass was still considered safe. In 1992, the overpass showed significant damage with defective expansion joints and cracks in the pavement. Concrete scaling and exposed reinforcements were also detected in the east abutment. In 2004, a major degradation of the beam seats and wide shear cracks in the cantilever were noted in the inspection report. The ultimate inspections in 2005 confirmed these findings and assessed the overall condition of the overpass as "mediocre." The collapse of the bridge on September 30th, 2006 was due to a shear failure in the southeast cantilever, caused by the deterioration of the concrete. The main cause of the deterioration was believed to be the freeze-thaw cycles and the use of de-icing salts.

The proposed method was applied to this case study, pictures and details reported in [24] and [25] have been analysed. Table 3 reports a synthesis of the results concerning each structural component and shows that the east abutment was the main problem, while damages in other parts of the bridge were not as severe.

The global CRN (defined in equation (1)) based on the above-mentioned inspection reports is shown in Figure 1. A dangerous situation after 1997 can be clearly seen, suggesting caution to the bridge conditions.

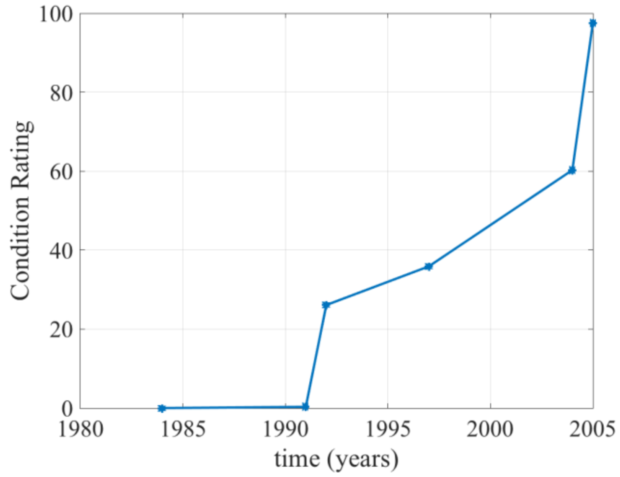

Figure 1 Condition ratings of the Concorde Bridge in Laval Quebec

3.2 The case of a Bridge net in Sardinia

The proposed method was applied to a group of reinforced concrete bridges located in Sardinia, Italy. All of these bridges have a similar structural design, consisting of a horizontal slab supported by transverse and longitudinal beams, which are simply supported by the lateral abutments, as shown in Figure 2.

Based on the number of transverse and longitudinal beams, the bridges were divided into three types, as indicated in Table 4. Since the design strength of the material was not available, the minimum resistance class of C30/37 was assumed as the reference strength, based on the exposure class XC4, as specified in [17]. For simplicity, the scale constant γ was set equal to 1.

A series of both destructive and non-destructive tests (such as the core compressive strength test, sclerometer, pull out, ultrasound wave velocity test, etc.) have been conducted on each bridge. To simplify the results, the average concrete resistance class determined from the experiments for each group is presented in Table 5. This is feasible because bridges of the same type tend to have similar characteristics. Generally, larger bridges (types T3-T2) have superior mechanical properties compared to smaller bridges (type T1).

Figure 2 T1 type bridge (up), T2 type bridge (middle), T3 type bridge (bottom).

Table 4 Description of Bridge net

Label	Long. Beam	Transv. Beam	Type
P01	2	1	T1
P03	2	1	T1
P07	2	1	T1
P08	2	1	T1
P10	4	2	T2
P12	8	2	T3
P13	8	2	T3

To demonstrate the method, let's consider the case of Bridge P08 which belongs to type T1. Figure 3 displays its geometric measurements and structural design. The visual inspection, along with the experimental tests, generated a damage identification form for each structural component. Examples for the abutment, beam, and slab are provided in Tables 6-8. These tables also include the values of the location coefficient (L_i) and the material degradation coefficient (Ti).

Moisture spots were found in every structural component, with concrete flaking occurring in the abutments and slab. There are also some transverse cracks present in the longitudinal beams. Most of the damages were not located in critical points (L_i=1), while the material analysis yielded a value of T_i=3 for the entire bridge.

Table 5 Bridge group Concrete class

Bridge Type	Concrete Class
T1	C16/20
T2	C25/30
T3	C25/30

The values of $\mathrm{K_m}$, as specified in equation (2), are: 0.4 for the abutment, 0.4 for the slab, 0.6 for the longitudinal beam, and 0.3 for the transverse beam. Table 9 displays the damage analysis for each structural component. In this particular case, it is evident that the Longitudinal Beam 1 and the slab are the most damaged elements.

The overall CNR for Bridge P08 is 6.72, which calls for immediate retrofitting. In particular, the previously mentioned structural components should be subject to more detailed analysis to monitor the damage progression and plan the retrofitting works.

The same process can be applied to all other bridges, and Table 10 summarizes the results of the network assessment. Bridges P13, P12, P07, and P08 require immediate retrofitting, while Bridge P10 is operational but requires urgent maintenance. Only Bridges P01 and P03 are in good condition.

