Robustness of RC girder bridges: the case of half-joint bridges

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ABSTRACT: Considerable research efforts have been made on the progressive collapse resistance of buildings. This effort is much more limited in the case of bridges, where robustness criteria are just as, or even more important than in buildings. Existing studies dealing with the robustness of bridges, although appreciable, often are limited to qualitative considerations that can provide designers with valuable pointers when designing new bridges. It is equally important to assess not only the safety but also the robustness of existing bridges through reliable metrics that can be used in the prioritization of interventions by the managing authority. According to this aim, this paper applies a selected measure of robustness to a particular type of reinforced concrete (RC) girder bridge, namely half-joint bridges. The Annone viaduct, which collapsed in 2016 after the passage of a heavy truck, is used as a case study.

1 INTRODUCTION

In the last decade, considerable research efforts have been made on the evaluation of the progressive collapse resistance – or equivalently in enhancing the structural robustness – of buildings with a particular focus on reinforced concrete frame buildings. Looking at the bridges, this research effort is much more limited and robustness criteria are just as, or even more important than in buildings. Existing studies on the robustness of bridges, although appreciable, are often limited to qualitative considerations that can provide designers with valuable indications for the design of new bridges. The dramatic series of bridge collapses that occurred in Italy in the last years has highlighted the need for urgent treatment of the worldwide infrastructural heritage, which consists mainly of reinforced concrete (RC) and precast concrete structures. In this regard, it is equally important to assess the safety and robustness of existing bridges. The robustness of existing bridges can be assessed through reliable metrics that can be used in the prioritization of interventions by the managing authority. In this context, the paper applies a measure of robustness available in the literature to a particular type of RC girder bridge, namely half-joint bridges. A simple methodology based on the notional removal of critical elements is applied for quantifying the structural robustness. Half-joint bridges represent a not insignificant amount of the Italian (and European) infrastructural stock and they are considered among the most critical infrastructures. The Annone viaduct, an RC half-joint bridge that collapsed in 2016 after the passage of a heavy truck, is used as a case study. The bridge was built in the early 60s in northern Italy and it consisted of a central suspended span and two side spans having a cantilever scheme, for a total length of 56.10 m.

2 BACKGROUND ON ROBUSTNESS INDICATORS

Objective measures of robustness are required to assess safety against progressive collapse, estimate losses, and decide whether a level of robustness is acceptable or not. Furthermore, a quantitative measure of robustness is useful for prioritizing maintenance and repair work on existing structures. Several indicators for measuring robustness are available in the literature, some of them formulated in deterministic terms, others in probabilistic or risk-based terms. A comprehensive selection of robustness measures proposed in the literature can be found in Adam et al. (2018). To date, there is no unanimous consensus on a univocal measure of robustness. This limits the adoption of robustness indicators on a large scale.

Deterministic measures of robustness are typically based on the consequences of an assumed initial damage in the structure. In particular, such measures quantify the change in structural properties, such as stiffness or strength, having in common one basic idea: the comparison between the intact and the damaged structure. Although there are no specific robustness indicators for bridges, a careful analysis of the metrics available in the literature allows some indicators to be favoured over others based on the type of collapse mechanism (Starossek 2017).

Among the deterministic indicators of robustness that can also be calculated using linear elastic material behaviour, the 'reserve-based measure' R_r proposed by Starossek (2017) is considered particularly suitable for the study of half-joint bridges. It is based on the redistribution of forces following the failure of a structural element. This measure of robustness is called a 'reserve-based measure' because the redistribution of forces is only possible when the structure has reserves of bearing capacity. Let us consider a beam suspended on rods in the intact initial configuration of Figure 1a. The load applied to the beam induces stresses in the rods. Assume a local failure in the rod j. When the rod j is removed (Figure 1b), the force in the rod j is redistributed in the system and the force in the adjacent rod k increases by the quantity ΔF_k. The maximum dynamic force in the rod k F_{kj} ($F_{kj} = F_k + \Delta F_{ki}$) can be compared with the bearing capacity of the rod ($F_{k,ult}$). The index R_r can then be expressed as:

$$R_r = 1 - max_{k,j} \frac{F_k + \Delta F_{kj}}{F_{k,ult}} \tag{1}$$

Positive values of the index R_r indicate robustness as failure does not propagate. Negative values of R_r indicate progressive collapse and lack of robustness. The maximum value of R_r in this formulation is of the order of $1 - \phi$ where ϕ represents the average resistance safety factor.

