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Robustness and redundancy finite element analysis of composite deck bridges

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Abstract. Structural robustness in bridges is a topic that still lacks consensus in both the literature and international standards such as AASHTO, ASCE, and Eurocodes. These standards do not provide clear guidelines for the application and validation of robustness and redundancy concepts in structures. Considering the need to prevent disproportionate or progressive collapse in bridges, the impact of these concepts on structural safety must be studied and included in the design phase. This paper proposes a finite element model of the deck of a multi-girder composite bridge composed of four simply supported I-beam girders with solid-web sections. The analysis, based on the conventional criteria of the ABNT NBR 16694:2020 standard and assuming linear-elastic behavior, introduces a discontinuity in the 3D element representing the girder to simulate a fracture in the mid-span region. Based on this premise, the study evaluates the redistribution of forces and the maximum vertical displacement of the girders under critical moving load conditions. The results are compared with theoretical definitions of robustness and redundancy, contributing to the enhancement of analytical methods in structural engineering.

Keywords: Robustness, redundancy, bridge, steel, composite deck.

1 Introduction

Throughout history, the causes and mechanisms of bridge collapse have attracted significant attention from researchers and engineers, with valuable knowledge and design experience gained from lessons learned through actual bridge failures [1]. Interest in this topic runs parallel to the increasingly strategic role that bridges play in territorial connectivity, the logistics of goods and people, and, consequently, the economic development of both developed and developing countries [2], [3]. The continuity of transportation services and the integrity of roadway networks directly depend on the structural performance of these structures under various natural and anthropogenic demands [1], as well as the progressive degradation of materials over time [4]. Events such as floods, earthquakes, extreme winds, and ground instabilities represent significant threats to the integrity of bridge foundations and superstructures [1]. In parallel, design flaws, inadequate construction practices, the use of low-quality materials, and accidental or intentional impacts constitute anthropogenic risks, especially in aging structures or those with insufficient maintenance [1]. Added to these are natural degradation mechanisms— such as corrosion, carbonation, and fatigue — which gradually compromise the load-carrying capacity and durability of bridges [4]. Within this context, preventing structural collapses becomes a technical and societal imperative, capable of significantly impacting public safety, system functionality, and the economic burden of highway infrastructure [1].
The structural efficiency of a bridge is not limited to its capacity to withstand immediate loads [4]. The redistribution of internal forces among structural elements, particularly under conditions of localized damage, is a key factor in determining its resilience [5]. In this regard, the concepts of robustness and redundancy assume a central role, as they promote the development of structures that are more tolerant of localized failures and capable of preventing progressive and disproportionate collapse [6]. However, the mitigation of such phenomena still lacks

consolidated guidance in current design codes, highlighting the need for more advanced and specific design strategies.

Regarding the concept of robustness, it is observed that, to date, there is no consolidated consensus, either in terminological terms or in relation to methodological procedures, within the specialized technical literature [7]. Although international standards, such as those issued by AASHTO, ASCE, and the Eurocodes, address this concept, they do not provide sufficiently detailed practical guidelines to enable its consistent application in structural analysis models. For example, in the *AASHTO LRFD Bridge Design Specifications* (2024) [8], fundamental aspects of structural modeling—such as the definition of loading scenarios, the consideration of dynamic effects resulting from failures, the stiffness assigned to numerical models, and the selection of structural elements— are delegated to negotiation between the designer and the bridge owner. This flexibility can compromise the uniformity and reliability of structural design in the face of extreme events.

The concept of structural robustness gained prominence after events such as the Ronan Point Tower collapse in 1968, which exposed the vulnerability of structures to accidental actions [7]. *Eurocode 1* (1994) defines robustness as the ability to withstand extreme events without disproportionate damage relative to the original cause, emphasizing the importance of continuity and ductility as essential properties for preventing the propagation of localized failures into global collapses [4]. Although ductility allows for plastic deformations without loss of stability, its effectiveness depends on the interaction among all structural components [7]. Robustness, therefore, is not associated with overdesign, but with the capacity for load redistribution and an insensitivity to localized damage, favors ductile failure modes with progressive load redistribution [5].