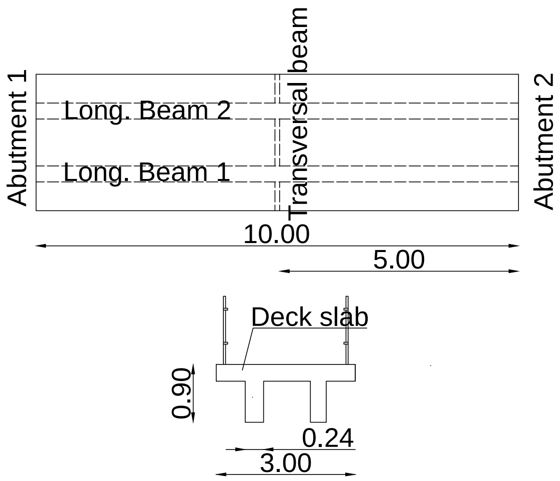

Figure 3 P08 Bridge Structural Scheme.

This outcome represents as a starting point for the network's bridge condition assessment. Further investigations are recommended for bridges that require immediate or urgent retrofitting to design the required refurbishments.

Table 6 Bridge P08 Abutment Damage identification form

Damage	B	K_2	K_3	L_i	R_{ck}^{exp}	R_{ck}^{d}	T_i
Humidity spot	1	1.5	1.5	1	20	37	3
Deteriorated Concrete	1	1.5	1.5	1			
Spalling	2	1	0.5	1			
Rusted reinforcements	3	1	0.5	1			
Web fracture	1						
Horizontal cracks	1						
Vertical cracks	3						
Inclined cracks	3						
Rusted stirrups	3	1	0.5	1			
Deformed reinforcements	4						
Construction joint deterioration	1						
Impact damages	1						
Support damages – top edge	2						
Support damages – bottom edge	4						
Out of plumb	2						

Table 7 Bridge P08 longitudinal beam damage identification form

Damage	B	K_2	K_3	L_i	R_{ck}^{exp}	R_{ck}^{d}	T_i
Humidity spot	1	1	1.5	1	20	37	3
Deteriorated Concrete	1						
Beam/Slab joint deterioration	2						
Freezing	2						
De-icing salts	2						
Spalling	2	2	2	1			
Reinforcements corrosion	3	2	2	1			
Tendons corrosion	4						
Duct deficiency	2						
Web fracture	1						
Longitudinal cracks	3						
Vertical cracks	3	1.5	1.5	2			
Inclined cracks	3						
Rusted stirrups	3	2	2	1			
Deformed reinforcements	4						
Constr. joint deg.	2						
Impact damages	1						

Table 8 Bridge P08 slab damage identification form

Damage	B	K_2	K_3	L_i	R_{ck}^{exp}	R_{ck}^{d}	T_i
Humidity spot	1	2	2	2	20	37	3
Deterio-rated Concrete	1	2	1.5	1			
Spalling	2	2	1.5	1			
Rein-force-ments corrosion	3	1.5	1.5	1			
Web frac-ture	1						
Longitu-dinal cracks	3						
Vertical cracks	3						
Inclined cracks	3						
Rusted stirrups	3	2	1.5	1			
Constr. joint deg.	3						

Table 9 Damage estimation for bridge P08 components

Element	F_{dm}	$F_{D,refm}$	100 F_{dm}/ $F_{D,refm}$
Abutment 1	10.20	512	1.99
Abutment 2	4.80	512	0.94
Slab	39.30	320	12.28
Lon.Beam 1	84.60	480	17.62
Lon.Beam 2	31.95	480	6.65
Transv.Beam	0.0	240	0.00

Table 10 Priority queue within the infrastructure network.

Order of urgency	Bridge Label	CNR
1°	P13	10.51
2°	P12	8.98
3°	P07	8.06
4°	P08	6.72
5°	P10	2.95
6°	P03	0.63

4 Conclusions

In this paper, a new method for conducting fast and low-cost condition assessments of Reinforced Concrete (RC) bridge networks has been introduced. This method is based on visual inspections and experimental on-site tests, and involves the following steps:

1. Identification of the structural components of each bridge or construction.
2. Weighing each structural component's importance to the overall structure's safety.
3. Conducting visual inspections and collecting information through experimental tests (when possible).
4. Evaluating damages based on their importance, extent, magnitude, position, and material degradation.
5. Ranking structures within the network using equations (1) to (2).

This ranking can aid Bridge Management System decision-makers in optimizing their allocation of funds for maintenance and management costs by allowing a focused approach on the most deteriorated parts of the bridge.

The key innovations of this approach are the parameters that take into account the damage's location at the structural element level and the mechanical degradation of materials.

The application of this method to a couple of real-life case studies has shown its efficacy in providing an "urgency ranking" for the retrofitting needs of a bridge network and a priority list of the most damaged elements within each structure. During the structure's lifetime, it is possible to track the evolution of damages by creating a useful database.

Future developments are expected to consider different materials (such as steel and masonry) and other types of constructions, such as general buildings, together with satellite territorial strategies of monitoring [26].

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