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The influence of the use of regulatory codes on the diagnosis of pathologies of reinforced concrete frame bridges

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Abstract
The duration of utilization of bridges leads to its aging and therefore to the loss of some of their characteristics. Hence, the importance of specialized auscultation so that they can perform their function during the lifetime assigned to them. The safety of these bridges must also be ensured in view of the increase in the load of the heavy goods vehicles actually travelling and passing through them. The aim of this research is to study the influence of the utilization of the Algerian and the European regulatory codes on the diagnosis of the pathologies of reinforced concrete frame bridges. To achieve this goal, a frame bridge of total length of 37.2 m, located in the wilaya of Tiaret in Algeria, is expertized. It consists of two parts, one cast on site and the other prefabricated in reinforced concrete; it is classified of low seismicity and as an important bridge according to the Algerian seismic regulation. A diagnosis of the condition of the bridge was carried out on the basis of a preliminary visual inspection by making a schematic and photographic finding of the main issues and another inspection based on experimentation by carrying out several tests in situ and in the laboratory, which led to its classification in 2E according to the manual image of the quality of works of art (IQOA). Two types of regulatory loads are applied in this article; the Algerian regulation in force and the European regulation Eurocode 1 in order to carry out a comparative numerical simulation using the lowest compression constraint found during the experiment on the cores taken from the expert bridge. The results of this work confirmed the safety side of the European regulation because it responds correctly to the evolution of loads, hence any future verification of the pathological state of existing frame bridges must refer to the Eurocode as the loading code.
Keywords: Bridge. Reinforced Concrete. Pathology. Auscultation. Diagnosis. Regulation.

1. Introduction

In general, the initial objective of a bridge is the crossing of an obstacle and then the continuation of the road, however, it can present critical pathological conditions, leading to risks to road safety and financial losses during maintenance work. Structural efficiency is not always considered as a quality, but as a prerequisite for correct design compared to other features such as functionality, hydraulics, safety and aesthetics.

Research on bridge pathologies requires significant resources in the form of machinery and experienced bridge inspectors who assess the condition of bridges on site (Catbas & Avci, 2022) and methods for digital transformation of bridge inspection, monitoring and maintenance processes (Bittencourt *et al*, 2021). Mascia and Sartorti (2011) studied bridge design and its relationship to pathological method and identification of pathologies in concrete, steel and wood bridges. Evaluations of bridge performance indicators have been proposed and implemented in case studies such as Marić *et al*. (2022), Mahdi *et al*. (2022) and Limongelli *et al*. (2018) and many European directives on the quantification of performance indicators to assess bridge quality control have been issued (Santos *et al*., 2022).

The aim of this research is to establish an assessment of the state of service of the frame bridges by using two different regulatory codes (Algerian and European) based on the degradations and disorders identified as well as the most singular anomalies observed through the inspections of the work and the tests carried out in situ and in the laboratory.

The inspections are based mainly on visual examinations of the various parts of the works, the banks and beds of the waders as well as on photographs of the deterioration found. In-situ and laboratory tests are carried out to diagnose the different pathologies of the expert bridges.

The Figure 1 illustrates in a diagram the principle of valuation, which begins with the use of the norms dimensioning in force which are in this article CPC fascicle 61 title II (Fascicle 61, 1961) and the Eurocode 1 (Eurocode 1, 1997) and which is identical to that of the dimensioning of new structures. In addition, this method generally has the advantage of facilitating the rapid identification of the key elements or sections of the construction.

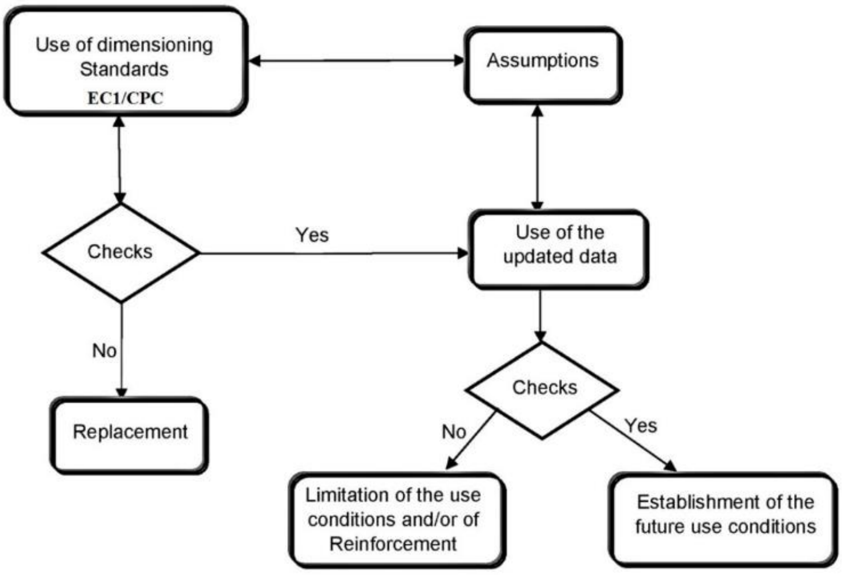


Figure 1 - Evaluation principal flow chart. Source: Authors (2023).

It is necessary to determine the updated data on loads, actions, strength and behaviour of the structure, on the basis of the user requirements agreed with the customer. When evaluating an existing bridge, the updated data shall include the new load model, the updated load factors for own weight and permanent loads, and the dynamic addition factor.

2. Bridge condition assessment procedures and regulations

Bridge design is a complex task requiring the synthesis of a great deal of information, and it plays a crucial role in the feasibility, cost, functionality and aesthetics of the bridge. The seismicity zone and the importance group of the structure influence the acceleration parameter, which is very important in the study and, above all, influences the degree of degradation of the structures over time. Any structure falling within the scope of Algerian seismic regulations must be classified in one of the three groups defined by the Algerian seismic regulations for engineering structures (RPOA, 2008). The RPOA (RPOA, 2008) divides the Algerian territory into five (05) zones of increasing seismicity (Figure 2).

