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Case study

Collapse of prestressed reinforced concrete jetties: durability and faults analysis

S. Tattoni [1], F. Stochino *

Department of Civil and Environmental Engineering and Architecture, University of Cagliari, Via Marengo 2, 09123 Cagliari, Italy

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1. Introduction

The durability of reinforced concrete in marine structures has always been an important topic. In quite a recent work [1], Song et al. investigate the factors affecting corrosions and the approaches for improving durability of marine structures. In this paper there are discussed environmental effects, construction quality, cover thickness, characteristics of concrete and structure type. In addition, the authors present the possible approaches to improve the durability of marine R.C. structures based on real cases and field surveys. An interesting experimental work [2] has been recently developed by Giordano et al. They investigate the effect of simultaneous corrosion and cyclic loading on reinforced concrete elements. The key role of the joint effect of those causes of degradations is highlighted in this work.

In the analysis of prestressed reinforced concrete, the number of the involved variables sharply rises along with the uncertainties of the factors affecting them. Biondini et al. [3] developed a fuzzy model to simulate the real values by means of bands bonded between suitable minimum and maximum extremes. Probabilistic methods are used also in [4], in which an optimization of R.C. cross-sections in aggressive environment is developed with a lifetime reliability approach. The authors demonstrate that the amount and arrangements of steel reinforcement and the value of the concrete cover thickness played a crucial role in the process.

One of the key parameters in lifetime assessment of marine R.C. structures is the chloride concentration. As regards its modeling, several papers have been written: in [5] Chatterji proposes severe criticism against the use of Flick's second law of

* Corresponding author. Tel.: +39 070 675 5410; fax: +39 070 675 5418.
E-mail addresses: stattoni@unica.it (S. Tattoni), fstochino@unica.it, fstochino@gmail.com (F. Stochino).
[1] Tel.: +39 070 675 5409; fax: +39 070 675 5418.

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diffusion regarding chloride ion migration through cement based materials. He developed this model only on an empirical basis, and it cannot be accepted from a theoretical point of view. Nevertheless, this popular model is still used nowadays [6–8]. Furthermore, the chloride deposition rate has been considered a benchmark for the environmental conditions. In [9], Meira et al. study the deposition of chlorides on wet candle devices, and its relation with chlorides accumulated in concrete. The authors claim to use the chloride deposition rate as an environmental indicator, in order to predict service life of constructions and concrete cover thickness for a required service life. As regards the durability of reinforced concrete structures, the effects of carbonation and chloride penetration play a key role in it. In a recent work [10], Bertolini investigates this issue and proposes possible approaches to the design of durable reinforced concrete structures.

2. Background

The prestressed concrete jetties analyzed in this paper are a part of a picturesque, fine marina in Italy. The structure was built in 1974 and has a capacity of 1500 berths. It can accommodate any boat up to 50 m length.

The jetties consist of precast decks and beams supported by R.C. piles. Referring to Fig. 1, the decks have a π-shaped cross-section of prestressed reinforced concrete. It is hinged on one side and simply supported on the other. The geometric characteristics of the cross-section are: 3.5 m width, 0.9 m height. The longitudinal span is 10 m.

Fig. 1 shows also the reinforcements' distribution. In the deck slab, the reinforcement is a double mesh with a step size of 150 mm and there are also bars of 4 mm diameter. The authors found out in the calculation report that there are also rebars of 8 mm diameter.

As regards the two ribs, the only ordinary reinforcements are those in the top part of the section: two 12 mm diameter bars that, indeed, are stirrups support bars. Actually, the tensile reinforcements of this element are the 4 × 0.6 in. prestressed strands in the bottom part of the cross-section. In other words, the stability of the whole element depends on the pre-tensioning.

It is important to highlight the desolidarization of some parts of the prestressing tendons. This technique, frequently used in this type of construction, has the function of modulating the prestressing force along the longitudinal axes, in order to serve the actual structural needs. In this case, desolidarization is obtained by inserting parts of the cable in a PVC corrugated duct. In this way the conglomerate, not being able to come in contact with steel, does not suffer the effect of prestressing. The ratio between the resistant bending moment and the design bending moment M_R/M_E must have been greater than

