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CORROSION FATIGUE INVESTIGATION ON THE POSSIBLE COLLAPSE REASONS OF POLCEVERA BRIDGE IN GENOA

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Abstract. *On August* 14, 2018*, a few spans of the cable-stayed viaduct crossing the Polcevera river (Genoa, Italy) collapsed, causing tens of fatalities along with considerable material damage and hundreds of people displaced. The viaduct, as well as many others belonging to the national road network, was built in the second half of the last Century, and has been in service for over fifty years. In the present paper, a possible scenario is proposed to put into evidence how the combined effect of fatigue at very-high number of cycles and corrosion could have been responsible for the sudden failure of one of the strands and the subsequent collapse of the so-called balanced system conceived by the designer Morandi. The analysis accounts for an actual estimation of the heavy lorries traffic and load spectrum, as well as the European standards prescription for the fatigue damage accumulation assessment. In addition, the effective construction phases of the viaduct are considered. The structural analysis is carried out by means of analytical models, in order to simplify the structure complexity without prejudice to the description of the most relevant aspects of the structural behaviour. The purpose of the present study is not to identify responsibility among the different actors involved with the infrastructure collapse, aspect to be addressed by the Italian magistracy. On the other hand, the main goal is to warn the scientific community and the public administrations that the combined effects of low amplitude fatigue and corrosion can be dangerously underestimated, and that the existing asset of last Century bridges deserves special attention in this respect.*

1 INTRODUCTION

Nowadays, the design of new bridges with respect to fatigue phenomenon, can be carried out according to the European standards [1], which basically assume the so-called stress-life approach with a Wöhler curve that attains an asymptotic fatigue limit when the number of cycles exceeds 10^8 [2, 3]. The standards provide also methods for the correct estimation of the load spectrum [4], together with prescriptions regarding the limitation of degradation, the monitoring of steel tendons, and the possibility of replacement of degraded elements.

Unfortunately, the case of existing last Century bridges is quite different, and the interplay of underestimated load spectrum, poor maintenance, and aggressive environments can combine in a very deadly way, as happened in 2018 to the Morandi cable-stayed bridge over the Polcevera river (Genoa, Italy).

Recently, it has been put forward that the collapse of the Polcevera viaduct could have been triggered by the fatigue failure of one of the cable-stays [5], causing the subsequent entire failure of one of the so-called self-standing systems and of the two adjacent Gerber beams (Fig. 1). This conjecture was based on the main hypothesis that the concrete covering of the strands was decompressed, at least in one or several sections close to the antenna.

In the following, the mechanical behaviour of the cable-stay is considered in detail. Although simple analytical models will be used, it will be accounted for the complex construction phases of the stay, the geometrical nonlinearities of the cables, the stress losses due to prestressing of the concrete covering of the steel tendons, and the mechanism of stress redistribution due to the degradation and corrosion evolution.

In this way, not only the congruence of the decompression hypothesis will be proven, but it will be shown also that the cable-stay could have retained almost its initial stiffness up to the failure moment, therefore resulting in a very brittle behaviour.

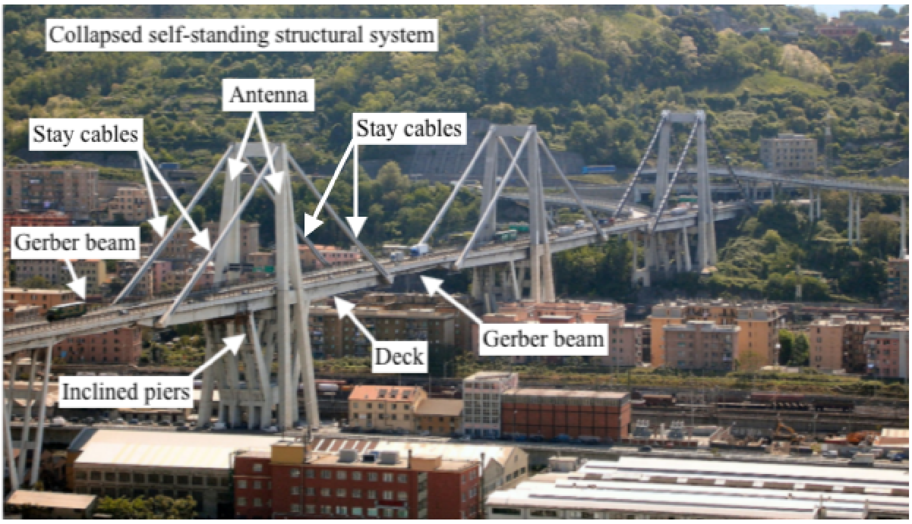

Figure 1: Three self-standing structural systems of Polcevera Bridge, with details of the structural elements of the collapsed part.

