Effects of corrosion on a steel bowstring bridge in marine environment: a case-study of assessment and retrofit

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Abstract

The case-study of a steel bowstring bridge set in a marine environment and highly damaged by corrosion is presented. The bridge was built in 2004 and was repainted for corrosion protection in 2010. Despite the recent construction and the maintenance interventions, many structural elements like hangers are highly damaged by corrosion with decreasing performance in terms of serviceability and ultimate limit states. A deep investigation was carried out in order to assess the bridge and to establish the necessary retrofit actions to be carried out in the near future. In-situ tests reveal the reduced performance of the original steel in terms of strength and corrosion protection, together with the inefficiency of the successive maintenance interventions. For this reason the author made drastic choices for assessing the bridge and for its retrofit, down to replacement of hangers and entire galvanization through thermal spray coating technology, in order to increase the expectations of durability in service life. The results of the investigations carried out on the bridge as well as the choices of intervention are presented and discussed.

Keywords: arch, tie, bowstring bridge, corrosion, steel bridge, hangers

Introduction

The corrosion of steel bridges in a marine environment is a challenge to the bridge engineer because many existing bridges show that the development of corrosion in steel members is more rapid than expected, even in the case in which the maintenance interventions of the owner are regular; when maintenance action is not regularly applied or it does not achieve the expected effectiveness, a decreasing performance occurs in terms of serviceability and in the extreme cases the ultimate limit state could be reached in the most stressed elements.

In its simplest form, corrosion of steel results from exposure to oxygen and moisture. Corrosion is accelerated in the presence of salt from roadway deicing, salt water, or perhaps salt deposited from other sources. Although steel corrodes readily in the presence of oxygen and moisture, the rate of corrosion is accelerated in the presence of chloride ions or other corrosive chemicals. Chloride ions result mainly

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from the use of deicing agents composed of materials with readily soluble chloride ions. These ions create an atmosphere in which unprotected steel corrodes very quickly. In order to improve durability, paintings are used as coatings for protecting steel from the impact of the environment [1]. Particularly, seawater consists of a solution of many salts and numerous organic and inorganic particles in suspension. Its main characteristics are salinity and chlorinity and, from the corrosion point of view, dissolved oxygen content, which ranges from 4 to 8 mg/l, depending on temperature and depth. Its minor components include dissolved gases – CO_2, NH_3 and H_2S – found in seawater contaminated by urban sewage or due to algae, bacteria, etc. [2]. An interesting review on steel corrosion in marine environments is reported by Alcantara et al. [3]. Steel bridge corrosion prevention and mitigation strategies are reported in Stephens et al. [4] while Kreislova and Geiplova [5] give useful indications on the evaluation of coatings for steel protection.

In this paper a case-study of a steel tied-arch bridge, known also as a bowstring bridge [6,7], with two inward inclined arches, in Southern Italy, over the mouth of a river, is presented and discussed. The main cause of the high corrosion level was the original lack of steel galvanization, because the designer was confident in a coating system composed only of an external epoxy-based painting. The damage due to corrosion is mainly concentrated in the hangers, whose cross-section was reduced with respect to the original one, with the danger of achievement of the local conditions for failure. The choices made by the author for the retrofit interventions are presented: particularly the replacement of hangers and a new strategy for coating and corrosion protection including thermal spray zinc coating in a duplex-kind coating with three-layer painting.

1 Geometry of the case-study bridge

The case-study is an existing arch bridge with inward arches in Sicily, over the mouth of the river Arena in Mazara del Vallo, along the seafront of the town.

The upper arch is composed of two arches, inclined inward by 18°. The arch cross-section is a steel box 700x900 mm. The two arches are stiffened by 11 transverse beams. The bridge has longitudinal and transverse symmetry with 27 hangers at each bridge side for a total number of 54 hangers made of circular steel bars, 70 mm in diameter. The deck is composed of two longitudinal side beams, with double T section, 900 mm wide and 1750 mm high, which are extradosed with respect to the platform and inclined 18°, lying in the same plane as the arches and hangers. The longitudinal beams are stiffened by transverse double T beams 800 mm high with L diagonal bracing and an upper reinforced concrete slab 25 mm thick. The upper slab is connected only to the transverse beams through Nelson rods, but not to the longitudinal ones, which are almost independent and linked only to the steel elements of arch, hangers and deck. The deck supports a single carriageway and two side footways. The total length of the bridge is 87 m, and the span between supports is 85.4 m. The arch rise is 16.5 m (f/l = 0.193) and the hanger spacing is 3.05 m. Each abutment is founded on 8 piles with diameter1.20 m. The geometry is shown in figure 1. Pictures of the bridge environment are shown in figure 2.

The design tensile yield strength of the steel is f_{yk} = 355 MPa and the concrete strength is f_{ck} = 35 MPa. The elastic modulus of the concrete E_c = 30500 MPa and the elastic modulus of the steel E_s = 210000 MPa.

