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Assessment and strengthening of reinforced concrete bridges with half-joint deterioration

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Abstract

Existing Niagara-type concrete cantilever bridges, which in Italy are called Gerber bridges, suffer from degradation due to reinforcement corrosion and a consequent reduced load-bearing capacity. The assessment of these structures is therefore an important part of the procedures for analyzing the state of existing bridges and any interventions required for their retrofitting. In particular, safety against failure of the half-joint must be evaluated with respect to the behavior of the entire bridge and its state of conservation, determining what the conditions are that lead to failure, in terms of loads, state of stress, and possibly cracking state. This evaluation has to be carried out considering the configuration of the bridge through local and global models, in the presence of the crossbeam between the main girders or the prestressing tendons that can modify the behavior of the Gerber saddle, compared to that expected for a simple dapped-end beam. To do this, it is necessary to combine the in situ investigations with reliable analytical and numerical models, which can correctly interpret the structural behavior of the bridge, determining the critical aspects for safety. The approach followed in this study was to combine the strut-and-tie models with nonlinear finite element models that can provide reliable information to the engineer who deals with the assessment and subsequently the conception of rehabilitation, in combination with in situ investigations and material tests. This was carried out through two different case studies in which the shape of the Gerber saddle, the presence of the crossbeam, and the presence of prestressing characterize the behavior and highlight the degree of safety and that of structural robustness. The results of the analyses carried out show that the presence of prestressing and a careful design with the use of large amounts of reinforcement can lead in some cases to high safety factors and consequently to a good level of robustness. They also show that the structural behavior can vary significantly according to the level of reinforcement corrosion: this is an aspect that can only be correctly assessed with the use of different models with different degrees of refinement. Finally, a proposal for

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global structural rehabilitation is presented by closing saddles, changing the static scheme to the continuous beam, and supplying external prestressing to strengthen the deck.

KEYWORDS

cantilever bridge, corrosion, Gerber saddle, half-joint, prestressing, reinforced concrete, structural assessment

1 | INTRODUCTION

In recent years, many concrete bridges on Italian roads that have a typical cantilever static scheme (also called Gerber-girder bridge) with half-joints have shown considerable deterioration problems due to reinforcement corrosion and concrete degradation. In some cases, the collapse of the Gerber bridge was recorded, due to maximum moving loads which brought the stress of the damaged Gerber saddles to the Ultimate Limit State. Similar problems were encountered in other nations.[1] This led the Italian Ministry of Infrastructures to include Gerber saddles among the most dangerous elements in the Guidelines for Assessment of bridges,[2] assigning the maximum level of attention to degraded saddles, regardless of the intensity and extent of the degradation. For this reason, bridges with the Gerber scheme and therefore the areas of the half-joints are the object of special attention during inspections of bridges for subsequent safety assessment and retrofit.

Gerber-girder bridges were popular in Europe around the middle of the 20th century.[3] Many of them were made of ordinary reinforced concrete without prestressing, but afterwards, from the 1960s to the 1980s, they were built with prestressed beams. Nowadays, several Gerber bridges have become obsolete due to concrete degradation leading to insufficient load-carrying capacity.

The problem, however, does not concern analysis of the classic dapped-end beams, which is the most common structural model studied in the literature,[4,5] but that of real Gerber saddles which, due to their geometry, the presence of crossbeams and/or prestressing tendons, significantly modify their behavior up to failure.

Reliable methods of assessment of these structures are of fundamental importance for the design of structural rehabilitation interventions, which can include both local strengthening of the single elements and global retrofit of the deck or even a change to its static scheme.

In the literature, there are several half-joint studies that also present quite vast and diversified experimental campaigns,[6–8] but they mainly concern classical geometries and configurations of Gerber saddles that move away from the real configurations of bridge girders. Some studies on real cases have instead indicated that often the real behavior of the bridge does not highlight the local failure of the element and therefore a criticality of the saddle itself, but rather shear/bending behavior of the main beams which shift the critical section from the saddle to the current section thanks to prestressing axial stress, the presence of the crossbeam or simply to overabundant amount of reinforcements.

However, this behavior depends on the state of degradation of the structural elements and above all on the corrosion of the reinforcements. Hence, a reliable evaluation has to be carried out through analytical and numerical models that also take deterioration into account.[9,10]

In this paper, evaluations of different cases of deteriorated concrete bridges are presented in order to show the specific properties and configurations of some Gerber saddles and the speedy assessment methods for existing half-joints for these bridges, taking into account the actual geometry, the presence of prestressing, and the consequences of deterioration due to concrete damage and reinforcement corrosion. It will be shown that in some cases saddles show unexpected intrinsic robustness despite the advanced state of decay because the designer introduced a larger amount of reinforcement than what was strictly necessary.

Following evaluation of the residual strength in the damaged state, it is necessary in many cases to proceed with rehabilitation of the Gerber bridge or strengthening of the half-joint. The usual retrofit methods generally provide for local strengthening of the saddles, by means of steel plating, insertion of vertical, and horizontal bars or high-performance concrete jacketing, and the use of carbon fiber fabrics.[11–15] From a global point of view, however, the overall scheme of the cantilever bridge does not allow for strength resources related to structural redundancy as is the case for continuous girders; hence, the problem of Gerber bridge retrofitting is treated here through a comprehensive rehabilitation strategy based on a change to the bridge static scheme.[16] This improves the overall robustness, with the closure of the saddles and the possible use of external prestressing.

2 | TYPOLOGIES OF GERBER-GIRDER BRIDGES AND LITERATURE STUDIES

Gerber-girder bridges are named for their inventor, Heinrich Gottfried Gerber, a German engineer and a professor at Münich. It has hinges at inflection points to reduce bending moments, taking advantage of continuity but maintaining the structure isostatic. This kind of girder was developed in response to failures caused by unequal foundation settlements in 19th-century railway bridges. In this connection, due to the lack of knowledge of geotechnics, settlement of piers and/or abutments, even by a few centimeters, was very common, inducing additional stresses in redundant structures. The Gerber girder scheme found great favor in bridge construction as it made it possible to combine the advantages of the continuous beam with those of isostatic structures. In Anglo-Saxon countries, such bridges are known as cantilever girder bridges, like the first large-span steel truss bridges done in the late 19th century. The concrete version of these bridges was built for short spans after the advent of reinforced concrete.

The hinges of the static scheme consist of Gerber saddles, where the classical theory of the Bernoulli–Navier beam cannot be applied. Hence, reinforcements designed in the past could be insufficient or ineffective, and later strut-and-tie models were found to be more appropriate for these structural elements sited in so-called D-regions.[17] Furthermore, the joint and open cracks allow water to enter more easily and accelerate progression of reinforcement corrosion and overall damage.

In Figure 1, a typical drawing of an old reinforcement arrangement on a Gerber saddle is shown.

In bridges, however, the half-joints, have a considerable variability of configurations, depending on the typology of the deck and the sensitivity of the designer. In particular, the configuration of the Gerber saddles is different for grid decks composed of main girders and transverse beams, for double-beam decks, for box girders, and for slab bridges and also depends on the presence or absence of prestressing tendons in the beam webs.

In existing bridges, it is possible to have half-joints of different types:

- single saddles on each beam in the absence of stiffening crossbeams, with or without prestressing (Figure 2a);
- saddles on reinforced concrete beams with or without prestressing connected by transverse beams present only on the corbels of the saddles (Figure 2b);
- saddles which involve the entire very rigid transverse beam and in which the main longitudinal beams fit into the crossbeam, whether they are prestressed or not (Figure 2c);
- saddles on box girders that involve the webs but in the presence of internal diaphragms (Figure 2d);
- transversely continuous saddles in slab bridges (Figure 2e).

For each case, the presence of prestressing is generally beneficial, also because the internal tendons have the anchor head right in line with the saddles, entering those areas with a favorable inclination for shear behavior, through the vertical component of inclined prestressing that is in the opposite direction to that of the external load (Figure 3). Moreover, the prestressing provides a further benefit through the presence of the axial force.