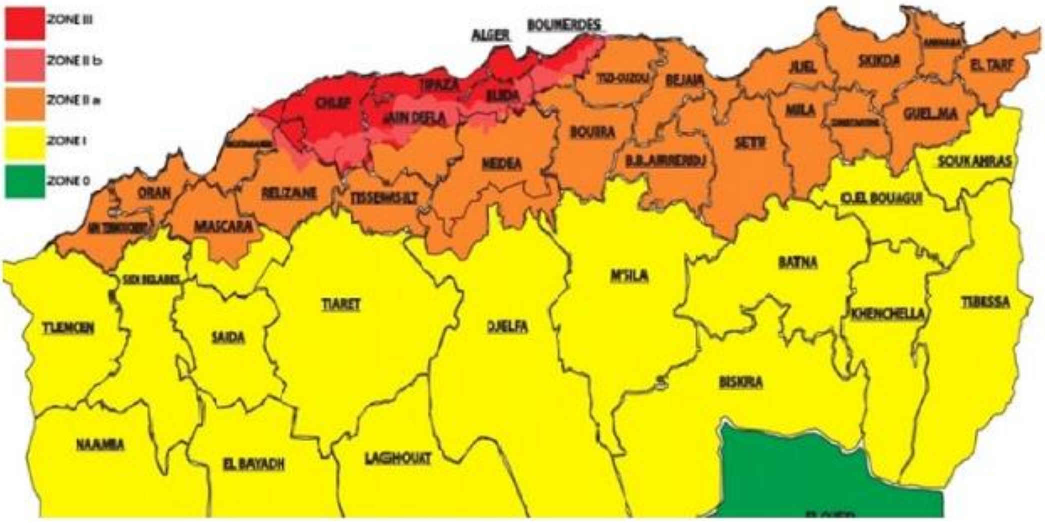


Figure 2 - Seismic zoning map of Algeria according to RPOA 2008. Source: (RPOA, 2008)

The minimum level of seismic protection afforded to a structure depends on its location and importance in terms of the objectives set by the community. Any structure falling within the scope of Algerian seismic regulations must be classified in one of the three groups defined by the RPOA version 2008 (RPOA, 2008). The condition of the bridge is characterized by the choice of one of five conditional classes, possibly supplemented by an "S" rating for user safety. These are defined in a specific guide called engineering structures quality index IQOA (SETRA, 1996).

A structure's IQOA rating is based on an analysis of its condition, carried out either on the basis of the structure's file if it contains a periodic detailed inspection report during the year, or on the basis of a summary inspection carried out in accordance with the IQOA inspection guide (SETRA, 1996) published separately. The IQOA (SETRA, 1996) classifies and provides an indicator of the medium condition of a portfolio of engineering structures, based on a technical assessment of each structure. This assessment involves assigning a condition class to each structure, based on the various defects and disorders affecting it. The overall grade is the worst of the elementary grades. The conditional class chosen from 1, 2, 2E, 3, 3U characterizes its mechanical or functional condition, in ascending order of severity (Figure 3).

The process used to determine the conditional class in which a bridge should be classified is summarized in the flowchart in Figure 3. The decision's elements are based on a set of notions and terms relating to the different parts of the bridge to be considered, and to the type of intervention required. Due to the imprecise nature of the terms "Maintenance" and "Repair" used in this flowchart, the classification of the structure should be carried out from top to bottom of the decision tree, rather than the other way round.

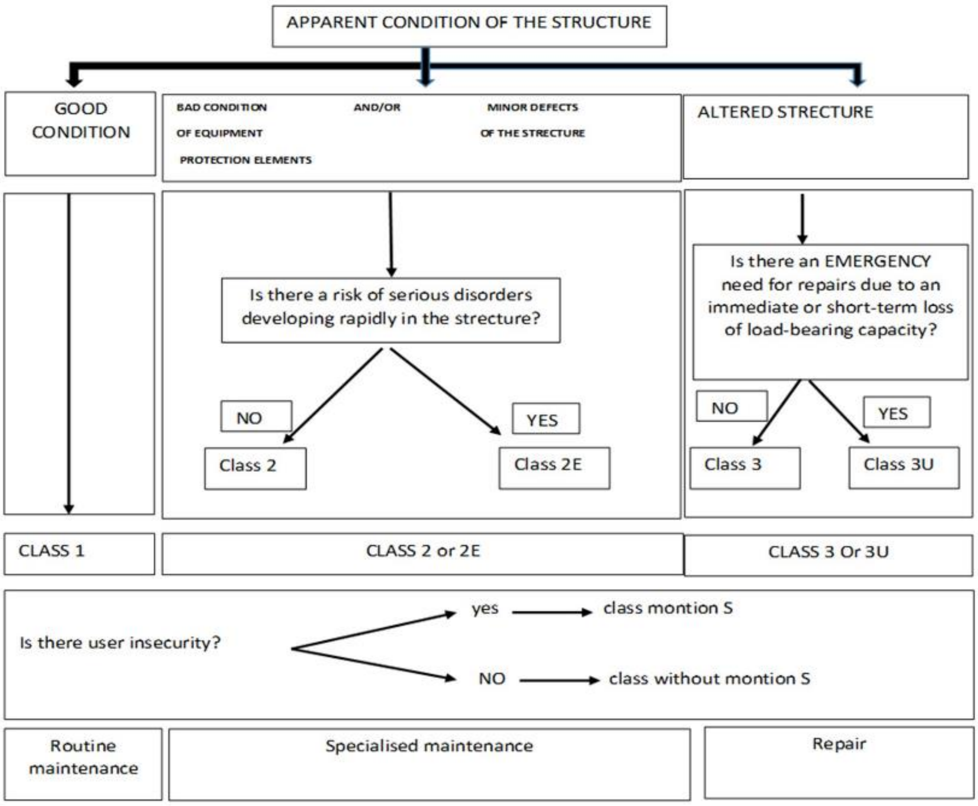


Figure 3 - The process of assessing status class according to the IQOA. Source: (SETRA, 1996)

Two types of loadings are applied in this article, the first loading is that of the Algerian regulation in force which is CPC fascicle 61 Title II (Fascicle 61, 1961) and the second loading is the European regulation which is Eurocode 1 part 3 (Eurocode 1, 1997).

With regard to CPC fascicle 61 Title II (Fascicle 61, 1961), two load systems A and B (articles 4 and 5) can be applied to bridge pavements. These systems are distinct and independent, in the sense that for the calculation of a given effect, the two systems cannot be applied simultaneously. The exceptional convoys to be taken into account when calculating the deck's load-bearing capacity are axle-mounted convoys and convoys on tank carriers (type D).

The exceptional convoy is assumed to travel alone, whatever the width of the bridge, and is arranged longitudinally to obtain the most unfavourable effect. On the contrary, the convoy on axles is a truck with three axles, all with single wheels fitted with pneumatic tires, and which meets the characteristics illustrated in Figure 4.

