18TH WORLD CONFERENCE ON EARTHQUAKE ENGINEERING
WCEE2024
MILAN

MILAN, ITALY
30TH JUNE - 5TH JULY 2024

www.wcee2024.it

CYCLIC TESTS AND RETROFIT STRATEGIES FOR PRECAST TWO-COLUMN BENTS OF RAILWAY BRIDGES

L. Carvalho[1], M. Pimentel[1], P. Delgado[2,1], A. Arêde[1], N. Vila Pouca[1] & J.R. Pinto[3]

[1] CONSTRUCT, Faculty of Engineering, University of Porto, Porto, Portugal, lucasc@fe.up.pt

[2] proMetheus, Instituto Politécnico de Viana do Castelo, Viana do Castelo, Portugal

[3] Mota-Engil, Engenharia e Construção S.A, Porto, Portugal

Abstract: *The use of precast structures in regions with high seismic activity is uncommon due to uncertainties in their seismic performance, particularly regarding column-to-cap-beam connections and the anchoring of column longitudinal bars into the cap beam or foundation. The construction of these joints often becomes impractical due to the high concentration of reinforcement required by current standards. Additionally, some design rules may be unclear enough for practical application and fail to establish a clear connection to the force flow within the joint. This case study focuses on the design of precast two-column bents for railway bridges in southern Europe. Nonlinear numerical models were developed to validate the strut-and-tie models used in the design process. The numerical results will be further confirmed through an experimental campaign, aimed at assessing the structural performance of the proposed solution, which consists of cyclic lateral loading tests with increasing displacements. Finally, a study of retrofit strategies for the substructure system will be performed, for the case in which the seismic action experienced by the structure is greater than expected in design. These strategies are based on the damage results observed on the structure, obtained from the numerical simulation and experimental campaign.*

1 Introduction

The pursuit of an enhanced construction process for bridges, aimed at ensuring the safety of workers while improving construction within the shortest possible timeframe to minimize traffic disruptions and costs, has been crucial. This objective is met through the implementation of industrialization in the construction process. Precast structures allow and show the greatest potential for achieving these goals. However, there are still uncertainties regarding their application in regions characterized by medium to high seismicity.

The connection zones are the key concern in the design and construction of precast bridge substructures in seismic zones, specifically connections between the column and the cap beam, as well as between the column and the foundation. Current standards require a significant amount of transverse reinforcement within these zones, resulting in reinforcement congestion and constructability challenges. Addressing these issues, this paper presents a fully precast columns and cap beams solution designed for railway viaducts. The seismic considerations for design encompass a maximum elastic spectral acceleration in the range of 0.7g to 0.75g for Type I and Type II, respectively. Moreover, in order to exploit different ductility classes, behavior factors ranging from q=3.0 to q=4.5 are under consideration.

The design of case study adheres to capacity design principles, ensuring energy dissipation at regions experiencing maximum flexural moments in the columns, avoiding brittle failure modes and safeguards the integrity of the cap beam supporting the deck. To further enhance understanding on the force transfer mechanisms in the column-beam joint zone and aiming to provide a more effective constructive arrangements while ensuring reliable forces paths, strut-and-tie models were developed to guide the cap beam design Carvalho, *et. al* (2023).

A preliminary numerical investigation, employing three-dimensional nonlinear finite element analyses, was conducted to evaluate force redistribution, confirm failure modes, and quantify the ductility and energy dissipation capacity of the proposed solution. Moreover, an experimental campaign based on cyclic loads with uniaxial directions is underway to confirm the design assumptions. In addition, structural retrofit strategies will be developed based on the damages observed in these experimental tests, for the case in which the seismic event experienced by the structure exceeds that foreseen in design.

2 Case Study

2.1 General

A double-track railway viaduct with 6 spans of 22m+4x30m+22m, see Figure 1, provides the case study for the solution developed. The continuous deck is bearing supported on 5 intermediate cap beams with two columns each. As shown in Figure 2, the columns are 10m high between the top of the foundation and the base of the cap beam. The deck is constituted by 4 precast longitudinal U-beams in each span. It was considered that the support devices on the cap beam do not transmit longitudinal forces. Thus, the substructure system is in charge of seismic action resistance in the transverse direction, being the transverse forces from the deck transferred through a corbel placed at the centre of the cap beam. The longitudinal seismic forces are balanced at the abutments by dedicated dissipative devices.

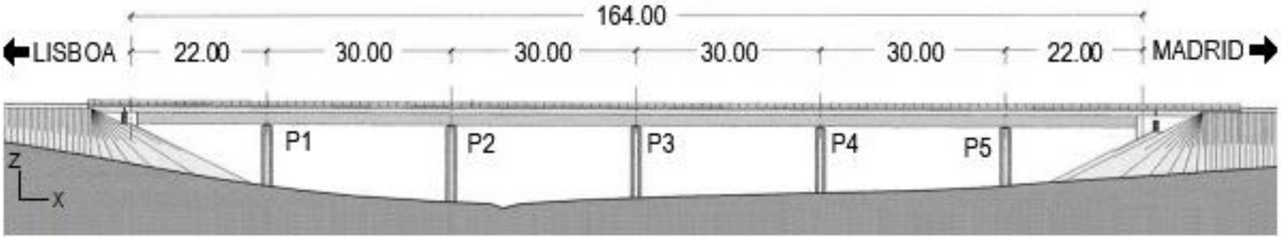


Figure 1. Analyzed Viaduct.