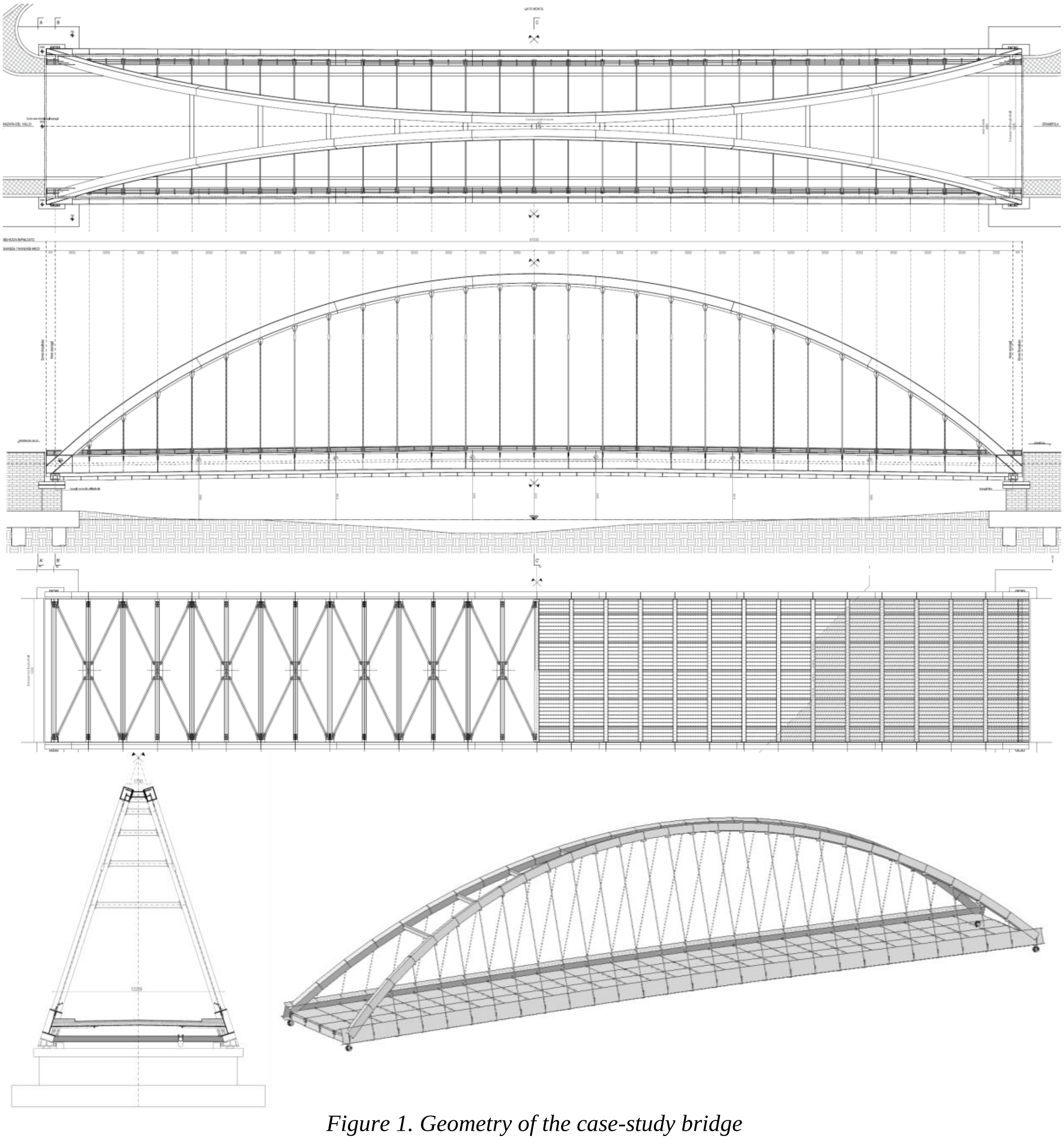
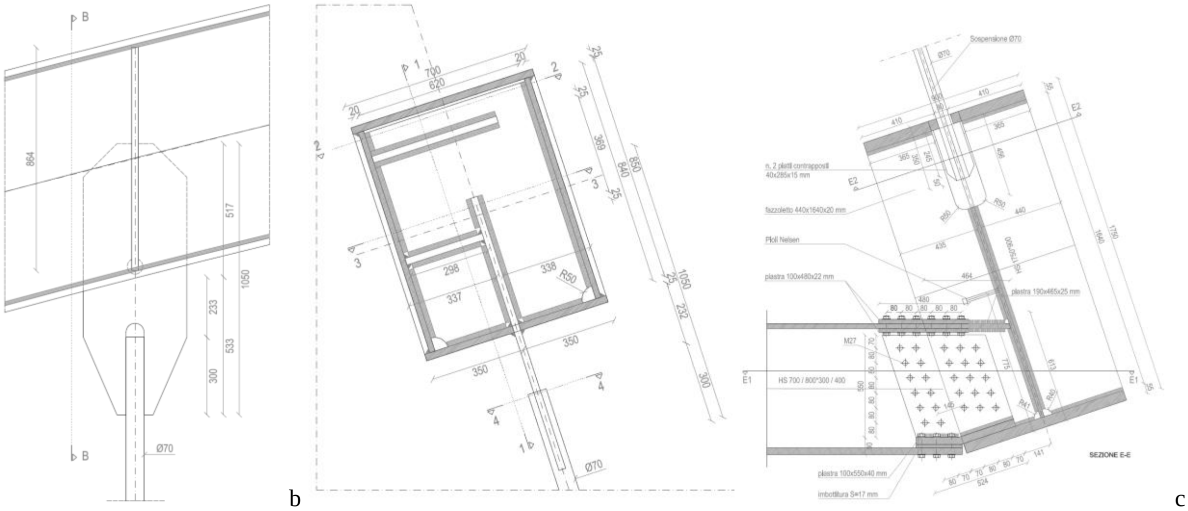


Figure 1. Geometry of the case-study bridge

An important characteristic of the bridge is that the hangers are made of steel bars with couplers for bars longer than 12 m; the connections welded to the arch and deck main beam are the critical points for corrosion and local damage together with the couplers. This kind of connections does not allow for hangers re-tensioning and/or simple replacement and it was the main difficulty to be faced for the retrofit design (fig. 3).

Figure 2. Aerial and perspective views of the bridge environment



a b c

d

Figure 3. Hanger attachment joints (welded connections) to the arch (a,b) and to the main beam (c). Views of corroded hangers and attachment joints (d)

2 Bridge assessment and onsite test evaluation

The outcome of the in-situ investigations and laboratory tests made it possible to determine the strength of the main elements, their degree of corrosion and damage as well as the high average rate of corrosion and the actual effectiveness of the maintenance intervention previously put in place. From these results it is clear that the most stressed and damaged elements are the hangers and especially those over 12 m long which have a bar coupler (fig. 4). Visual and instrumental investigations agree in diagnosing a reduction of cross-section in many hangers, especially at the base, in the hole of the principal beam which allows the hanger bar to go down and to be welded at the centroid of the steel beam (fig. 3d). Moreover, many other areas of water stagnation present a high rate of corrosion: couplers, areas for attaching road signs on the hangers, etc.

Figure 4. Pictures of hanger corrosion: bar coupler and upper joint welded to the arch

Figure 5. Pictures of member corrosion: joint of hangers to the principal beam, pitting in the principal beam, transverse beam bolt connection

Corrosion is also localized in some areas of the beam (inner edge of the upper flange, attachment areas of the hangers, lower transverse beams and bolted connections, etc.) and of the arch (connections with the transverse beams, drains on the inner area and outside of the web). In some points, pitting phenomena were detected, i.e. very localized corrosion that goes deep inside the steel section, compared to the surrounding areas [8].

The deterioration of structural elements is mainly caused by corrosion in the marine environment. In this specific case, the high rate of corrosion is the essential element to be faced in the assessment because after fifteen years of life the bridge shows signs of serious damage, especially for the hangers, despite the fact that maintenance has already been carried out. Corrosion proceeds in these elements very quickly, even under the skin, that is, under the detached coating, with inhomogeneity on the surface, and concentration of deep corrosion in some points up to pitting phenomena (fig. 5), exfoliation and fractures with reductions of steel cross section.

Atmospheric corrosion occurs in the layer of humidity on the metal surface, often not visible to the naked eye, and the rate of corrosion is also affected by various factors such as relative humidity, condensation and the increase in the rate of pollution in the atmosphere. In this case the presence of rotting algae (with probable sulfate-reducing bacteria) could be an accelerating factor, together with the high humidity and average temperature [2,9].

In order to correctly assess this situation, it was necessary to identify and classify the corrosiveness of the environment in the area where the structure is located and the consequent identification of the durability of the corrosion protection systems according to the type of coating chosen.

From the investigations carried out, it appears that the bridge, despite being in an area with high susceptibility to corrosion, has not undergone galvanizing treatments during construction, hence protection has always been entrusted (starting from the original project) to epoxy-based painting cycles. Since durability is the expected time of duration of the effectiveness of an anticorrosive protection until the first important maintenance intervention, this is a case of very low durability: 5 years of life before the intervention made in 2010 and 15 years of life at present with the need for a major maintenance intervention, less than ten years from the first.

In the following sections the investigations carried out on the bridge and their results are discussed, highlighting the most important elements for assessment. Investigations were carried out with dynamic tests on the hangers and bridge deck; samples of steel elements being taken from various structural members for direct tensile tests (strength determination), hardness and resilience tests in the laboratory, as well as for performing chemical-metallographic tests and microscope investigations; in-situ hardness tests; thickness of the elements and protective coating; combined visual, magnetoscopic and ultrasonic investigations on welded and bolted joints. Investigations are carried out by the author in cooperation with 4EMME Service s.p.a., national test laboratory authorized by the Italian Ministry of Infrastructures for on-site investigations and laboratory tests on structures and specialized on bridges.