In the case of bridges, applying the concept of robustness requires a differentiated approach, given their predominantly series-type behavior, where the failure of a single element can be potentially critical for the entire structure [7]. Robustness assessment should start with an analysis of the structure's criticality and the consequences of its collapse [7], weighing the technical and economic feasibility of its implementation. Adopting a systemic approach—decomposing the structure into subsystems and identifying alternative load redistribution paths—is fundamental to quantifying robustness under diverse exposure scenarios [7]. The main challenge lies in formulating objective design and assessment criteria that reflect the diversity of typologies and ensure performance levels appropriate to roadway infrastructure safety [7].

The concept of structural redundancy gained significant attention after the collapse of the Silver Bridge in Point Pleasant, over the Ohio River, in 1967. The failure was caused by the brittle fracture of a non-redundant eyebar, highlighting the vulnerability of systems lacking load redistribution capacity. Following this collapse, the concept of *Fracture Critical Member* (FCM) was introduced, leading to the publication of the first Fracture Control Plan by AASHTO in 1978 [13]. Redundancy is a fundamental characteristic of complex structures, such as bridges, as it provides safety, reliability, and strength against localized failures [7]. According to the *National Steel Bridge Alliance* (NSBA), continuous structures should make use of their multiple load paths unless otherwise justified—a position supported by standards such as the *AASHTO LRFD Bridge Design Specifications* [8] and the *Manual for Bridge Evaluation* [9], which defines redundancy as the ability to maintain functionality even in a damaged condition. However, the *Steel Bridge Design Handbook* [10] itself acknowledges the absence of a universally accepted definition for redundancy. In practice, redundancy can be observed, for example, when a steel girder damaged by fatigue or impact does not compromise the global stability of the bridge due to the presence of alternative load paths [8]. Its application, however, is limited by technical and economic factors, especially in systems with low geometric or structural redundancy, such as suspension or truss bridges [2].

To address this complexity, *Title 23 Code of Federal Regulations* (CFR), Part 650.305, Subpart C – *National Bridge Inspection Standards* (NBIS) [11] adopted, in 2022, an operational classification of redundancy into three categories: (a) Load path redundancy, based on the existence of multiple primary supporting members capable of redistributing loads after failures; (b) Structural or system redundancy, which considers the global stability of the superstructure even after the failure of individual components, as in the case of continuous girders; and (c) Internal redundancy, related to the capacity of a single structural member to resist the propagation of fractures through appropriate cross-sectional detailing. This segmentation allows for more accurate analyses and contributes to the standardization of design, inspection, and rehabilitation criteria. Considering redundancy throughout the entire life cycle of a bridge is especially critical for elements classified as *fracture-critical members* (FCMs), common in steel structures, given their vulnerability to brittle fracture.

Beyond the regulatory challenges outlined above, bridge design is an iterative process that combines creativity with technical rigor in the pursuit of structurally viable, economically sustainable, and functionally appropriate

solutions [12]. While modern structural calculations incorporate safety factors and conservative assumptions that enhance structural reliability [12], the operational reality of such structures imposes additional challenges: continuous exposure to harsh environmental actions, the cumulative effect of repetitive loads, and the possibility of localized damage—often not immediately detectable or repairable.

Modeling the structural response under localized damage scenarios, such as cracking of one of the main elements of a bridge, represents a particularly challenging gap in the field of structural engineering [11]. In theory, systems composed of decks with simply supported girders have the capacity to redistribute loads through the interaction among girders, slabs, and bracing systems; however, the effectiveness of this mechanism depends on specific connection conditions that, in conventional design studies, are not always thoroughly assessed [13].

Given this context, this paper proposes to investigate, through three-dimensional numerical simulations using the finite element method in ANSYS software version 2025 R2, the structural response of a deck composed of simply supported girders, designed in accordance with [14] and [15] standards, when subjected to localized damage at midspan of one of the longitudinal girders.

The occurrence of a fracture in the longitudinal girder is attributed to variations in fatigue-induced stresses, leading to a reduction in its load-carrying capacity. This condition necessitates the redistribution of load demands to other structural members to maintain operational functionality until repair, thereby preventing collapse. In the present analysis, the spatial structural system—including girders, deck slab, and vertical bracing—is modeled under a linear-elastic assumption, with the aim of evaluating the robustness and redundancy of a conventionally designed bridge system. The study further seeks to quantify the residual load-carrying capacity and assess the associated risk of progressive collapse.