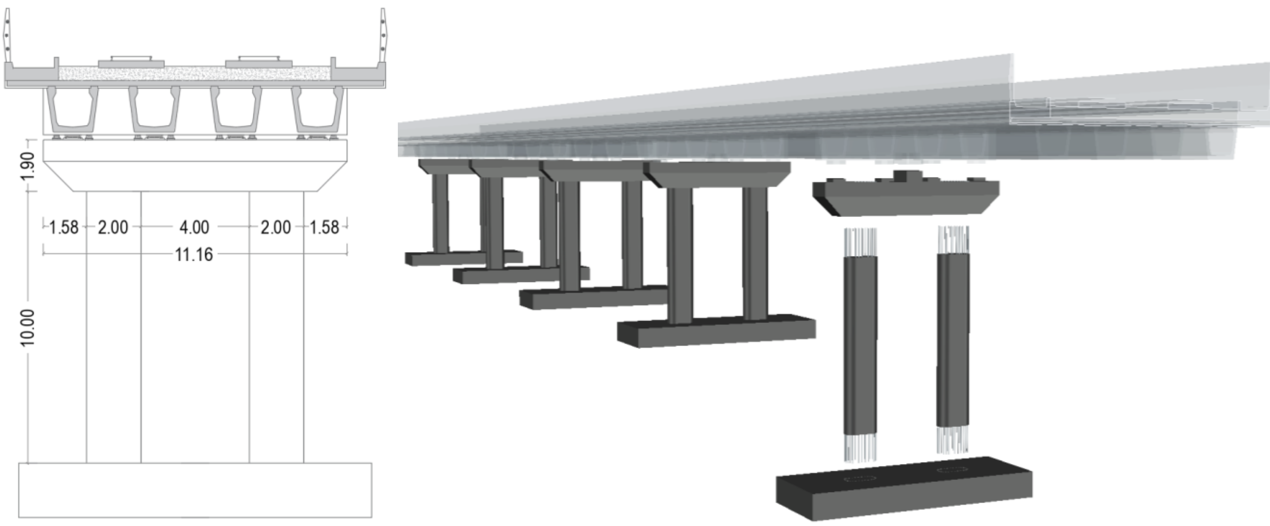
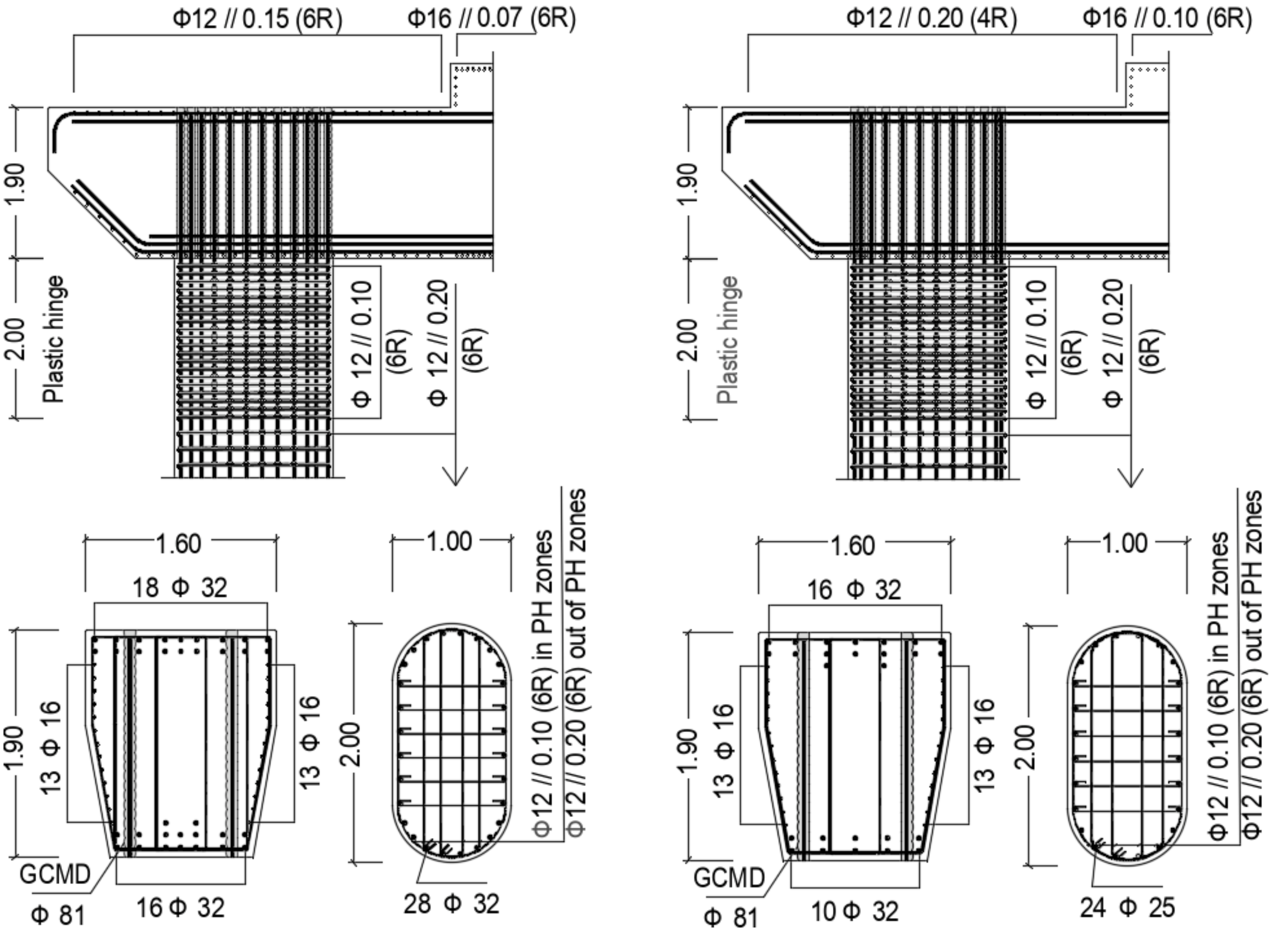


Figure 2. Two-column bents layout – frame and 3D perspective.

2.2 Substructure Design for Performance and Constructability

In the design, a force-based design approach was used, which meant that the columns were designed for maximum forces determined by a response spectrum analysis in SAP2000 (2020). Two design scenarios were performed to determine the seismic forces: an inelastic design spectrum with a behavior factor q=3.0, resulting in a maximum transversal seismic design force of 3.16MN in the governing alignment, and a behavior factor q=4.5, resulting in a maximum transversal seismic design force of 2.11MN in the governing alignment. The resulting reinforcement layout for both scenarios are presented in Figure 3.



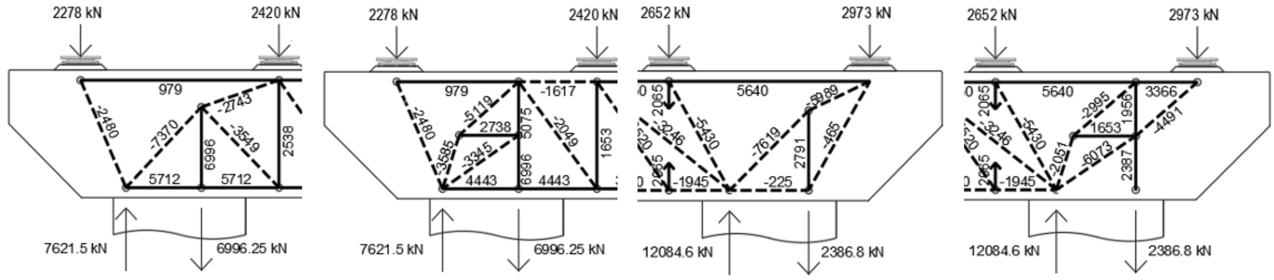
(a) design scenario with q=3,0 (b) design scenario with q=4,5

Figure 3. Column and cap-beam cross-sectional and reinforcement layout.