2.1 Material testing, steel corrosion and metallographic investigation

2.1.1 Coating thickness

The coating thickness values were measured on all the samples of structural elements. Very variable values of the coating thickness were found from a minimum of 542 μm to a maximum of 1181 μm above a hanger, a value which is however considered misleading as detachment from the support by internal oxidation of the hanger may have led to an alteration of the measures. An average value of about 700 μm is therefore believed to be likely, which is quite high compared to the ordinary values of the in-situ paints. Of course, it should be considered that the intervention, carried out in 2010 with epoxy paints of the Surface Tolerant type, is mostly superimposed on the previous one, because it was only done with water cleaning of the surfaces and without sandblasting. For this reason, although the detected thickness of the coating is considerable, it does not seem that it was effective for protection from corrosion, especially in the points where the marine aerosol has most attacked the structural elements due to the persistence of water, favored by the prevailing winds (joints, surfaces and base of the hangers, couplers, edge of the beam and arch inclined inwards, etc.).

Metallographic analyses were carried out under a microscope on samples taken onsite.

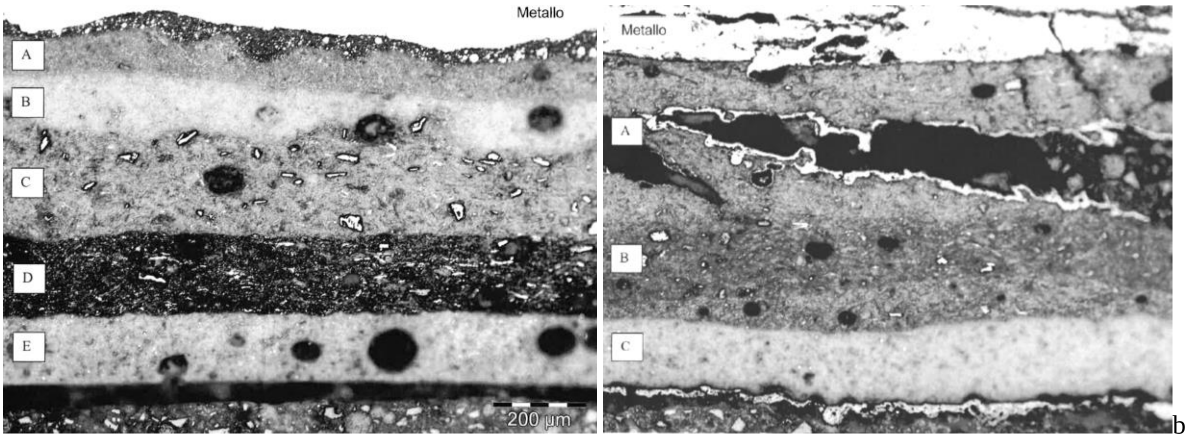
a  b

Figure 6. Microscope sections (100X) of two samples: a) deck beam; b) hanger

The results of the samples always show five different coating layers indicated with the letters A to E starting from the steel towards the outside (fig. 6a). Layer A is always an attack; in general it appears that the first three layers A, B, C refer to the period of construction, and the layers D and E to a later period, referable to the maintenance intervention carried out in 2010. The thickness of layers A and B is always around 100 μm, layer C doubles to about 200 μm, and layers D and E are instead about 140 μm thick each. The overall thickness of the coating package is approximately 700 μm, compatible with the average of the coating thicknesses detected by the in-situ tests. Evidently in these cases, the 2010 intervention only cleaned the surfaces of the previous coating and the epoxy-based Surface Tolerant paints overlapped those already present; here when layers are well bounded, the steel is well protected from corrosion. No traces of previous galvanizing layers were detected.

The result of the hanger samples instead shows 3 layers (fig. 6b):

- the inner layer is indicated by A and it is very thick (about 350 μm) with showy inclusions of oxides, hence it refers to the three original layers detected in the other samples covered in turn by the post-construction intervention. Here the painting was carried out on an already oxidized surface, in the absence of a thorough cleaning of the steel support, or without sandblasting.

- The outer layers B and C are therefore attributable to the maintenance intervention carried out in 2010.

Unfortunately, the investigation on the hanger shows that the cleaning was not thoroughly carried out; hence the application of the paints included the oxidized layer and the progression of corrosion continued in time. This may explain the advanced state of corrosion suffered by the hangers compared to other elements.

2.1.2 Steel member thickness

As for the thickness of the steel elements, it was detected by means of an ultrasonic thickness gauge, after cleaning the investigation surfaces. The investigated thickness concerned the arch and deck sections (flanges and webs) and the residual thickness of the hangers in the areas that appeared most oxidized and easily accessible during on-site operations. The results confirm the geometry of the steel plates of the original design and of the accounting drawings.

As for the hangers, while in the still intact areas, hanger diameters equal to 70 mm were found, and therefore, in line with the original nominal diameter, the measurements made in the corroded areas showed major section reductions for significant extensions and on many hangers. The average value of the section reduction is 5.5 mm (residual diameter 64.5 mm) which corresponds to a reduction of 7.78% in terms of diameter, 14.95% in terms of area and 21.56% in terms of strength modulus of the cross-section. The most corroded hangers show section reductions of almost 10 mm in diameter with area reductions of more than 24% and strength modulus reductions of over 34%.

2.1.3 Tensile steel strength and hardness

The hardness tests were carried out in a diffuse way on all the elements in situ by means of a portable hardness tester and in the laboratory, on the steel samples taken for the tensile break test. A more extensive campaign was repeated on the hangers after the results of the first tests had been acquired (HB = Brinell hardness).

The average of the hardness on the lower transverse beams of the deck was HB = 166 corresponding to an equivalent steel strength of f_t = 556 MPa, in line with a S355 grade steel.

The average of the hardness on the principal beam of the deck beams was HB = 135 corresponding to an equivalent steel strength of f_t = 452 MPa, not compatible with a S355 grade steel but with a S275 grade steel.

The average of the hardness on the steel members of the arch was HB = 144 corresponding to an equivalent steel strength of f_t = 482 MPa, not compatible with a S355 grade steel but with a S275 grade steel.

The average of the hardness on the hangers was HB = 127 corresponding to an equivalent steel breaking tension of f_t = 424 MPa, not compatible with a S355 grade steel but, at the limit, with a S355 grade steel but with a S275 grade steel. It is worth noting that many values of hardness onsite are below a S275 grade for hangers, down to the minimum value of HB = 115, corresponding to a steel strength of f_t =380 MPa.

By using the direct tensile strength test for calibration of the correlation coefficient between the hardness measured in the laboratory and those obtained in situ, the result is an average correlation coefficient f_t/HB = 3.35, which confirms the value of the literature curve adopted for the hardness tester in situ. The hardness-strength correlation is therefore confirmed by laboratory tests and the correlation coefficient is in the range suggested by the Standard ASTM A370, which reports a table of correlation between hardness and steel strength [10]. From this table the current range of values for S275 grade steel is HB = 125÷140 (f_t=430 MPa) while for S355 grade steel is HB = 150÷165 (f_t=510 MPa).

Regarding the tensile strength values determined by direct laboratory tests, they confirm the indirect evaluations made through the hardness test campaign.

