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Numerical Simulation of Full-Scale Load Tests on 50-Year-Old PC Bridge Deck Beams Under Flexural- and Shear-Dominant Failures

Simulación numérica de ensayos de carga a escala real en vigas de tablero de puente de hormigón pretensado (PC) con 50 años de antigüedad bajo fallos dominados por flexión y por cortante.

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ABSTRACT

Computational methods and modeling criteria for life-cycle design, assessment, maintenance, and management of aging structural systems require robust calibration and validation based on data and information gathered from existing structures and experimental tests. This paper provides a contribution along these lines based on criteria, methods, and tools for computational modeling and experimental validation of nonlinear finite element analysis of reinforced concrete (RC) and prestressed concrete (PC) structures. Structural modeling was developed with RC/PC beam finite elements and bi-dimensional finite elements for plane-stress analysis, formulated in accordance with the Modified Compression Field Theory. The formulations were applied to numerical simulation of full-scale load tests on 50-year-old PC bridge deck beams under different loading conditions intended to promote flexural- or shear-dominant failures. The models were informed by the results of laboratory tests on material mechanical properties and residual prestressing stress. The comparison of numerical and experimental results of full-scale load tests allows validation of the nonlinear analysis methods and structural modeling strategies and contributes to the successful implementation in practice of life-cycle-oriented models for deteriorating RC/PC structures.

KEYWORDS: PC bridge deck beams; full-scale load tests; nonlinear finite element analysis; experimental validation.

RESUMEN

Los métodos computacionales y los criterios de modelización para el diseño, evaluación, mantenimiento y gestión a lo largo del ciclo de vida de sistemas estructurales envejecidos requieren una adecuada calibración de parámetros y la validación de los modelos a partir de datos obtenidos de estructuras existentes y de ensayos experimentales.

Este trabajo contribuye en esta línea mediante la aplicación de criterios, métodos y herramientas para la modelización computacional y la validación experimental del análisis no lineal mediante elementos finitos de estructuras de hormigón armado (HA) y hormigón pretensado (HP).

La modelización estructural se realiza empleando elementos finitos de viga para HA/HP y elementos finitos bidimensionales para análisis en estado plano de tensiones, formulados de acuerdo con la Teoría Modificada del Campo de Compresiones (Modified Compression Field Theory).

Estas formulaciones se aplican a la simulación numérica de ensayos de carga a escala real realizados sobre vigas de tablero de puente de hormigón pretensado con 50 años de servicio, sometidas a distintas condiciones de carga destinadas a provocar modos de fallo dominados por flexión o por cortante.

Los modelos se calibran a partir de los resultados de ensayos de laboratorio sobre las propiedades mecánicas de los materiales y sobre el nivel residual del esfuerzo de pretensado.

La comparación entre los resultados numéricos y experimentales obtenidos en los ensayos de carga a escala real permite validar los métodos de análisis no lineal y las estrategias de modelización estructural, contribuyendo a la implementación práctica de modelos orientados al ciclo de vida para estructuras deterioradas de hormigón armado y pretensado.

PALABRAS CLAVE: Vigas de tablero de puente de hormigón pretensado; ensayo de carga a escala real; análisis no lineal por elementos finitos; validación experimental.

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1. INTRODUCTION

Bridges and infrastructure facilities are frequently exposed to aggressive environments, leading to aging and structural deterioration processes that may seriously affect their life-cycle performance and residual lifetime (Biondini & Frangopol 2016, 2019). This critical situation is reflected in the high costs involved in many countries to restore or enhance the structural capacity and functionality of existing bridges and infrastructure facilities that are currently rated as structurally deficient (ASCE 2025). Risk-based methodologies for effective bridge prioritization are therefore fundamental to support a rational allocation of resources for inspection and diagnostic activities (Biondini et al. 2022). Moreover, the urgency of this situation is emphasized by bridge failure events that occurred worldwide in recent years with alarming frequency and involving different flexural- and shear-dominant failure mechanisms. Robust and efficient life-cycle-oriented design, assessment, and maintenance methods have been established and consolidated over the past decades to address these problems. However, for a successful implementation in practice and a reliable use to inform the evolution of standards and codes, life-cycle-oriented models and methods still require robust validation and accurate calibration based on experimental tests and data gathered from existing structures. These procedures are particularly challenging for reinforced concrete (RC) and prestressed concrete (PC) structures, especially bridges, mainly due to a wide range of deterioration processes and associated uncertainties.