2 ANALITICAL MODELS FOR THE MORANDI BRIDGE

As shown in [5], the Morandi Bridge can be analysed with rather simple analytical models regardless of the structural complexity. In particular, the axial force in the cable-stays at the connection with the deck can be estimated with no special efforts, given that the cross section of the main deck is assumed constant, and considering that the 352 steel strands, which were put in place, were continuously re-tensioned during the removal of provisional tendons [6], in order to vanish the vertical displacement of the hanging section (point A in Fig. 2a).

On the other hand, it is crucial to our purposes to obtain the cable-stay geometry and distribution of stresses with sufficient detail. According to the construction phases, first of all the 352 inner steel strands were put in place. Note that at the connection with the antenna, the kinematic boundary condition is known, while at point A the vertical static component and the kinematic boundary condition are known from the previous analysis of the deck. In this case, the analytical solution for the elastic catenary [7] can serve to our goal, provided to assume as unknown discrete parameters the initial length of the cable and the horizontal component of the force at point A. Those two unknowns can be determined by iterative calculation, and substituted into the general solutions for the axial force, and for the elastic line (Fig. 2b, red line). At the second stage of the cable-stay construction, the concrete covering was added. Since the weight of the concrete severely modifies the cable-deformed configuration, the concrete was cast in several distinct segments, and joints were filled only after the segment curing. The final configuration can be obtained directly from the previous elastic catenary equation, just increasing the dead weight of steel with the amount due to the concrete segment, and recalculating the discrete unknowns. Given the deformed configuration (shown in Fig. 2b with red line), it is possible to obtain the stress in the cables as well as in the concrete.

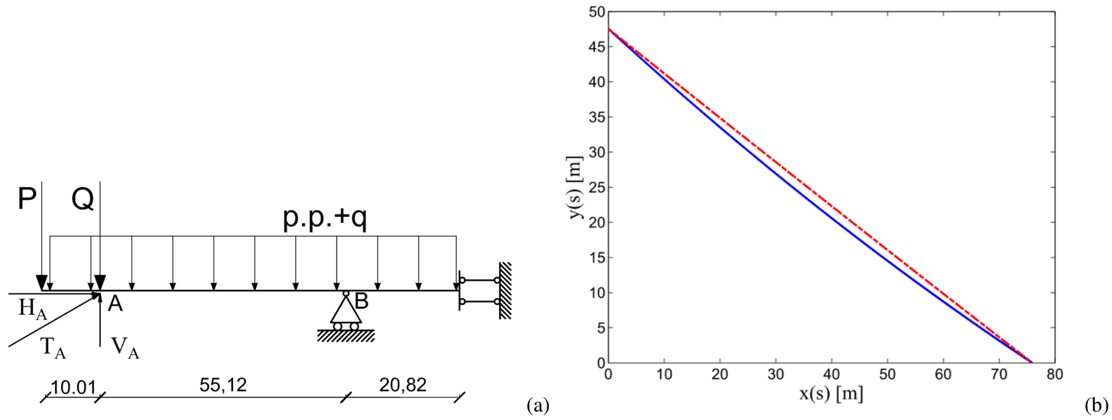

Figure 2: Static scheme used for the evaluation of the normal stresses in the 352 strands due to the self-weight of the deck (a); Deformed configuration of the 352 strands during the construction phase of the stay cables (b).

During the third construction phase, 112 additional outer strands were added to the concrete covering and pre-stressed in order to set a compressive coaction state in the concrete and improve the inner cable protection. Note that the effect of the stresses induced in the concrete by prestressing is symmetrical, therefore it is reasonable to assume that the curvature of the cable-stay keeps unvaried. Finally, the system is injected and connected to the transverse connection beam, realising (in principle) a perfect homogenised cross-section.