The configuration of the crossbeam is also very important because it can allow or prevent redistribution of the reactions between the saddles in the critical phase near failure, although often its presence only in line with the corbels makes the attachment section of the main beam on the transverse beam critical, especially in the web area. However, it should be considered that many reinforced concrete bridges also have a lower slab in the cantilever sections which guarantees compressive strength in the areas of negative bending moment, and

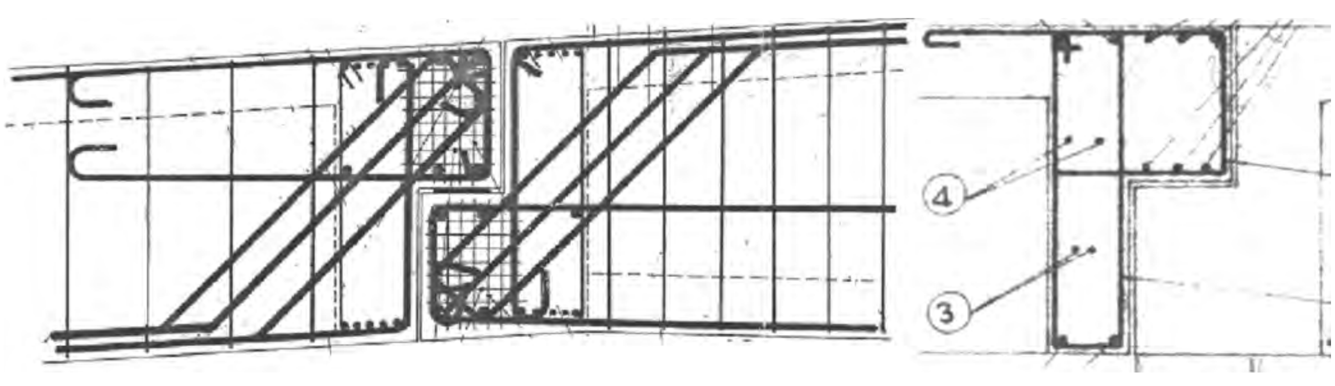


FIGURE 1 Gerber saddle and transverse beams (original drawings of typical concrete bridge girders in the 1960s)

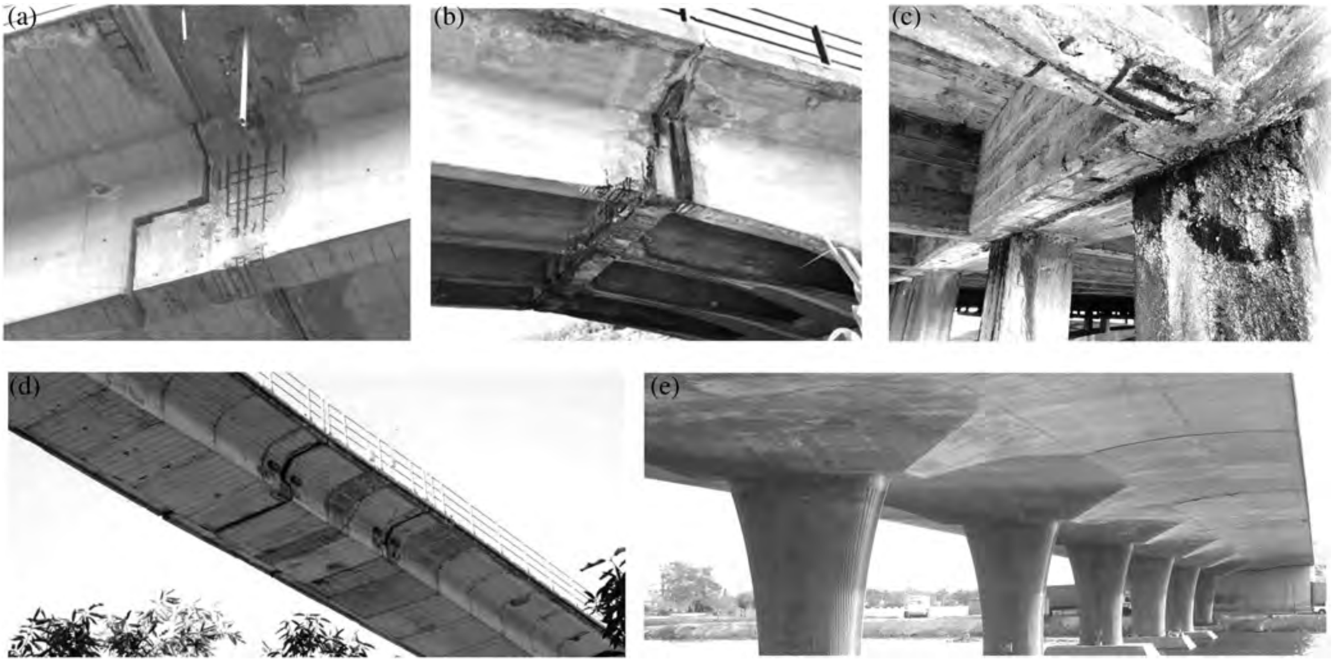


FIGURE 2 Configurations of Gerber saddles on bridges

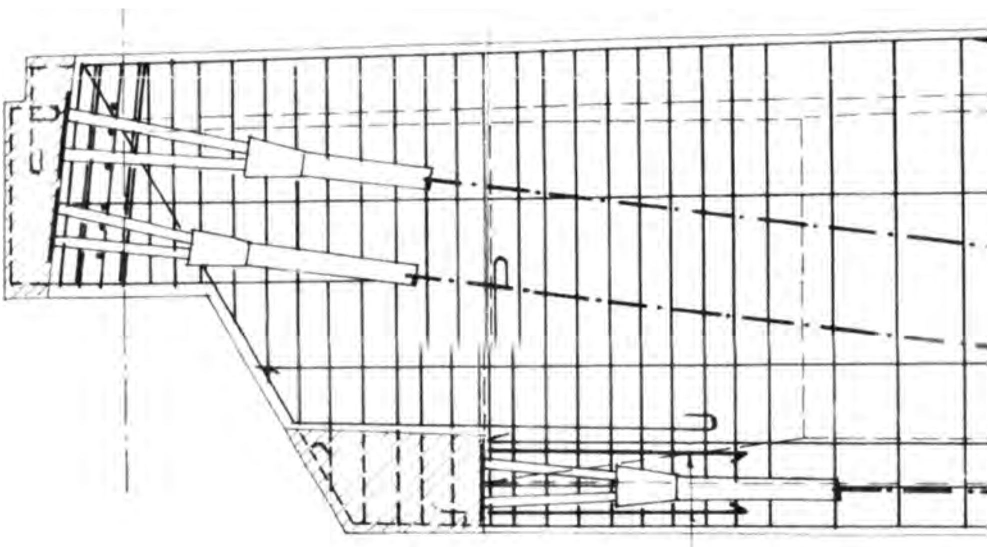


FIGURE 3 Saddle with prestressing tendons

this further increases the strength of the saddle in the section where the main girder attaches to the crossbeam.

As can be seen from Figure 2, many of these bridges show signs of advanced concrete degradation and corrosion of reinforcements. The deterioration of concrete Gerber bridges affects a significant number of bridges on main and secondary roads, with the different configurations presented above.

This is generally caused by percolation of water into the joint from the deck surface, which over time causes spalling of the concrete cover, corrosion of the reinforcement, and overall reduction of the bearing capacity. In prestressed bridges, an important effect of these phenomena can be an advanced state of corrosion of the tendon anchors, which has the consequence of cross-section reduction of wires and strands. There is also a more general loss of prestressing, that is a reduction of the axial force and then a reduction of the overall beneficial effect of prestressing on the saddle and on the entire girder. Under these conditions, diagonal cracking of the saddle between the corbel and the whole current cross-section of the beam can appear and it essentially depends on the mild reinforcements and their configuration. In this case, therefore, inspection of the existing saddles can allow the engineer to make an evaluation of the type of corrosion (uniformly diffused or from pitting) and of the cross-section loss of reinforcements. However, it should be considered that an accurate inspection of Gerber saddles is often not possible due to the limited spaces that many bridges present in service and the consequent difficulty of access, so evaluation of the innermost reinforcements is extremely difficult and in most cases, it is only possible to estimate the reduction of the reinforcement section and to evaluate the state of degradation of the concrete, its on-site strength, the overall behavior of the half-joint and the presence or absence of cracking due to overstress states in the damaged configuration. In this case, safety assessment of the bridge can be carried out referring to global and local models of the deck, although the first approach is generally to adopt strut-and-tie models to know the saddle's strength in the ultimate conditions. These models, which were conceived for design, such as those recommended in Eurocode 2,[18,19] can be modified for specific cases of existing Gerber saddles with different configurations, and detailed local models with Finite Elements can also be used. Strut-and-tie models generally fail to hold the strength and redistribution potentials associated with the presence of transverse beams and prestressing. In many cases, therefore, for these bridges, a complete assessment implies the use of two-dimensional (2D) or three-dimensional (3D) nonlinear FE detailed models, which however show variability of the results depending on the modeling (nonlinear constitutive laws, cracking model of the concrete, mesh used, etc.) and require calculation tools that are not always available in engineering practice.

Many efforts have been made by researchers[6,7,17] to find models for evaluating the strength of half-joints, although the difficulty of simulating the transverse behavior of the deck has conditioned the experimental campaigns limiting the investigation to the behavior of classic Gerber saddles, composed of two dapped-end beams.

Through several experimental campaigns, the studies by Desnerck et al.[8–10] have provided knowledge of the failure behavior of half-joints for different reinforcement configurations in reinforced concrete saddles, also including the variable of corrosion and concrete degradation. Based on these studies, the recent English recommendations for checking existing half-joints[20] give useful indications on the behavior of Gerber saddles in old bridges, also taking degradation into account, even though they all refer to classical schemes of two dapped-end beams with a short half-joint.

In the literature, for similar cases, the most important influence of damage is attributed to concrete cracking due to corroded bars, since concrete strength reduction causes both a reduction of strut capacity in the strut-and-tie models, and a significant modification in tie capacity due to the loss of anchorage owing to reduction of bond strength. No significant variation of steel strength is instead suggested in the literature, considering only the reinforcement area loss as the main effect of corrosion.[10]

Considering the present state of the art, in the following sections, the safety assessment and retrofit hypotheses are applied and presented for two case-studies, different for typology and features.