By integrating the theory of load redistribution and the normative criteria with the practical study of a critical scenario, this work contributes to the technical-scientific advancement of bridge engineering, providing a basis for improving modeling considerations in design and contextualizing such concepts to the Brazilian reality, characterized by specific challenges related to maintenance, inspection, and structural safety.

Following the Introduction, Section 2 provides a detailed description of the geometry of the deck components used in the numerical analysis, including the longitudinal girders, the angles forming the trussed floor beams, and the plates used to secure the bracing members. The modeling parameters are also presented, encompassing the boundary conditions, the adopted code-based assumptions, the procedure for determining the applied loads and their placement on the structure, as well as the mesh discretization and other relevant aspects of the Finite Element (FEM) model. Section 3 presents the analysis of the results obtained from the computational simulations, correlating them with concepts established in the technical literature, the requirements of applicable standards, and the observed structural responses. Finally, Section 4 summarizes the conclusions reached and provides recommendations for the continuation and further development of the research.

2 Structural Characterization and Load Modeling

The superstructure considered in this study consists of a reinforced concrete deck with a thickness of 30 cm, supported by four longitudinal steel girders spaced 3,5 m apart. The geometric properties of these girders are shown in Figure 1 and comply with the requirements of [14]. The system's transverse stiffness is ensured by the use of vertical diaphragms spaced regularly at 5,60 m intervals placed along the entire bridge length. It is worth noting that the structural design of the deck follows the guidelines of [14], but its detailing is beyond the scope of this study and is therefore omitted.

The analyzed bridge has a total length of 40 m and a width of 12,80 m, in accordance with the standards established by the Brazilian National Department of Transport Infrastructure (DNIT) [16] for Class II highway bridges. The cross-section includes two traffic lanes and lateral shoulders, without pedestrian walkways, as illustrated in Figure 1. The selection of a 40-meter span is justified not only by the applicability of the homogenized vehicular load model, as allowed by [15] for spans equal to or greater than 30 meters, but also by the economic competitiveness of the composite steel–concrete section compared to a full concrete solution beyond this span length. The vehicle load considered corresponds to the TB-450 load train, which simulates a standard vehicle composed of three axles spaced 1.5 m apart, with 75 kN applied at each wheel. The vehicle occupies an area of 18 m², additionally subjected to a uniformly distributed load of 5 kN/m², in accordance with the aforementioned standard.

The contact area of each wheel was determined according to [15], using a 45 degree spread angle from the top of

the asphalt layer to mid-depth of the concrete deck slab. For this calculation, a 10 cm thick asphalt pavement layer with a specific weight of 24 kN/m³ was considered, along with an additional resurfacing load of 2 kN/m², as required by [14]. The final spread area corresponding to each wheel was calculated as 1,0 × 0,7 m, differing from the original nominal area of 0,5 × 0,2 m, Figure 2.

In the numerical model, non-structural elements such as concrete parapets and the asphalt layer were modeled exclusively as permanent loads and not as three-dimensional elements. This modeling choice aims to prevent interference in the stress redistribution results caused by the moving loads, as such components are not considered structural members according to Brazilian bridge design standards.