Having determined the precise deformed configuration of the cable-stay, and being known the initial state of imposed deformations, it is possible to determine the stress in the concrete covering, as well as in external and internal steel strands not only accounting for the relaxation of steel, but also for the friction stress losses due to the strands' curvature. The results are summarised in Tab. 1.

Curvilinear coordinate along the cable [m]	Normal stress in the concrete [MPa]	Normal stress in the 352 strands [MPa]	Normal stress in the 112 strands [MPa]
0	-0.37	583	743
10	-0.83	577	768
20	-1.28	571	794
30	-1.73	565	819
40	-2.18	558	845
50	-2.64	552	871
60	-3.09	546	897
70	-3.54	540	922
80	-3.99	534	948
89.45	-4.42	529	973

Table 1: Normal stresses in subsequent sections of the stay-cables (curvilinear coordinate measured from the antenna).

3 STRESS REDISTRIBUTION DUE TO CORROSION

The cross section of the cable-stay, represented in Fig. 3a, can be considered made up of three main components: the 352 inner strands, the concrete covering, and the 112 outer strands. According to the sequence of the construction phases, the cable-stay is not subjected to sensible bending moments. Therefore, each component is subjected to uniform stress and strain conditions. In addition, each of the three components can be effectively considered acting like the system of springs in parallel shown in Fig. 3b. The axial force in each cross section of the cable-stay has been obtained in the previous section. Therefore, it is possible to calculate the axial force partition among the three components according to the well-known stiffness criterion, not only in the undamaged configuration, but also for different corrosion evolution scenarios.

The stiffness of the inner and outer steel strands is proportional to their cross section, which decreases linearly with the increase in the corrosion. The concrete stiffness remains constant until a compression stress field is maintained. On the contrary, assuming that the tensile strength of concrete could be disregarded, the stiffness of the concrete covering segment vanishes as soon as, due to the increasing corrosion level, the compressive stress field is lost. Therefore, the axial force in decompressed concrete segments is completely carried out by steel tendons alone.

It is worth noting that, considering Tab. 1, the cross sections, where the compression prestressing of concrete is less effective, are located in the region close to the antenna. Nevertheless, if no corrosion is considered, the compressive stress field in the covering concrete is guaranteed along the whole length of the cable-stay.

On the other hand, if corrosion takes place, the steel cross section diminishes, and the stresses are redistributed accordingly.

The diagram of Fig. 4a shows the evolution of tensile stress in the inner and outer tendons, for increasing levels of corrosion, considering a cross section of the cable close to the antenna. Three different scenarios are considered, conjecturing that the corrosion could proceed only in the outer strands, only in the inner strands, or uniformly in the whole steel reacting section. Note that, when the tensile stress in the steel reaches the ultimate tensile strength, the corresponding strands are removed since brittle failure is likely to occur, and the stresses are redistributed. Fig. 4b shows the corresponding evolution of compression stress in concrete. Note that, regardless of the adopted corrosion scenario, the concrete is completely decompressed already for a

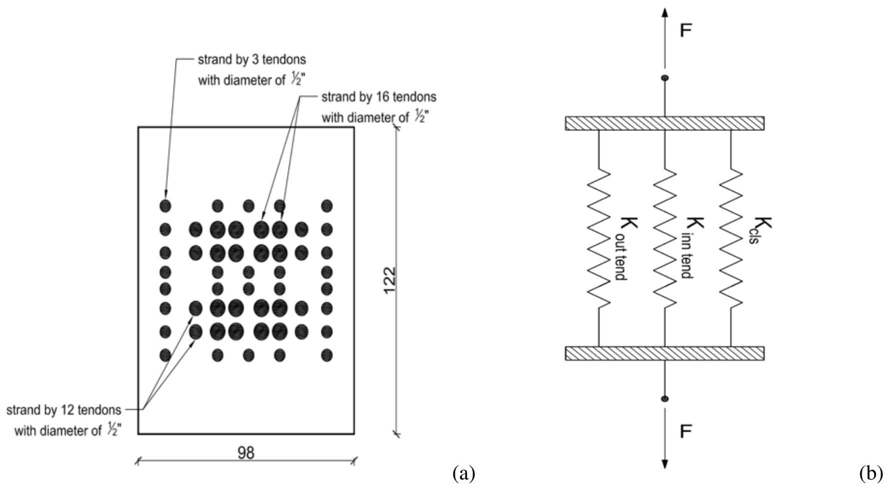

Figure 3: Cross-reacting section of the stay cable (a); simplified structural scheme used to evaluate the redistribution of the normal stresses among the three components of the stay cable (b).

low corrosion level approximately equal to $7-8\%$.