3 | CASE-STUDIES OF HALF-JOINT CONFIGURATION AND DETERIORATION

In this section, two case studies are presented for the assessment of Gerber bridges. They were chosen among those addressed by the authors for their peculiar characteristics, representative of two possible configurations among those listed in the previous section. The first is a Gerber bridge designed by Riccardo Morandi, which presents a reinforced concrete saddle with very particular geometry, while the second case is a motorway overpass which presents a Gerber saddle with prestressing tendon anchorages and separate transverse beams behind the two corbels. Both cases show deterioration of concrete and corrosion of reinforcements to be taken into account in the safety assessment. The demand was evaluated according to the loads indicated by Eurocode 1.[21]

3.1 | Bridge over the Salso river

The first case is a bridge over the Salso river in Licata (Sicily, Southern Italy), designed by Riccardo Morandi. It is a three-span bridge with a Niagara-type cantilever girder scheme, with side spans of 33.10 m and a central span with a total length of 49.60 m, the central beam between Gerber saddle being 32 m long (Figure 4). The overall width of the deck is 19.00 m, while the total length is 115.80 m. A view of the bridge is shown in Figure 1.

In the two side spans and in the cantilevers, the reinforced concrete deck is composed of eight side-by-side asymmetrical T-type beams with a web thickness of 400 mm, height variable from 1.40 to 2.50 m and upper slab 200 mm thick. The girder has three transverse beams for each span and a bottom slab with thickness varying from 220 to 300 mm near the piers. The midspan is made up of eight prestressed reinforced concrete beams with variable height and double T section, completed by a reinforced concrete slab with a thickness of 200 mm and

FIGURE 4 View of the bridge

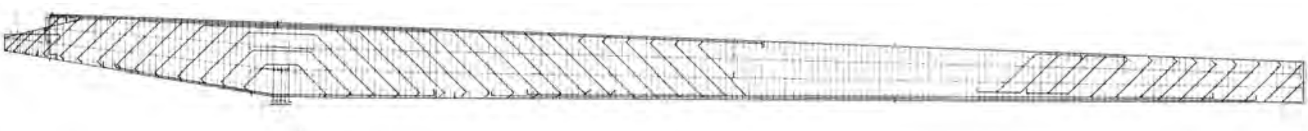


FIGURE 5 Arrangement of side span and cantilever reinforcements with Gerber saddle

FIGURE 6 Deterioration of the Gerber saddles

three transverse beams. The central beams are prestressed through Morandi's patent M5-type tendons. The materials are the following: concrete for side spans and cantilever with $f_{ck} = 25$ MPa; concrete for prestressed girder with $f_{ck} = 35$ MPa; reinforcement steel type ALETOR with yielding strength $f_{yk} = 440$ MPa; prestressing steel with tensile failure strength $f_{ptk} = 1700$ MPa.

Visual inspections, in-situ investigations, and material tests made it possible to develop a finite element structural model that would allow reliable evaluation of the current state of the bridge.[22]

Figure 5 shows the geometry of the cantilever and the arrangement of the reinforcement bars. Here, only the

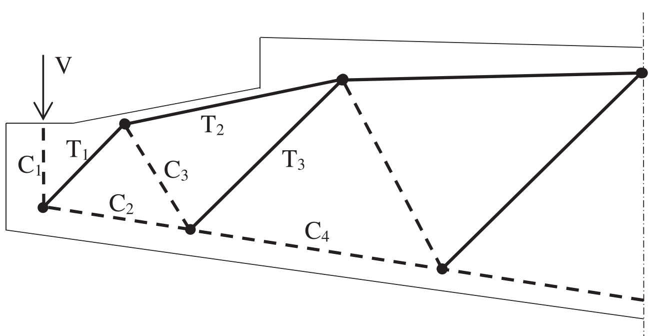


FIGURE 7 Strut-and-tie model of Gerber saddle specifically adopted in the case-study

lower corbel of the saddle is considered because it was judged the weak element, the upper corbel of the central beam being prestressed, with a greater capacity.

For safety assessment of the Gerber saddle, the provisions of Eurocode 2 were initially considered, using the strut-and-tie models provided in section 10.9.4.6 of EC2.[18,19] Since in the present case the problem is checking an existing saddle and not designing a new half-joint, the schemes suggested by the codes cannot be applied immediately. The particular geometry of the saddle designed by Morandi, which presents a soft connection between the beam cross-section and the corbel, instead of the classic dapped-end beam, leads to the need to conceive a specific strut-and-tie scheme for this case, taking into account the layout of reinforcements too.

In the original design, the saddle was well dimensioned and now it does not show a significant crack pattern with wide cracks, apart from a few superficial cracks due to concrete spalling (Figure 6).

The strut-and-tie model specifically conceived is shown in Figure 7 and it takes into account the actual arrangement of reinforcements and the related location of the struts.

Initially, the classical check at ULS, using partial safety coefficients of loads and materials provided by Eurocode, was carried out; the results showed that the weakest element is tie T_2 with capacity (in terms of maximum allowable reaction) $V_{Rd} = 1939$ kN and demand $V_{Ed} = 1189$ kN, the safety coefficient being 1.63. Considering that the application of partial safety coefficients at the ULS cannot give an effective measure of the global safety coefficient at saddle failure, the check was repeated taking all the partial safety coefficients to 1, using the yielding strength of the reinforcements. In this way a direct check was carried out when failure occurred: the estimated capacity without partial coefficients was $V_{Rd} = 2230$ kN with demand $V_{Ed} = 873$ kN, and the actual safety coefficient of the saddle was equal to 2.55, which can be considered satisfactory.

Deterioration of materials due to weathering and consequent corrosion of the most exposed reinforcements modify the previous results, especially considering the environment in which the bridge is placed. The distance from the sea and the investigations carried out on reinforcements, however, excluded pitting phenomena and highlighted almost uniform corrosion of the bars. Moreover, the saddles most exposed to water drainage are those of the outermost beams, which are also the most stressed for moving loads. It is essential to know the actual state of reinforcements and the level of corrosion during the safety assessment, in order to establish the safety coefficient against sudden failure of the saddle, due to degradation of the weakest element. From the previous evaluations, in the original undamaged conditions, the weakest element of the truss in the proposed model is tie T_2. The area A_s of the tie, corresponding to the tensile stressed reinforcement, was considered totally effective in the previous evaluation and the initial corrosion-free value of this reinforcement is A_0. Then, for a first approach, the effects of corrosion can be considered simply by reducing the cross-section area of the reinforcements due to widespread and uniform corrosion induced by water drainage inside the expansion joint and the saddle. Hence, it was possible to evaluate the reduced area value of the corroded reinforcements A_{cor}, which leads to a unitary safety factor, using an inverse method, where the maximum sustainable corrosion level is assessed. Hence, the unitary safety factor was here considered as the condition of saddle failure. The relation that links the corrosion level CL to the safety factor SF is the following:

$$\mathrm{CL} = 1 - \frac{A_{cor}}{A_0} = 1 - \frac{\frac{A_0}{\mathrm{SF}}}{A_0} = 1 - \frac{1}{\mathrm{SF}} \tag{1}$$

Using the inverse procedure, from the safety factors previously evaluated the related maximum allowable corrosion is obtained: with SF = 2.55, the result is CL = 60%, which means that a 60% mass loss for a uniform corrosion of the main reinforcements would lead to a unitary coefficient, that is to an insufficient safety factor. It is evident that this value is very high since no corrosion of this magnitude was found through on-site investigation, the maximum steel loss being about 10%–20%. This first evaluation was carried out in the simple hypothesis that corrosion only affects the reduction of the area of the reinforcement and not the concrete strength, bond or steel strength.

However, in the present case, the bond of the smoothed rebar is ensured by the end hook, and concrete cracking causes reduction of the concrete compressive strength of the struts only. Thus, the following concomitant effects could be realistically assumed: the effective reinforcement area of ties and stirrups decreases through mass loss, due to corrosion; and the concrete strength decreases due to the struts.

According to the results of in-situ inspections a reduced reinforcement area of 0.85 A_0 could be considered, together with an average strength of concrete $f_{cd}' = 0.80\ f_{cd}$, as suggested in literature studies[10] and ACI318 recommendations.[23] With these assumptions, the weakest element is again corroded tie T_2 and a check on the saddle supplies a minimum safety factor SF = 2.04 with $V_{Rd} = 1784$ kN. The latter is fully acceptable, since the struts are always the elements with greatest capacity, even with a reduction in the concrete strength. This confirms the robustness of the saddle designed by Morandi, both in terms of reinforcements (ties) and concrete struts,

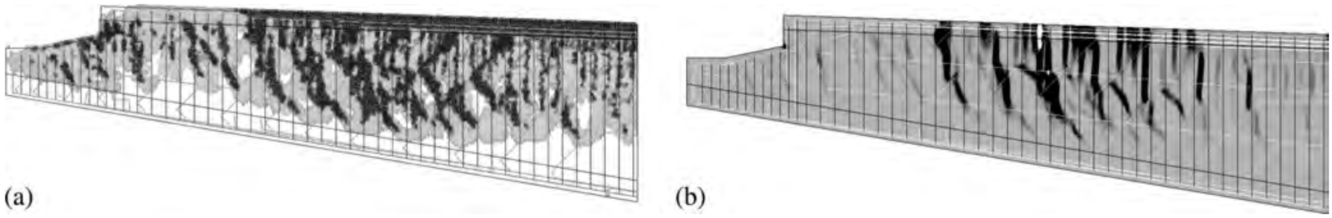


FIGURE 8 Finite element (FE) model of the cantilever with original geometry and reinforcement. (a) Cracking pattern; (b) strain field.