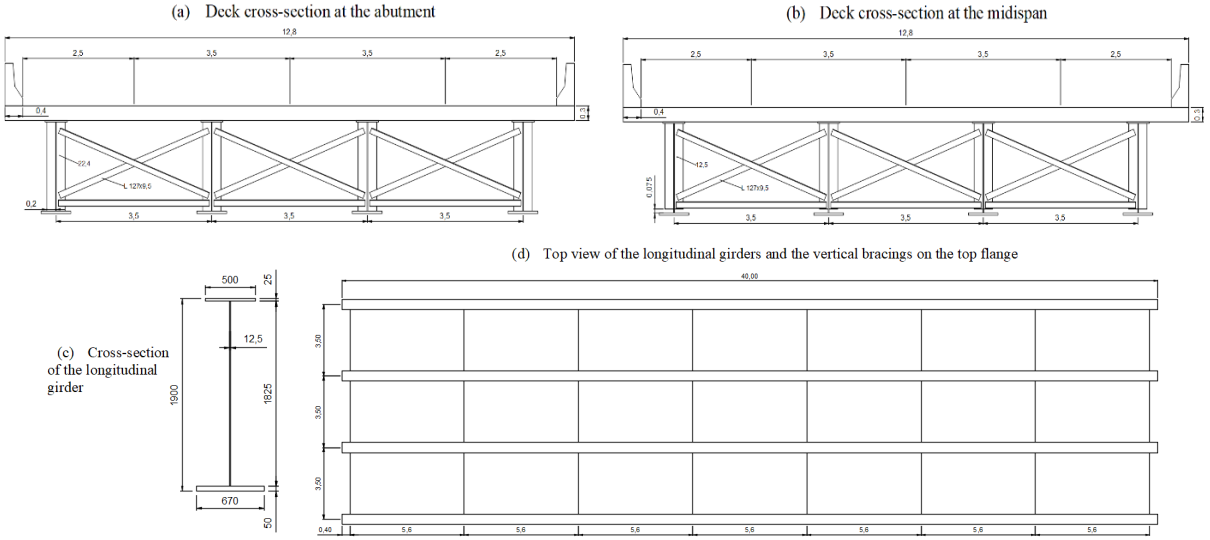


Figure 1. Main cross-sections of the bridge deck - (a) dimensions in millimeters, (b) and (c) dimensions in meters

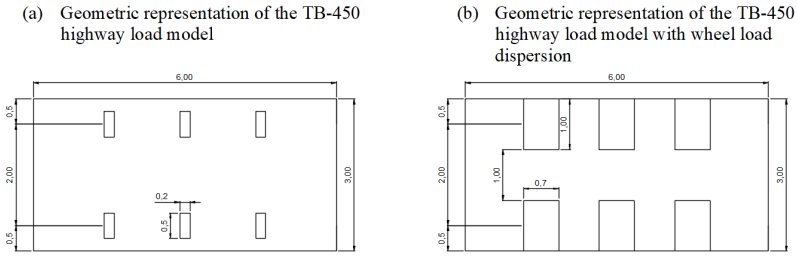


Figure 2. Geometric representation of the TB-450 highway live load for the normal and dispersed configurations (dimensions in meters)

For the analysis of structural robustness and redundancy during the design phase, live loads were amplified using the normative coefficients defined in [15], namely: the Vertical Impact Coefficient (CIV) and the Lane Number Coefficient (CNF). The Additional Impact Coefficient (CIA), in turn, was not applied, as it is a factor used to increase the characteristic live load due to imperfections or discontinuities in the roadway surface, such as expansion joints and structure ends. The application of these factors aims to realistically simulate, from the static load, the dynamic effect of the moving load and the effective distribution of live loads in the structural model. The calculation methodology of these coefficients is beyond the scope of this study and is therefore omitted; however, the loads applied in the numerical model are presented below in Table 1.

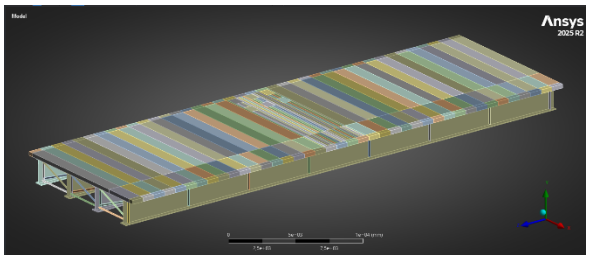
It is noteworthy that the deck slab was specified with concrete having a characteristic compressive strength of f_{ck} = 35 MPa, in compliance with the requirements of [14], which establishes a minimum value greater than 30 MPa. For the longitudinal girders, crossbeams, and plates, ASTM A572 Grade 50 steel was adopted, with a yield strength of f_y = 345 MPa and an ultimate tensile strength of f_u = 450 MPa, also meeting the limits set forth in [14].

Table 1. Considered loads and their magnitudes

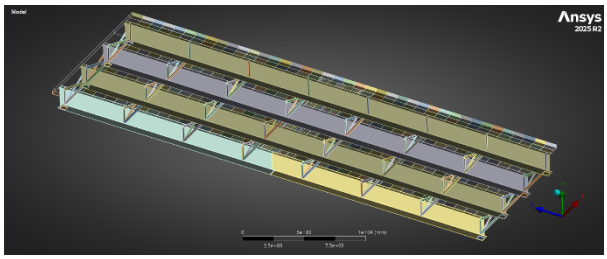
Load	Magnitude
Asphalt surfacing and resurfacing (thickness according to DNIT Pavement Manual)	4,4 kN/m^2
Concrete parapet	14,468 kN/m^2
Q = P.CIV.CNF = 60×1,236×1	74,16kN/car wheel
q = p.CIV.CNF = 5×1,2361x1	6,18 kN/m^2