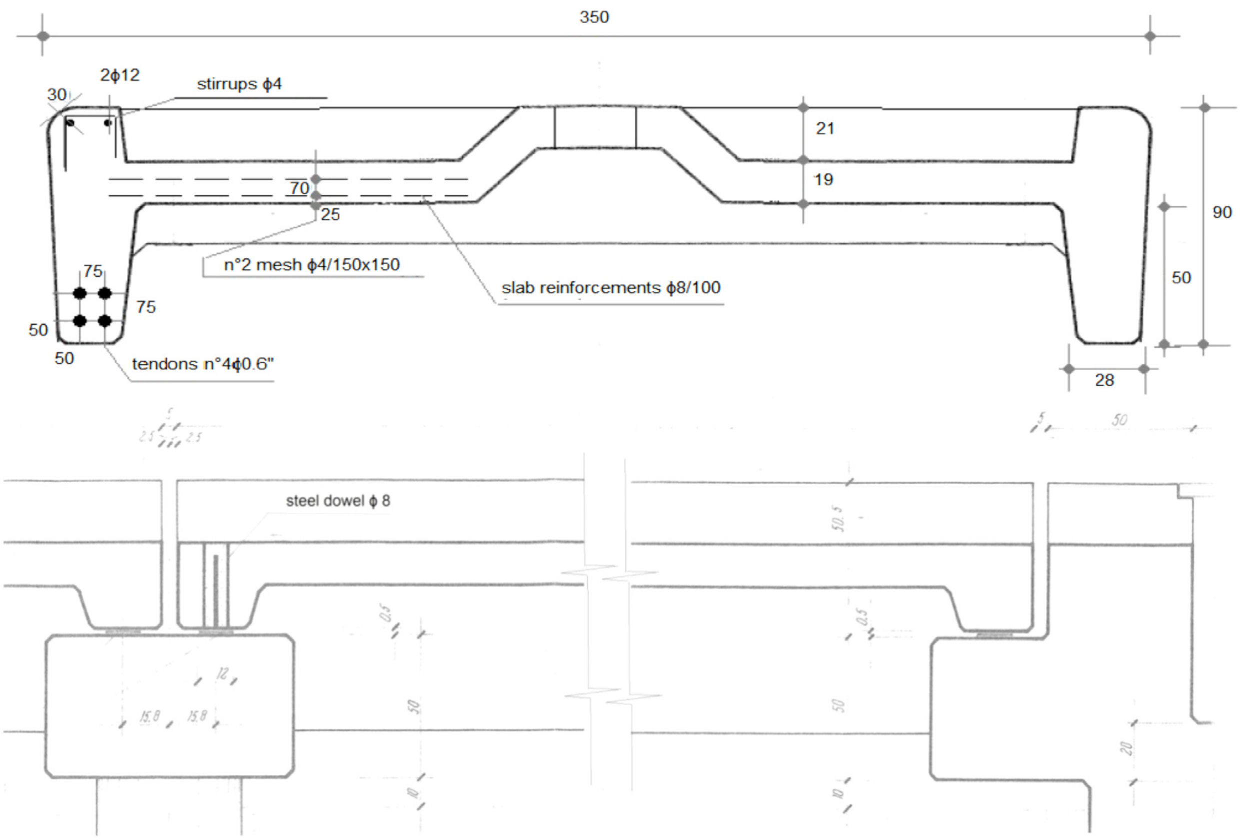


Fig. 1. Cross (top) and longitudinal (bottom) sections of the jetty.