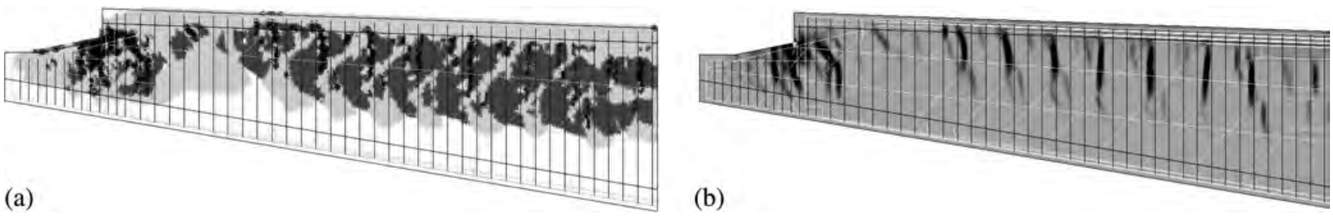


FIGURE 9 Finite element (FE) model of the damaged cantilever with deterioration of concrete and reinforcement corrosion in the saddle area. (a) Cracking pattern; (b) strain field.

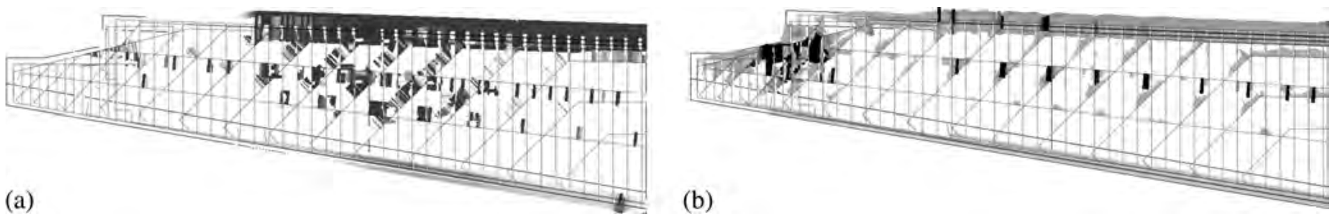


FIGURE 10 Finite element (FE) model of the cantilever. Reinforcement stress near failure. (a) Original configuration; (b) damaged configuration with deteriorated saddle

taking into consideration the possibility of redistribution offered by the transverse beam, which was disregarded in the strut-and-tie model.

A nonlinear finite element model of the entire cantilever was then developed with MIDAS software to compare the results with the strut-and-tie model of the saddle and to evaluate the cracking pattern, taking into account the upper and bottom slabs and the actual arrangement of reinforcements. The model had 21,536 DOF with 10,564 plane-stress 2D elements and 11,386 truss elements for reinforcements. The smeared cracking model was adopted with an average dimension of elements and crack band width 40 mm. The tensile constitutive law of the concrete was bilinear with fracture energy $G_F = 0.142$ N/mm while the compressive law adopted was the Thorenfeldt one, with a smooth exponential post-peak branch; all properties adopted were those suggested by Model Code 2010 and the Dutch Guidelines for Nonlinear Finite Element Analysis of Concrete Structures.[24]

Figure 8 shows the results near failure in terms of concrete cracking and strain field; failure was found when $V_{R,FE} = 2850$ kN, which is greater than the value $V_{R,ST} = 2230$ kN found by the strut-and-tie model, with a difference of 28%. The reason is that the entire cantilever model with the actual reinforcement shows a different failure mode linked mainly to web cracking through flexure and shear of the cantilever subjected to negative moments. The geometry of the saddle and the reinforcement arrangement play a crucial role in this behavior.

The failure mode changes when reinforcement corrosion is taken into account. In the same hypotheses as for the strut-and-tie model, that is the reduction of the reinforcement area and of the concrete strength for the saddle area only, it is found that the weakest area precisely becomes that of the saddle, leading failure towards a more usual modality with crack opening at the corbel corner (Figure 9).

This change of behavior, due to reinforcement corrosion and in general to a lower capacity of the saddle, is also highlighted by a different stress field of reinforcements (Figure 10): for the original case yielding of upper reinforcements of the cantilever is obtained while in the case of a damaged saddle, yielding occurs earlier in the saddle inclined bars, agreeing with the result of the strut-and-tie model for which tie T_2 is the weakest element.

FIGURE 11 View of the overpass

FIGURE 12 Views of the saddles

For the case of the damaged saddle of course the overall capacity was reduced to $V_{R,FE} = 1880$ kN, while with the strut-and-tie model a limit value $V_{R,ST} = 1784$ kN was found. Hence, the value of the FE model was more or less in accordance with the strut-and-tie model, with a difference of just 5%. This result confirms the robustness of the entire cantilever-saddle complex and the goodness of the reinforcement arrangement, as well as the decisive contribution of the actual geometry, with transverse elements like the upper and bottom slabs and the transverse beam.

3.2 | Motorway overpass

The second case-study is an overpass in Sicily on the motorway connecting Palermo to Messina. It has a classical Niagara-type scheme with the central beam between half-joints and two symmetrical cantilevers. The side and central beams are prestressed, as are the cantilevers; the deck is composed of four beams with three intermediate transverse beams, deck width 8.50 m, total longitudinal length of 89 m between abutments, side span length of 22.50 m, central span length 44 m between piers and central beam length 31 m between half-joints.

The double-T cross-section of the beam is slightly variable in height, from 130 to 170 cm, with the upper slab 16 cm thick. Figure 11 shows a global view of the overpass, while Figure 12 shows views of the saddles.

For the present case, the properties of the materials are the following: concrete strength $f_{ck} = 33$ MPa, reinforcement yield strength $f_{yk} = 440$ MPa, and prestressing steel strength $f_{ptk} = 1700$ MPa.

In this case, the Gerber saddle configuration is the classic one with two dapped-end beams that meet on the bearing axis. Both corbels host two prestressing tendons, while the transverse beams are present behind the saddles and affect the entire height of the beam. This configuration of transverse beams implies that the lower corbel of the saddle is only on the beam width, and hence the crossbeam does not contribute to the strength of the saddle. Figure 13 shows the arrangement of tendons and reinforcements for the two dapped-end beams.

The two strut-and-tie models recommended by Eurocode 2 can be adopted here (Figure 14), in order to take into account the strength contribution of the inclined tie and of the horizontal and vertical ones, proportionally to the reinforcement layout. Prestressing has an important role in the overall strength because the inclined tendons supply a vertical component, opposite to the shear, which is highly beneficial. Naturally, only the two tendons anchored in the saddle were considered effective. The results are given for the cantilever saddle (lower corbel).

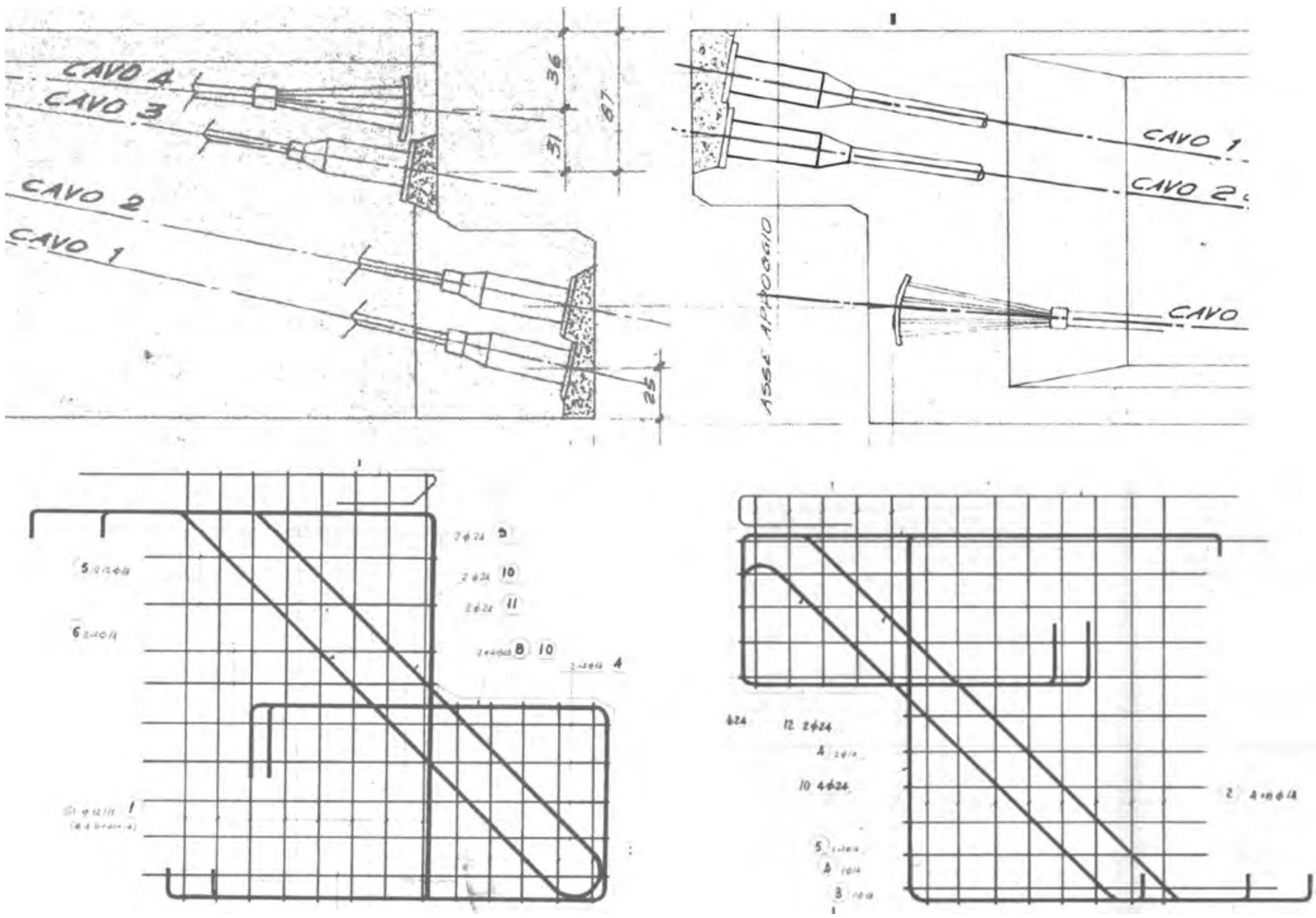


FIGURE 13 Prestressing tendon configuration in the saddles and ordinary reinforcement arrangement

