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Health assessment of road bridges with Gerber saddles: non-linear planar models

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Abstract

A correct assessment of the structural behavior and performance of deteriorated structures is crucial both for a proper planning of maintenance works and for a correct design of new structures in aggressive environments. The aim of this work is to investigate the response of Gerber saddles to the variation of the two main parameters representing typical damaged scenarios: the concrete cover and the diameter of the steel reinforcements. The analyses are conducted through Finite Element two-dimensional non-linear models based on the compatible stress field; the results are compared with the relevant 'Strut and Tie' models. At first, parametric analyses are carried out, in order to evaluate the sensitivity of load capacity to the position of steel rebars and to the dimension of concrete cover; this first part of the contribution aims to highlight the importance of an adequate level of knowledge. Then, the contribution focuses on the reduction of saddle capacity due to degradation conditions, referred to both the loss of concrete cover and the corrosion of reinforcing bars. The analyses closes with a comparison among the results provided by the two-dimensional investigations and the ones provided by the relevant 'Strut and Tie' models.

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Keywords: road bridges, Gerber saddles, non-linear analysis, planar models, health assessment

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1. Introduction

Many Italian and worldwide reinforced concrete bridges realized between the ‘50s and ‘80s of the last century present a typical Gerber static scheme. By adopting such a structural solution, indeed, internal forces (and thus dimensions of structural elements) were reduced but the static scheme was kept isostatic. In recent times, the significant fragility of Gerber saddles (i.e., half-joints of the decks) has been widely recognized due to two main causes: (i) the notable exposure to water contact (since the deck discontinuity eases the water percolation in these zones), and (ii) the stocky nature of the structural element that favors the occurrence of fragile collapses (i.e., due to shear). This is highly testified in several literature studies (Campione, Granata, Papia, & Maria, 2022; Desnerck, Lees, & Morley, 2017; Santarsiero, Masi, & Picciano, 2021) as well as in infamous bridge collapses such as the Concorde overpasses in Canada (Mitchell, Marchand, Croteau, & Cook, 2011) and the Annone overpass in Italy (Bazzuchi, Restuccia, & Ferro, 2018).

Despite the wide development of computational tools, for discontinuity regions (D-regions), “Strut and Tie” models are still essential in many applications (Di Carlo, Meda, Molaioni, & Rinaldi, 2023; Spinella & Messina, 2023). Usually, manual calculations or implementation in spreadsheets are adopted in these cases. Their application to real structures is however time-consuming due to the need for iterations with global models and the necessity to consider different load cases, as also suggested by Eurocodes (EN1992-1-1, 2004) and Guidelines (Ministero delle Infrastrutture e dei Trasporti, 2020).

Furthermore, this method is not suitable for checks under serviceability conditions (deformations, crack widths, etc.). For these reasons, a method known as the Compatible Stress Field Method, CSFM, has been developed by ETH Zurich and the software company IDEA StatiCa as part of the DR-Design Eurostars-10571 project (Kaufmann, et al., 2020), see https://www.ideastatica.com/walls-and-details (accessed 30.11.2023) This is a continuous stress field analysis method based on FEM and plane diagrams, in which classical stress field solutions are supplemented by kinematic considerations on allowable deformations.

In this paper, parametric assessments are carried out for Gerber saddles to evaluate the effect of rebar placement and concrete cover dimensions on load capacity. This first section of the paper is intended to highlight the importance of having a sufficient level of knowledge. The focus then shifts to the reduction in saddle capacity caused by deterioration conditions, including loss of concrete cover and rebar corrosion. The analysis concludes with a comparison of the results from the two-dimensional investigations with those from the relevant Strut and Tie models.

2. Assumptions of the 2-D model

The effective compressive strength of concrete is calculated using the basic assumption of no-tension in concrete but taking into account the stiffening effect that stressed (undamaged) concrete between two adjacent cracks applies on the reinforcement bars undergoing tension stresses. The CSFM therefore uses common constitutive laws provided by design standards for concrete and reinforcement. The advantage for the designers is that there is no need to provide additional, often arbitrary, material laws, as often done when non-linear finite element analyses based on fracture energy are adopted.