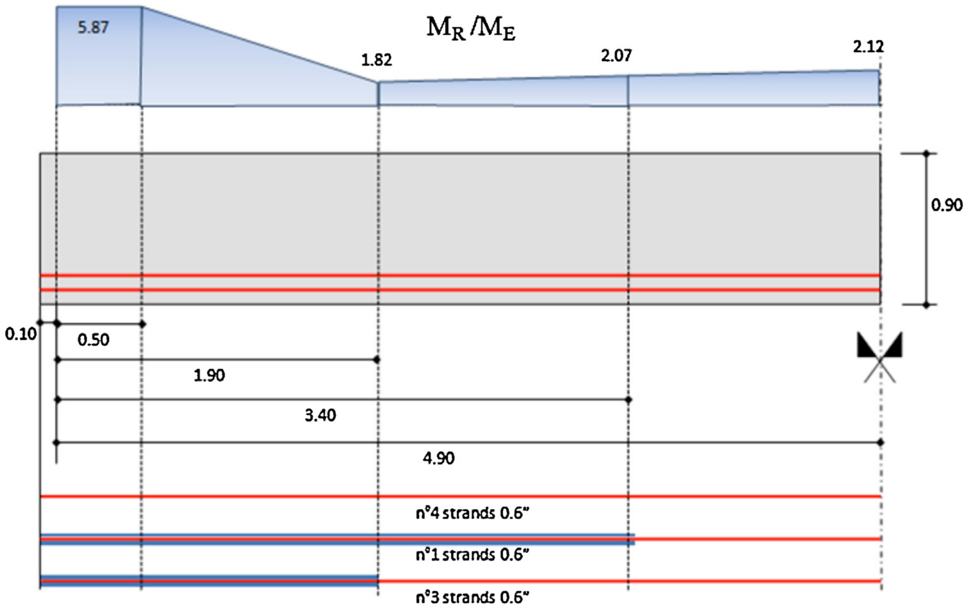


Fig. 2. Ratio between resistant bending moment and design bending moment along the longitudinal axes.

1.3, according to the technical standards of that time [11] (see Fig. 2). It is evident from an examination of Fig. 2, that this ratio is always observed, but with different values. The points of least resistance (minimum values of the ratio M_R/M_E) are 2 m far from the support and not in the midspan section.

From the original design report, the original experimental test and inspections in situ, it is possible to deduce the following mechanical characteristics. The concrete proved to be a very good material with a characteristic strength of at least 30 MPa. The ultimate characteristic strength of steel tendon is 1800 MPa while its strength at 0.1% strain is 1540 MPa. The concrete cover varies from 20 mm for the deck slab to 50 mm for the protection of the strand.

Several cracks have appeared along different jetties in the marina starting from the nineties. The collapse, as shown in Fig. 3, occurred in autumn 2011.

After the collapse, the damaged jetty was recovered and this was the starting point of the post-failure analysis. It is clear (see Fig. 3(b) and (c)) that this is a durability problem, or more precisely, a corrosion problem. The aggressive marine environment and some execution flaw caused the failure.

3. Durability analysis

The first analysis conducted by the authors regards the depassivation limit state for carbonation induced corrosion. Fig. 4 shows a sketched carbonation parabolic model (see Eq. (1)) adopted by the Model Code 2010 [6]. It is visible that after 25–30 years the carbonation depth is greater than the concrete cover of 20 mm.

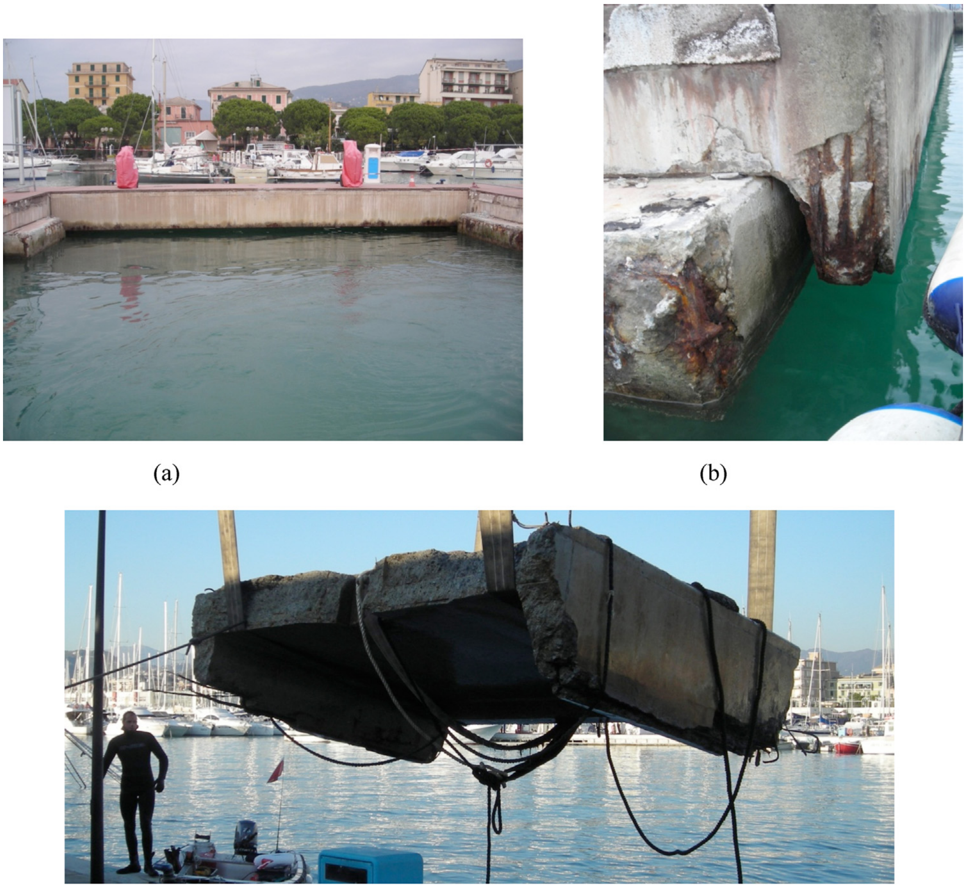
$$x_c(t) = \left(\frac{t_0}{t}\right)^w \cdot k \cdot \sqrt{t} \tag{1}$$