Concerned with constructive issues, a longitudinal reinforcement ratio of about 1.2% was considered, dictating the column geometry in the case with q=3.0. The geometry was maintained for the scenario with q=4.5. Also, the longitudinal reinforcement ratio ensures the viability of using corrugated ducts with diameters of 80mm to 100mm. The columns act as cantilevers in the longitudinal direction, and the corresponding second-order effects control the columns' minimum dimension. The minimum transversal stiffness of the railway bridge complying with the Eurocode 1 (2017) requirements was also verified. The cap beam geometry was designed with weight constraints in mind, as well as the dimensions required to accommodate the final and temporary bearings during the assembly process, all while maintaining a reasonable reinforcement ratio and avoiding dense reinforcement layouts. The cap beam includes corrugated ducts distributed accordingly with the layout of the column vertical bars allowing a streamlined on-site assembly process. These ducts are posteriorly grouted for the anchorage of the vertical bars. As a fully precast solution is envisaged, special attention was paid to the reinforcement layout at the cap-columns joints. These are discontinuity regions, and the design was performed resorting to strut-and-tie models, as will be presented next.

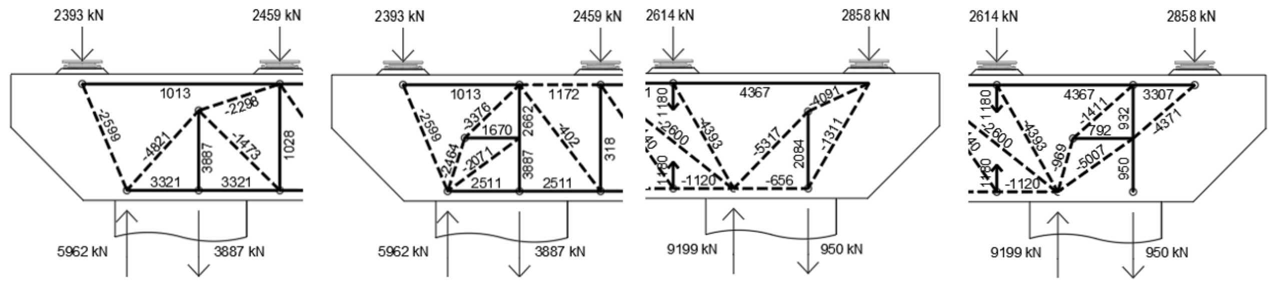
2.3 Strut-and-Tie Models

Strut-and-tie models were developed, exploiting alternative load paths and load-bearing mechanisms, justifying the reinforcement layout to allow better constructability of the precast solution, and ensuring the envisaged structural performance. Among these models, the showed in Figures 4 and 5, for design scenarios with q=3.0 and q=4.5, respectively, were the ones used for cap-beam design purposes. The reinforcement layouts of Figure 3 were obtained by combining the results of models 1 and 2, for both design scenarios. The contribution of each model was set at 50%. The transfer of the tensile forces coming from the column is made partially by straight anchorage of the vertical bars (Model 1) and by overlapping with stirrups (Model 2).



(a) ST Model 1 – opening moments (b) ST Model 2 – opening moments (c) ST Model 1 – closing moments (d) ST Model 2 – closing moments

Figure 4. Strut-and-tie models for design scenario with q=3,0.



(a) ST Model 1 – opening moments (b) ST Model 2 – opening moments (c) ST Model 1 – closing moments (d) ST Model 2 – closing moments

Figure 5. Strut-and-tie models for design scenario with q=4,5.

For opening moments, regardless of whether design scenario with q=3,0, or q=4,5, model 1 allows the vertical bars from the column to be anchored in a straight manner, however, this requires additional vertical stirrups placed outside the column alignment to balance a fraction of the tensile force coming from the column, and additional longitudinal reinforcement in the bottom of the cap beam. Model 2 allows using horizontal reinforcement placed along the height of the cap beam to gradually reduce the vertical tensile force in the column bars. For closing moments, model 1 allows the vertical bars from the column to be anchored in a straight manner. Model 2 allows partial anchorage of the force in the vertical bars coming to the column due to the activation of the horizontal reinforcement crossing the joint.

It is noted that the vertical tie located at the left of the right column allows partial suspension of the shear forces coming from the leftmost part of the cap beam, with a fraction of the load being carried by direct strut action towards the column. This allows defining the same amount of transversal reinforcement in this region irrespective of the opening or closing rotation.

A more comprehensive study can be found in Carvalho, *et. al* (2023), where some strut-and-tie models were developed aiming to provide a deeper understanding of the several force transfer possibilities that can occur within a column-cap beam joint during seismic events. It enables the reconfiguration of reinforcements as needed, facilitating assembly and maintaining structural safety.

3 Nonlinear Analysis of the Reduced Scale Model

3.1 General

Three-dimensional finite element models were developed in the software DIANA 10.5 (2012) to perform numerical simulations of 1/3.6 reduced scale prototypes, which were designed following Cauchy's similitude relationship. The objectives of these numerical simulations were to assess the redistribution of forces, mainly on the cap beam and joint zones to confirm the adequacy of the strut-and-tie models used in the design, to confirm the failure mode, quantify the ductility/energy dissipation capacity, and explore the results of both design scenarios. These prototypes such as the numerical models are hinged, at mid-height at the section of null flexural moment and can be seen in Figures 6 and 7.