In detail, the CSFM method assumes fictitious, rotating, stress-free cracks that open without any friction (Fig. 1a), and considers the equilibrium of the cracks together with the average deformations of the reinforcement. Therefore, the model considers the maximum stresses of the concrete (σ_{c3r}) and of the steel reinforcement (σ_{sr}) at the cracks while neglecting the tensile strength of the concrete ($\sigma_{c1r} = 0$), except for the mentioned stiffening effect on the rebars. The consideration of tensile stiffening allows the average deformations of the reinforcement (ε_m) to be simulated (Fig. 1b).

The implemented concrete model is based on the constitutive laws for uniaxial compression prescribed by the design codes for cross-sections, depending only on compressive strength. The parabola-rectangle diagram specified in (EN1992-1-1, 2004) and shown in Fig. 1c is used by default in the CSFM, but the designer can also choose a simplified ideal elastic-plastic relationship. According to (ACI Committee 318, 2014), only the parabola-rectangle stress-strain diagram can be used. As mentioned above, tensile strength is neglected as in classical reinforced concrete design.

The effective compressive strength is automatically evaluated for cracked concrete based on the principal tensile strain (ε_1) using the reduction factor k_{c2}, as shown in Figg. 1c and 1e. The implemented reduction relation (Fig. 1e) is a generalization of the 2010 FIB Model Code proposal for shear checks, which includes a limit of 0.65 for the maximum ratio of effective concrete strength over the concrete compressive strength (International Federation for Structural Concrete - fib, 2010).

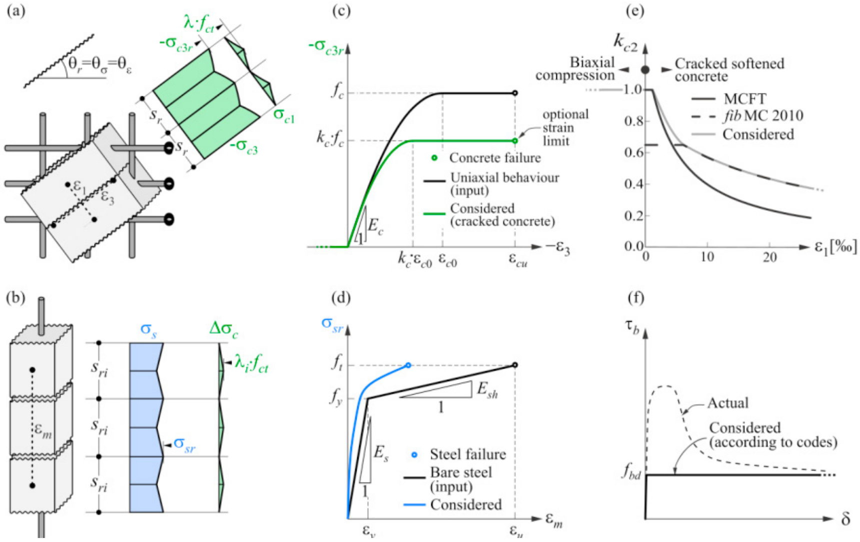

Fig. 1. Assumptions of the 2-D model (Kaufmann, et al., 2020)

For reinforcements, the idealized bilinear stress-strain law (typically defined by design standards, Fig. 1d) is considered by default. Lastly, the bond slip between reinforcement and concrete is introduced into the finite element model for ultimate limit state load cases by considering the simplified rigid-perfectly plastic constitutive relationship shown in Fig. 1f, where f_{bd} is the design value of the ultimate bond stress specified by the design code for the specific bond conditions.

When applying the tension stiffening effect, a distinction is made between stabilized and un-stabilized cracks: in both cases, the default is to assume that the concrete is fully cracked prior to loading.