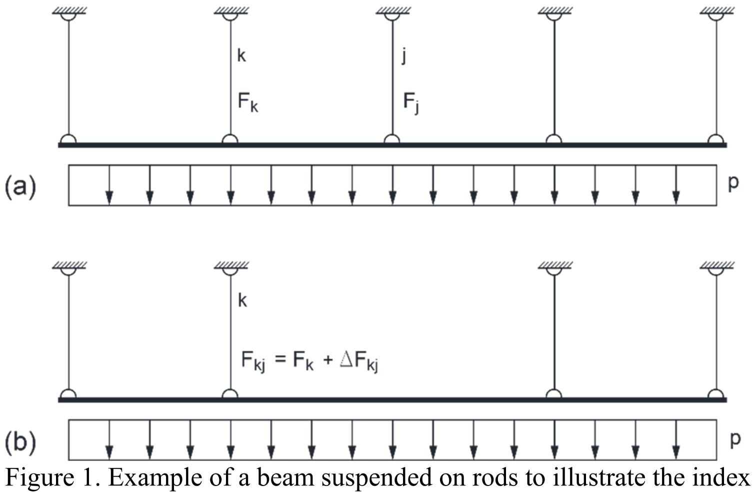

Figure 1. Example of a beam suspended on rods to illustrate the index R_r (adapted from Starossek 2017).

Although the index R_r is referred to by Starossek (2017) as a measure of robustness, it is perhaps mainly connected to structural redundancy since it shows the ability of the system to redistribute loads after damage to a single or a few members. The terms structural robustness, redundancy, and static indeterminacy are often used as synonymous in the literature but they denote different properties of the structural system. The fundamental difference between these terms is addressed in Biondini et al. (2008) where multiple examples are given as well. It has been also demonstrated that the degree of static indeterminacy is not a consistent measure of structural redundancy (Frangopol & Curley 1987; Biondini et al. 2008). Although structural robustness strongly depends on redundancy and alternative load paths may enhance structural redundancy, the latter does not necessarily lead to increasing structural robustness (Biondini 2009, André 2020). In fact, the redistribution of internal actions on the damaged system may promote the evolution of damage reducing robustness.

3 CASE STUDY DESCRIPTION

3.1 *Geometry*

The bridge case study is the Annone overpass, a precast concrete structure built in the 1960s and collapsed in 2016 due to the shear failure of a half-joint during the passage of a heavy truck. The bridge was located in Annone, a small town in the province of Lecco (northern Italy), and designed as a second category bridge according to Circular 14 February 1962 n.384 (1962). The bridge consisted of a central suspended span (drop-in) and two side spans having a cantilever scheme, for a total length of 56.10 m and a total width of 7.4 m. The two side spans were simply supported on the abutments and on the intermediate walls each having a cantilever of about 2.80 m at the end. The central beams were simply supported on the two lateral beams employing half-joints thus reproducing a statically determinate condition. The span of the central beams was equal to 19.0 m. The geometric details of the bridge are here omitted but can be found in di Prisco et al. (2018, 2023). Five prestressed prefabricated beams (PC beams in the following), with a spacing between their axes of 1.35 m and disposed along the longitudinal bridge direction, supported a continuous cast in situ RC slab. The RC top slab was clamped at the abutments. RC transversal beams – five in the central span and four in the side spans – were cast in situ in the orthogonal direction. A schematic plan view of the central span is shown in Figure 2a together with the nomenclature of the 10 half-joints (R1 – R10), while a vertical section of the bridge is illustrated in Figure 2b.