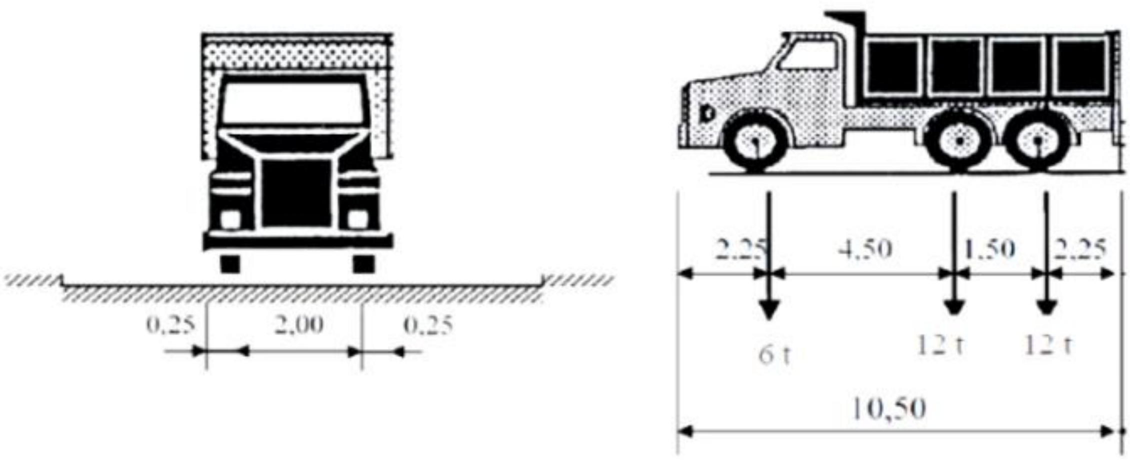


Figure 4 - Type truck Bc according to the CPC. Source: (Fascicle 61, 1961)

For loading in accordance with Eurocode 1 part 3 (Eurocode 1, 1997), traffic on bridges generates a spectrum of stresses that can lead to fatigue. This stress spectrum depends on vehicle geometry, axle loads, vehicle spacing, traffic composition and dynamic effects. The use of different fatigue load models is defined by Eurocode 1 Part 3 (Eurocode 1, 1997). Fatigue load models 1, 2 and 3 are to be used to determine the maximum and minimum stresses resulting from the various possible load arrangements of the model under consideration on the bridge, while fatigue load models 4 and 5 are to be used to determine stress variation spectra resulting from the passage of trucks over the bridge.

The fatigue load model 1 has the same configuration of the main loading system (characteristic load model 1 defined in 4.3.2 of Eurocode 1 part 3). Fatigue load model 2 consists of a set of idealized trucks, termed "frequent" trucks, to be used as defined in the Table in article 4.6.3 of Eurocode 1 part 3 (Eurocode 1, 1997). Load model 3 consists of four axles, each with two identical wheels. Its geometry is shown in Figure 5.

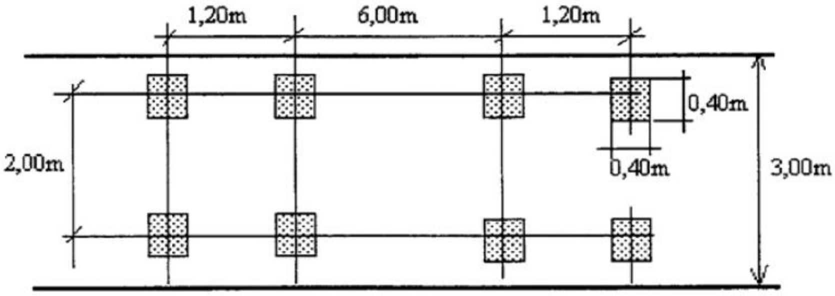
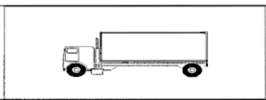
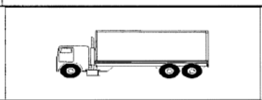
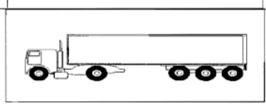
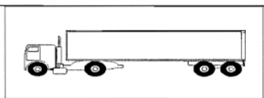
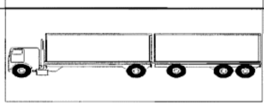


Figure 5 - Fatigue loading model 3. Source: (Eurocode 1, 1997)

The weight of each axle is equal to 120 kN and the contact surface of each wheel is a 0.40 m square. The fatigue load model 4 consists of a set of standardized trucks which together produce effects equivalent to those of typical European road traffic. Unless otherwise specified, a set of trucks should be considered to have a composition similar to that of the traffic expected on the road concerned, as defined in Table 1 (article 4.6.5 of Eurocode 1, part 3).

Table 1 - Set of “frequent” lorries. Source: (Eurocode 1, 1997)

1	2	3	4
LORRY SILHOUETTE	**Axle spacing** (m)	**Frequent axle loads** (kN)	**Wheel type** (see Table 4.8)
	4,5	90 190	A B
	4,20 1,30	80 140 140	A B B
	3,20 5,20 1,30 1,30	90 180 120 120 120	A B C C C
	3,40 6,00 1,80	90 190 140 140	A B B B
	4,80 3,60 4,40 1,30	90 180 120 110 110	A B C C C

3. Investigative inquiry on the expertized bridge

Location of the bridge

The bridge (Lat: 34°58'57.21 "N; Long: 2°20'33.70 "E) is located on National Road N°120 (RN120 ex CW77) at PK 97+000 and serves as a connection for the districts of the Daïra of Chellala as well as for the agricultural farms and regional douars of the Rechaigua district (Figure 6).

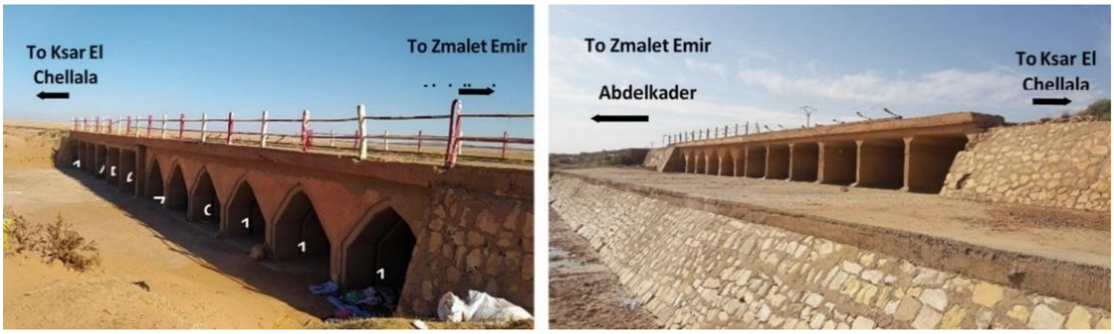


Figure 6 - The surveyed bridge located on the RN 120 national road at KP 97+000. Source: Authors (2023).