The three-dimensional modeling of the superstructure was initially developed in AutoCAD 2025 and subsequently exported to the ANSYS 2025 R2 simulation environment, where the structural analyses were performed. Considering the bridge's symmetry in both plan and elevation, and following the premise established by [15] that the live load must be positioned on the roadway with the wheels in the most unfavorable configuration — including the shoulder and safety lanes —, a discontinuity was introduced at midspan of the outermost longitudinal girder of the deck. This discontinuity simulated a failure caused by stress variation (fatigue), with the live load being longitudinally centered over this failure (Figure 3) and positioned so that the affected girder carried the majority of the live load resulting from the six wheels of the design vehicle, in addition to the corresponding portion of the pedestrian (crowd) load.

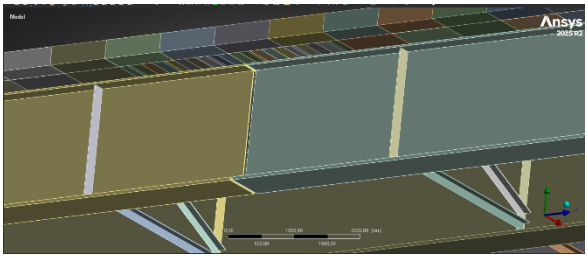
As illustrated in Figure 3, the concrete deck was subdivided in its three-dimensional geometry to enable the generation of a **uniformly distributed quadrilateral finite element mesh**. Preliminary tests were conducted to assess the influence of mesh refinement, and it was found that the smallest feasible element size, considering the available computational capacity, was **100 mm** for the slabs and longitudinal girders, and **50 mm** for the cross girders and stiffeners. However, the final model adopted in this analysis employed a mesh with elements measuring **200 mm** for the slabs and longitudinal girders and **100 mm** for the cross girders and stiffeners. The difference in vertical displacements between the evaluated mesh sizes was only **0.31%**, indicating adequate convergence of the results.



(a) Top view of the deck highlighting the wheel spread area of the TB-450 vehicle and the slab segmentation performed to assist in mesh generation.



(b) Botton view of the deck highlighting the bracings.



(c) Detail showing the facture (discontinuity of the longitudinal girder) imposed on the numerical model

Figure 3. Positioning of the design vehicle, fracture on the deck and botton view of the deck

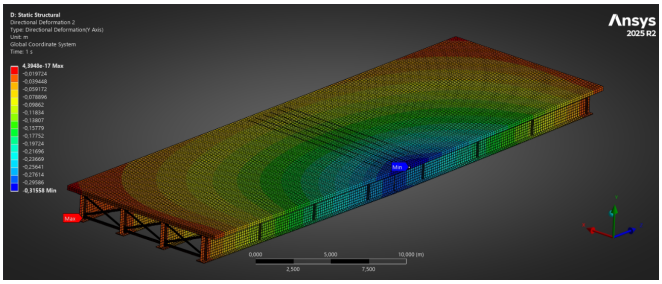
To enable the assessment of structural redundancy, the requirement established by the FHWA [13] was followed, which specifies that, for redundancy evaluation purposes, all three components of the girder cross-section — the tension flange, the web, and the compression flange — must be considered fractured. The defect thickness was set at 50 mm, a value determined based on the bottom flange thickness itself. To assess the redistribution of internal forces among the structural elements, a bonded contact condition was exclusively applied. This type of contact permanently joins two distinct parts, preventing both separation and relative sliding. Such a configuration simulates the condition of components that are welded or permanently fixed, ensuring rigid behavior and enabling a linear solution, since the contact area and length remain unchanged throughout the loading process.