3.2 *Load configuration*

This section describes the nominal applied loads on the elements of the central span due to their self-weight and live loads according to the NTC 2018 (Infrastructure Ministerial Decree, 2018). For the sake of brevity, bending moments and shear forces at mid-span are omitted and only the load acting on the half-joints is evaluated.

Regarding the central span, the overall self-weight of the bridge, including safety barriers, pavement, curbs, beams, and RC slab, was equal to 1690 kN, a value higher than that assumed by the designer, set to 1521 kN. The dimension of conventional lanes, the load positions, and the amplitudes are described in Figure 3 according to NTC 2018 (Infrastructure Ministerial Decree, 2018). Two load configurations are assumed to maximize the load on supports R1-R5 (symmetrical load with respect to the mid-span can be assumed to maximize the load on supports R6-R10): (i) lane 1 loaded according to the diagram in Figure 3 and (ii) lanes 1 and 2 both loaded according to the diagram in Figure 3. The first load configuration is schematized in Figure 2a as the load area highlighted in green – and half of the light grey area – resulting in a total live load of 1132 kN, a combination of concentrated and distributed loads. The second load configuration is schematized in Figure 2 as the load area highlighted in green, grey, and red, for a total live load of 1693.5 kN.

It should be emphasised that these live loads are significantly higher than those envisaged in the design phase. The bridge was in fact designed following the provisions of Circular 14 February 1962 n.384 (1962) as a bridge of second category. Even assuming that the distance between two subsequent vehicles is equal to zero, the maximum overall live load applied on the central span is equal to 88 t. This load must be increased with a dynamic amplification coefficient that depends on the bridge span and that can be considered equal to 1.284 if the span between the central piers is considered resulting in a maximum live load of 113 t (1109 kN vs 1693.5 kN).