In Eq. (1) (taken from [6, Eq. (7.8.2)]), x_c represents the carbonation depth, k is a factor reflecting aspects like the execution and basic resistance of the concrete against ingress of carbonation; w is the weather exposure ($0 < w < 1$, 0 for indoor conditions and 1 for wet conditions); t_0 is the time of reference in years. Some useful indications to evaluate the values of t_0 and k can be found in [12, Annex B]. Regarding the graphs shown in Fig. 4, the authors assume $k = 7.1102$, $t_0 = 0.0767$, $w = 0.1062$.

As mentioned in Section 1 an important issue is the chloride attack (see Fig. 5 and Eq. (2)). The authors studied the depassivation limit states for chloride induced corrosion. As stated by Model Code 2010 [6], the ingress of chlorides in a marine environment may be modeled by the modified Fick's second law of diffusion.

$$C = (C_{\Delta x} - (C_{\Delta x} - C_i)) \cdot \left[\mathrm{erf}\left(\frac{x}{2 \cdot \sqrt{D_{\mathrm{app}}(t) \cdot t}}\right)\right] \tag{2}$$

In Eq. (2) (taken from [6, Eq. (7.8.11)], C represents the content of chlorides in the concrete at a depth x, $C_{\Delta x}$ is the chloride concentration at concrete surface [wt.% binder content], C_i is the initial chloride content of the concrete [wt.% binder content]



(a) (b)

(c)

Fig. 3. Empty space left by the collapsed jetty (a), degradation of concrete (b), the recovery of the collapsed jetty (c).

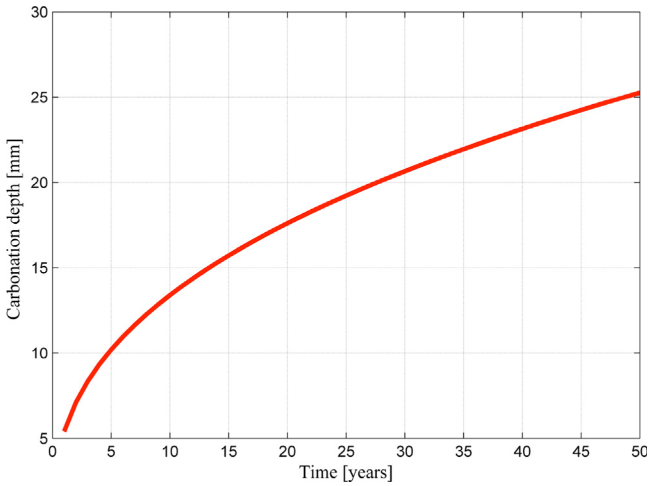
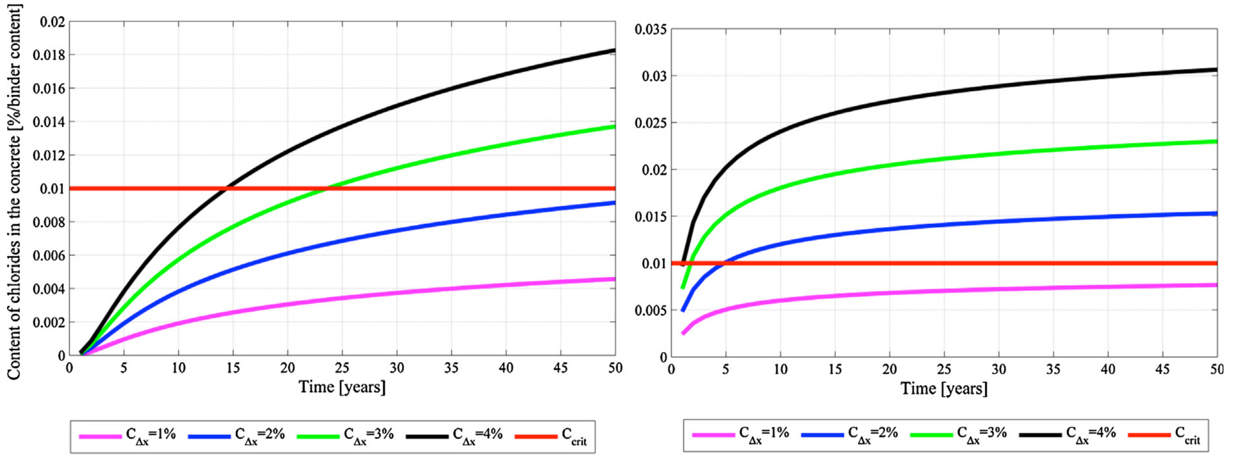