Its main traffic consists of cars, tractors and trucks. The bridge, which has been surveyed, is classified according to the Algerian seismic regulations RPOA version 2008 (RPOA, 2008) in zone I, meaning low seismicity, and in group 2 of importance class (major bridge). The total length of the bridge is 37.2 m, and it consists of two sections: the first, built in 1994, has a length of 20.40 m and comprises 6 cast-in-place reinforced concrete scuppers, and the second, built in 1997, has a length of 16.80 m and comprises 6 prefabricated reinforced concrete scuppers. The longitudinal section and two cross-sections of the bridge are represented in Figure 7.

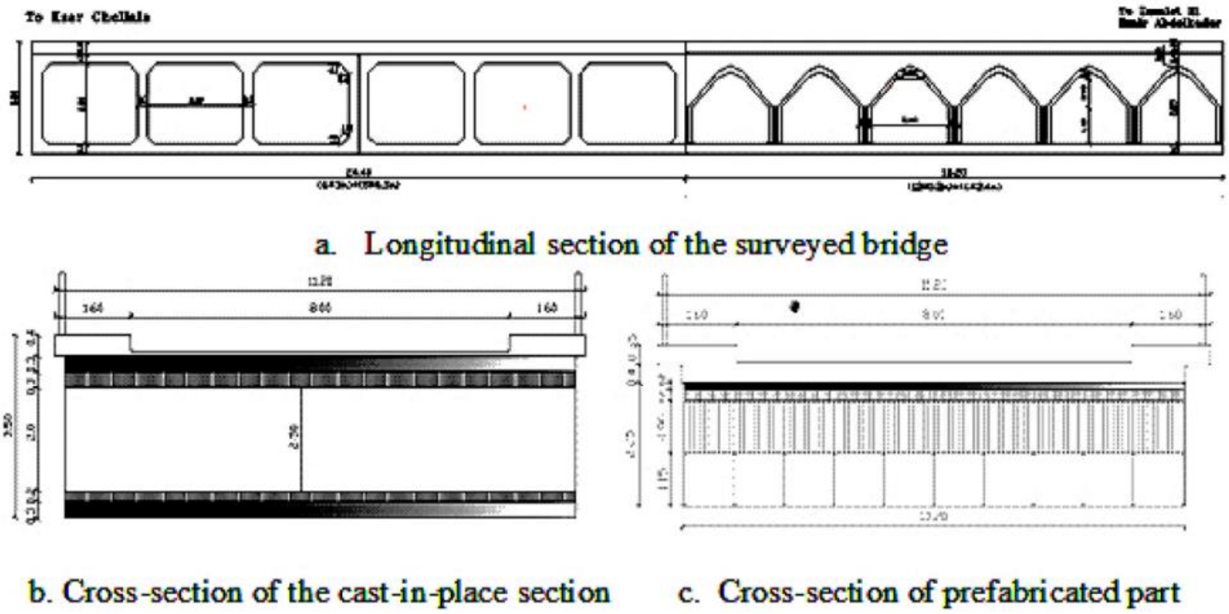


Figure 7 - Sections of the tested bridge. Source: Authors (2023).

Realization of experiments

For this article, the diagnosis of the bridge's condition is based on two criteria, a preliminary visual inspection and an experimental one. The preliminary visual inspection is based on a visual diagnosis of the bridge's condition, including a schematic and photographic record of the principal visible disorders, an opinion on the apparent condition of the structure, working conditions, accessibility to the bridge and a collection of available information on the bridge and its environment.

For the inspection based on experimentation, the tests were realized to diagnose the pathologies of the appraised bridges. Dynamic ultrasonic testing in accordance with standard EN 12504-4, which consists in measuring the propagation time of a train of radio waves between two points, knowing the distance between transmitter and receiver, from which the propagation speed is deduced (Figure 8).

Figure 8 - Multi-detector wall scanner. Source: Authors (2023).

The measuring equipment used is a TICO-type ultrasound (Figure 9), which enables measurements to be taken using the surface method (indirect measurement).

Figure 9 - Measurement of propagation speed (TICO-type ultrasound). Source: Authors (2023).

The second test determines the rebound index of a hardened concrete surface in accordance with NA 2786, which can be used to assess the homogeneity of concrete in situ. Simple compression tests in accordance with NA 5075 determine the compressive strength (Figure 10).

Figure 10 - Concrete strength measurement apparatus (concrete sclerometer). Source: Authors (2023).

Carbonate thickness at the facing is measured in accordance with standard NF EN 14630 in the laboratory on freshly broken core samples (Figure 11) taken from the bridge (split along a generatrix). Phenolphthalein is colourless at pH below 8.2 and deep pink at pH above 9.9.

Figure 11 - Concrete core samples. Source: Authors (2023).

The final test consists in determining the presence of chlorides in the mortar facing, in accordance with UNI 7928 standards. This measurement is carried out in the laboratory on freshly broken core samples taken from the bridge (split along a generatrix). In the presence of free chloride, the mortar takes on a light grey colour (Figure 12).

Figure 12 - Analysis of ion chromatography for free chloride ions. Source: Authors (2023).

4. Results and Discussion

Preliminary visual inspection results

The survey of the main degradations includes a photographic description of the disorders observed in the bridge structures, in particular the main structure consisting of the masonry pedestals, the reinforced concrete slab and the embankment protection walls upstream and downstream of the bridge, as well as interpretations of the probable causes of these degradations.

Following a visit to the structure on 13-11-2022, the damage was divided into three categories: superstructure, infrastructure and equipment, the results of which are summarized in Tables 2, 3 and 4, and Figures 13, 1, and 15.

Table 2 - Recap of visual inspections of the superstructure of surveyed bridges. Source: Authors (2023).

	Nature of degradation	**Probable causes**
SUPERSTRUCTURE	- Incipient concrete spalling without visible reinforcement (green arrow) and localized concrete spalling with exposed reinforcement (red arrow) (Figure 13.a). - Exposed reinforcement without concrete spalling (blue arrow) due to insufficient embedding or faulty shimming of reinforcement. (Figure 13.b).	-The thrust exerted by the oxidation of the reinforcement on the concrete cover, due to the porosity of the concrete. - Insufficient coating thickness. - Concrete carbonation.
	- Areas of segregation without exposed reinforcement (Figure 13.c). - Traces of rust due to reinforcement corrosion. - Traces of previous concrete repair (Figure 13.d).	- Faulty concrete placement or bad concrete quality.