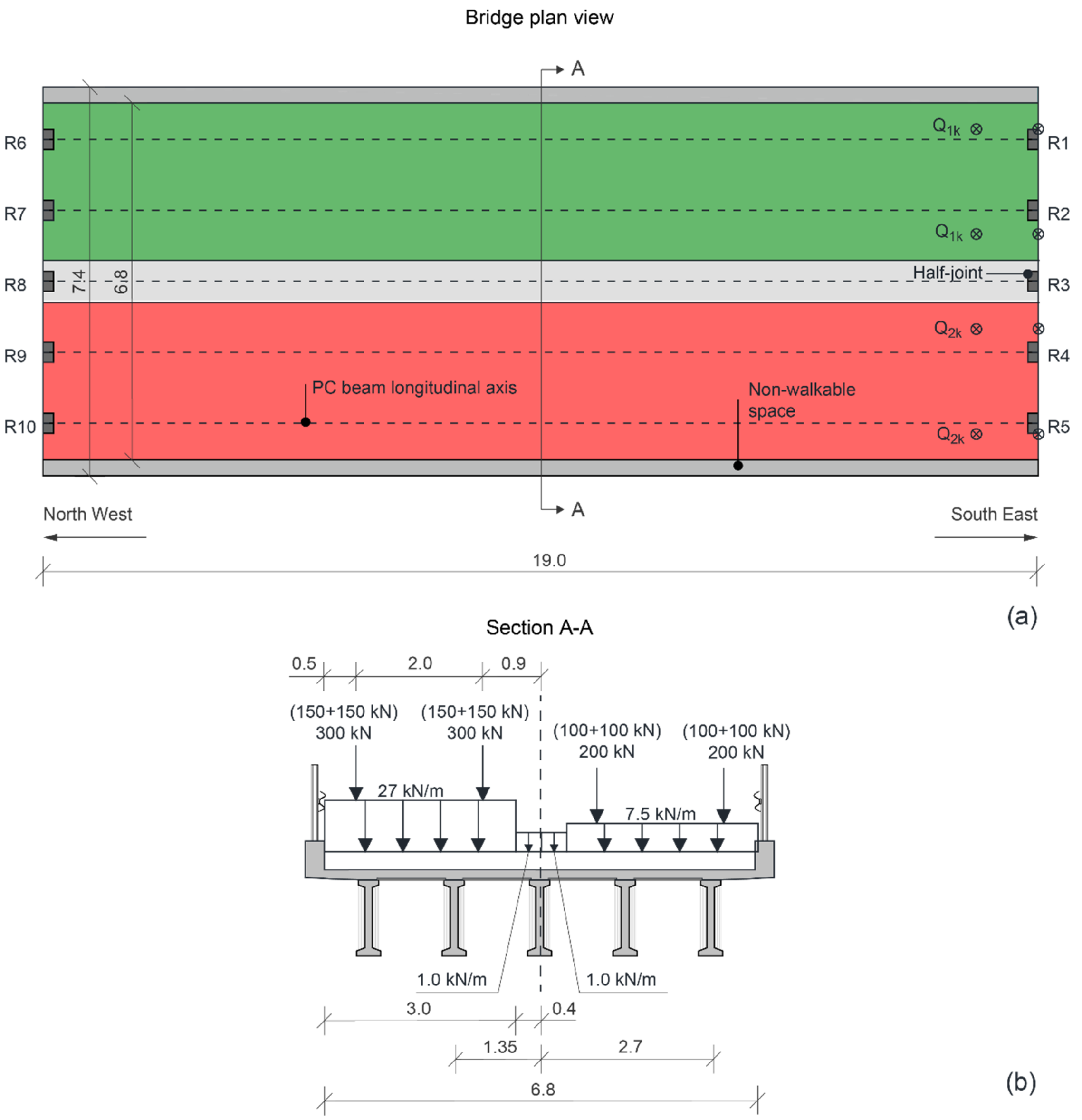

Figure 2. Loads on the central suspended span according to NTC 2018 (Infrastructure Ministerial Decree 2018): (a) plan view and (b) vertical cross-section.

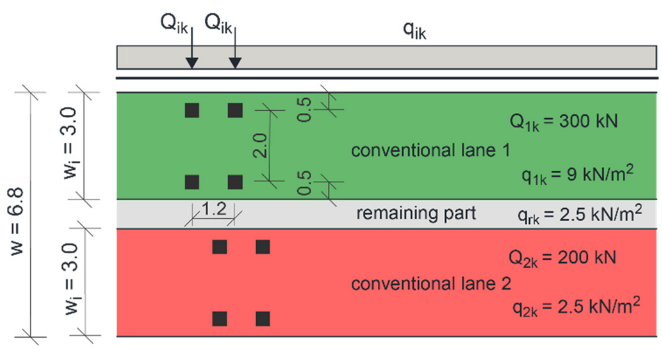

Figure 3. Dimension of conventional lanes and loading scheme according to NTC 2018 (Infrastructure Ministerial Decree 2018).

3.3 *Half-joint resistance*

Equation (1) involves calculating the ultimate capacity of the half-joints ($F_{k,ult}$), which are considered the critical element of the entire system. The estimation of $F_{k,ult}$ is based on the strut-and-tie models presented in di Prisco et al. (2023) which reproduced the half-joint geometry and reinforcement details. On this last aspect, an as-built reinforcement situation with nominal bar diameters is assumed. The ultimate load for each half-joint is then estimated at 578.5 kN.

4 FINITE ELEMENT MODELS

The robustness quantification is carried out using linear elastic finite element (FE) models. The basic – intact – model used in this study is the same as that adopted in di Prisco et al. (2023) and named 'num. 2A'. This FE model is a 3D simplified model in which the side spans are not reproduced. This model is therefore only used to conduct analyses on the bridge central span. The side spans are assimilated to elastic constraints for the central span according to the loading scheme shown in Figure 4a. Such a loading scheme provides a stiffness k equal to 6.06E7 N/m. Values of elastic constants for PC beams, transversal RC beams, and top RC slab are here omitted but are given in di Prisco et al. (2023).