2.1.4 Tests on welded and bolted joints

The tests on the connections between the various elements were of different types: extensive visual inspection, removal of bolts and nuts from bolted joints of the deck, and in-situ investigations of bolted and welded joints with combined visual, magnetic and ultrasound methods.
From the visual inspection, many bolts present high states of surface corrosion; the ultrasound investigation extended to the beam-arch connection bolts did not identify breakages or deep cracks in the bolts for shear strength. Instead some nuts and the heads of some bolts were affected by the oxidation layer with exfoliation, generally superficial and not deep.
Considering that the nominal tensile strength of a class 10.9 bolt is f_t =1000 MPa, the results obtained by direct and indirect tests can be acceptable and compatible with the design strength class.
Furthermore, bolts of the arch-beam joint were tested through ultrasounds: the length of each bolt was equal to an average of 157 mm and no internal defects were observed, after eliminating the surface area damaged by corrosion. It therefore seems possible to preserve the existing bolts after cleaning with sandblasting to white metal and creating a new layer of protection, without altering its functionality.
No significant defects were found on the welded continuity joints of steel arch and principal beam, confirming the results of the tests on the original welds already available in the original documentation.
About bars and couplers of hangers, the investigations were carried out in-situ with a magnetic method and ultrasounds in several sample points. In this case, three types of joint are distinguished:
a) the lower junction of the hanger welded to the web of the main beam (fig. 3c);
b) the upper joint of the hanger welded to the connection plate to the arch box (fig 3a,b);
c) the coupler joint with welds between the coupler and the bar of the hanger (fig. 4).
In the lower junction of the type (a) there were many surface defects due to the high degree of corrosion with some cracks, mostly superficial. No deep cracks emerged that seem to compromise the tightness of the joint.
For the upper joint of type (b) there were surface defects and advanced corrosion in some upper plates, especially in the lower area of the connection (lower point of attachment between the hanger bar and the connection plate) with exfoliation and deep incision of the weld due to oxidation. The tightness of some connections is therefore to be considered compromised.
For the joint of type (c) between the hanger and the coupler, very significant defects were found due to the extremely high state of corrosion, especially in the upper attachment areas of the coupler, with exfoliation of the coupler body, fractures on the welds and reduction of the cross-section at the attachment of the hanger bar. This situation made it impossible to carry out an internal investigation of the welds, since cleaning with sandblasting would be necessary before reaching the still effective area. As a result, many coupler joints are limited in functionality and unreliable.
From the analyses it can be concluded that while the joints of the hanger with the deck beam, although corroded, are not of particular concern, the situation of the joints at the top of the hanger with the arch and the coupler joints are to be considered damaged by corrosion with some elements that cause particular concern for the joint strength. In addition, at the base of the hanger high corrosion damage is located when the hanger passes through the hole in the beam upper flange.

2.1.5 Chemical and metallographic analyses

Chemical and metallographic analyses under the microscope were carried out on different elements, aimed at identifying the chemical composition of the steel used for construction and the analysis of the crystalline structure of the steel as well as the stratigraphy of the coating. The samples are intact without significant corrosion.

It should be noted that the percentages of carbon, silicon, phosphorus and sulfur are lower than those declared in the sheet metal alone tested during the works, while the percentage of manganese appears comparable. In particular, the reduced amount of carbon and silicon can influence the mechanical characteristics of both strength and hardness of steel, since the reduced percentage of carbon is generally responsible for reduced tensile strength, such as that found by the strength tests carried out.

Sample	%Fe	%C	%Si	%Mn	%P	%S
W1	98.15	0.1185	0.1863	1.807	0.0111	0.0098
W2	98.04	0.1363	0.1861	1.122	0	0.0007
W3	98.10	0.1393	0.2414	1.356	0	0
W4	98.15	0.1879	0.1628	1.333	0	0
Average of in-situ tests	98.11	0.1455	0.1942	1.4045	0.0028	0.0026
Original steel tests	98.10	0.1930	0.2450	1.4350	0.0215	0.0120

Table 1. Chemical analysis of steel samples

Metallographic analyses were carried out under the microscope on the four in-situ samples. Samples W1, W2 and W3 appear with ordered structure without defects and with medium-sized grains. Samples W2 and W3 show a band structure typical of structural elements with hot laminated steel, with longitudinal arrangement of the pearlite. From the metallographic point of view, the samples have comparable characteristics in the distribution of pearlite within the ferritic matrix. Sample W4 (taken from the deck steel beam) has a larger size and quantity of pearlite grains than the other samples. The strength and hardness of this steel were higher than the others, as was the percentage of carbon. From the comparison it can be said that the steel used for the main beam is different from that used for the other elements. The best results provided by the mechanical tests on sample W4 (deck beam) seem to be confirmed by the metallographic analyses (fig. 7); comparison of the metallographic analyses, strength, hardness and chemical compositions seems to assign a reduced mechanical performance to the other elements, in line with the onsite investigations.

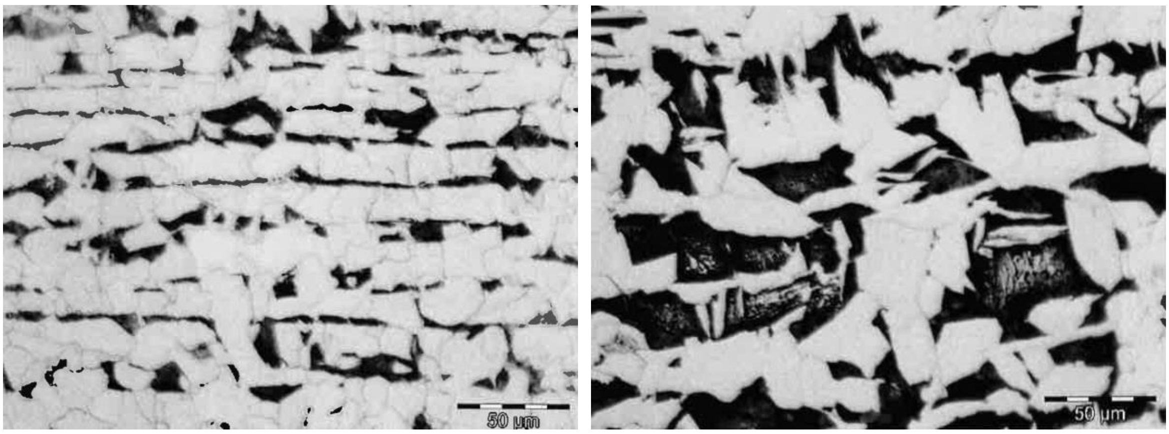
a  b

Figure 7. Metallographic views under the microscope (500X). a) steel with minor size of pearlite grains; b) steel with major grains and best strength properties

2.2 Dynamic investigation

Dynamic tests were carried out on 7 different hangers. They were investigated by means of three accelerometers located at 1/6, 2/6 and 3/6 of the length in the perpendicular direction to the arch plane (transverse y direction) for identifying the modal shape and frequencies. On some hangers the test was also repeated in an orthogonal direction to the hanger but on the arch plane (global x direction), in order to verify the geometric symmetry and the constraint degree to arch and deck beam.

The characteristic behavior of the hanger, consisting of a 70 mm diameter steel bar, is that of showing the equal importance of the bending (beam) behavior and the typical axial behavior of the tie rod. For this reason, the theoretical research of the modal frequencies cannot rely on the usual equations of the beam or vibrating string, since the frequencies that depend on flexural inertia and axial force are different from those of a rope or a flexional stiff beam. Hence the comparison of the frequencies obtained by onsite investigations must be carried out with the results of the fourth-order differential equation which governs the problem of the dynamics of elements having a bending and axial stiffness and subjected to a significant axial force, which modifies the vibrational characteristics [11,12].

The equation which governs the problem is the following one:

$$T\frac{\partial^2 y}{\partial x^2} - ESj^2\frac{\partial^4 y}{\partial x^4} = \rho S\frac{\partial^2 y}{\partial t^2} \qquad (1)$$

where j is the radius of inertia, ρ is the mass per unit of volume, S the cross section area, (ρS the mass per unit length), E is the elastic modulus and T the value of axial force.