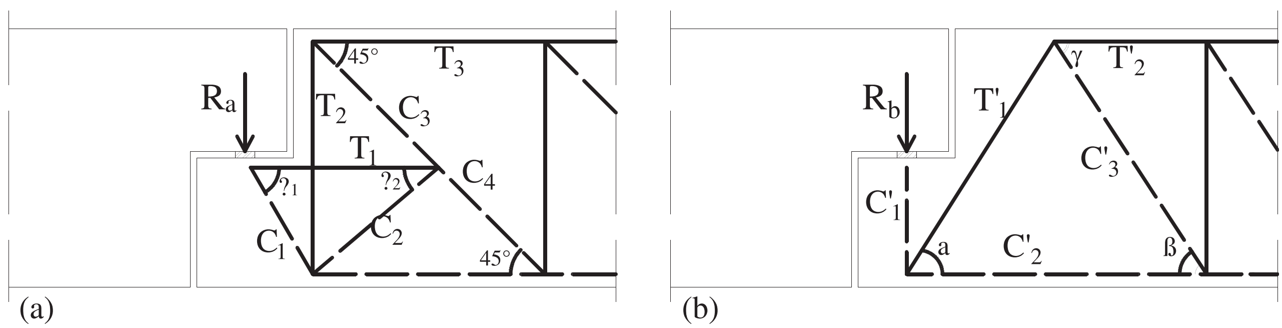
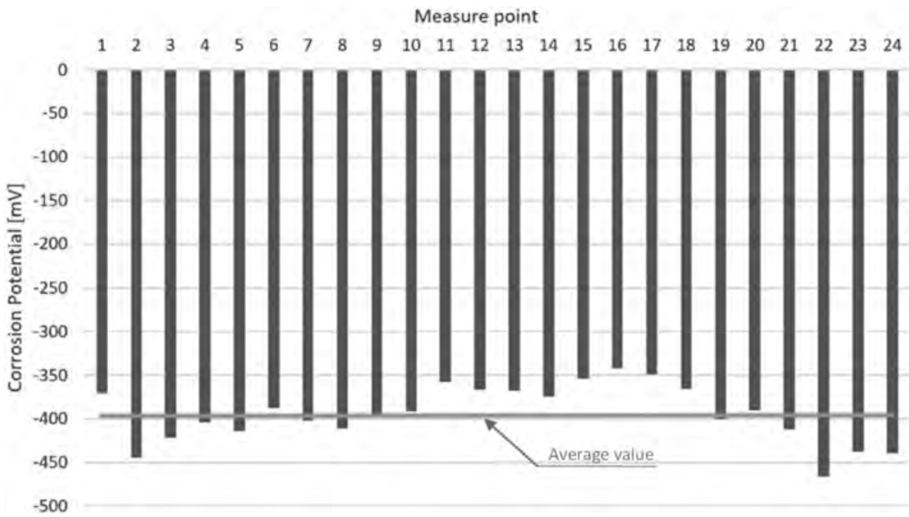


FIGURE 14 EC2 strut-and-tie models of the half-joint. (a) Model with horizontal and vertical ties; (b) model with inclined tie

With a check at ULS, using partial safety coefficients of loads and materials provided by Eurocode, the weakest elements were always found to be the vertical tie T_2 and the inclined one T_1', with capacity $V_{Rd} = 2016$ kN and demand $V_{Ed} = 1549$ kN, leading the safety coefficient to 1.30, considering the contribution of prestressing. The evaluation with all partial safety coefficients set at 1 gives an estimated capacity $V_{Rd} = 2542$ kN with demand $V_{Ed} = 1133$ kN, making the actual global safety coefficient equal to 2.24, which is rather high. The contribution of the prestressing tendon considered as reinforcement (steel cross-section area) is neglected in this evaluation. Consequently, the contribution to tension of the tendon in the failure mechanism is not considered, because in the anchoring area it cannot be considered as effective as in the areas where compatibility between concrete section and tendon is achieved thanks to sheath grouting (bonded tendon). In this connection, the present evaluation was carried out locally on the saddle considering prestressing with unbonded tendons.

In this case, it was possible to proceed to a more detailed investigation campaign. Initially, the inspection

FIGURE 15 Results of corrosion potential on the most damaged saddle



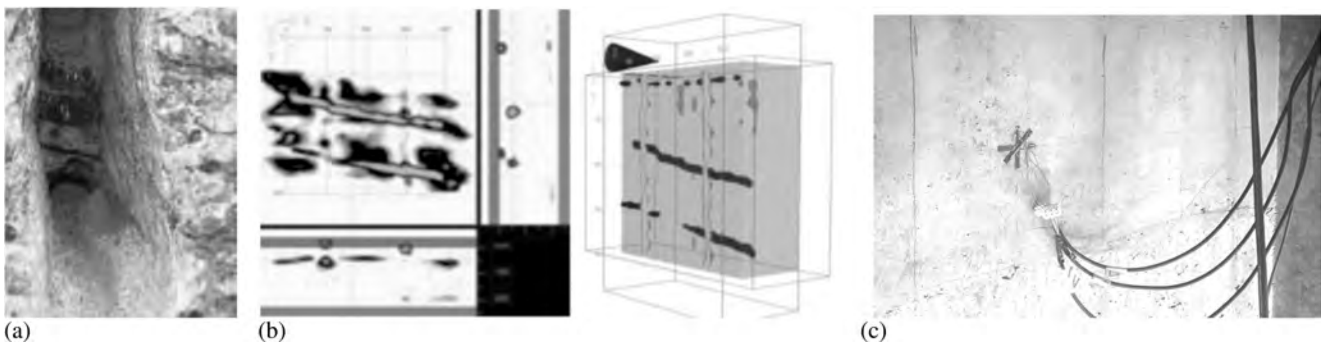


FIGURE 16 Onsite investigation and tests on Gerber saddles. (a) Direct inspection of reinforcement; (b) two-dimensional (2D) and three-dimensional (3D) geo-radar images of tendons; (c) stress relaxation test in prestressing concrete.

was carried out through corrosion potential mapping, in order to evaluate the status of the reinforcements, that is, whether the rebar was subjected to a generalized or localized corrosion attack or not. This method does not provide any quantitative information on the corrosion rate or the extent of the damage that has already been produced but is particularly suitable for a first definition of the corrosion potential of the structure, coupled with further investigations. The potential method is based on the fact that any condition of corrosion (non-corroded, generalized, or localized corrosion) corresponds to a variation of potential within typical intervals.[25–27] In the case study, some Gerber saddles showed a corrosion potential much lower than −300 mV, with minimum values of −450 mV, indicating a high risk (Figure 15). In other saddles of the same bridge, on the other hand, a moderate potential between −50 and −300 mV was found, with a more limited risk. The overall situation still highlights risks from moderate to high due to the exposed reinforcements of the Gerber saddles.

Furthermore, destructive and nondestructive tests were carried out on the concrete and steel, for direct evaluation of the reinforcement status; geo-radar surveys were carried out on the beam to identify prestressing tendon paths together with tests for stress relaxation through strain gauges for evaluation of the efficacy of prestressing, compared to the initial hypotheses based on the original drawings and calculations relating to the bridge (Figure 16).

The damage to the saddle was again considered here, through a reduction in the effective reinforcement area due to corrosion, reduction of the concrete strength, and reduction of the prestressing force. In this case, the loss of reinforcement area due to corrosion was set to 20% while a prestressing loss of 30% was considered for two reasons: the loss due to corrosion of anchorages, subjected to deterioration together with that of the ordinary reinforcements; and the loss due to uniform deterioration of the tendon along the deck attributable to an inadequate sheath grouting. It is to be pointed out that in this case too, no pitting of the reinforcements was found and the bond was guaranteed thanks to the reinforcement hooks.

With these assumptions, which are very penalizing for the saddle strength, and maintaining all partial coefficients at the unitary value, the total capacity drops to

$V_{Rd} = 1969$ kN with demand $V_{Ed} = 1133$ kN, bringing the safety coefficient to 1.74, which can be considered fully satisfactory. The weakest elements of the strut-and-tie model are always the ties, while the struts show greater capacities. If the evaluation with corroded reinforcements and concrete damage is repeated at ULS, that is, with partial safety factors, the capacity/demand ratio is near to 1: this implies that the damaged saddle satisfies the EC1 safety check at its minimum value.