Elastic constraints (translational springs in the Y direction) are applied to the ends of the beams of the central span as shown in Figure 4b. A 3D mesh view of the simplified FE model is shown in Figure 4c. Two types of 3D finite elements are used for different regions of the bridge: the top RC slab is discretized with shell-type elements, while the longitudinal and transversal beams are discretized as linear beam-type elements.

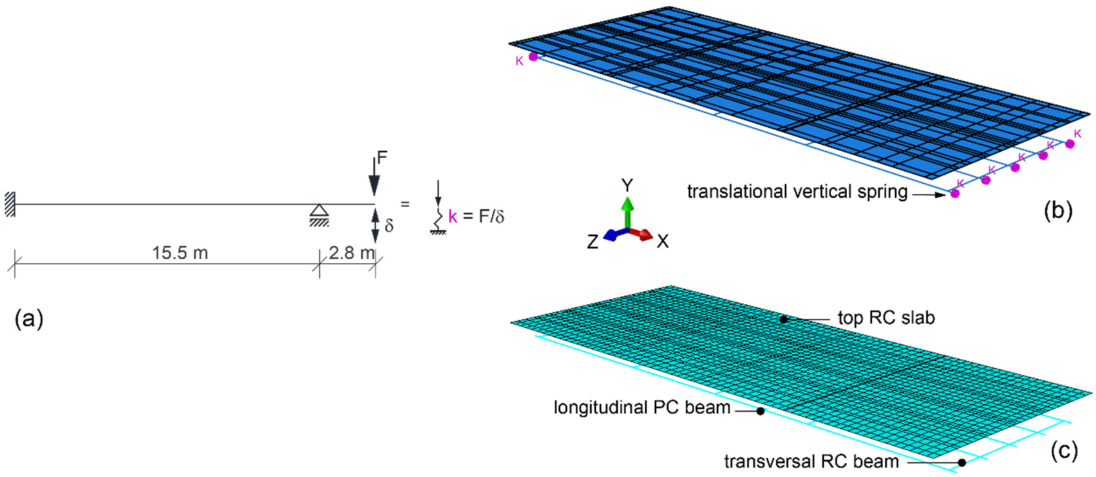

Figure 4. Simplified FE model: (a) lateral span loading scheme used for estimating the stiffness (k) of half-joints (b) details of the boundary conditions and (c) mesh view (adapted from di Prisco et al. 2018).

5 NOTIONAL REMOVAL APPROACH FOR HALF-JOINT BRIDGES

Design from actions resulting from unspecified hazards can be treated with a notional damage scenarios approach. According to the draft version of the *fib* Model Code 2020 document, notional damage scenarios can include (i) notional deterioration scenarios or (ii) notional removal scenarios.

In case (i), the geometrical and/or material properties of one or more structural elements are notionally reduced and the structure is checked for disproportionate consequences.

In case (ii), structural elements are notionally removed and the structure is checked for disproportionate consequences, for example by conducting an alternative load path design. Columns (one or more), panels/walls or nominal wall length (one or more) and any other elements judged vital to the structural performance are among the structural elements that can be notionally removed.

Notional removal scenarios – usually considered to be related to the failure of a connection or structural member due to an unidentified hazard – have been largely applied to evaluate the robustness of buildings (see for example Rodriguez et al. 2021 and Martinelli et al. 2022, among the others). The notional removal scenarios strategy is here applied to the bridge under study by removing the half-joints, considered to be the most critical elements of the entire bridge (di Prisco et al. 2023). It is worth mentioning that the notional removal scenarios approach herein adopted is more conservative than the notional deterioration scenarios approach.

The scenarios that will be considered include the removal of a single half-joint at a time. Given the symmetry of the loads described in §3.2 with respect to the centreline of the bridge, it will be sufficient to consider the loss of the half-joints R1-R5 (loss of half-joint R1, loss of half-joint R2, etc.).