Finally, it should be noted that the live loads — corresponding to the design vehicle and the pedestrian (crowd)

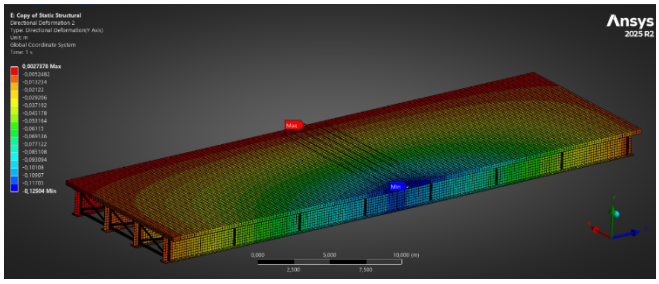
load — were distributed over the superstructure, during the design phase, according to the influence lines of the deck cross-section specific to each girder. Considering that the analysis of load redistribution in the event of a fracture is proposed at the design stage, with the purpose of evaluating the efficiency of a structure designed in accordance with national standards regarding robustness and redundancy, the live load applied in the numerical models used in this study followed the influence line corresponding to the fractured girder. The subdivisions shown in the images of Figure 3 are intended to standardize the finite element mesh generated by the software, ensuring greater uniformity and accuracy in the results obtained.

3 Computational results

Once the numerical model was developed, the vertical displacements of the bridge deck were evaluated under two distinct loading conditions. In the first condition, all permanent loads were considered — including the self-weight of the structure, the weight of the asphalt pavement, and the curb units — in addition to the live load corresponding to the design vehicle and pedestrian (crowd) load. In the second condition, the permanent loads were disregarded, and only the vertical displacement resulting from the live load application was evaluated. In both cases, the live load was distributed according to the influence line of the deck cross-section corresponding to the outermost longitudinal girder, where the fracture was imposed. It is important to emphasize that the purpose of this analysis was not the structural design of the bridge deck, but rather the evaluation of the numerical model's response under a damaged condition, aiming at the subsequent assessment of the concepts of robustness and redundancy.
According to Annex C of [14], the maximum allowable vertical deflection for the deck under live load is 49 mm, obtained by dividing the span length between supports by 800. The numerical results showed that the deck with the imposed fracture, considering all permanent loads and neglecting any camber effects, exhibited a maximum vertical displacement of 294,57 mm in the fractured longitudinal girder. For the condition involving only the live loads, the maximum displacement observed was 115,35 mm for the same girder.



(a) Total deck displacement considering permanent and live loads.



(b) Total deck displacement considering only live loads.

Figure 4. Displacement of bridge decks

Table 2. Displacements considered per girder

Load	Magnitude
Failed girder	294,57mm
First intact girder	192,31mm
Second intact girder	132,82mm
Third intact girder	83,4mm

Table 3. Displacements considered per girder

Load	Magnitude
Failed girder	115,35mm
First intact girder	67,67mm
Second intact girder	36,62mm
Third intact girder	9mm

These values contrast significantly with the allowable displacements of the intact deck subjected to the same load configuration. It was observed that the fracture, located in the girder positioned directly beneath the vehicle load, not only compromised the integrity of that element but also induced additional demands on the adjacent girders. These neighboring members exhibited high vertical displacements, indicating that, although the bracing system in combination with the deck slab allowed the activation of alternative load redistribution paths — a characteristic of structurally robust systems — the remaining members did not possess sufficient stiffness or strength to meet serviceability limit state requirements. Therefore, it can be concluded that, although the structure exhibited

behavior indicative of robustness, it lacks effective redundancy, making it susceptible to a significant loss of performance in the event of a localized failure.

4 Conclusions

The numerical model demonstrated that the Brazilian standards [14] and [15] do not, independently, establish adequate requirements to ensure structural robustness and redundancy in composite bridge decks. This deficiency is confirmed by the insufficient capacity of the remaining elements to absorb the incremental load resulting from the load redistribution following a failure scenario. The inability of the structural components to withstand this additional load represents a failure to meet the robustness criteria and the three typologies of redundancy previously addressed. To ensure the deck's post-damage operationality, it is imperative that the live load be repositioned and safety verified considering the use of two traffic lanes, based on influence line analysis. It is emphasized that the stress analysis in the steel elements was disregarded in this study. The justification resides in the understanding that, given the high displacements observed, the composite action (structural integrity) between the slab and the girders, which is fundamental for the numerical model formulation, would not be maintained in the actual structure, thus invalidating the stress results provided by the software.
It is suggested, for subsequent studies, to implement models that incorporate the consideration of hinged (or pinned) connections between the bracings and the main longitudinal girders. This modification aims to enhance the fidelity of the numerical simulation concerning the actual structural behavior. Such a methodological refinement will enable a more accurate quantification of load redistribution and offer robust technical support for the improvement of current Brazilian standards, notably regarding the explicit incorporation of robustness and redundancy criteria into bridge design.

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