Fig. 4. Depassivation limit state for carbonation corrosion according to Model Code 2010 [6].

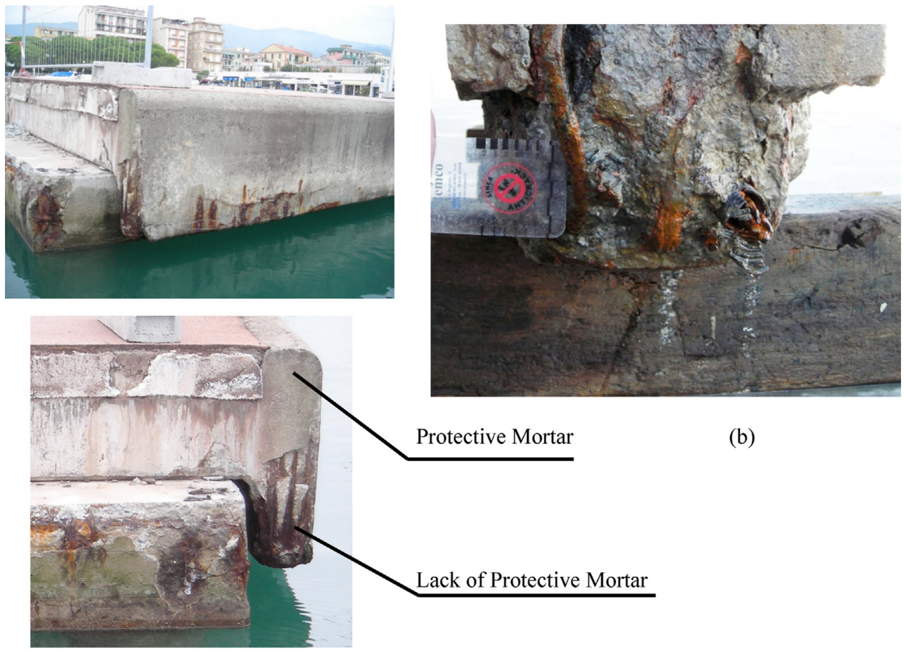
(in Fig. 5 it is assumed equal to 0), $D_{app}(t)$ is the apparent coefficient of chloride diffusion through concrete [m/s^2] at time t, its expression can be found in [6, Eqs. (7.8)–(12)].

According to this approach, Fig. 5 represents the content of chlorides in percentage of binder content. Two concrete cover depths with different $C_{\Delta x}$ chloride concentration at the concrete surface are being considered here. In case of 50 mm thick concrete cover (Fig. 5(a)), the critic chloride concentration of 0.01% can be reached for a concentration of chloride at the



(a) (b)

Fig. 5. Depassivation limit state for chloride induced corrosion with 50 mm thick concrete cover (a) and 20 mm thick concrete cover (b), according to Model Code 2010 [6].



(a) (b)

Fig. 6. Spalling of the protective mortar (a) exposes the prestressed cable ducts to weathering. In (b) the sea water comes out from the duct of the recovered collapsed jetty.

concrete surface of 3% after 23 years or in less time, if $C_{\Delta x}$ is greater than 3%. The situation changes in case of the top part of the cross-section where the concrete cover is 20 mm thick (see Fig. 5(b)). In this case, the critic chloride content is reached in different scenarios after a few years. Taking into account that the real concrete cover may vary at different positions of the jetties, it is likely that a chloride induced corrosion could have occurred.