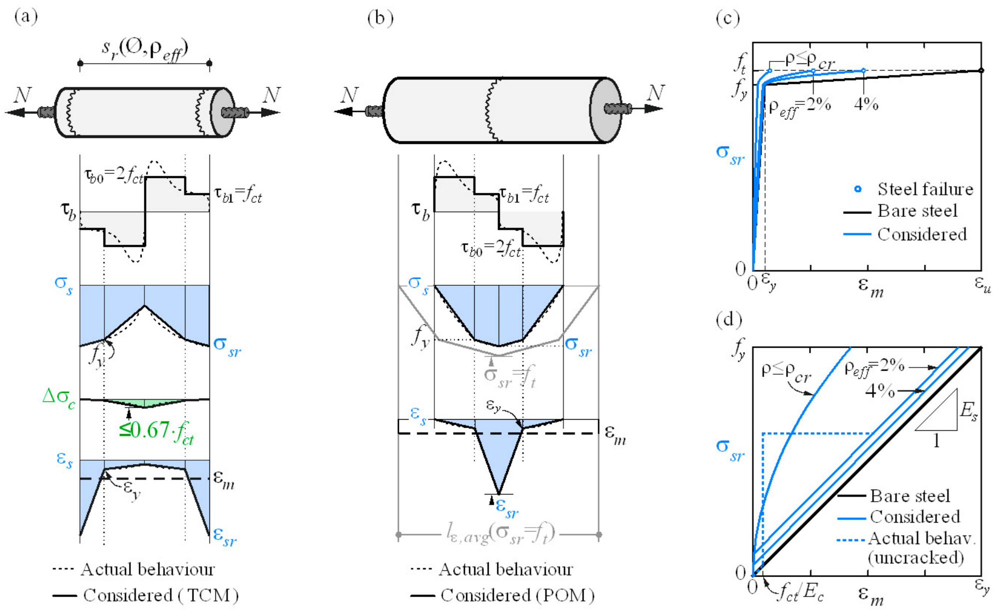

Fig. 2. Tension stiffening models (Mata-Falcón, 2022).

3. Case study

The case study concerns a Gerber saddle belonging to a 40cmx70cm concrete bridge beam; the concrete has a cylindrical strength of 30 MPa and reinforcements are Feb38K steel (f_y = 430 MPa). The reinforcements consist of: stirrups ϕ 10 with pitch of 20 and 10 cm, longitudinal bars of 3 upper and 3 lower bars ϕ 20, 3 hook bars ϕ 12 and 4 inclined ϕ 12, see Fig. 3. The saddle is representative of geometric configuration and reinforcement of dapped-end supported beams long almost 15.00 m.

The assumed scenarios relate to:

1) arrangement of reinforcement with reinforcement cover equal to 25 – 15 – 5 mm;
2) arrangement of reinforcement with cover of 25 – 15 – 5 mm and uniform corrosion of lower bars, stirrups, hooks and inclined bars (that is, upper longitudinal bars are supposed healthy);
3) arrangement of reinforcement with bar cover equal to 25 - 15 - 5mm and uniform corrosion of lower bars, stirrups, hooks and inclined bars (as scenario 2) with a reduction of cross-section due to loss of concrete cover in the lower part (that is, upper longitudinal bars are supposed healthy).

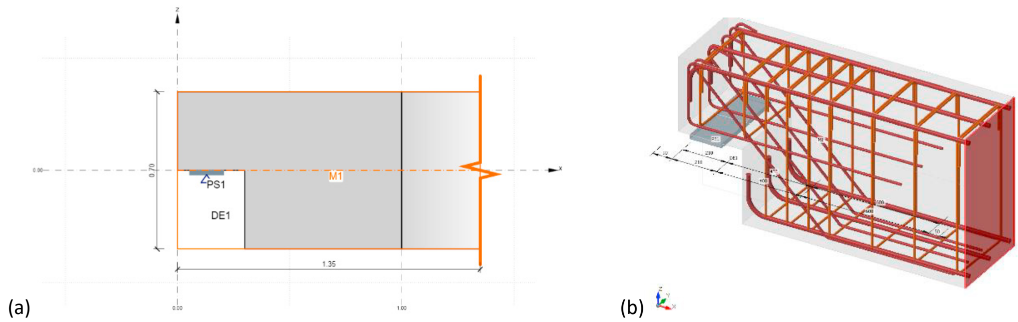

Fig. 3. Case study: (a) geometry, and (b) reinforcement placement.