Since the numerical resolution of eq. (1) and identification of the eigenvalues is rather complex [13], approximate solutions have been developed in the literature, for which the n^{th} modal frequency is given by the relationship [14]:

$$f_n = \frac{n}{L}\sqrt{\frac{T}{\rho S}}\left(1 + 2\sqrt{\frac{ES\,j^2}{TL^2}}\right) \qquad (2)$$

where L is the length of the element, in the case of clamped ends.

A comparison of the frequencies found by eq. 2 was made with those found through an FE model of the hangers carried out in the software MIDAS Gen 2019, applying an initial prestress through the specific function of the software for determining the modal frequencies; this function gives an FE solution of eq. 1 to the specific case of boundary conditions. In fact the free length is here reduced of about 50 cm with respect to the overall length because of the connection welded to the deck beam and, at the hanger top, to the arch, as well as the passage from the hole of the beam flange at the base of the hanger. This modifies the constraint conditions: a clamp is approached in the x direction (longitudinal direction) while an intermediate behavior between clamp and hinge is registered in the y direction (transverse direction).

Since in eq. 2 the value of the axial force is an important variable to be known a priori, the value T introduced into the equation is the one determined through the global bridge model already carried out in the Finite Element software. The comparison between the results of onsite tests and the theoretical-numerical elaborations is reported in table 2 for the first modal frequency and in table 3 for the second one.

Hanger	Axial force T	L	Theoretical frequency	FE half clamp	FE clamp	Experimental frequency	Deviation
	kN	m	Hz	Hz	Hz	Hz	
08 M	232.7	12.65	3.99	3.87	4.24	3.66	-5.426%
09 M	223.5	13.50	3.65	3.53	3.86	3.70	1.370%
09 M dir. X	223.5	13.50	3.65	3.53	3.86	3.70	1.370%
13 M	206.3	15.30	3.06	2.96	3.21	3.17	-1.246%
13 M dir. X	206.3	15.30	3.06	2.96	3.21	3.32	3.427%
19 M	223.5	13.50	3.65	3.53	3.86	3.30	-6.516%
09 V	223.5	13.50	3.65	3.53	3.86	3.70	1.370%
14 V	206.0	15.40	3.03	2.94	3.18	2.90	-1.361%
14 V dir. X	206.0	15.40	3.03	2.94	3.18	3.10	-2.516%
22 V	269.0	10.40	5.32	5.16	5.74	5.30	-0.376%

Table 2. First modal frequency of hangers

Hanger	Axial force T	L	Theoretical frequency	FE half clamp	FE clamp	Experimental frequency	Deviation
	N	m	Hz	Hz	Hz	Hz	
08 M	232.7	12.65	7.99	8.42	9.20	7.76	-2.879%
09 M	223.5	13.50	7.29	7.65	8.33	-	-
13 M	206.3	15.30	6.11	6.34	6.85	6.54	3.155%
13 M dir. X	206.3	15.30	6.11	6.34	6.85	6.84	-0.146%
19 M	223.5	13.50	7.29	7.65	8.33	6.90	-5.350%
09 V	223.5	13.50	7.29	7.65	8.33	7.80	1.961%
14 V	206.0	15.40	6.06	6.29	6.79	6.25	-0.636%
14 V dir.X	206.0	15.40	6.06	6.29	6.79	6.35	0.954%
22 V	269.0	10.40	10.64	11.45	12.64	-	-

Table 3. Second modal frequency of hangers

The results show good agreement between the theoretical value of the frequencies of the 1st and 2nd modes and what is determined by onsite tests. The comparison between the theoretical values of frequencies and those obtained by the FE model allowed the author to calibrate the FE model and to consider it reliable for the evaluation of tensile axial forces of hangers due to dead loads. The greatest deviation (in reduction) shown by hangers 19 M and 08 M can be attributed to different factors:

- a different value of axial force in that hanger, different with respect to the theoretical one (which would lead to a lack of homogeneity in the distribution of the hanger tensions and therefore in the actual behavior);
- a different constraint condition with reduction of stiffness at the ends (which would result in less functionality of the end welded connections);
- a concentrated lack of stiffness due to the coupler condition (which would lead to damage to the coupler with reduced functionality).

Particular attention should be paid to hanger 19M because the reduction deviation is strong on both the two frequencies and also with respect to the symmetrical 09M and 09V hangers, which have higher frequencies, in line with those determined by the theoretical-numerical evaluations. This implies an asymmetry of behavior on both the bridge sides to be attributed to possible reductions in tension or stiffness of the hanger constraints with respect to the symmetrical ones. Figure 8 shows the typical result of a dynamic analysis on a hanger.

The dynamic tests on the deck were later repeated, placing three accelerometers at 1/6, 2/6 and 3/6 of the deck span, on the edge of the principal beam. The measurement was carried out in the *z* direction (orthogonal to the road surface) and the excitement was given through the jerk of a truck obtained with the introduction of an artificial bump in the middle of the bridge. The measurements confirmed a value of the first modal frequency, on both occasions, equal to approximately 1.8 Hz. The value obtained was compared with the theoretical model, with very good agreement.

Considering the excited mass and the overall result of the frequencies recognized with onsite tests it is believed to be possible to validate the FE model of the bridge and to consider the values obtained acceptable.

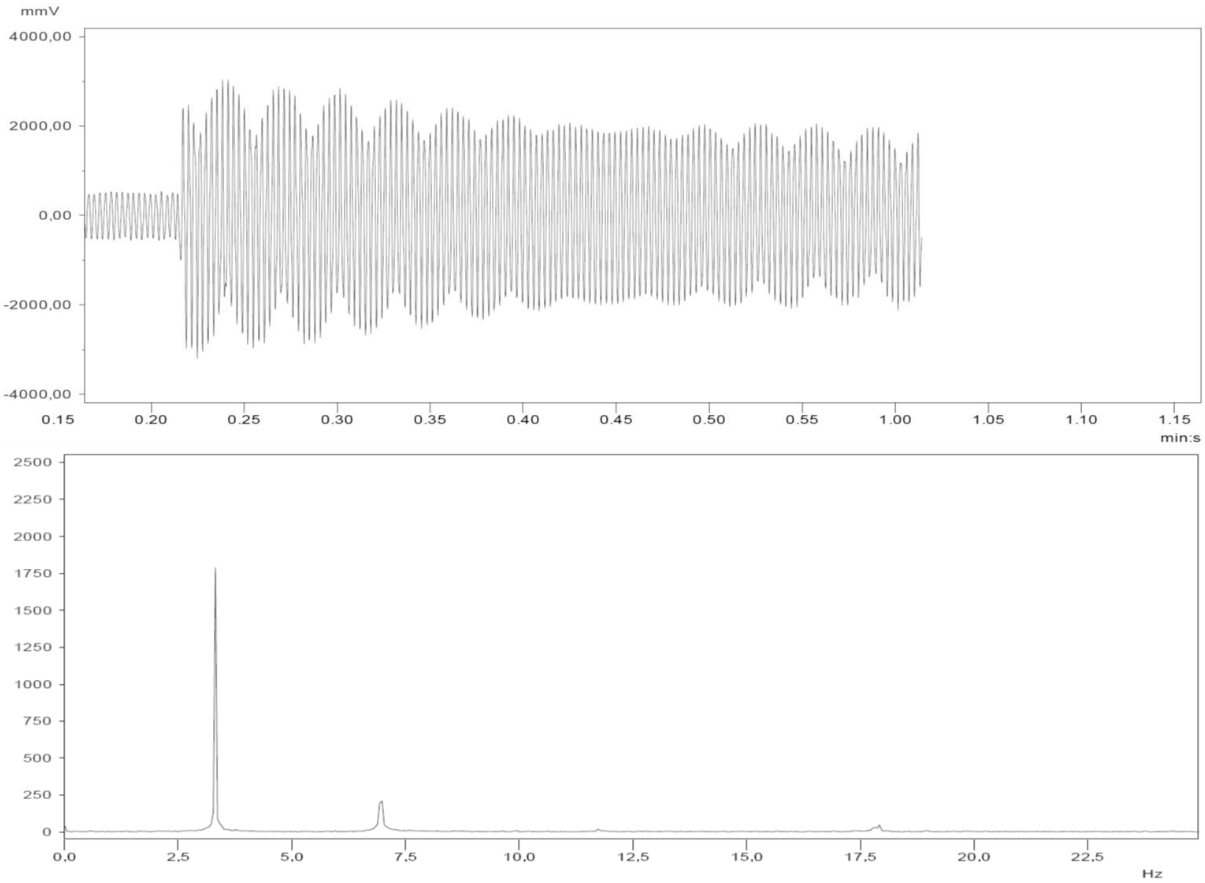


Figure 8. Dynamic tests on hangers: oscillography and analysis in the frequency domain