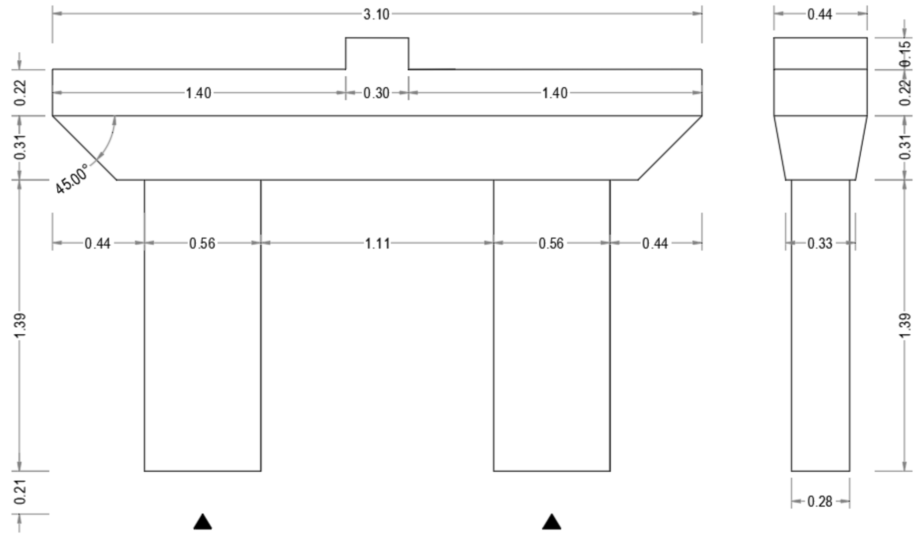
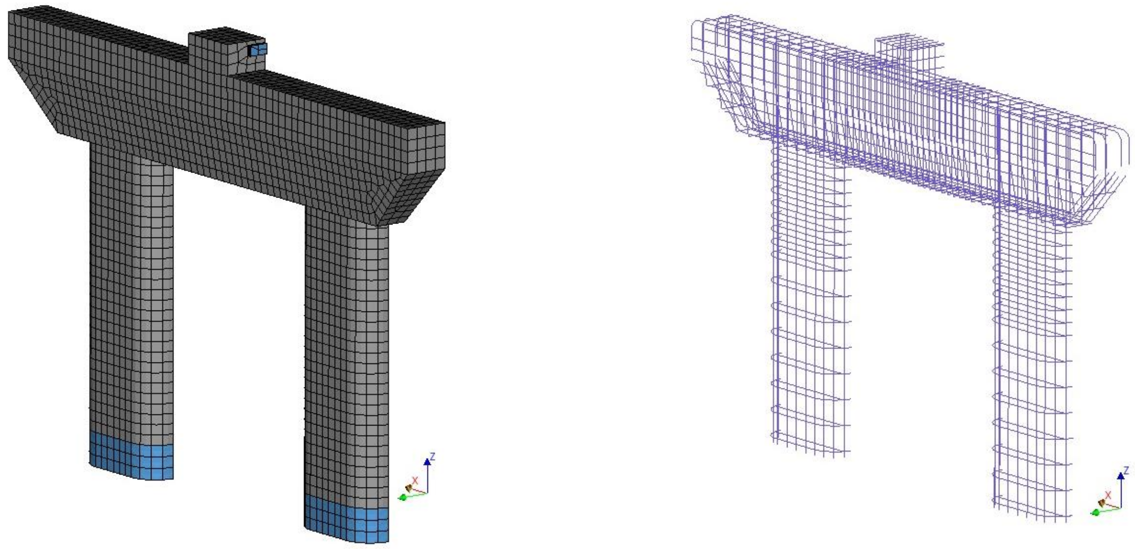


Figure 6. Geometry of the reduced scale prototypes.



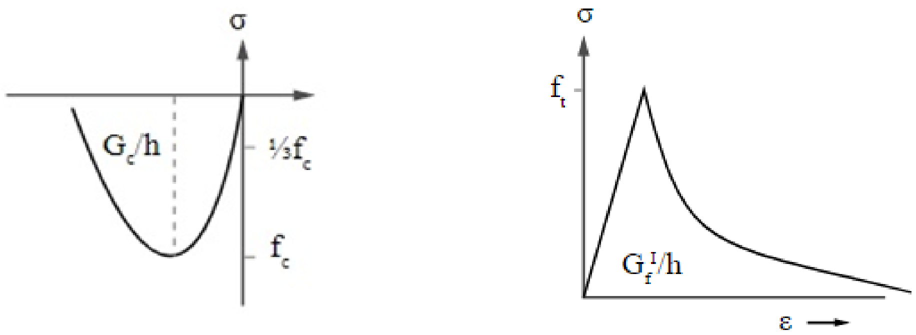
(a) finite element mesh. (b) reinforcement rebars.

Figure 7. Nonlinear model concerning to design scenario with q=3,0.

Noting that, in Figure 7, that ZX is a symmetry plane, only half the structure is modelled using quadratic elements. The vertical forces in the four bearings were applied in a first step. Afterwards, the horizontal force at the corbel is increased monotonically until failure.

3.2 Modelling Strategies and Constitutive Models

A total strain based rotating smeared crack model is used. A parabolic uniaxial compressive stress-strain curve Feenstra (1993) was selected, including a regularized post-peak branch defined by the compressive fracture energy, G_C=60N/mm. The Hsieh-Ting-Chen failure envelope and multiaxial compressive stress states was adopted, as described in Selby, R. G., and Vecchio, F. J. (1993). The crack band model by Hordijk (1991) is used to model tensile fracture. The strain softening curve is that proposed in fib Model Code (2013), defined by the tensile strength and by the fracture energy, G_F=0.09N/mm. The mean material properties corresponding to the concrete strength class C40/45 and steel class A500 were adopted. The bond-slip curves were defined according to the fib Model Code (2010), assuming the maximum bond shear stress τb max equal to 9.5 MN/m^2.



(a) parabolic curve (compression response). (b) Hordijk curve (tensile response).

Figure 8. Constitutive curves for compression and tensile response, respectively.

3.3 Results

The lateral force vs. displacement curves is shown in Figure 9. Failure occurs with concrete crushing in the inner face of the right column, in both scenarios, after yielding of the flexural reinforcement in the columns. A yielding plateau is clearly visible in these curves. The straight dashed lines represent the lateral failure load F_{pl} corresponding to the plastic mechanism involving the formation of the plastic hinges at the top of the columns, obtained by considering the flexural strength of the columns determined by conventional sectional analysis methods considering the axial forces from the finite element model. The agreement with the maximum force achieved in the simulation is satisfactory.