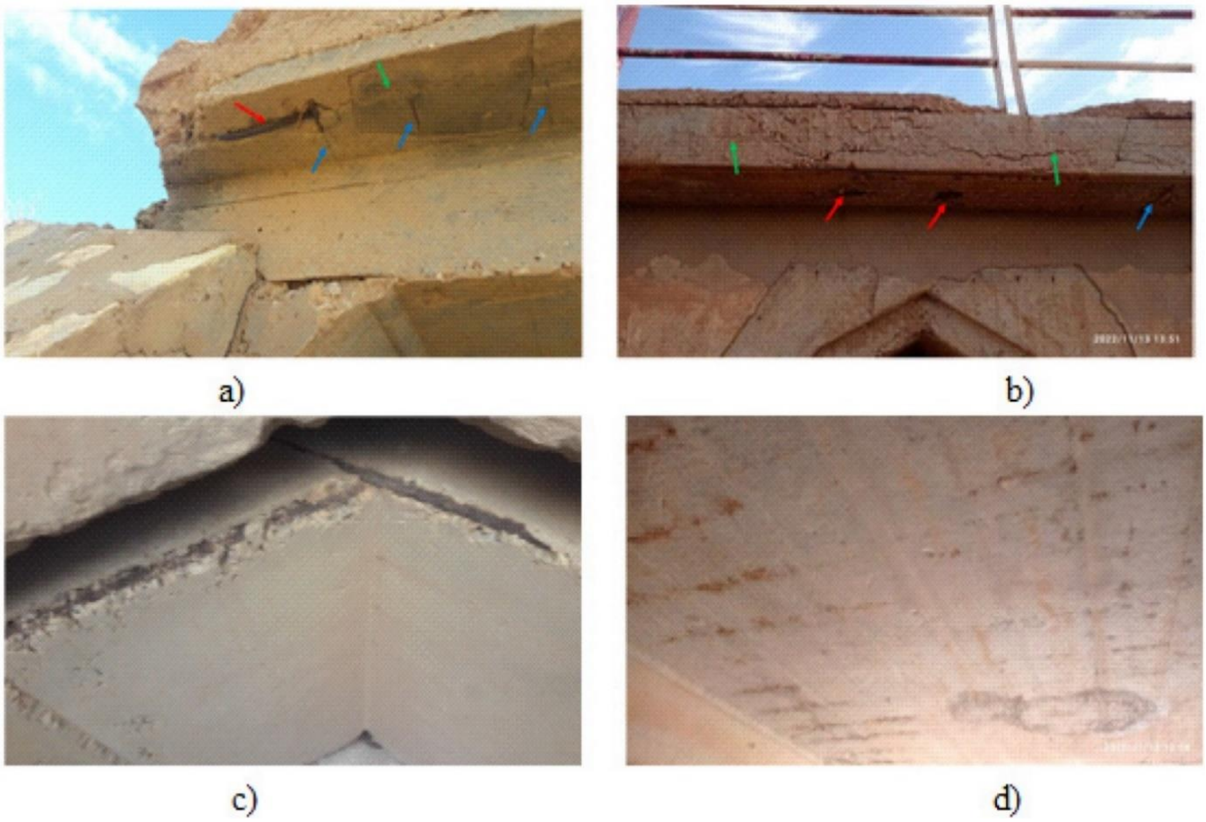


Figure 13 - Visual damage to the bridge superstructure. Source: Authors (2023).

Table 3 - Recap of visual inspections of the infrastructure of the bridges surveyed. Source: Authors (2023).

	Nature of degradation	**Probable causes**
INFRASTRUCTURE	- Segregation zones (Figure 14.a). - Spalls without exposed reinforcement (Figure 14.b). - Exposed reinforcement without concrete spalling (Figure 14.c). - Evidence of previous concrete repair (Figure 14.d). - Concrete spalling without exposed reinforcement (Figure 14.e). - Cracks in spandrel walls of precast elements on upstream side.	- Due to faulty concrete placement or bad quality concrete. - Due to impact of floating objects. - Insufficient embedding or faulty wedging of reinforcement. - The thrust exerted by the oxidation of the reinforcement on the embedding concrete, due to the porosity of the concrete. - Shock from floating objects.

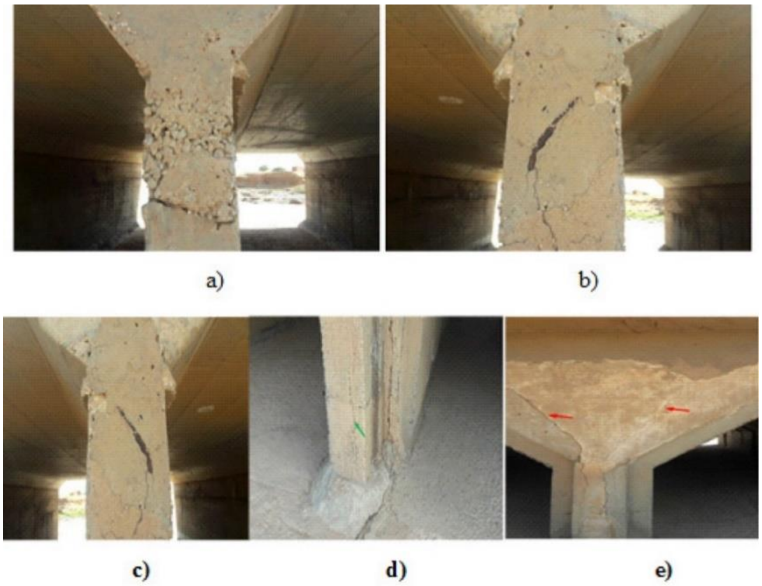


Figure 14 - Visual damage to bridge infrastructure. Source: Authors (2023).

Table 4 - Recap of visual inspections of equipment on surveyed bridges. Source: Authors (2023).

	Nature of degradation	**Probable causes**
EQUIPMENTS	- Damaged guard rail (Figure 15). - Damaged paintwork on guard rail. - Corrosion of guard rail. - Localized alignment fault. - Missing tubular elements.	- Lack of maintenance

Figure 15 - Visual damage to bridge equipment. Source: Authors (2023).

During this visual inspection and after examination of the various parts of the bridge, the banks and the wadi bed, the bridges assessed were classified as class 2E according to IQOA. The bridge was found to be in good condition, with no mechanical damage indicating that its capacity had been exceeded under load. The disorders observed are localized in areas of segregation and spalling on slabs and walls, and efflorescence on slabs.

Experimental Results

The experimental program focused on the analysis of concrete in walls and slabs, and for this reason eight (08) samples were taken, the locations of which are shown in the schematic plan of the core samples (Figure 16).