6 RESULTS AND DISCUSSION

The simplified FE models described in §4 allow the computation of the index R_r (Starossek 2017) with the load configurations described in §3.2. Figure 5 shows the reactions in half-joints R1-R5 in the case of intact situation and for the half-joint loss scenarios described in §5. Figure 5a considers the loads on lane 1, whilst Figure 5b considers the loads on both lanes 1 and 2. As one would logically expect, the reaction in the removed half-joint is nil. The most severe condition occurs when the end half-joint R1 in lane 1 is removed. In this condition, the adjacent half-joint R2 registers the maximum reaction. For both load configurations (Fig. 5), another severe load condition compared to the intact condition is the loss of the half-joint R2. When loads are applied on both lanes (Fig. 5b), another severe load condition compared to the intact condition is the loss of the other end half-joint R5. The loss of centre half-joints allows a more homogenous redistribution of reactions, making these scenarios less critical than the loss of end half-joints.

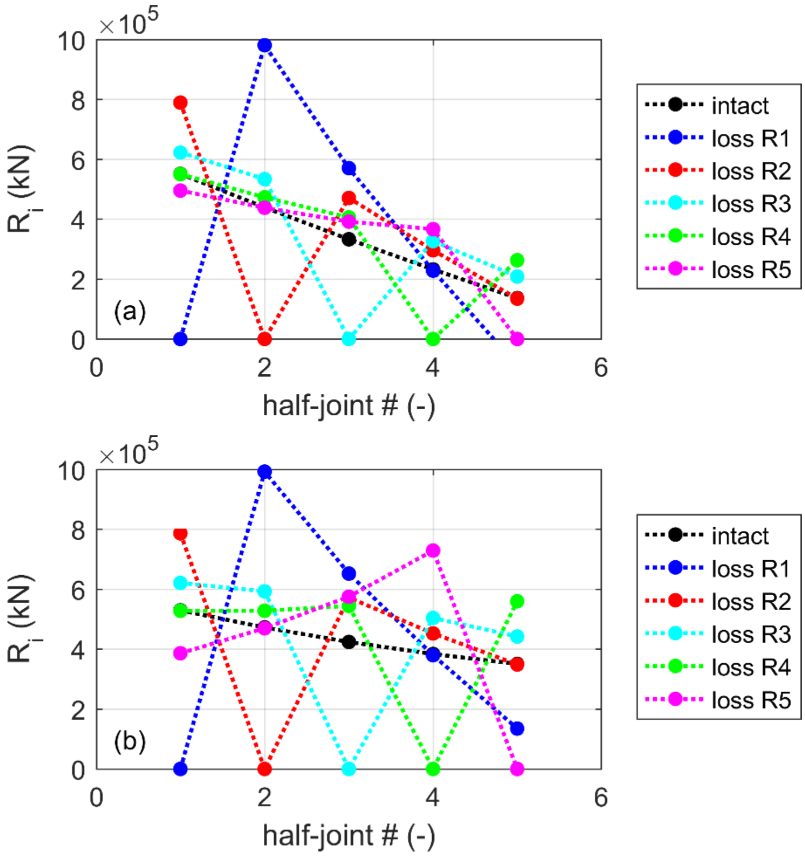

Figure 5. Reactions in half-joints R1-R5 in case of intact situation and for several half-joint loss scenarios: (a) loads on lane 1 and (b) loads on lanes 1 and 2.

Figure 6 shows the index R_r calculated with eq. (1) for several half-joint loss scenarios. Regarding loads applied on lane 1 only, positive values of R_r are obtained when half-joints R4 or R5 are removed. Looking at the load configuration where loads are applied on both lanes, positive values of R_r are obtained only when half-joint R4 is removed. In both load situations and in agreement with the results of Figure 5, the most unfavourable situation is the loss of the half-joint R1.