4. Root cause of the failure

To sum up the situation: reinforced concrete structures built in 1974 showed first cracks after almost 10–15 years, and some collapse occurred after 20–25 years. Probably, corrosion (due to carbonation or to chloride concentration) has reduced reinforcement resistant cross-section.

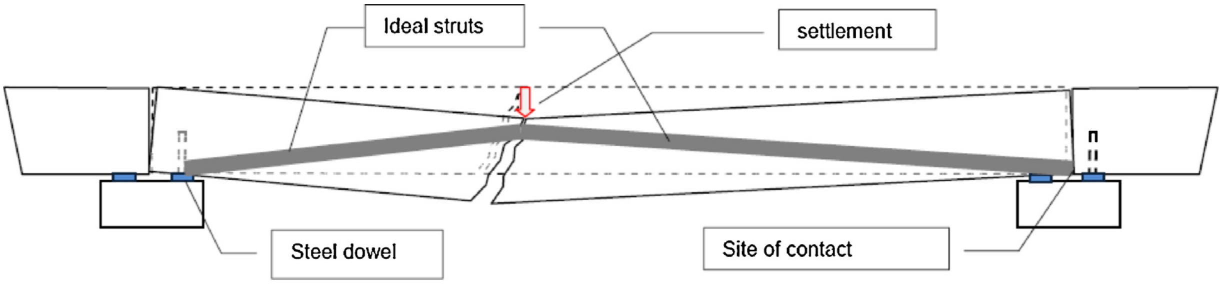


Fig. 7. Mechanism of collapse.

Furthermore, some minor details and the lack of preventive measures may have exacerbated the situation. Fig. 6(b) shows the condition of the collapsed jetties after its recovery: the terminal section of the duct did not have an adequate protection. Actually only a thin cover of mortar (almost 10 mm) protected the head of the ribs in almost all jetties (see Fig. 6(a)).

The spalling of the protective mortar due to the corrosion of the stirrups exposed the prestressing cable ducts to weathering. Corrosion of the tendons could have started immediately. In this case the durability of a prestressed concrete structure is jeopardized by the lack of an adequate external protection of the strands.