The assumed loads are: a distributed load of 130 kN/m representing permanent loads (to add to self-weight), 210 kN/m for variable loads, Fig. 4. These loads are then combined as Ultimate Limit State (ULS). The result in terms of required equilibrium actions are automatically evaluated by the calculation software, see again Fig. 4, where V_z and M_y are the shear and the bending moment. In the following section, the ULS scenario is considered.

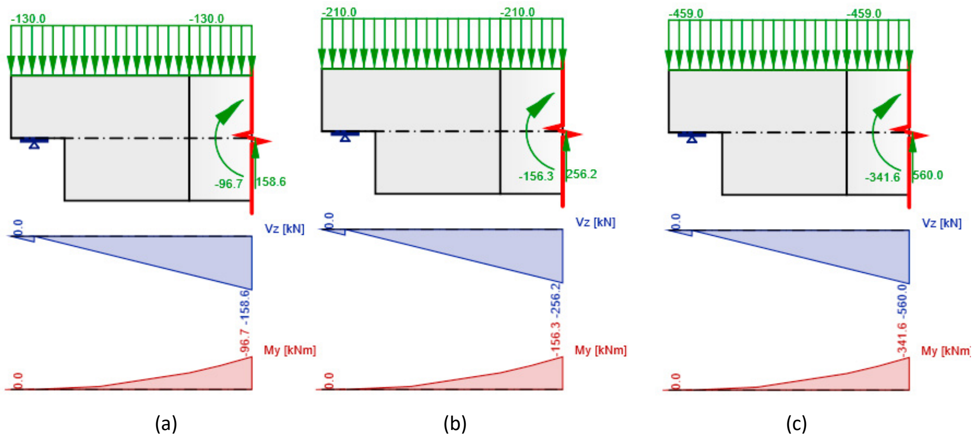

Fig. 4. Loading scenarios: (a) permanent load case; (b) variable load case; (c) Ultimate Limit State (ULS).

4. Results and comparisons

A sensitivity analysis of the maximum working rate in concrete when the concrete cover changes from 25 mm to 15 and 5 mm is showed as histograms in Fig. 5. The three discussed scenarios are considered; analyses in the corroded steel configuration (scenario 3) refer to a time interval of 20 years and assume a uniform corrosion propagation rate equal to i_{corr} = 0.5 μA/cm^2, corresponding to a uniform cross-sectional diameter loss of about 1 mm.

The comparisons show a reduction in working rate of about 8%, for scenario 1, 5%, for scenario 2, and 3%, for scenario 3, when the concrete cover is increased from 5 to 25 mm. Comparing the various scenarios, the effect of the damages introduced in scenarios 2 and 3 can lead to an increase up to about 10%. These are not negligible values, considering both the peculiarity of these elements and their brittle structural behavior.