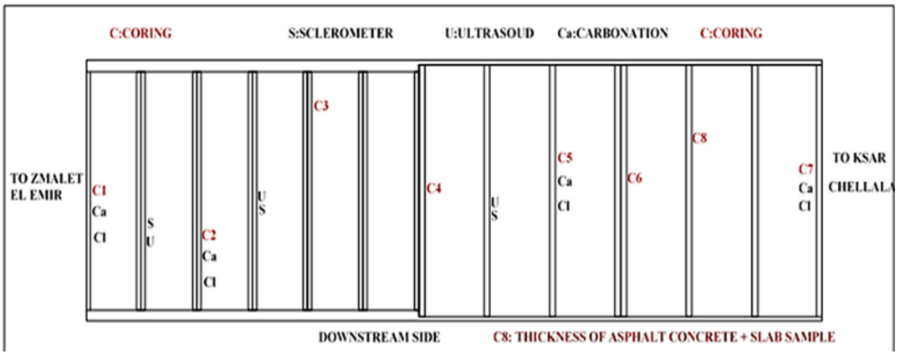


Figure 16 - Schematic plan of the inspection program. Source: Authors (2023).

The samples taken in situ are shown in Figure 17 and consist of a core (Ø 50 mm) taken from the abutment on the Zmalet El Emir Abdelkader side (C1), 5 cores (Ø 50 mm) taken from the uprights (C2, C3, C4, C5 and C6), a core (Ø 50 mm) taken from the carriageway on the upstream Ksar Chellala side (C7) to measure the asphalt thickness and a core (Ø 50 mm) from the slab (C8).

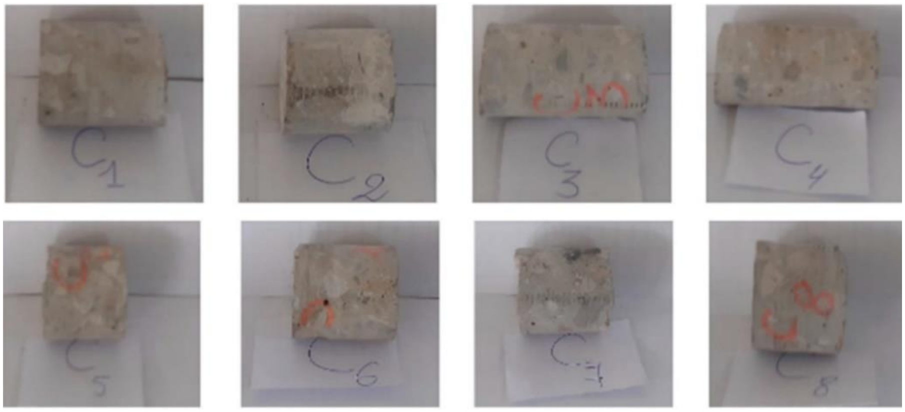


Figure 17 - Photos of core samples. Source: Authors (2023).

The simple compression tests carried out on the various cored specimens gave the results summarized in Table 5.

Table 5 - Compressive strength of core samples. Source: Authors (2023).

Part of the work	**Compressive strength MPa**
C1	24,6
C2	23,1
C3	25,0
C4	22,9
C5	19,9
C6	23,7
C7	18,2
C8	26,7

The carbonation test produced the results shown in Figure 18, where the reddish area is the non-carbonated zone. Table 6 summarizes the results.

Table 6 - Phenolphthalein test results. Source: Authors (2023).

Designation	Immediate reading (mm)
C1	33
C2	25
C5	50
C7	43

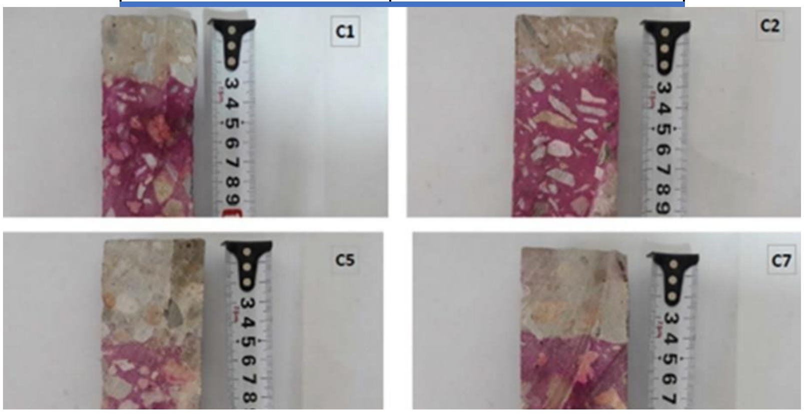


Figure 18 - Phenolphthalein test on carrots. Source: Authors (2023).

For the silver nitrate test, the result was negative as there was no light grey coloration, as shown in Figure 19.

Figure 19 - Silver nitrate test on core samples. Source: Authors (2023).

The results of the ultrasound measurements are summarized in Table 7.

Table 7 - Ultrasound test results. Source: Authors (2023).

Designation	Speed (m/s)
U1	3856
U2	4110
U3	1808

Sclerometer tests were carried out on the elements of the surveyed structure, and the results are summarized in Table 8.

Table 8 - Sclerometer auscultation results. Source: Authors (2023).

Designation	Rebound index
S1	48
S2	50
S3	33

The results of the compressive strength averages on the reinforced concrete cores, the sclerotic measurements and the ultrasonic velocities show that the concrete of the auscultated bridge in place is of average quality. As a result, there is no cause for concern, as no deterioration attributable to a strength defect in the concrete has been detected (no cracks). The thickness of carbonated concrete measured (25 to 50 mm) is overcritical, as the steels are not protected from carbonation corrosion, particularly in areas of low embedment. On the other hand, the thickness of concrete containing free chlorides is zero, indicating that the steels are well protected against pitting corrosion.

After examination of the various parts of the structure by visual auscultation and experimentation carried out in the laboratory and in situ, the structure is classified in IQOA class 2E, as it shows no mechanical degradation indicating that its capacity has been exceeded under the effect of loads. The bridge does not require any reinforcement with regard to current regulatory overloads. It only requires regular and specialized maintenance to avoid passing out to the third Class 3. (Repairing the deteriorated zones).

Simulation and Numerical Results with Robot Structural

The surveyed bridge was classified as Class 2E according to the IQOA, and to clarify the use of regulatory codes for diagnosing the pathologies of reinforced concrete frame bridges, it was necessary to carry out a numerical study using Robot Structural Analysis Professional (Robot, 2022) for both the cast-in-place and precast sections of the bridge under investigation, using the Algerian code CPC fascicle 61 title II (Fascicle 61, 1961) and the European code Eurocode 1 part 3 (Eurocode 1, 1997). In this simulation, the compressive stress used in the calculation is the lowest finding during experimentation on the core samples, equal to 18.2 MPa. Since the bridge is made up of two distinct sections, it was necessary to carry out a numerical study for the cast-in-place section and the pre-cast section. After regulatory loading according to Eurocode 1 part 3 (Eurocode 1, 1997) and CPC (Fascicle 61, 1961), the results of the calculations concluded that Fatigue Load Model 2 is the most unfavourable for Eurocode 1 (Eurocode 1, 1997) and System Bt for CPC (Fascicle 61, 1961). Among the results deduced from this calculation, Figure 20.a illustrates the most unfavourable moment curves recorded according to CPC (Fascicle 61, 1961) and Figure 20.b according to Eurocode 1(Eurocode 1, 1997) for the cast-in-place section.