3.3 | Discussion of results

For the case studies seen above, satisfactory safety coefficients were obtained, for the calculation with both ULS and unit coefficients, adopting the average values of material strength and introducing corrosion.

In the first case, the saddle is not prestressed, but structural robustness is provided by the particular geometry; it is worth noting that the original design reinforcements well adapt to tension isostatics and this optimizes the structural behavior and the strut-and-tie model, taking advantage of the large amount of design reinforcement, arranged in accordance with the half-joint behavior. The saddle, despite the absence of prestressing and disregarding the redistribution ability supplied by the crossbeam on the corbel, provides good performance.

In the second case, the role of prestressing becomes essential to the overall strength of the saddle, because the contribution of the inclined tendon to the capacity is about 30% of the total value V_{Rd}. The mild reinforcements are however well arranged and abundant even in this case; hence, the contribution to strength provided by the strut-and-tie model is significant compared to the maximum stress evaluated with the service loads provided by the current EC1 recommendations.

The results obtained in the two case-studies highlight that in some cases good performance of the half-joints of existing bridges in service life is maintained even with deterioration due to reinforcement corrosion. Indeed, in the presence of a careful original design with specific reinforcements, continuous crossbeam on the saddle and prestressing anchorages, a moderate state of degradation does not involve imminent dangers of failure and there is a good local degree of robustness of the Gerber saddles. This is demonstrated by the value of the safety coefficient, which is initially 2.55 (for the first case-study), much higher than the value 1.50 which is generally considered in safety checks. Furthermore, in order to achieve the failure condition, the corrosion of the reinforcements should reduce the cross-section by at least 50%–60%. Furthermore, for robust half-joints, failure shifts towards the cantilever sections in the webs of double-T sections, changing the failure behavior from the local one due to saddle cracking to the shear-flexure one due to web cracking of the current beam section.[1] For this reason, the maximum degree of criticalness for these elements provided by the Italian Guidelines for Assessment and Maintenance of Bridges[2] does not allow the engineer, even in the first phase of inspection and safety assessment, to be able to distinguish cases that appear more critical (absence of crossbeam and prestressing with degraded saddles only on beam webs) from less critical situations, like those highlighted in this study.

The guidelines consider the level of the defect of Gerber saddles and its extent through the so-called "weight of the defect," that is, the dangerousness parameter, but it is always set at its maximum value (G = 5), regardless of the extent and intensity of the actual damage. This choice, although it is reasonable in light of the risk that a brittle and sudden break of the saddles can entail losses in terms of human life or in any case attainment of the Ultimate Limit State, immediately leads, through the procedure supplied by the Guidelines, to a maximum threshold of attention for the bridge that has such a defect. In fact, the procedure entails that to each defect identified during the inspection a numerical weight is attributed and, through the total grade of defects found, the defect class is assigned, contributing decisively to the definition of the global attention class. In the case of Gerber saddles, every defect found, whether of minor or major importance, immediately leads to a definition of maximum alert with a high attention class. The immediate consequence of this evaluation is that the bridge must be moved to a higher level of evaluation, in which material tests and in-situ investigations have to be carried out together with analytical and numerical evaluations on the structural model for definition of retrofitting interventions. Awaiting these interventions and depending on the deficit found in the analysis, a transit ability assessment of the bridge is also necessary with loads and strength partial coefficients referred to a 5-year reference time.

When the saddle is on the web of the girder only the contribution of mild reinforcement and concrete strength are the fundamental features of the half-joint capacity and the likelihood of a brittle failure with catastrophic events is very high: this situation has to be considered extremely dangerous. In other cases, with the presence of crossbeams and prestressing, the consequences of damage are less important, because corrosion is generally concentrated in the outermost elements, where water drains away or enters the joints while the contact between corbels occur on the entire deck width, thanks to the crossbeam.

These considerations suggest differentiating the weights of the defect found in Gerber saddles considering two aspects:

TABLE 1 Proposal for classification and defect grading of Gerber saddles

Configuration	Grade G	Extent k_1	Intensity k_2
Saddle with open cracks, regardless of the girder typology	5	1	1
Saddle on the girder web only, without crossbeams and prestressing	5	0.7 ÷ 1	0.7 ÷ 1
Saddle on the girder web only, without crossbeams and with prestressing	4	0.7 ÷ 1	0.7 ÷ 1
Saddle with crossbeams in the corbels only, with or without prestressing	4	0.7 ÷ 1	0.7 ÷ 1
Saddle on box girder webs without diaphragms, with prestressing	5	0.7 ÷ 1	0.7 ÷ 1
Saddle on box girder webs with diaphragms and prestressing	4	0.5 ÷ 1	0.5 ÷ 1
Saddle with rigid crossbeams on the corbels and on the girders, with or without prestressing	3	0.5 ÷ 1	0.5 ÷ 1
Saddle transversely continuous on slab bridges, with or without prestressing	3	0.5 ÷ 1	0.5 ÷ 1

- the consequences of the defect due to the actual static scheme and to the arrangement of the Gerber saddle, which can have different effects on the structural behavior and on the probability of achieving the ULS;
- the intensity and extent of damage due to corrosion, concrete spalling, and reduced contact area of the structural members.

For this reason, it would be appropriate to introduce two weights in the defect sheet of Gerber saddles annexed to Italian Guidelines, as is done for other defects (extent k_1 and intensity k_2) to take into account different situations, grading different levels of risk. In the present form of the Guidelines, they are always set at 1, bringing the total degree of the defect to 5 regardless of these aspects. Hence, the inspector cannot grade the level of damage and cannot give any judgment on the incidence of the defect on the structural behavior. It could be appropriate, instead, to differentiate the values of k_1 and k_2; in this way, when corrosion is advanced in reduced corbels the maximum level of risk can be assessed. Otherwise, the defect has less importance.

Grading of the defect in the phase of visual inspection might seem at first sight contrary to a criterion of prudence in terms of safety. In reality, it must be considered that the Guidelines are created to carry out a quick classification of the vulnerability of bridges, considering the defects detected in the visual inspections. Afterwards, with the assignment of a class of attention, each Road Authority can draft a priority list of analysis for the major risks detected. When a single defect immediately leads to maximum attention, without the possibility of grading its real importance, the consequence is that a massive number of structures would immediately be classified as being at maximum risk due to a defect that is not necessarily globally dangerous for the structure. A large number of these cases could lead to an excess of work for the technical offices of managing bodies, making the procedure too onerous, although it was initially thought to be quick and easy. This circumstance could slow down actual execution of maintenance work and interventions on the most dangerous bridges, by obtaining the opposite effect to what was wanted, in relation to the budget available and risking leaving some bridges in critical conditions because each one was treated in the same way.

A proposal for saddle classification and defect grading is given in Table 1.

However, the proposal made here concerns priority in the planning of maintenance interventions, but it remains undeniable that the failure of a Gerber saddle is one of the most severe aspects in the assessment of bridges, due to the possible consequences. Hence, although a safety assessment against failure reassuring, in a bridge in service it is necessary that the Code checks at ULS are also satisfied. This, in addition to the fact that the Gerber bridge shows an intrinsic lack of overall structural robustness, being longitudinally isostatic, always suggests eliminating the deficiencies encountered by the ULS checks and improving the behavior of the Gerber saddles by rehabilitation and retrofit interventions. This is therefore necessary both when an important deficit of the safety factor is found and when the deficit is limited and code checks are partially not satisfied. The two cases can be treated differently: while in the first case the strength of the saddle region governs the design of the retrofit intervention due to corrosion. In the second case, by contrast, the overall behavior and the need to provide the Gerber bridge with robustness become fundamental and local strength has minor importance. In the following sections, these two aspects of structural rehabilitation will be taken into account.

4 | REHABILITATION OF GERBER BRIDGES

Usually, the first-choice solutions for the rehabilitation of Gerber saddles are those of a local strengthening intervention. These can be carried out through one of the following:

1. Concrete jacketing with or without FRP, which is aimed at improving the concrete properties, restoring the cover, and inserting new reinforcing bars (Figure 17).
2. Steel plating for external strengthening[12]; it provides a higher degree of reinforcement than the previous interventions but is very invasive and requires adequate spaces for rehabilitation works (Figure 18).
3. Strengthening by inserting vertical or inclined Dywidag bars,[13] which limits diagonal cracking and increases the overall strength through local prestressing of the saddle (Figure 19).

All these methodologies involve local reinforcement and improvement of the saddle's behavior, which maintains its initial geometric configuration, increasing its capacity.[11] In these interventions, the intermediate bearings are also usually replaced, lifting the supported deck. This operation is often very complicated due to the housing of the contrast and the lifting jacks. At the end of the intervention, the local capacity of the saddle is actually increased but the overall behavior of the bridge does not change, maintaining the initial static scheme. This means that the overall structural robustness is not increased, and the deck remains longitudinally isostatic, although adequate to the loads and improved in its critical areas.

Furthermore, a local intervention may have increased the security coefficient against the failure of the half-joint, but in many cases, the problem moves to the bending/shear failure of the current cross-section of the beam (especially in the cantilever area), effectively frustrating the effect of the local increase in strength, because failure can occur before, for a lower level of load, in another section. Hence, local reinforcement of the Gerber saddle could in some cases not be appropriate, especially in the presence of the crossbeams.