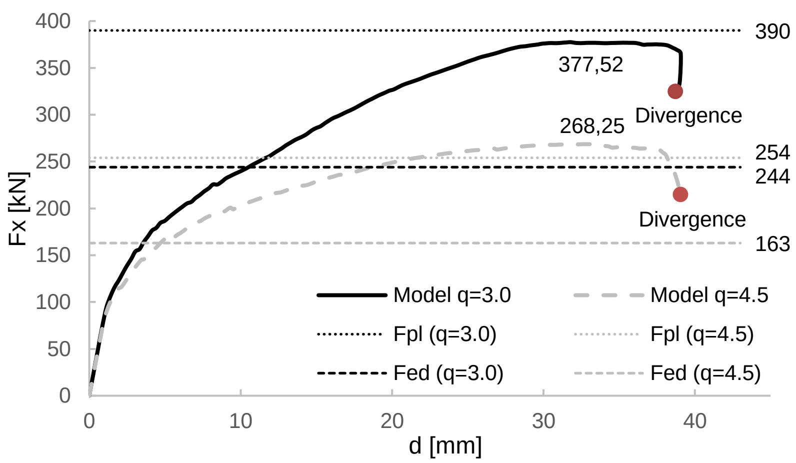
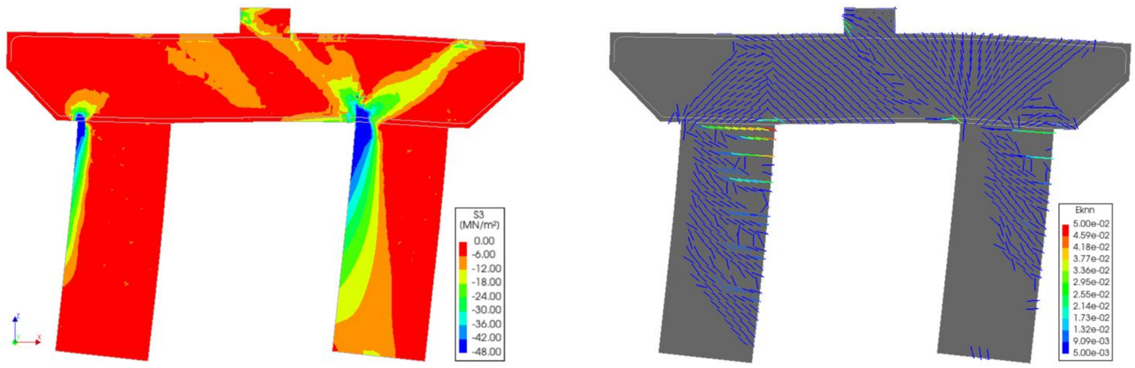


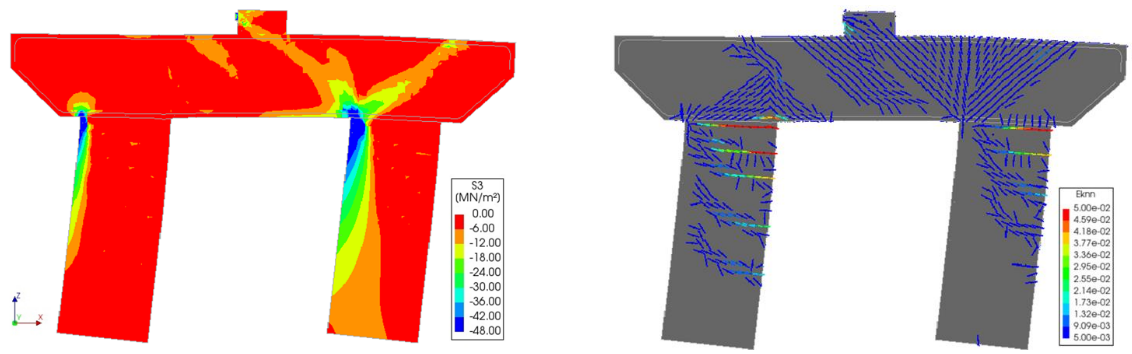
Figure 9. Force vs. displacement.

Figures 10 and 11 show the colour maps with the principal compressive stress values field and the cracking pattern right after the plastic mechanism has developed, for both design scenarios. In these results, a concentration of compression stresses in the inner face of the right column and a cracking distribution slightly accentuated in the left column can be observed.



(a) principal compressive stress field. (b) cracking pattern.

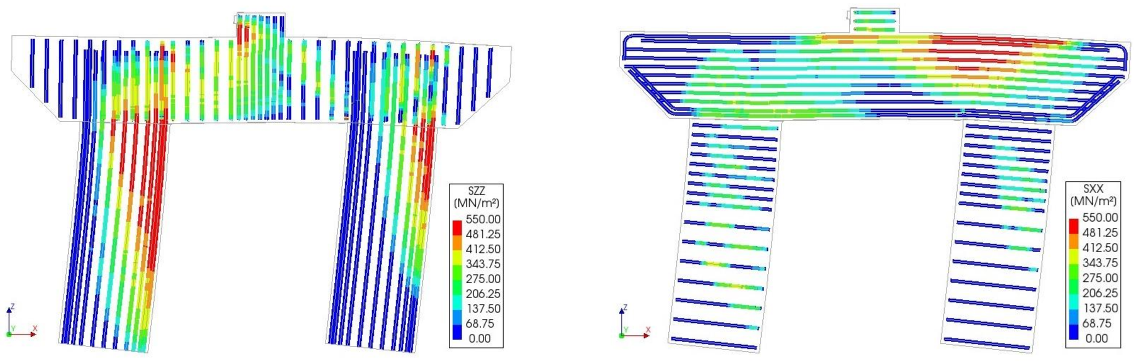
Figure 10. Numerical results at the plastic mechanism stage for design scenario with q=3,0 – concrete.



(a) principal compressive stress field. (b) cracking pattern.

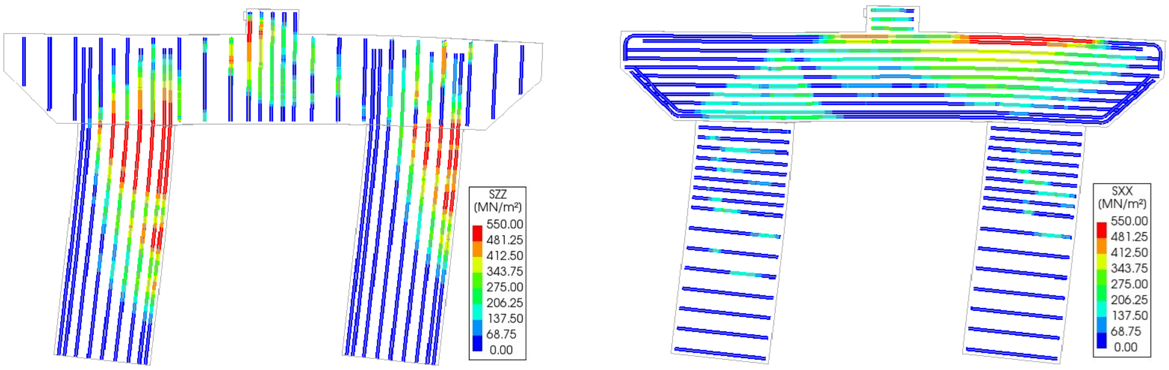
Figure 11. Numerical results at the plastic mechanism stage for design scenario with q=4,5 – concrete.