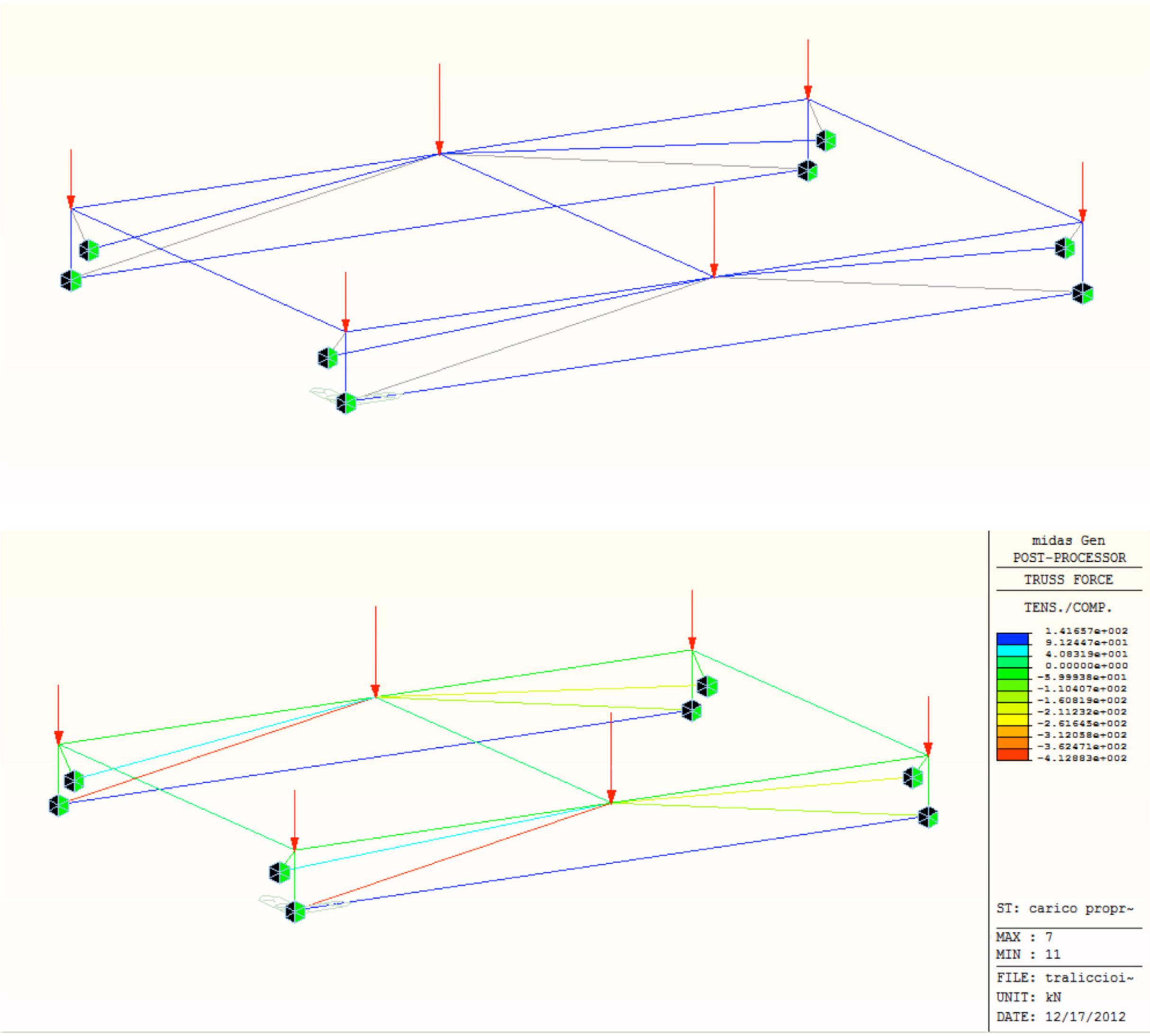


Fig. 8. Strut and tie model of the jetties before the tendon collapse.

5. Description of the mechanism of the failure

It is interesting to outline what probably happened:

(1) The ordinary reinforcement in the terminal cross-sections, characterized by moderate concrete cover (in theory 30 mm, in reality even less than 20 mm) was affected by the first corrosive processes. For this reason, the formation of iron oxide hydrates ($Fe_2O_3{\cdot}nH_2O$) and the development of cracks begun.
(2) The sea water could have penetrated the structure, especially within the desolidarization duct. The situation was aggravated from the cycle of immersion and emersion due to the wave motion, which allowed the continuous supply of Cl^-, O_2 and H_2O.
(3) The corrosion started to affect the prestressing reinforcement. Then the expansion of the iron hydroxides caused occurrence of cracks parallel to the longitudinal axes of the beam. The reduction of the cross-section of the reinforcement tendons continued until they were gradually broken up.
(4) The collapse of the jetties took place in the section of least resistance, at about 1/4 of the span length distance from the support.
(5) Thanks to the non-contemporeaneous failure of the two ribs, the jetty could have resisted by means of a strut and tie mechanism (see Fig. 7). It is obviously neither reliable nor permanent. The jetty remained in place with a settlement. The latter is clearly visible in other still resisting jetties where presumably a rib is close to collapse, while the adjacent one is still resistant.