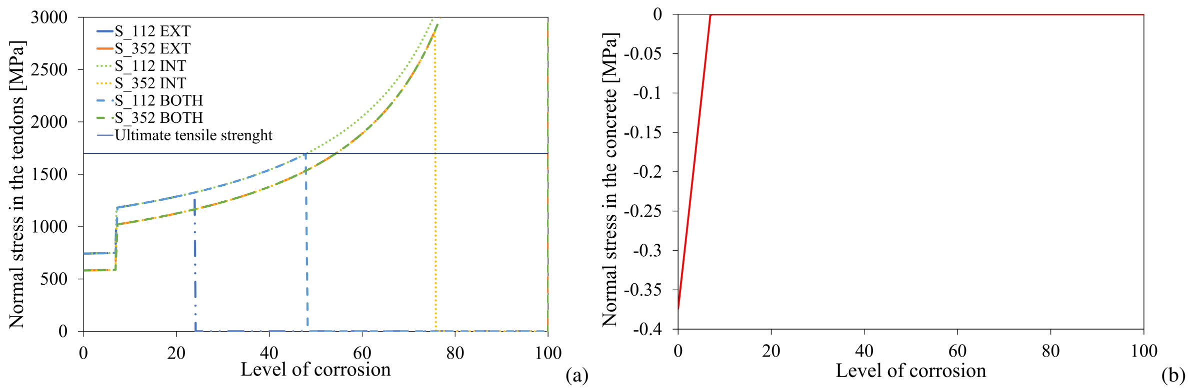

Figure 4: Stress redistribution due to different corrosion scenarios in the steel tendons (a), and in concrete (b).

Morandi [8], in the late '70s, was already quite worried about the unforeseen rapid degradation of the bridge. Therefore, it is reasonable to assume that the cross sections close to the antenna could have been decompressed during the last forty years. As a result, it is possible to conjecture that the very-high cycle fatigue phenomenon has concerned only the strands of the cable-stay.

Eventually, it is very important to check the decrease in the cable-stay axial stiffness due to the decompression of concrete in the region close to the antenna. In fact, if decompression had caused sensible decrease in the cable-stay stiffness, anomalous deflections and localised damage of the main bridge deck would have been detectable. Once a certain level of corrosion is adopted, not only the redistribution of stresses among the three reacting components can be calculated as shown before, but also the decrease in the local cable-stay stiffness as a function of the curvilinear coordinate. In particular, a sensible change takes place when the concrete covering is decompressed. The calculation can be performed iteratively by uniformly increasing the level of corrosion and evaluating for each scenario the length of decompressed concrete covering. After that, the elongation of the cable-stay due to the decrement in the axial stiffness can be obtained by integration along the axis. It is worth noting that uniform corrosion is assumed for the sake of simplicity, but it must be clear that the provided deflection is an overestimation of what is obtained in case of localised, and more realistic, corrosion. The results, summarised in Tab. 2, show that a sensible deflection of the cable-stay can be obtained only for uniform levels of corrosion above 30% circa.

The results reported in Tab. 2 allow us also to assess some of the hypotheses adopted to formulate the simplified model. Since the self-standing system is statically indeterminate, the stiffness decrement of the cable due to corrosion could have implied some sensible stress redistribution towards the bridge deck. As far as this is concerned, it is possible to consider a structural scheme for the whole bridge deck and to apply the settlement of 159 mm in correspondence of the cable stay connection (corresponding to Point A in Fig. 2a). The calculation provides that the vertical reaction force and the axial force transmitted to the stay-cable are decreased less than 8%. This variation is negligible and the proposed simplified model appears to be effective to our purposes.