Figures 12 and 13 show the reinforcement stresses for both design scenarios right after the plastic mechanism has developed. At this stage, in both cases, most of the longitudinal reinforcement of the left column is yielding, as seen in Figure 12 and 13 (a). It is remarkable that more longitudinal bars are yielding in the right column in the case corresponding to the larger behaviour factor. Regarding the horizontal reinforcement, in the scenario q=3.0, the longitudinal reinforcements of cap beam exhibit more pronounced yielding. In both cases, the column transverse reinforcements remain in the linear elastic range, as determined by seismic design.



(a) vertical bar stresses. (b) horizontal bar stresses.

Figure 12. Numerical results at the plastic mechanism stage for design scenario with q=3,0 – reinforcements.



(a) vertical bar stresses. (b) horizontal bar stresses.

Figure 13. Numerical results at the plastic mechanism stage for design scenario with q=4,5 – reinforcements.

The reinforcements responsible for the transfer of forces between the column longitudinal bars and the cap beam, in the joint zone, ensure the formation of the plastic hinges in the columns, avoiding joint failure. Confirmation of this is pending experimental tests. The evolution of the bending moments and axial forces in the columns are shown in Figure 14. The red line shows linear elastic distribution of forces and the black lines the results from the nonlinear simulation. The model predicts that in the pushover analysis the right column is stiffer and therefore the moments increase more than estimated by the linear elastic analysis. Reversed cyclic analyses are now underway to confirm is this also occurs for this more realistic loading protocol.

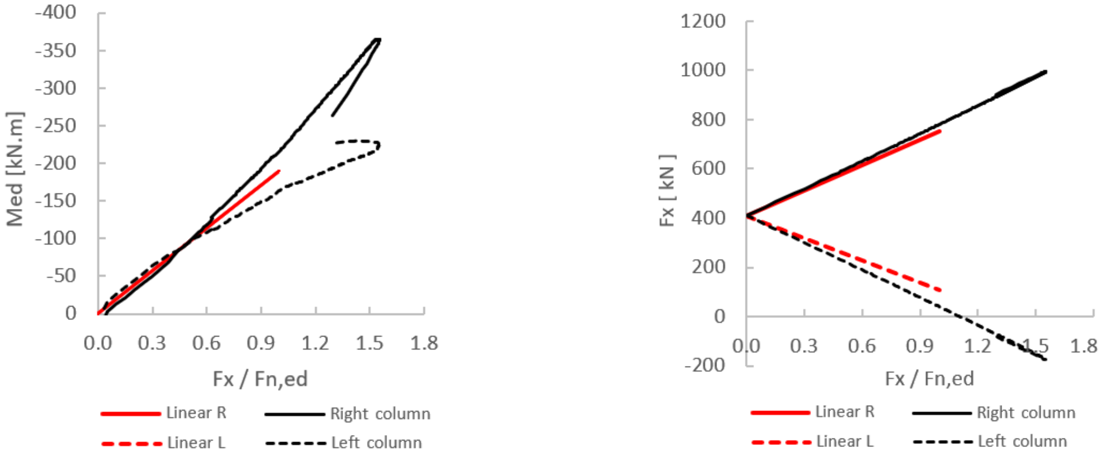


Figure 14. Evolution of the flexural moments and axial forces in the columns, respectively.

4 Experimental Campaign

4.1 General

An experimental campaign will be conducted with the aim of assessing the reduced-scale prototypes under cyclic loading, in the Laboratory of Structural and Seismic Engineering (LESE) at the Faculty of Engineering of the University of Porto (FEUP). The main objective of this experimental campaign involves assessing the behavior of the column-to-cap beam system through three reduced-scale prototypes. These include a cast-in-place (CIP) reference specimen and two precast specimens (PE1 and PE2), differing each other according to the design scenario considered, all subjected to cyclic loads with uniaxial directions.

4.2 Test Setup

Once the prototypes columns were considered with half of its height, i.e., from the top to mid length of the column, the rotation should be allowed at the inferior extremity of the column (prototype base). Therefore, on the setup layout presented in Figure 15, the column bases are screwed into freely rotating steel plates, placed on mechanical devices with free rotation capacity, and designed to withstand shear forces in both directions and the resulting axial forces.

The axial load applied to the columns, aiming to simulate the load mobilized during the seismic action, is performed with the assistance of prestressing bars. The scaled force resulting from these actions takes a total value of 800 kN (400 kN in each column), distributed in four points on the cap beam, aiming to represent the support devices. This load distribution is achieved using two HEB profiles supported by neoprene pads located in the intended places for vertical load application.

The application of the lateral load, which is intended to represent the action of the inertial forces developed during the seismic event, is applied to the corbel through a hydraulic actuator, extended through two CHS profiles hinged at their ends. The maximum expected horizontal force to be applied is 400 kN for the test specimen designed for the most demanding scenario.