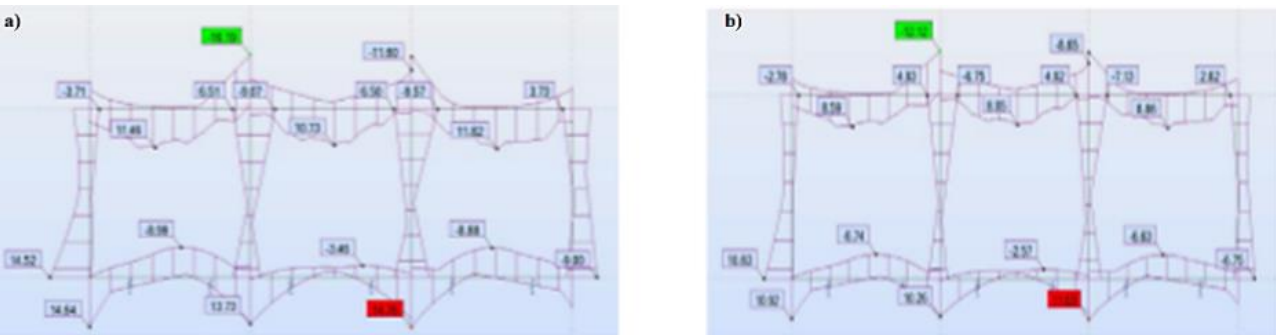


Figure 20 - Most unfavourable moments for cast-in-place sections according to Eurocode 1; (Eurocode 1, 1997) and CPC (Fascicle 61, 1961). Source: Authors (2023).

The calculation is also carried out for the prefabricated part of the bridge, and Figure 21.a shows the curves of the most unfavourable moments recorded according to CPC (Fascicle 61, 1961) and Figure 21.b according to Eurocode 1 (Eurocode 1, 1997).

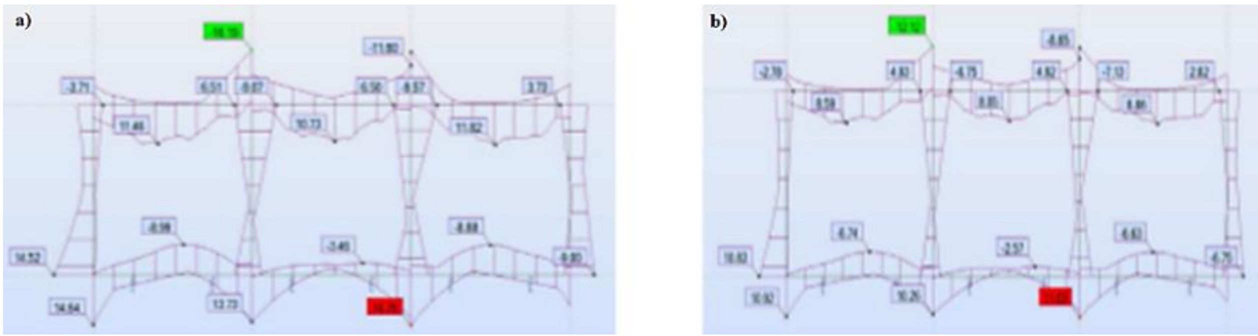


Figure 21 - Worst-case moments of precast sections according to Eurocode 1 (Eurocode 1, 1997) and CPC (Fascicle 61, 1961). Source: Authors (2023).

Figure 22 shows the various maximum moment values calculated using Eurocode 1 (Eurocode 1, 1997) and CPC (Fascicle 61, 1961) with updated data and compared with the moment values initially calculated for the cast-in-place section of the bridge, and Figure 23 that corresponding to the precast section of the bridge.

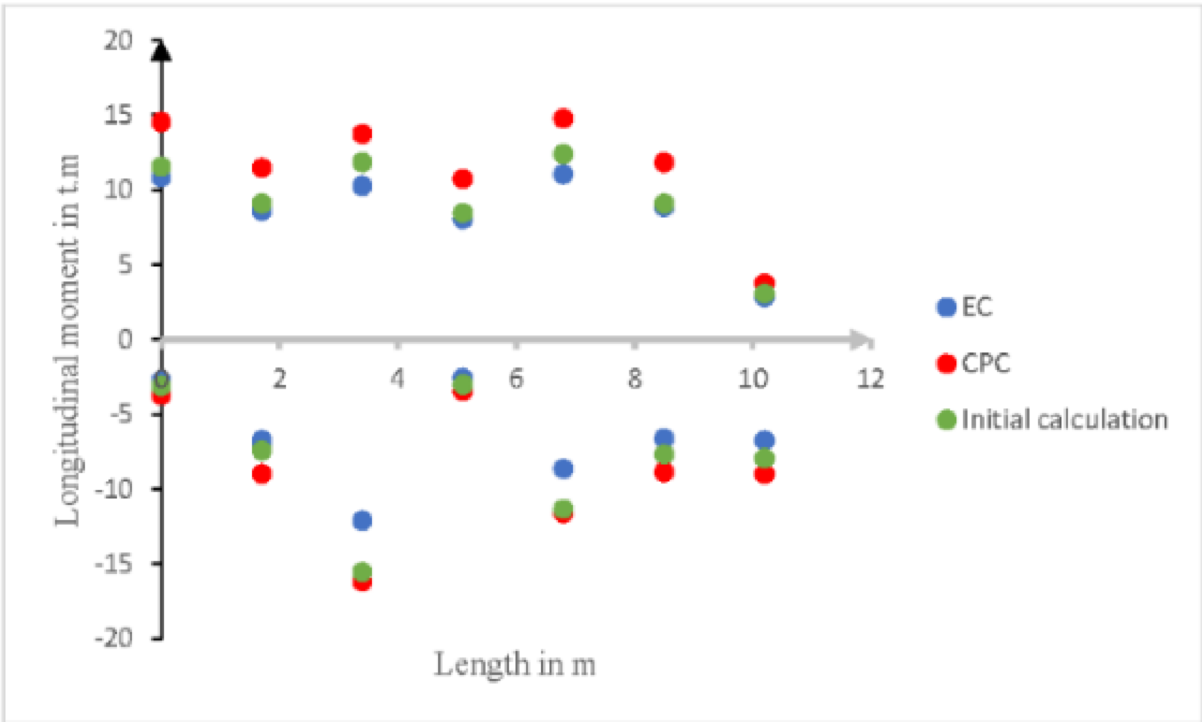


Figure 22 - Maximum moments calculated by EC1 (Eurocode 1, 1997) and CPC (Fascicle 61, 1961) compared with moments calculates by the initial data (cast-in-place section). Source: Authors (2023).