2.3 Conclusions of investigations and test analysis

From the results of onsite investigations and laboratory tests, from the correlations between direct and indirect measurements, from the investigations on the welded and bolted connections and from the comparisons with the original data of construction, it can be concluded that:

a) the chemical and mechanical characteristics of the steel used for most of the structural elements do not coincide with those declared in the original documentation; in particular the mechanical characteristics of the steel are lower than those of S355 grade and near those of S275 grade: for this reason the latter one was the strength class used for all checks.

b) The high degradation due to corrosion of hangers (reduction of cross-section) obtained from ultrasonic thickness tests, associated with the reduced strength of steel, can lead to a danger of susceptibility to stress peaks for maximum loads and/or fatigue failures.

c) The high corrosion degradation of the welded joints, especially at the top of the hangers (connections to the arch) and at the bar couplers, leads to a partial loss of functionality with the danger of breakage or failure for some hangers due to the tensile axial stress induced by the maximum loads.

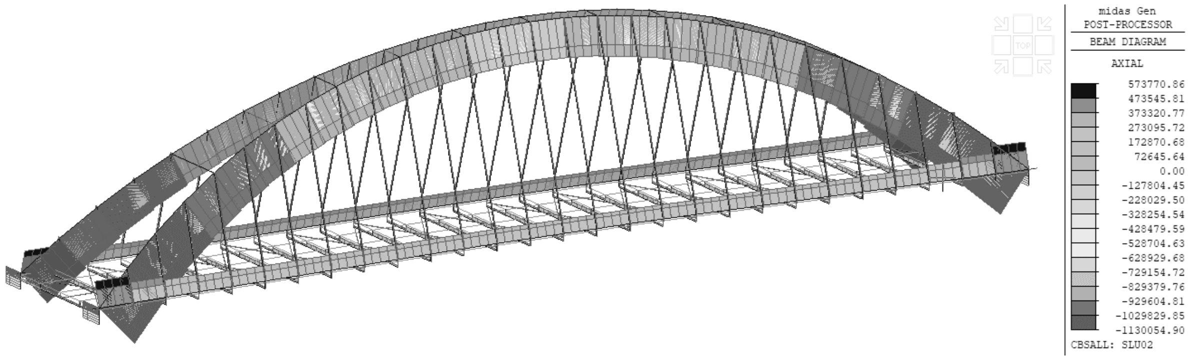
d) The thickness of coating in the main elements seems to be adequate for the steel protection but in many points the corrosion under the skin is still in progress as the 2010 maintenance intervention does not seem to have stopped the corrosion in the points of greatest attack due to insufficient cleaning of the support from oxides before applying new layers of paint.

d) The susceptibility to corrosion of the carbon steel used in the marine environment is very high and the protective layer must be improved to increase the durability of the existing elements but it is not possible to preserve the existing hangers because the state of degradation at present is too high, with the danger of local and/or global collapse of elements (in the case of progressive failures) for maximum service loads.

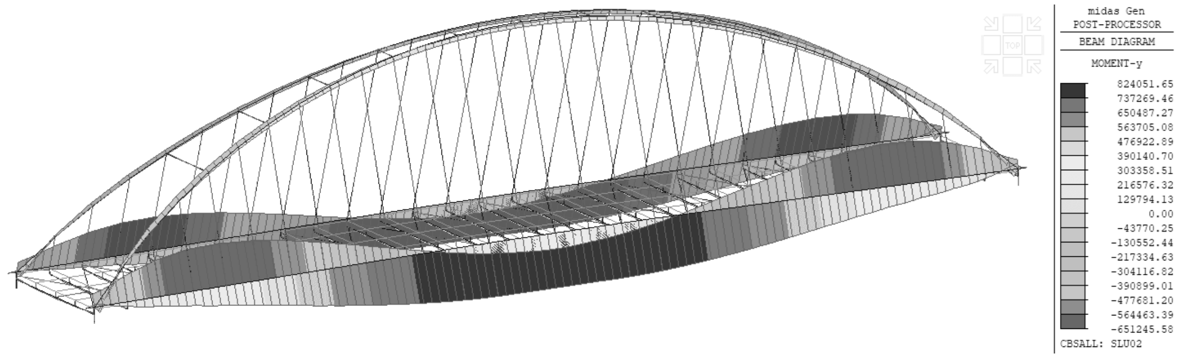
3 Structural evaluations

For the structural analysis, a Finite Element model was carried out using MIDAS Gen 2019 software. The model is three-dimensional, with beam type elements for the steel members and shell type for the deck slab: the full model is composed of 636 joints, 761 beam elements and 112 shell elements. Two analyses were performed: a linear one and a geometric nonlinear one. The results of the two analyses agree with slight differences for the undamaged bridge; for this reason only the results of the linear analysis are shown here. Figure 9 shows the main results of the analysis of the undamaged bridge for the combination of dead load and moving loads. If the bridge was in the ideal undamaged conditions of the original project, all the checks would be satisfied. Unfortunately, the state of degradation due to corrosion of elements does not make it possible to evaluate the global and local safety coefficient with the stress checks; it is also necessary to consider the reduced section of some hangers, heavily damaged by oxidation. Since the reduction of the steel cross-section is not homogeneous on all the hangers and does not show the same extent, it is not possible to hypothesize homogeneous redistribution of the stresses between the elements of the bridge, hence the most important effect is the local one due to stress peaks. The central hangers have been reduced in their section for an average value of 5 mm to 10 mm:

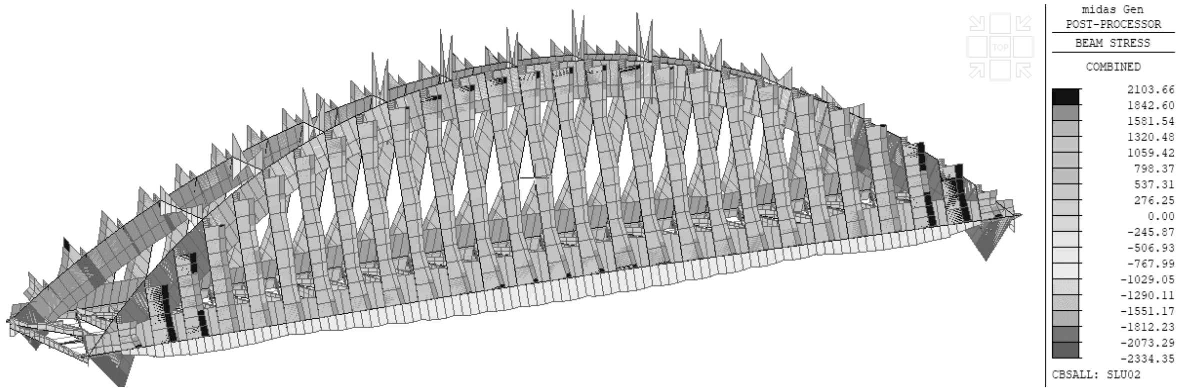
i.e. the nominal diameter of 70 mm is reduced to 65 mm and in some cases to about 60 mm.