Indeed, in nonprestressed concrete bridges, it often happens that the overall vulnerability of the bridge does not depend only on the saddles but also on other areas that present deficiencies, such as the cantilever sections near the pier (which can suffer from maximum shear and negative bending moments) or middle areas of the drop-in span, for maximum positive moments. In these cases, local retrofitting must be carried out at several points of the bridge and with different methodologies,[14] leading to a multiplication of local interventions and significant logistical complications for the execution of the works.

Alternatively, a solution that can easily be adopted in both ordinary and prestressed concrete bridges is to change the static scheme by closing the Gerber saddles and eliminating the joint, as well as strengthening the bridge sections in the new configuration by introducing prestressing through external tendons or Dywidag bars. This solution involves a change in the behavior of the bridge under service loads and in particular under moving loads, whereby the sections partially modify their behavior.

In this connection, a cost–benefit analysis provides that local reinforcement with FRP is not always helpful due to the difficulty of accessing the internal area of the saddle or for steel-plating interventions, which are actually advantageous only on dapped-end beams without transverse elements. The use of vertical or inclined Dywidag bars can instead be combined with a global intervention, involving closure of the saddle, which always remains the most advantageous solution.

Indeed, it is convenient that the bridge becomes a continuous beam deck with the following effects:

- reduction of shear force in the saddle section;
- increased bending moment for moving loads in the saddle area;
- slight reduction of negative bending moment in the cantilever sections;
- increase of overall resistance to bending moment and shear thanks to the effect of the additional axial force due to external prestressing.

In this case, the saddle is completely filled and locked, creating a new deck section, which can generally be configured as a new crossbeam, making it unnecessary to change the internal bearing of the saddles. By contrast, it is necessary to change the constraints of the piers and/or abutments, since the change in the static scheme implies the need to have the sliding bearings in line with the piers to favor the effectiveness of prestressing; in these cases, seismic action is generally faced by the fixed abutment, which must be adequate or protected through the use of dampers. The final result is that of obtaining a

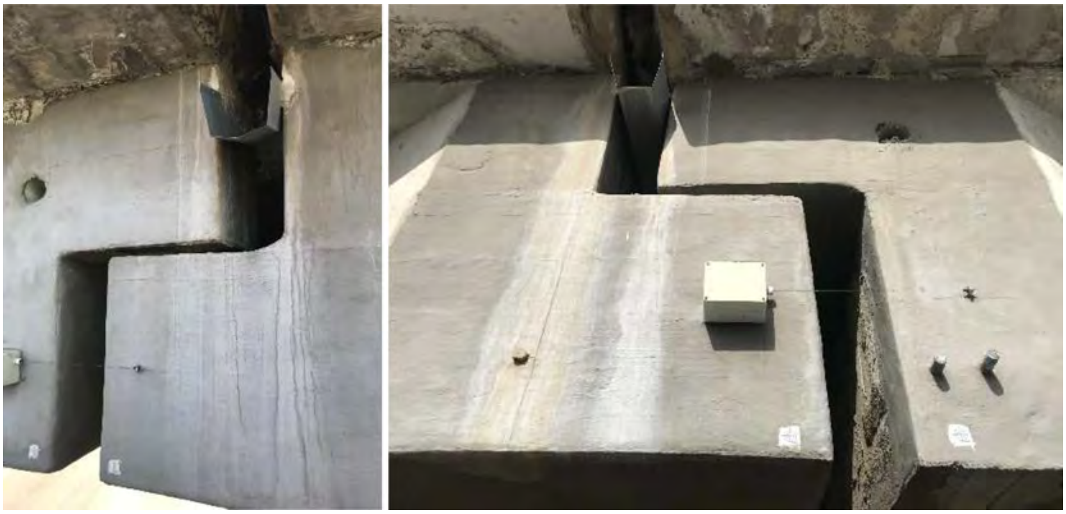


FIGURE 17 Concrete jacketing of Gerber saddle with CFRP fabrics

FIGURE 18 Steel plating of Gerber saddles of Sabbione and Generale Franco Romano viaducts (Italy)[12]

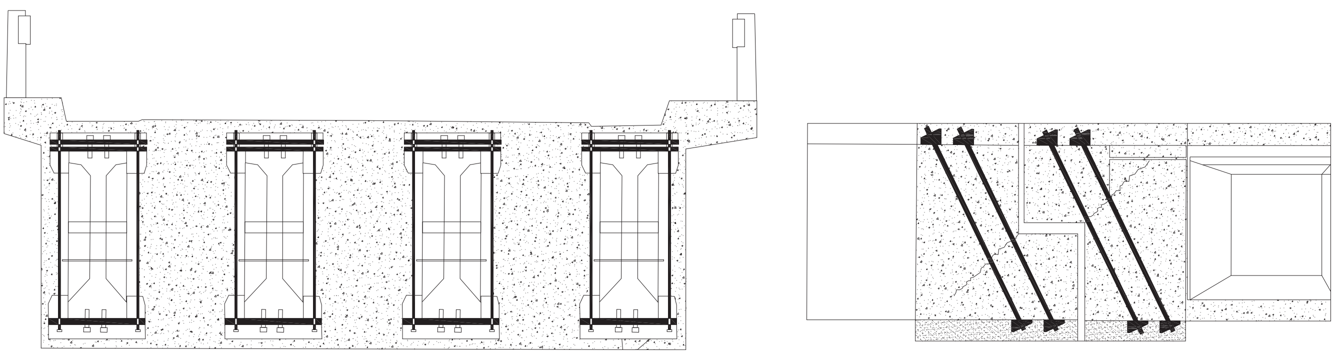


FIGURE 19 Gerber saddle retrofitting with prestressing bars in Vargas bridge[13]

deck of greater robustness thanks to the degrees of hyperstaticity introduced with the new static scheme.

In addition to these aspects, from a seismic point of view, elimination of the articulation due to the saddle improves the entire behavior and eliminates the unpleasant effect of both horizontal and vertical hammerings between the saddle and the drop-in span. In this regard, the effects of the vertical component of an earthquake, which is often responsible for the outflow of the supported span from the bearing seat, are greatly reduced. Hence, for all these reasons, saddle closure is always a good choice, as long as the continuous beam bearings are adequate. Often local strengthening gives a false feeling of increased safety because it is based on the belief that the saddle area is always the critical element. Instead, it often happens that the critical condition is caused by

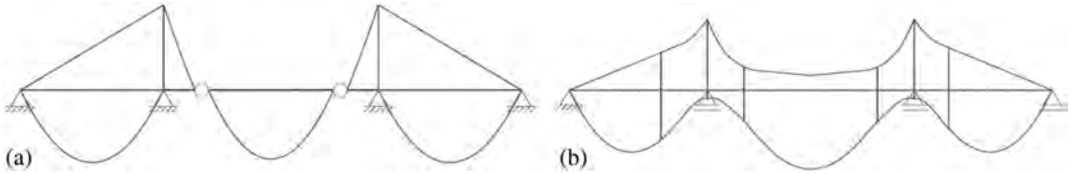


FIGURE 20 Comparison of max-min bending moments of moving loads in a Gerber bridge (a) and in the continuous girder (b)

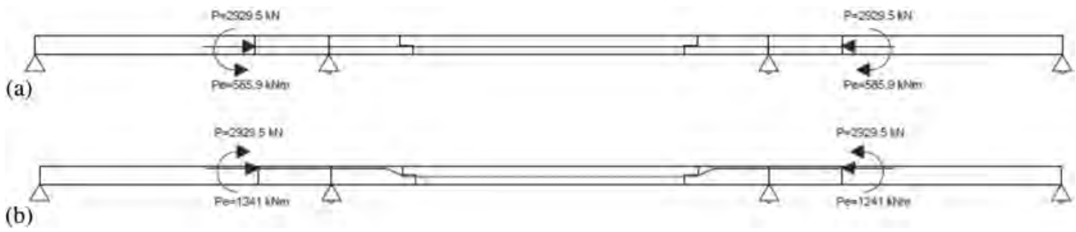


FIGURE 21 External prestressing introduced in a Niagara-type bridge. (a) Slightly eccentric tendon; (b) eccentrically shaped tendon

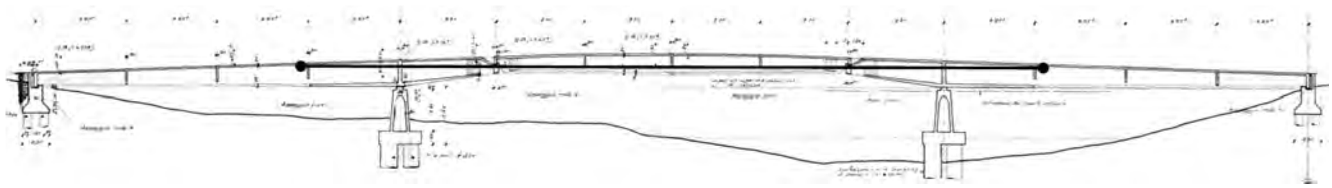


FIGURE 22 Rehabilitation of the first case-study bridge. Configuration of the new external tendons