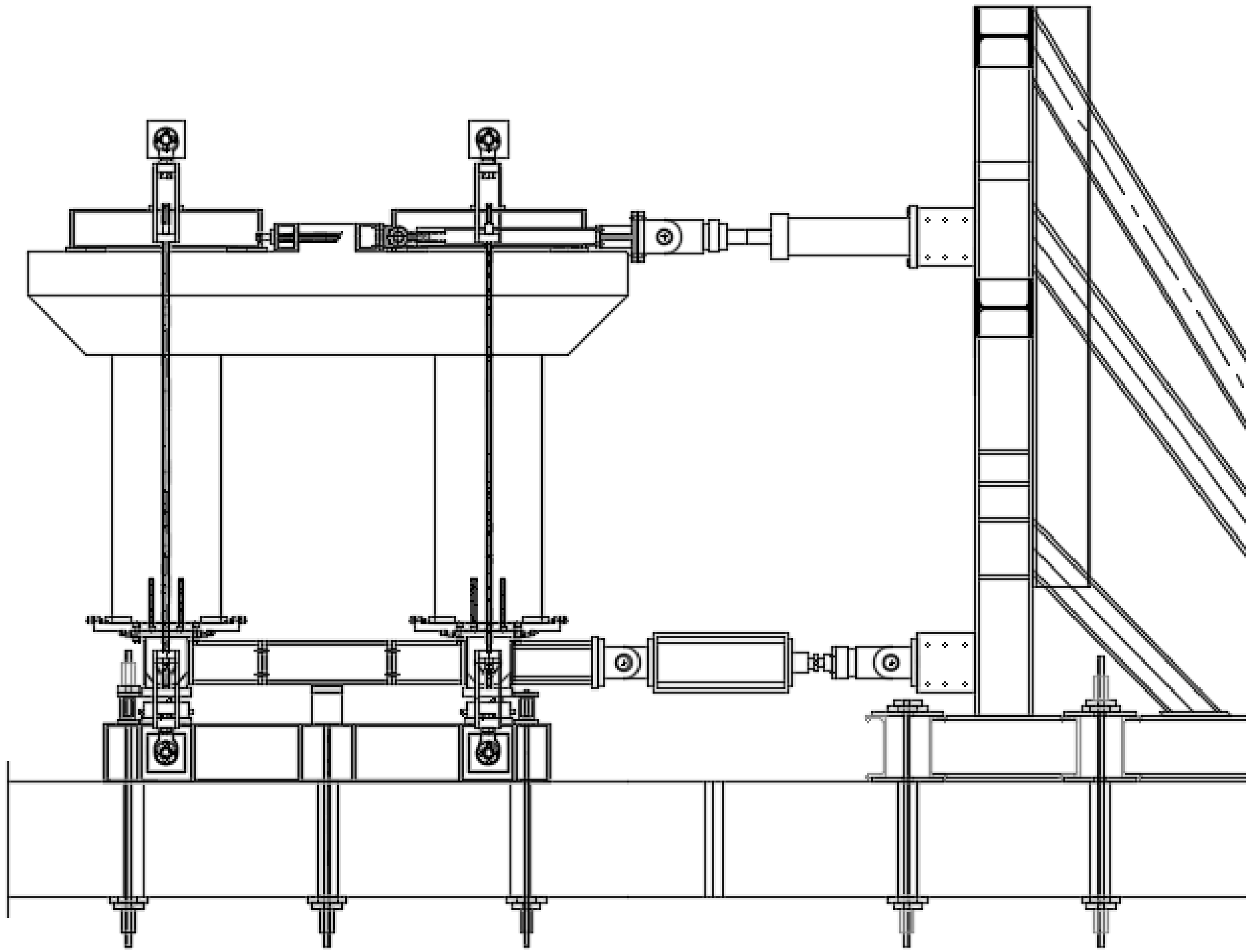


Figure 15. Test setup.

4.3 Monitoring System

As part of the monitoring system, a set of Displacement Transducers (LVDTs) will be utilized to track displacements at various points of the structure. Additionally, strain gauges (SG) will be affixed to the two CHS profiles, the prestressing bars, and the lower beam of the force retention system, facilitating the evaluation of bending and shear deformations. Moreover, imposed loads and reaction forces will be recorded by load cells (LC) strategically placed throughout the entire structure, including in the horizontal actuator, the horizontal force retention system, and at each column base. Figure 16 provides a comprehensive layout of the monitoring system that will be employed in the setup.

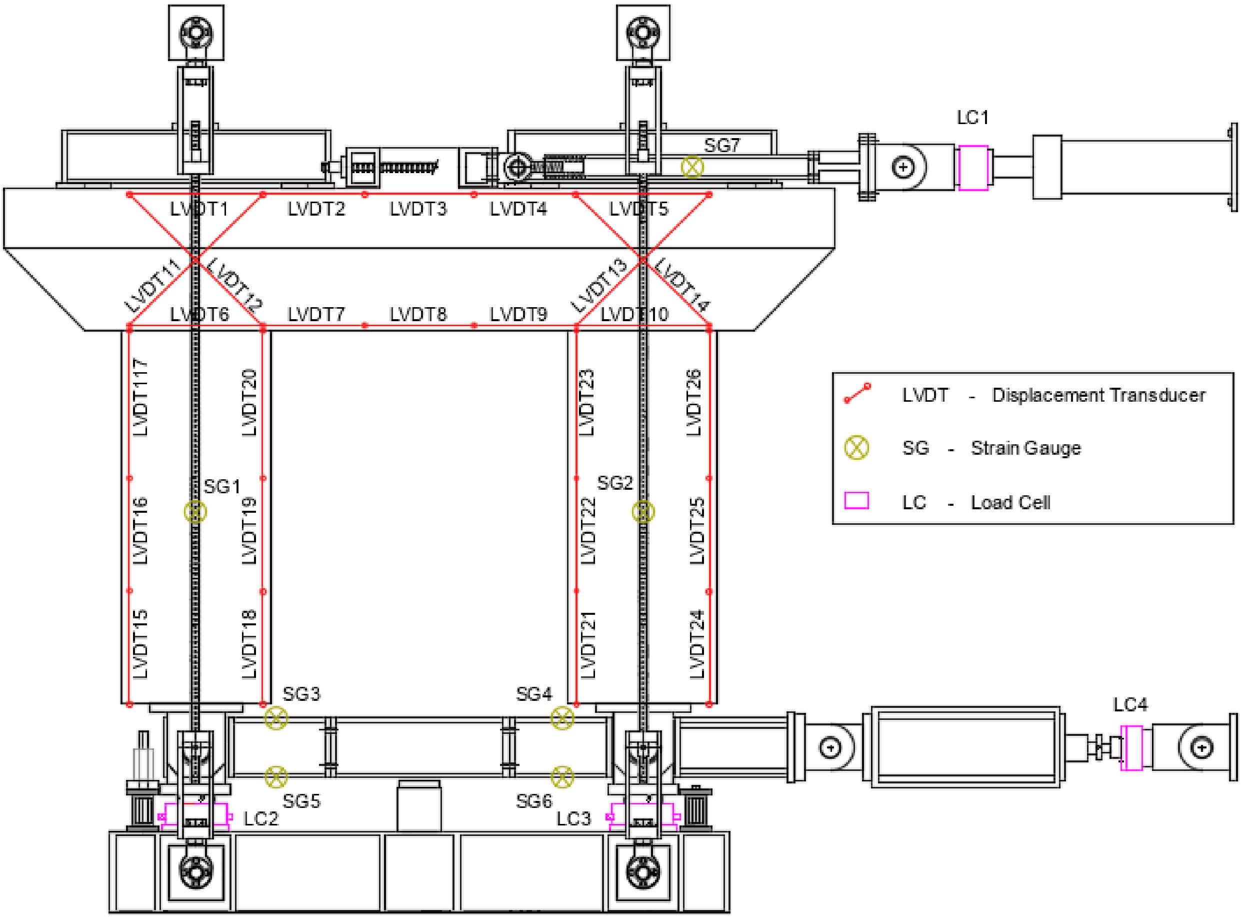


Figure 16. Schematic overview of the monitoring system.

5 Retrofit Strategies Proposal

In the event that a bridge or viaduct is subjected to seismic action beyond what was predicted in design, it is crucial to consider retrofit proposals for the substructure. The goal is to restore the functionality of the bridge quickly and safely, reducing downtime and avoiding additional costs associated with such situations.