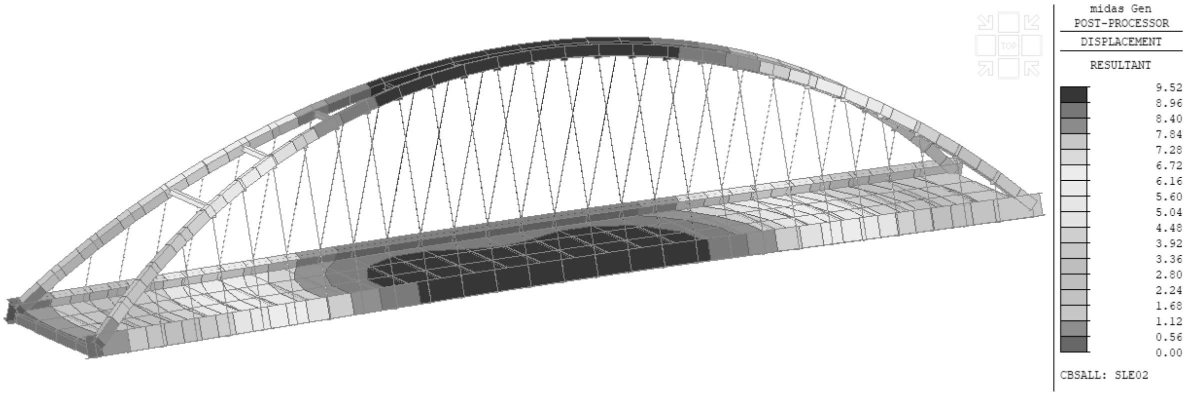
a



b



c



d

Figure 9. Results of FE analysis of the undamaged bridge for dead loads + moving loads. a) Max-min axial force (kN); b) Max-min bending moments(kNm); c) Max-min stress in steel elements (MPa); d) Max displacements (cm)

Therefore, the checks for stress peaks, based on the resultant forces obtained from the FE model that are to be expected in the hangers, are shown in table 4.

Values above 1 for moving load combinations computed according to Eurocode [15] show that the checks are not satisfied, associated with the steel strength properties obtained by onsite tests. The checks were carried out with Eurocode provisions [16].

If cross-section reduction due to corrosion increases, reaching the diameter of 55 mm, the checks show the achievement of the ultimate limit state and the danger of local failure for sudden breakage for the combination of dead loads.

For this reason, the bridge is currently subject to limitation of heavy traffic. Further damage due to corrosion would lead to total closure.

HANGER	d	A	J	W
14V	[cm]	[cm^2]	[cm^4]	[cm^3]
	6.0	28.274	63.61725	21.206
	Load Combinations			
FORCES	SLU01	SLU02	SLU06	SLU09
N [N]	287720	418740	419650	419510
M [Ncm]	290240	287650	297480	296570
σ [MPa]	238.6	283.7	288.7	288.2
Check σ/f_k	0.9111	1.0834	1.1023	1.1005

HANGER	d	A	J	W
18V	[cm]	[cm^2]	[cm^4]	[cm^3]
	6.0	28.274	63.617	21.206
	Load Combinations			
FORCES	SLU01	SLU02	SLU06	SLU09
N [N]	303780	438430	438790	439200
M [Ncm]	251490	247600	258530	257800
σ [MPa]	226.0	271.8	277.1	276.9
Check σ/f_k	0.8630	1.0379	1.0580	1.0573

Table 4. Examples of stress check for two hangers damaged by corrosion. d: diameter, A: area, J: moment of inertia, W: strength modulus

4 Bridge retrofit: replacement of hangers and corrosion protection

For the retrofit of the bridge, the most complex intervention is replacing the existing hangers [17]. It is planned to replace 46 hangers out of the total of 54, maintaining onsite the first two existing hangers on each side of the bridge. The replacement must take place with adequate supports and retaining tools (retaining cables, shoring tower, etc.) in order to also replace the anchor joints of the hanger on the arch and beam. The entire welding area of the hanger to the beam will then be cleaned and the upper plate anchoring to the arch will be partially cut to make way for the new plates. The hanger, currently embedded at the ends through the welded connections, will become a hanger-rod with forks and final pin on both sides (pinned-pinned tie rod), in order to reduce the stress on the end welds. The replacement will take place for one hanger at a time with a precise sequence, from the ends towards the center of the bridge both longitudinally and transversely.

The existing hanger will be divided into several parts and during cutting stresses and deformations must be monitored, both on the hanger itself and on the adjacent ones, also evaluating the displacements induced by the release of the cut hanger. The new hanger will then be assembled, and equipped with a tensioner for adjusting the axial stress, in order to recover the pre-stress and previous deformed configuration under dead loads. The whole operation will be monitored with strain gauges on the old and new hangers and with measurement of displacements along the bridge.