Level of corrosion $[\%]$	Decompressed length of the stay cable $[\mathrm{m}]$	Variation of the displacement of the deck due to the corrosion $[\mathrm{mm}]$
0.00	0.000	0.0
4.31	0.000	2.6
7.11	0.400	5.7
8.19	2.199	13.5
9.27	4.049	21.7
12.93	10.296	51.2
18.32	19.593	100.7
23.71	29.140	159.4
29.09	38.836	229.1
35.00	49.682	321.7

Table 2: Evaluation of the variation of the vertical deflection of the cable-stay due to the corrosion.

4 FATIGUE ASSESSMENT OF CABLE-STAY

4.1 Evaluation of the fatigue load spectrum

The fatigue load spectrum is estimated, as detailed in [5], based on the FLM4 fatigue model [4] and on heavy lorries traffic information reported by Autostrade per l'Italia [9]. The amplitude of the load cycle related to the passage of each lorry of a certain category can be obtained analysing the influence line of the bridge [5], resulting equal to about 1.3 times the weight of the truck. Finally, assuming that a heavy lorry is traveling along the bridge only in the slow lane, and considering that the load cycle is entirely supported by the nearby cable stay, the load spectrum is obtained.

4.2 Fatigue damage accumulation

Since the stress range is not constant, a rule for the damage accumulation must be adopted [1, 10]. For the sake of simplicity, we assumed Palmgren-Miner's rule [11]. Being n_i the absolute frequency of each applied stress range $\Delta\sigma_i$, and N_i the corresponding number of cycles to failure for constant stress range, the fatigue failure occurs when the accumulated damage reaches the unity. In addition, corrosion causes the decrement in both the resisting cross-section, and the modification of the Wöhler's curve [3], which is translated downwards and loses the horizontal asymptote [12]. Tab. 3 summarises the damage contribution provided by the different lorry categories in correspondence to the critical corrosion level approximately equal to 22%. Finally, note that analogous modification of Wöhler's curve must be considered if the influence of structural size is additionally considered [13].

The obtained critical corrosion percentage is very close to the values detected during a survey carried out in 2015 [14] as well as with the analyses of the bridge's stay cable ruins [15]. On the contrary, the estimated critical level of corrosion corresponding to static failure is almost three times higher [16].

If the pre-stressed state in the concrete covering would have been preserved, thanks to a more effective maintenance and retrofitting program, the fatigue strength of the system would have benefit for different aspects. First of all, the main fraction of the fatigue loadings would have been withstood by the concrete covering, in place of being redistributed to the steel tendons alone. Secondly, the preservation of the steel tendons cross section would have resulted in lower stress amplitude cycles. Finally, if the covering system would have been as effective as conceived by Morandi, even the effect of modification of the Wöhler's curve due to the aggressive environment, would have been largely avoided. In fact, the estimated fatigue damage accumulation in absence of aggressive environment, and consequent corrosion evolution, would have never approached unity, as estimated in [5].

Stress range [MPa]	Applied number of cycles	Number of cycles to failure	Partial accumulated fatigue damage
19.5	1.582×10^7	1.838×10^9	0.009
30.2	3.954×10^6	3.185×10^8	0.013
47.7	3.954×10^7	5.102×10^7	0.775
37.9	1.186×10^7	1.271×10^8	0.093
43.8	7.908×10^6	7.173×10^7	0.110
	$\sum n_i = 7.908 \times 10^7$		$\sum D_i = 1.000$

Table 3: Application of Palmgren-Miner's rule according to fatigue load spectrum with level of corrosion equal to 22%.

5 CONCLUSIONS

The collapse of the Polcevera Bridge in Genoa can be ascribed to un-predicted interaction between very-high cycle fatigue and corrosion, based on the hypothesis of concrete decompression in the cable-stays. This crucial conjecture has been investigated in detail, by means of an analytical model for the cable stay that is able to account for detailed construction phases and the evolution of the internal stresses of the strands, due to relaxation and friction losses. In addition, the evolution of concrete decompression and cable-stay axial stiffness decrement have been obtained as a function of increasing corrosion levels.

In presence of aggressive environments, the modification of the Wöhler's curve compromises the common concept of fatigue limit, just as in the case of structural size effects. In fact, if very-high cycle fatigue is approached, even rather limited stress-range cycles can accumulate substantial damage.

The described phenomenon is worth for further investigation, not only as far as the Morandi's Bridge is concerned, but also for the large assets of existing structures and infrastructures that were built during the last Century.

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