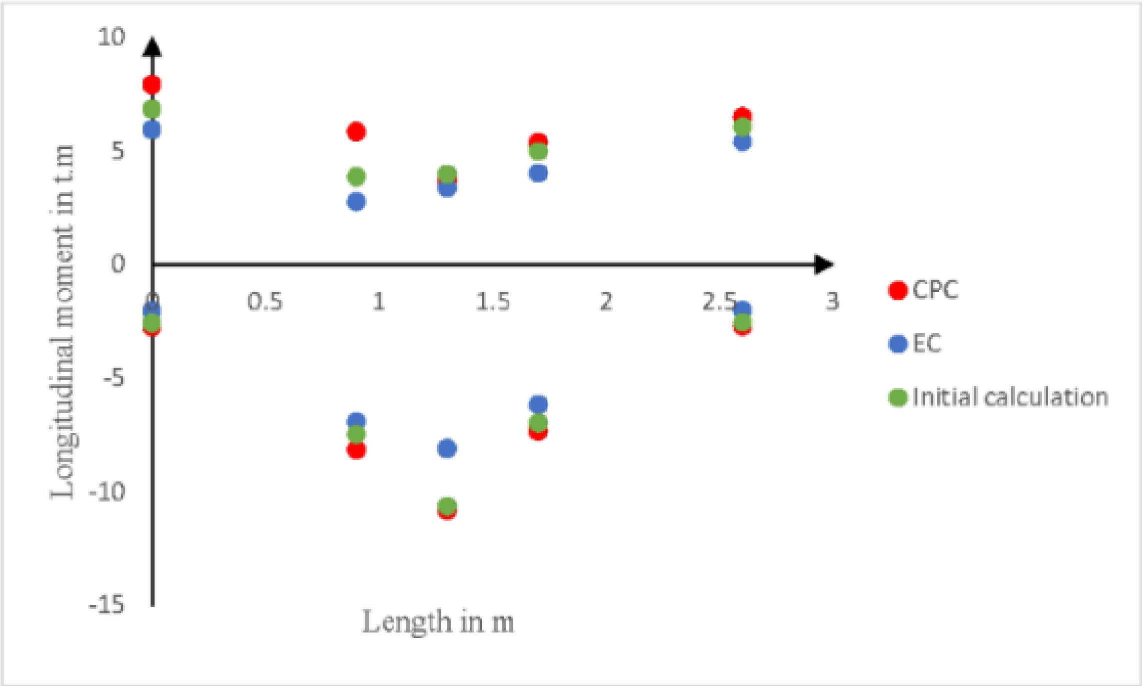


Figure 23 - Maximum moments calculated by EC1 (Eurocode 1, 1997) and CPC (Fascicle 61, 1961) compared with moments calculates by initial data (pre-cast part). Source: Authors (2023).

The results of the extreme bending moments at supports and spans for both parts of the bridge with the current data are grouped for CPC (called CPC/Cur) and for Eurocode 1 (called EC/Cur) in Table 9 below with the moment values of the initial bridge design (called Calc/Ini).

Table 9 - Maximum moments under EC1 (Eurocode 1, 1997) and CPC (Fascicle 61, 1961). Source: Authors (2023).

		Maximum negative moment (t.m)	**Maximum positive Moment (t.m)**
Cast-in-place bridge	CPC/Cur	-16.19	14.76
	EC/Cur	-12.12	11.03
	Calc/Ini	-15.56	12.38
Pre-cast bridge	CPC	-10.82	7.90
	EC	-8.10	5.91
	Calc/Ini	-10. 64	6.82

The results of the global extremes show that the Eurocode moment values with the current bridge data remain lower than the global extremes calculated with the initial data, in contrast to those calculated by CPC (Fascicle 61, 1961) with the current data. Since the moments calculated by Eurocode 1 (Eurocode 1, 1997) are lower than the values calculated by the bridge's initial data, the variation will be counted as negative for the calculation made by Eurocode 1 (Eurocode 1, 1997) and positive for the calculation made by CPC (Fascicle 61, 1961). These variations are listed in Table 10 below.

Table 10 - Variation maximum moments EC1 (Eurocode 1, 1997) and CPC (Fascicle 61, 1961). Source: Authors (2023).

		Maximum negative moment (t.m)	Maximum positive Moment (t.m)
Cast-in-place bridge	Variation - CPC/Cur (%)	4.05	19.22
	Variation - EC/Cur (%)	-22.11	-10.90
Precast bridge	Variation - CPC/Cur (%)	1.69	15.84
	Variation - EC/Cur (%)	-23.87	-13.34

It is clear that the calculations carried out by Eurocode1 (Eurocode 1, 1997) are lower than the values calculated by the initial bridge data with rates varying from 10.09% to 23.87%, unlike the calculations carried out by CPC (Fascicle 61, 1961) which exceed the initial calculations by a rate varying from 1.69% to 19.22%.

5. Conclusions

Following visual auscultations and experimental results in situ and in the laboratory, the two-part frame bridge, one cast in situ and the other prefabricated, in its current state, was classified as Class 2E according to the IQOA. These results showed that the moment values according to Eurocode 1 (Eurocode 1, 1997) with the current data are lower than the global extremes calculated with the initial data, in contrast to those calculated by the CPC (Fascicle 61, 1961), so it is clear that EC1 (Eurocode 1, 1997) correctly meets the classification given by the IQOA (SETRA, 1996) in contrast to the CPC (Fascicle 61, 1961).

Numerical results have shown that EC1 (Eurocode 1, 1997) responds correctly to changing loads, and it is therefore necessary to adopt the European code EC1 (Eurocode 1, 1997) as the reference code for numerical calculations of the pathological state of frame bridges, in order to confirm its classification as provided by the IQOA (SETRA, 1996).

Following the assessment work carried out on the frame bridge, which has been classified as 2E according to the IQOA, the bridge does not require any reinforcement with regard to current regulatory overloads, and no conservatory measures need to be taken, but repair work is compulsory in degraded areas. This approach confirms the safety aspect of European regulations, so it is clear that for future verification of the serviceability of existing frame bridges, reference should be made to EC1 only, as EC1 (Eurocode 1, 1997) was in line with the pathologies identified on site and in the laboratory, unlike the Algerian regulatory code, which gave results far very superior about the actual pathological state of the expertized bridge.

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