While the results in Figure 6 show a lack of robustness for most of the scenarios considered (negative values of R_r), it should be remembered that the unit load multiplier ($\alpha = 1$) applied to the load configurations described in §3.2 is very severe. Concerning the load configuration involving loads on both lanes, the ratio between the live loads calculated according to Circular 14 February 1962 n.384 (1962) – and for which the bridge was designed – and according to NTC 2018 is 0.66. Applying a load multiplier equal to this ratio ($\alpha = 0.66$), Figure 7 compares the values of R_r for loads applied on lanes 1 and 2 and load multipliers $\alpha = 1$ and $\alpha = 0.66$. The two curves show very similar trends but with higher values of R_r for $\alpha = 0.66$. This case shows how the loss of central half-joints R3 or R4 – with positive values of R_r – does not lead to a propagation of failure. The most unfavourable situation is the loss of the end half-joint R1.

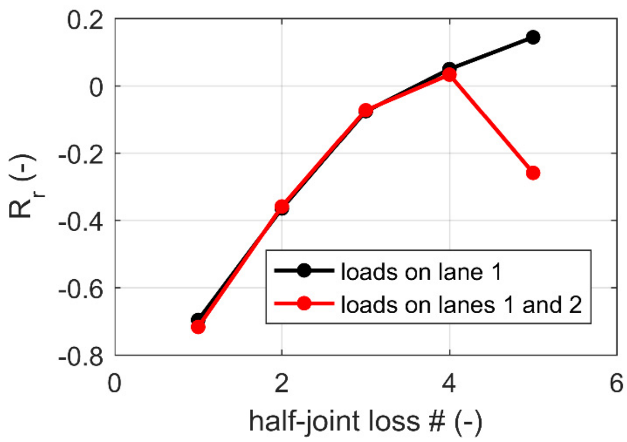

Figure 6. Index R_r (Starossek 2017) for loads applied on lane 1 only and loads applied on lanes 1 and 2.

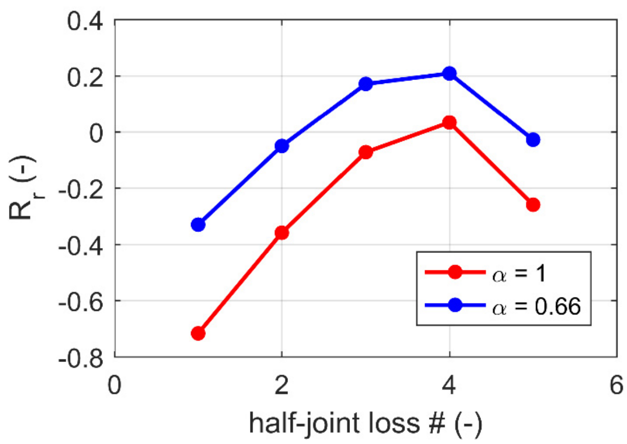

Figure 7. Index R_r (Starossek 2017) for loads applied on lanes 1 and 2 and load multipliers $\alpha = 1$ and $\alpha = 0.66$.

7 CONCLUSIONS

In this paper, a selected measure of robustness – typically adopted for buildings – is applied to a particular type of reinforced concrete (RC) girder bridge, namely half-joint bridges. A simple methodology based on the notional removal of half-joints – which represent the critical elements for this type of bridge – is applied. The analysis of the structural system under notional removal scenarios is herein executed through linear static analyses where dynamic effects are neglected.

The Annone viaduct, an RC half-joint bridge that collapsed in 2016 after the passage of a heavy truck, is used as a case study. Using two of the heaviest load combinations in accordance with the current standard NTC 2018, the following conclusions can be drawn.

The results obtained show greater robustness when central half-joints are removed compared to when end half-joints are removed. In the former case, the loss of a half-joint is followed by a redistribution of actions in the adjacent Gerber saddles that do not propagate the initial damage. Conversely, the loss of an end half-joint results in an absence of robustness for both load configurations considered. This result is in line with the prescriptions adopted for some bridges whereby the transit of exceptional heavy trucks must take place in the centre of the carriageway. This requirement was also foreseen for the Annone bridge, but it was unfortunately disregarded by the heavy truck that then triggered first the rupture of the half-joint R1 and then the collapse of the entire bridge.

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