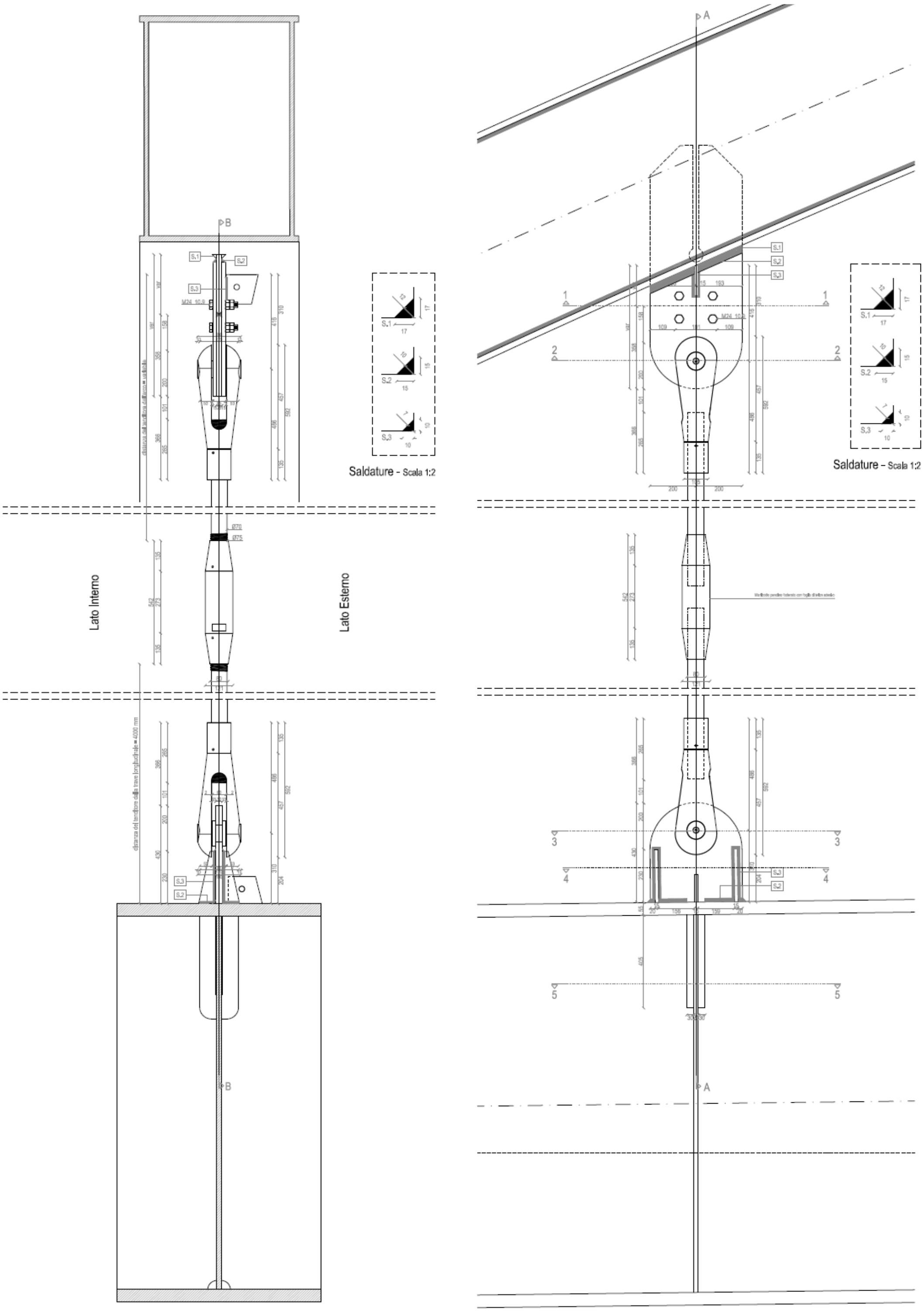


Figure 10. Geometry of the new hanger

Figure 10 shows the geometry of the new hanger. It will be made of a 70-mm diameter bar like the existing one, with hot galvanized steel S355JR type; the intermediate tensioner will be able to adjust the hanger after it has been put into position providing pre-tension that brings the state of stress of the new hanger back to that of the existing one. The aim is not to change the original state of the bridge after the work was completed in 2004, maintaining the states of stress and deformation of the final construction stage.

The choice of maintaining the rigid bar like in the original design was taken carefully by the designer under request of the Owner, after checking hanger for fatigue failure following the recommendations of Eurocode. In fact, an hanger made of cables would perform better with regard to deterioration and redundancy, because a crack developed at the surface of the 70-mm bar would grow under fatigue to cause failure of the cross-section of the hanger. Generally the deterioration of a hanger made of high-strength steel wires, would be highly more redundant and perform better under fatigue. However in this case the stress variation was rather low thanks to the small distance between the hangers; furthermore, the choice of creating a pinned-pinned connection made it possible to eliminate the welds at the top and bottom of the hangers, improving their performance also in terms of fatigue. The redundancy is therefore entrusted to the spacing of hangers and to the low value of the service stress.

The bar of the new hanger, as well as forks, pins and tensioners, will be hot-dip galvanized in the factory, taking care that the threads undergo effective galvanization to guarantee rubbing strength. The anchor plates will instead be welded and galvanized onsite. All the elements will then be protected with the same painting cycle: high adhesion zinc-based epoxy primer suitable for already galvanized surfaces, high solid epoxy intermediate layer, and white polyurethane finishing layer.

It is necessary to insert a Teflon tape around the pin and between the adhering plates for disconnection of the contact between the hot-dip galvanized and the elements galvanized onsite, in order to avoid concentrated corrosive phenomena due to possible dielectric couplings. The same must be done outside the tensioner by sheathing the area of the tensioner and the threads with a Teflon sheet so that the flowing water does not infiltrate the tensioner and localized stagnation and corrosion does not occur.

For protection of the existing structures, a sandblasting cycle of SA3 grade, onsite thermal spray galvanization (zinc coating) and subsequent painting cycle are planned. In fact, it is not believed that the owner can rely again exclusively on an epoxy-based cycle, since this has twice proven to be ineffective in the short span of 15 years. Hence a zinc based cycle must be implemented. There are two ways to deal with the problem:

- one would be to proceed with a "thermal spray zinc" type treatment, used in all English and Norwegian bridges as a rule for over thirty years, as well as in the USA, precisely for steel bridges by the sea [18]. This technology is the safest and best performing, with a high technological level for installation on existing structures.
- The alternative is to treat the surfaces, after sandblasting and complete cleaning, with a painting cycle that includes a zinc-based primer, also called "cold galvanizing", i.e. pseudo-metallization with a layer of paint with zinc oxide sprayed or brushed on, with high protective thickness and a subsequent layer of epoxy paint only for coverage, for additional protection of the sacrificial layer.

In addition to the two galvanizing alternatives, there is the option of the duplex, i.e. a first metallization and a second covering with painting cycle [19]. The choice made in this case is that of duplex technology associated with thermal spray galvanizing and therefore actual in-situ metallization.
Thermal spray galvanizing consists in spraying the molten zinc, finely pulverized, on the surface to be suitably protected with sandblasting for white metal (grade SA3). The zinc is sprayed with guns having a melting and spraying device and a zinc feeding device. The fusion is ensured by an electric arc, and the spraying fluid is generally dry and oil-free air, at a pressure of 0.27 / 0.54 MPa. Zinc is used in the form of wire, with a diameter between 1.5 and 2.5 mm. Metallization must be carried out immediately after the preparation of the surface, so that it is still perfectly clean, dry and not oxidized. This is essential for electrical contact and therefore anodic protection of the zinc coating with respect to the steel of the support. The minimum layer of galvanization when finished must be 100 μm. The painting cycle planned after galvanizing is the following.
1) Painting with medium-high thickness modified epoxy-polyamide primer with high content of non-toxic active pigments (zinc phosphates) with high adhesion and chemical resistance, particularly suitable for the protection of surfaces of already galvanized steel, and also suitable for retouching welding joints or for repairing damage to the epoxy coating during construction. Quite long recoating intervals with epoxy or polyurethane coatings are possible. It can be covered with a variety of products: chlorinated rubber, alkyd, vinyl, polyurethane, synthetic and oven-baked polyester, epoxy and epoxy. Compatible with cathodic action systems, it has good resistance to water and corrosion. The final thickness of the dry film must be at least 80 microns.
2) Painting with two-component, high solid epoxy intermediate layer, formulated with micaceous iron oxides which increases the barrier effect and the long-term protection characteristics of the painting cycle. Used as a high thickness intermediate, it provides excellent barrier protection when used in aggressive environments, such as bridges, chemical plants. The minimum final thickness of the dry film must be 80/100 microns.
3) Painting with two-component satin polyurethane enamel, high gloss and surface hardness, which shows excellent resistance to atmospheric agents with the use of an unalterable catalyst. The minimum final thickness of the dry film must be 80/100 microns, and the color white.

5 Conclusions

The assessment and retrofit of a steel arch bridge highly damaged by corrosion in a marine environment have been shown and discussed. The results of onsite investigations showed that the steel strength was lower than the expected one and that hangers were the most damaged structural members. Corrosion had reached a very high rate, leading to high level of damage 15 years after construction, in spite of a maintenance intervention already carried out. The lack of galvanized steel in the original project and the ineffective maintenance provided imply that many hangers decrease their serviceability performance till there is a danger of achieving the ultimate limit state in the case of maximum live loads. For this reason it was necessary to design a drastic maintenance intervention with replacement of hangers and sandblasting of all the bridge surface for onsite thermal spray zinc coating and duplex protection through a new cycle of three-layer painting, following the recommendation of American and Northern European Standards. The results of the investigations, assessment and structural analysis are shown, as well as the main intervention planned, in order to give useful tools to engineers dealing with steel bridges in a marine environment.

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