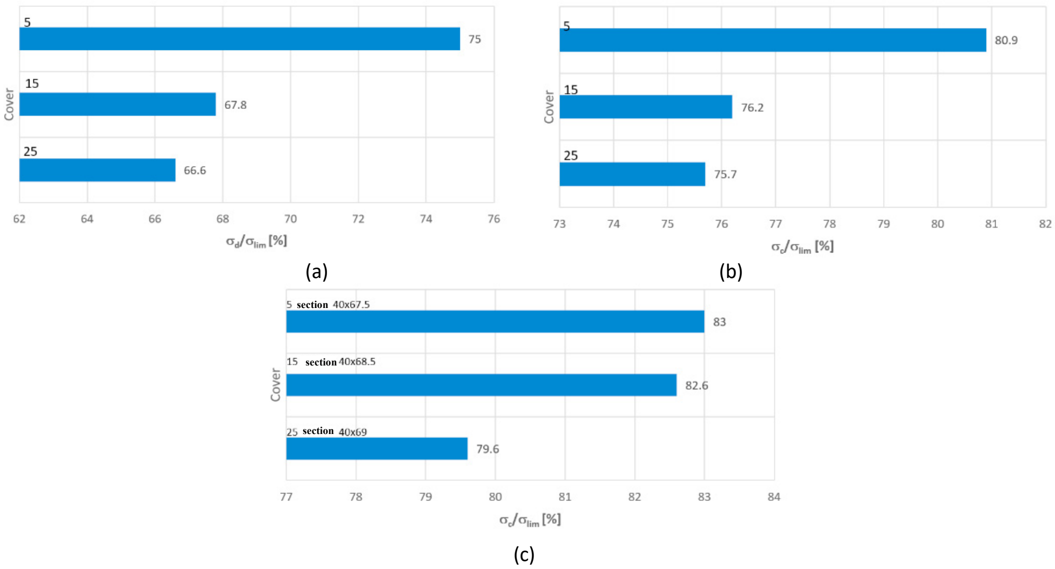

Fig. 5. Sensitivity analysis of the maximum working rate (as percentage) in concrete changing concrete cover (25 – 15 – 5 mm): (a) undamaged scenario (scenario 1), (b) corroded steel (scenario 2), and (c) corroded steel and loss of concrete cover (scenario 3).

The comparisons in terms of working rate of concrete, steel and anchor strength are show in Fig. 6. The results show a small reduction in working rate of steel and anchor mechanism when the cover is varied (always less than 2%).

In order to compare the ultimate loading capacity of the considered Gerber saddle, Table 1 shows a direct comparison in terms of allowable permanent and variable loadings.

It is shown that for scenario 1 the structure can always carry the assigned loads, whatever the size of the concrete cover. On the contrary, in the presence of damages (scenarios 2 and 3), with the initiation of corrosion first and the corrosion and loss of the concrete cover then, the structure has a variable load carrying capacity of 88.2% to 87.8% and 87.9%, scenario 2, and 77.7%, 82.1% and 87.9% respectively. Basically, by also considering the loss of concrete cover in addition to corrosion, that is, moving from scenario 2 to scenario 3, there is always a reduction in loading capacity, except for the case where the concrete cover is very small (5 mm), where there is no significant change.

In order to evaluate the accuracy of the proposed modeling techniques, scenarios 1 and 3 of the Gerber saddles with 25 mm of concrete cover (boldface data in Table 1) are also analyzed by 'Strut and Tie' models. These two cases were chosen because they resulted in the maximum reduction of load capacity.

For this purpose, a homemade spreadsheet is under development, where all the necessary verifications are performed according to Eurocode 2 and the two combined 'strut-and-tie' models of Fig. 7. The relevant results will be shown during the conference.