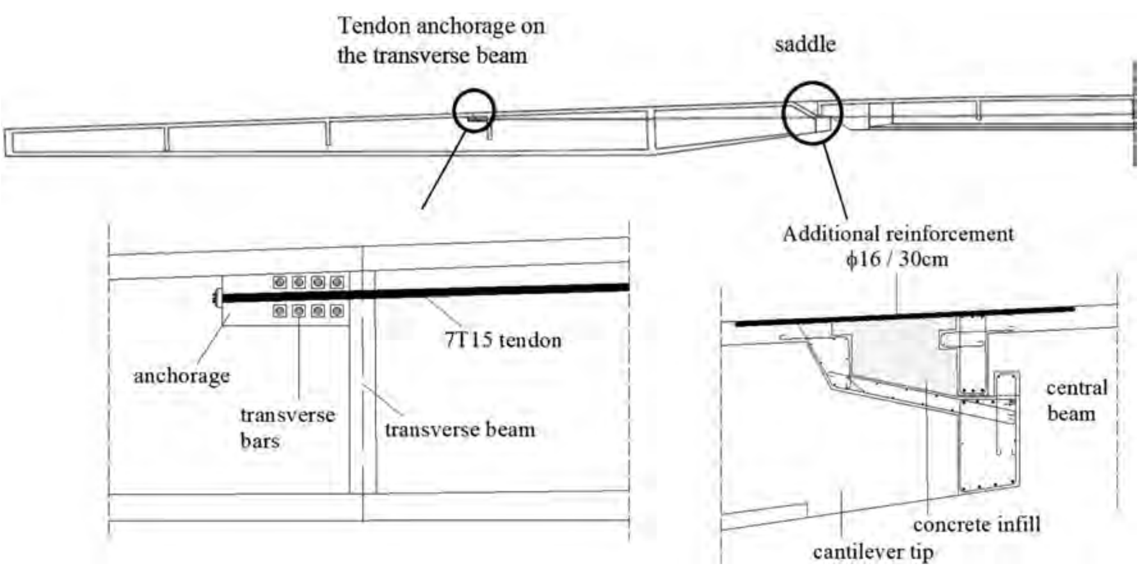


FIGURE 23 Details of retrofitting with external prestressing of the first case-study

bending/shear failure of a different cross-section, far away from the saddle. Without a global intervention with saddle closure and overall strengthening of the deck, this failure mode cannot be avoided.

Figure 20 shows a comparison of max–min diagrams of bending moments for moving loads in a three-span Gerber bridge and in the equivalent continuous girder, while Figure 21 shows two hypotheses of adopting external prestressing in a Niagara-type bridge, using the presence of crossbeams for anchoring and deflecting the tendons.

The efficiency of the shaped tendon is greater than that of the straight one; in fact, the straight tendon with small eccentricity maintains an adequate level of axial

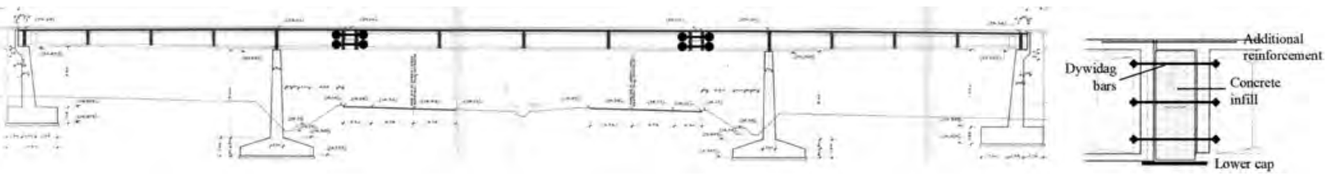


FIGURE 24 Rehabilitation of the second case-study bridge. Introduction of Dywidag bars across the half-joint

force to close the saddle and counteract the additional bending moments due to moving loads in the intermediate areas of the central span. In addition, the shaped tendon provides a further strength contribution to the negative bending moment in the cantilever area and to the positive bending moment in the central span, significantly increasing the capacity with respect to the demand due to the service loads. It follows that the benefit is not limited only to the introduction of the additional prestressing in terms of axial force but also to the eccentricity of the tendon, which improves the shear behavior in the cantilever tip and the bending moment behavior in the areas of maximum positive and negative moment. Naturally, evaluation of the moments due to prestressing must take into account the hyperstatic effects due to the changed scheme of the continuous beam. In order to choose the tendon layout, a cost–benefit evaluation has to be carried out because the costs and complications of having deviators at deflection points in the tendon as well as the increased effects of friction must be counterbalanced by a significant increase in efficiency of the intervention with the shaped tendon.

In the first case-study presented, it is possible, for example, to exploit the natural camber of the bridge, inserting a straight tendon that is almost centered in the saddle area and rises in the cantilever area, at the same time remaining below the cross-section centroids of the central span. This simplifies introduction of the tendon into the girders and optimizes the eccentricity of prestressing (Figure 22), without the need for deviators. The presence of crossbeams, spaced at 8 m, is beneficial for hosting sliding guides, maintaining the tendon straight along the bridge and avoiding intermediate deflections and accidental deviations.[15,16]

The criterion used for prestressing dimensioning was to nullify the tensile stress due to the load combination at the lower edge of the saddle for the positive moments induced by moving loads in the continuous girder, keeping the tensile stresses at the upper edge below the allowable tensile stress f_{ct} in concrete. To achieve this result, 2 tendons 7 T15 (7 strands 0.6″) were considered for the outermost beams and 2 tendons 4T15 (4 strands 0.6″) for the other beams, introducing the prestressing force of 2400 kN on the external beams and 1500 kN on the internal ones.

Details of the proposed solution are shown in Figure 23.

In the second case-study, the presence of the original prestressing, which is still effective, limits the problem to the deteriorated Gerber saddles; hence, the sections above the piers and in the midspan do not require further strengthening, even following the change in the static scheme. In this case, it is more advantageous to fill the saddle and to introduce Dywidag bars to tighten the joint. In this way, it is possible to take advantage of the configuration of the crossbeams which are located just behind the corbels on the two sides of the saddle, with an intervention that is easy to implement at a limited cost (Figure 24).

Naturally, this is possible because the actual value of prestressing and the arrangement of internal tendons is sufficient to supply the required strength in the new continuous beam configuration for the additional stresses due to moving loads in the saddle area. Alternatively, when prestressing losses are too high and the internal tendons cannot guarantee an adequate quote of prestressing with likelihood of cracking for maximum loads, external unbonded prestressing can be provided in addition to the original internal (bonded) one, as in the previous case-study, following the two general tendon arrangements of Figure 21.

5 | CONCLUSIONS

In this study, the problem of assessment and retrofitting of Gerber bridges and in particular of deteriorated half-joints in existing bridges was investigated.

Two case studies of bridges with a Gerber scheme have been presented, typical of the wide variability of this bridge typology, which can occur in road and railway infrastructures. The results of the analyses carried out on the half-joints show that the presence of a well-studied geometry in the design stage and a large amount of reinforcements ensure a certain robustness of the saddle even though partially deteriorated due to corrosion of the reinforcements and degradation of the concrete. In saddles hosting prestressing tendons, the contribution of the tendon inclination can be significant and in any case the presence of the axial prestressing force is always beneficial. In these cases, also considering the design reinforcements, it was found that the saddles of the examined case-studies maintain high safety coefficients even in the presence of corrosion and in very penalizing conditions.

The large number of bridges of this type present on Italian roads recommends creating a list of priorities, which can be carried out through careful assessment of structures with in situ inspections and investigations and structural modeling. This has to be based on the real conditions and severity of the corrosion process as well as on the possible structural consequences, considering the configuration of the saddle, the layout and amount of reinforcements, and the presence of crossbeams that allow redistribution of forces or prestressing.

A proposal for classifying the different typologies of saddles and for grading the importance of the defect due to corrosion of reinforcements is given, in order to help in the massive work of inspections and first-level assessment of existing bridges for effective creation of the priority list. Naturally, the half-joints that are on the beam web only and in which the absence of crossbeam and prestressing does not provide redundancy factors must be considered the most critical ones.

From the cases analyzed, it can be concluded that Gerber saddles are not always the most critical elements of the Gerber bridge, because the current cross-sections, especially in the cantilever area near the saddle and in the drop-in span can be critical, showing early failure due to the combination of shear and bending.

In any case, two rehabilitation strategies can always be considered: local saddle strengthening interventions or global rehabilitation interventions with a change of static scheme and strengthening of the structure, as a whole, through additional prestressing.

The global strategy was preferred here, because, whatever the real safety degree of the structure, closure of the saddles, and creation of structural continuity is always beneficial. The fact is that they eliminate the joints and therefore the causes of localized degradation due to water drainage, introducing a beneficial hyperstatic effect. Through external prestressing, the performance of the current cross-sections is improved throughout the bridge and especially on the cantilever. In addition to these aspects, from a seismic point of view, elimination of the articulation due to the saddle improves the entire behavior eliminating the hammering between the saddle and the drop-in span and the risk of outflow of the supported span by the bearing seat. Furthermore, local strengthening gives a false feeling of increased safety because it is based on the belief that the saddle area is always the critical element while the critical condition may occur for flexure/shear failure of current cross-sections. Hence, for all these reasons, saddle closure is always a good choice, as long as the continuous beam bearings have to be adapted.

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DATA AVAILABILITY STATEMENT

The data that support the findings of this study are available on request from the corresponding author. The data are not publicly available due to privacy or ethical restrictions.

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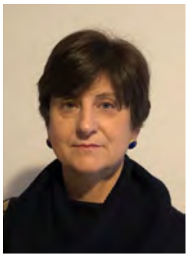

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