Post-seismic strengthening proposals may involve the application of innovative techniques, such as the incorporation of energy dissipation devices, the installation of additional structural elements, and/or the optimization of construction materials to withstand seismic loads. A careful analysis of the specific conditions of each bridge, taking into account factors such as geometry, construction material, and seismic history, is essential for the effective design of these reinforcement measures.

In a subsequent phase of this work, after the accomplishment of the experimental tests outlined in point 4 of this document, post-seismic reinforcement proposals will be carefully studied to restore the functionality of the bridge quickly and safely. The aim is to minimize post-seismic impacts on substructure and mobility. These retrofit measures not only aim to repair damage but also contribute to improve strength and resilience of bridges, preparing them to face future seismic challenges with greater performance.

Nevertheless, some preliminary retrofit strategies can be proposed, with base on the results of the numerical simulations already carried out in section 3, namely, the CFRP confinement of the columns top end and the longitudinal and transverse CFRP retrofit of the cap beams, as showed in Figure 17

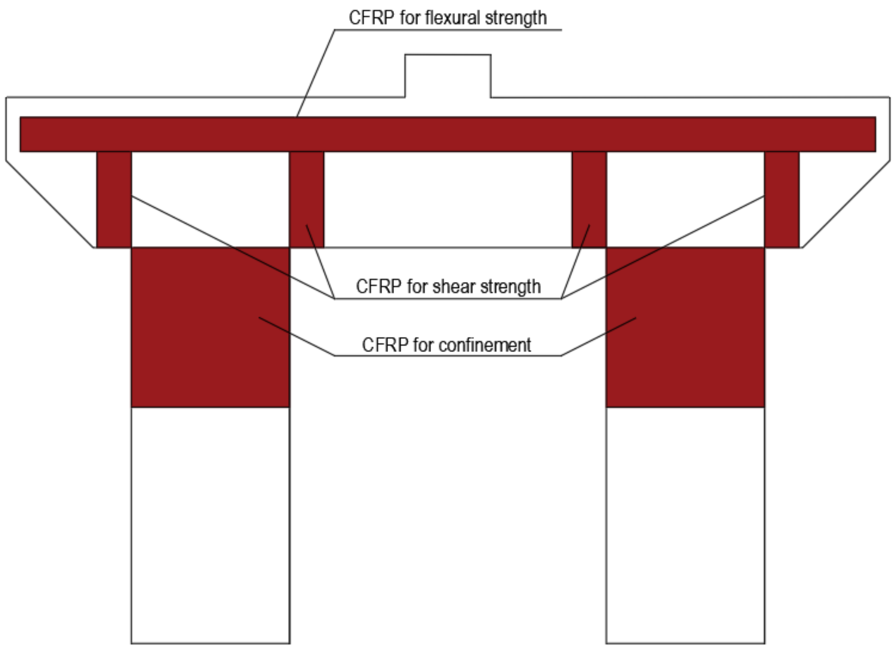


Figure 17. Retrofit strategies proposal.

6 Conclusions

A fully precast column and cap beam solution for railway viaducts was designed to withstand medium to high seismic excitations. For this, seismic design strategies and approaches were adopted to ensure adequate energy dissipation through plastic hinges located on the column ends. Strut-and-tie models were developed for the cap beam design, thus providing a better understanding of the force transfer mechanisms in the column-beam joint zone, and making it possible to obtain different possibilities for reinforcement layout and constructive arrangements, maintaining reliable force paths and avoiding constructability problems.

From the numerical investigation carried out, satisfactory structural behavior could be evidenced, being possible to observe the formation of the plastic mechanism, thus achieving the yielding plateau at the force-displacement curve, while the column longitudinal reinforcements were yielding, and the shear and confinement reinforcements were in the elastic domain. Failure is predicted to occur with concrete crushing in the inner face of the column subjected to the largest compressive axial force. Moment redistribution between the two columns leads to larger demand on the stiffer column, which is that subjected to higher compressive forces. Still, the plastic load could be achieved.

An experimental investigation is underway to confirm the design assumptions as the numerical simulation results. In a next phase, after the accomplishment of the experimental campaign, a further numerical simulations will be performed considering the cyclic loading protocol, and the experimental results will be used to calibrate parameters and refine modelling strategies, ensuring that the numerical model accurately reproduces what was observed during the experimental tests.

In a subsequent phase to this, proposals for substructure strengthening will be conducted with the aim of implementing effective measures in the bridge substructure. The intention is to restore structural integrity and ensure the ongoing safety of infrastructures while also seeking to strengthen the substructure's capacity to withstand future events.

7 Acknowledgements

This work is a result of project iPBRAIL - Innovative Precast Bridges for RAILways, with reference POCI-01-0247-FEDER-039894, co-funded by the European Regional Devel-opment Fund (ERDF), through the Operational Programme for Competitiveness and Interna-tionalization (COMPETE 2020) and the Lisbon Regional Operational Programme (LISBOA 2020), under the PORTUGAL 2020 Partnership Agreement. The authors also acknowledge the financial support by: Base (UIDB/04708/2020) and Programmatic (UIDP/04708/2020) funding of CONSTRUCT financed by national funds through the FCT/MCTES (PIDDAC).

This work was also developed within the scope of the project proMetheus – Research Unit on Materials, Energy and Environment for Sustainability, FCT Ref. UID/05975/2020, financed by national funds through the FCT/MCTES.

8 References

Carvalho, L., Pimentel, M., Arêde, A., Vila Pouca, N., Delgado, P., Pinto, J.R. (2023). *Design of pre-cast two-column bents using strut-and-tie models and nonlinear finite element analysis.* fib Symposium – Structural Performance and Design. Istanbul, Turkey.

CSI. *SAP2000_v22.1.0 (2020). Finite element program for modelling, analysis, and design of any type of structure*. Help documentation of the program, https://www.csiportugal.com/.

CEN European committee for standardization (2017). *Eurocode 1 - Actions on structures. Part 2: Traffic loads on bridges*. Brussels.

DIANA (2012) Release 10.4, TNO DIANA BV, Delft, The Netherlands.

Feenstra, P. H. (1993). Computational Aspects of Biaxial Stress in Plain and Reinforced Concrete. PhD thesis, Delft University of Technology.

Selby, R. G., and Vecchio, F. J. (1993). Three-dimensional Constitutive Relations for Reinforced Concrete. Tech. Rep. 93-02, Univ. Toronto, dept. Civil Eng., Toronto, Canada.

Hordijk, D. (1991). Local approach to fatigue of concrete. Doctoral Thesis Universiteit Delft.

Fib (2013) International Federation for Structural Concrete. Fib Model Code for Concrete Structures 2010. Ernst & Sohn, Switzerland.