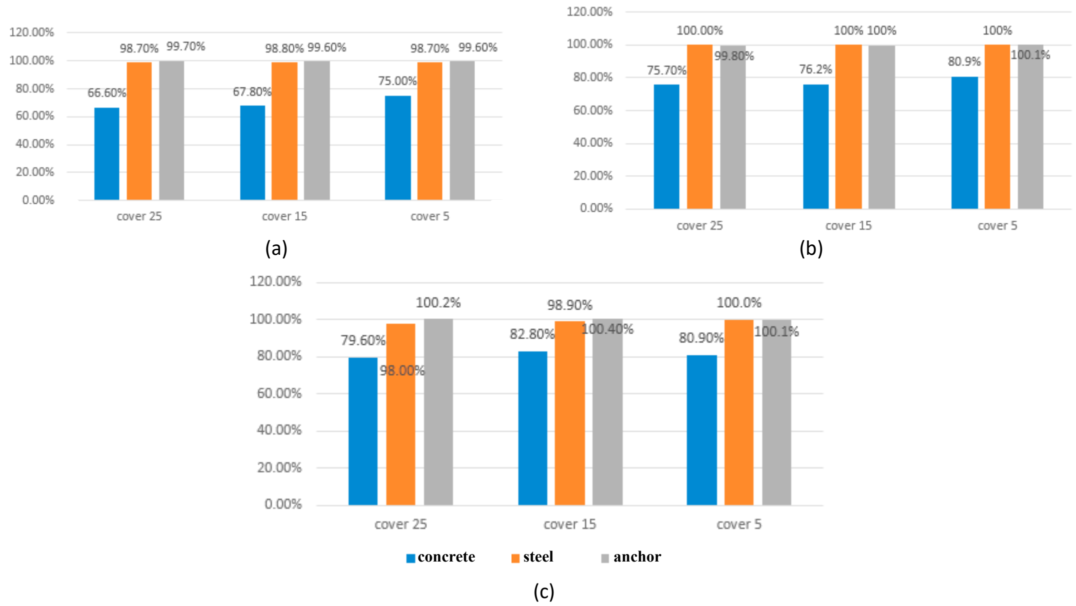

Fig. 6. Results for the maximum working rate (as percentage) changing concrete cover (25 – 15 – 5 mm): (a) undamaged steel (scenario 1), (b) corroded steel (scenario 2), and (c) corroded steel and loss of concrete cover (scenario 3).

Table 1. Summary of the estimated loading capacity.

Scenario	Concrete cover		
	25 mm	15 mm	5 mm
1	**100 % permanent + 100% variable**	100 % permanent + 100% variable	100 % permanent + 100% variable
2	100 % permanent + 88.2% variable	100 % permanent + 87.8% variable	100 % permanent + 87.9% variable
3	**100 % permanent + 77.7% variable**	100 % permanent + 82.1% variable	100 % permanent + 87.9% variable

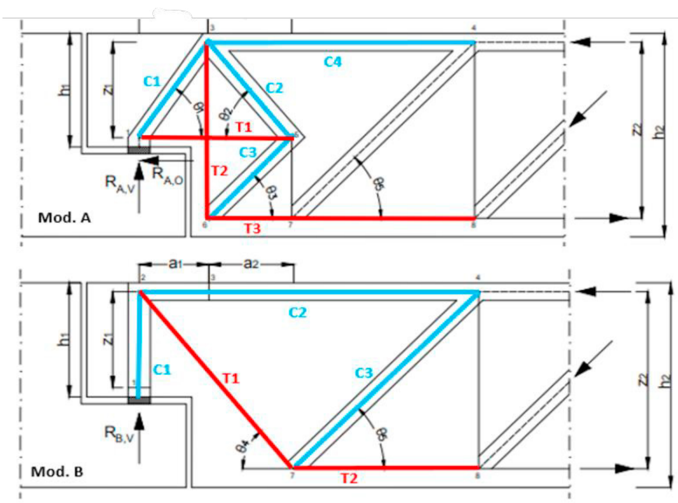

Fig. 7. Adopted 'Strut and Tie' models.

5. Conclusions

This paper deals with a topic that is highly relevant due to recent collapses in road bridges. Several studies have being carried out by the academic community on the analysis and design of D-regions. Here, after a preliminary introduction in road bridges with Gerber saddles, a sensitivity analysis has been performed through a Finite Element two-dimensional non-linear models based on the compatible stress field. The usual critical issues related to Gerber saddle have been considered: changing in concrete cover (and then in reinforcement placement, assuming that the same cross-section dimensions is preserved), corrosion of steel reinforcements, and loss of concrete cover.

The first study, the one concerning the dimensions of concrete cover, confirms the importance of an adequate level of knowledge of such structural elements, since variations up to 8% can be gathered for the stresses in concrete. When damaged scenarios are considered, the study shows a loss of loading capacity of the saddle up to 22%, a value that is certainly not negligible, especially considering that the reduction assumed for the rebar cross-section was only 1 mm and that only the loss of the concrete cover was assumed for the concrete. Comparisons with simplified 'Strut and Tie' models are in progress and will be presented during the conference. Future developments concern the extension of the analyses to different geometries and modeling techniques.

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