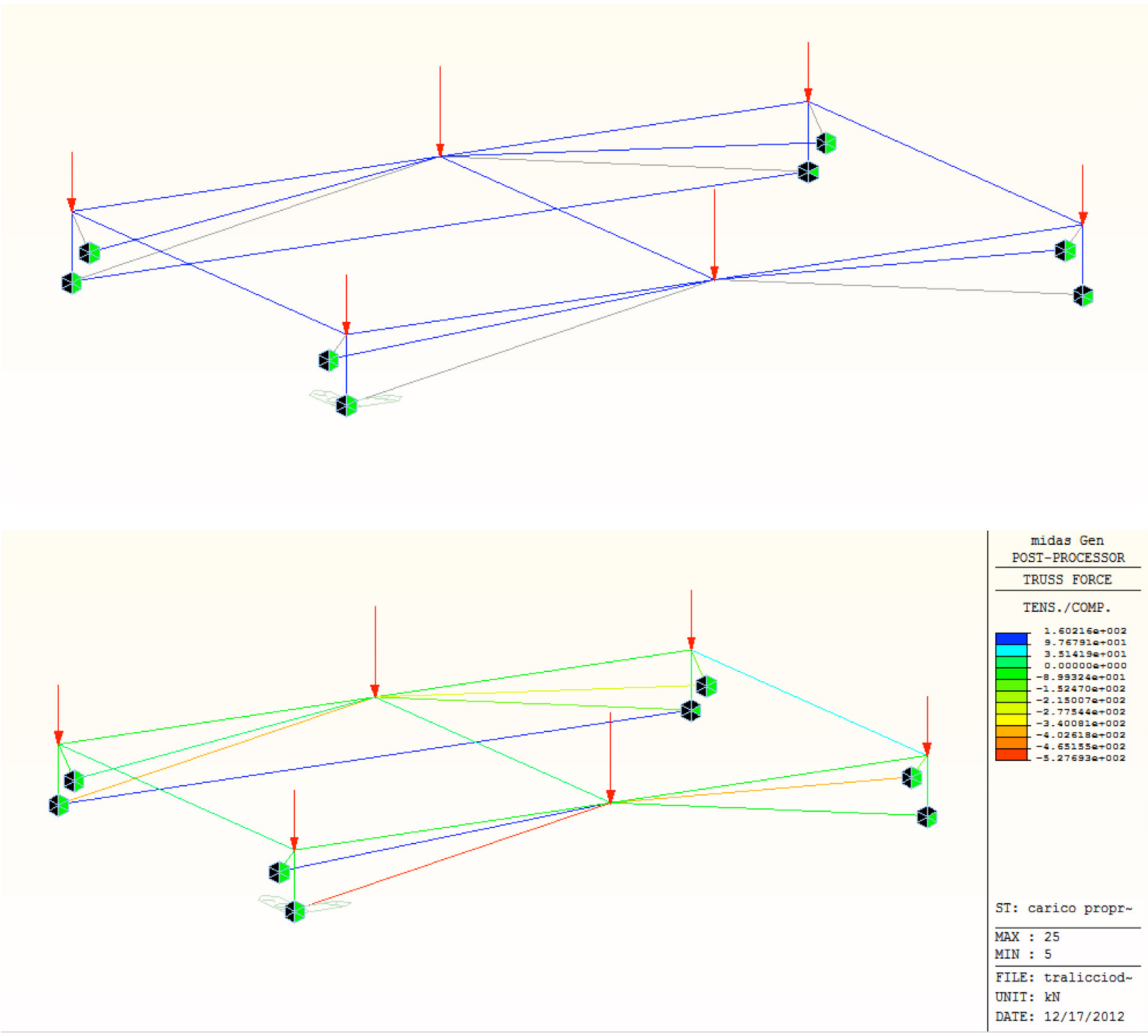


Fig. 9. Strut and tie model of the jetties after the tendon collapse.

(6) The failure of the steel dowel (see Fig. 7) or of parts of the jetty in contact with the adjacent element, or other causes (e.g. shock, high shots mooring, and piling-induced movements of the waves) suddenly made impossible the balance of the element, which fell down into water.

Fig. 8 shows the strut and tie model that can represent a resistant mechanism of the jetty in ordinary condition. In particular, the ribs on the sides of the element can be noted. There are prestressing tendons on the bottom part and sloping concrete strut on the top part.

Constraint system simulates the actual supporting condition: hinged on the left and simply supported on the right. An elastic linear structural analysis was performed. It is important to consider the value of the force acting on the ties: 142 kN plus the prestressing force (see Fig. 8). This result shows the paramount relevance of the prestressing strands. During the corrosion process the tendons of one rib were being gradually broken.

Fig. 9 presents the strut and tie model of the jetty after the collapse of the tendon of one rib. The result of the structural analysis highlights that the force acting on the last prestressing tendon is equal to 160 kN. Therefore, in this damaged system, an increase of the 11% of the stress on the prestressing strand is expected. It seems not such a big rise, but the corrosion of the other strand has reduced the resistant cross-section of the last reinforcements. Thus, this system is neither reliable nor permanent and the total collapse is pending.

6. Conclusions and recommendations

In conclusion, this is an exemplar case study regarding R.C. corrosion in marine environment. From the first analysis it seems that the depassivation limit state could have been reached since the first decades. The Italian standards of the time of the construction did not pay much attention to durability problems, so the original designer arranged the preventive measures on the basis of his experience (for example: the use of a very good concrete, concrete cover sufficient for almost all cross-sections, even if in reality this formula was not observed at all). Unfortunately, these measures were not sufficient. Apart from the replacement of all the remaining jetties, it seems that the solution for the refurbishment is not connected to the residual prestressing (in any case destined to vanish in time), and it consists in building supports or reinforcements external to the elements. Further developments of this research are expected with regard to the corrosion model, in order to better define when the depassivation limit state occurred.

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