Considering the structural analysis methods available in the literature, life-cycle analysis is applied in practice by using a variety of tools, models, and resources, whose representativeness and accuracy require proper verification. Advanced structural modeling and nonlinear structural analysis methods are frequently indispensable tools to accurately assess the lifetime structural performance, as well as to identify damage features and investigate the attainment of multiple limit states that characterize the structural response at the material and component level, such as concrete cracking, steel yielding, and concrete crushing. Moreover, structural modeling based on finite element formulations should guarantee the versatility of the methods to account for both flexural and shear mechanisms (Vecchio & Collins 1986; Kaufmann & Marti 1998; Collins, Bentz, & Sherwood 2008; Marí et al. 2015). However, robust validation and accurate calibration of these methods are generally difficult tasks because of the limited availability of experimental data on the long-term performance of in-service structures. In fact, despite experimental tests of corroded RC/PC beam specimens have been conducted and documented in the literature, experimental campaigns on existing bridges and full-scale members removed from in-service structures are very limited. There is therefore a strong need to validate life-cycle models properly accounting for the magnitude and spatial distribution of the uncertainties associated with geometrical quantities, mechanical properties, and exposure conditions that are typical of existing structures (Anghileri & Biondini 2025b). It is also important to establish and calibrate methodologies for daily engineering practice (Messina & Proverbio 2023). In the assessment of the structural performance of existing systems, discrepancies between the specified design properties and the actual characteristics can be significant due to several factors, including aleatory uncertainties, instantaneous and/or long-term variations, design variants and human errors in the construction phase. Gathering new data from both experimental tests and inspections of existing structures is therefore essential for the successful practical implementation of life-cycle methods (Biondini & Frangopol 2018).

In this paper, computational methods for nonlinear analysis of RC/PC structures are calibrated and validated using experimental results from the BRIDGE|50 research project, which includes multiple full-scale load tests on 50-year-old PC bridge deck beams under different loading conditions (Anghileri & Biondini 2021, 2022, 2023, 2025a, 2025b). Structural modeling is developed with two approaches associated with different levels of complexity and computational cost based on RC/PC beam finite elements and bi-dimensional finite elements for plane-stress analysis accounting for material nonlinearities. The numerical analyses are calibrated by the results of experimental laboratory tests on material mechanical properties and

(a) (b)

Figure 1. PC bridge deck beams: (a) Dismantling of the grillage bridge deck beams of the 50-year-old Corso Grosseto viaduct; (b) Storage of the beams at the testing site (corrosion damage visible at the beam ends).

residual prestressing levels. The results of the experimental validation are complemented by further numerical simulations aimed at investigating the residual structural capacity of the tested PC bridge deck beams and support proper planning of the ongoing full-scale load tests.

2. EXPERIMENTAL CAMPAIGN ON 50-YEAR-OLD PC BRIDGE DECK BEAMS

2.1. BRIDGE|50 research project

The BRIDGE|50 research project was established jointly by Politecnico di Milano and Politecnico di Torino under an agreement with public authorities and private companies to conduct a wide experimental campaign investigating the residual structural performance of a 50-year-old double-deck road viaduct located in Turin, Italy (Biondini, Manto et al. 2021; Biondini, Tondolo et al. 2021). The 80-span simply supported grillage bridge deck was formed by precast PC beams, including ten inner I-beams and two lateral U-box beams, with a top cast-in-situ RC slab (Savino et al. 2021). During the demolition of the viaduct after 50 years of service, several structural members were dismantled and preserved at a testing site, including 29 PC bridge deck beams (25 I-beams and four U-box beams) and two PC pier caps (Anghileri et al. 2020).

2.2. PC bridge deck beams

The dismantled PC bridge deck beams (Figure 1) are characterized by a length of about 19.50 m and a composite cross-section made of a precast PC I-beam and a top cast-in-situ RC slab. The precast beams are prestressed with twenty 7-wire steel strands arranged straight along the longitudinal axis of the beam. The nominal diameter of the steel strands is 12.7 mm (effective area 99 mm²). Stirrups with a diameter of 8 mm and spaced at 250 mm in the inner I-shaped cross-section and 100 mm in the rectangular cross-section at the beam ends have been evaluated based on data reported in the design documentation and results of pacometer tests. Visual inspection activities conducted on the dismantled beams allowed the identification of local damage in the end regions due to corrosion, with steel mass loss and concrete spalling and delamination, attributed to the inadequacy of bridge water conveyance system and use of road salts during the bridge lifetime (Beltrami et al. 2021; Carsana et al. 2022; Carsana, Redaelli, & Biondini 2023; Carsana, Biondini, & Redaelli 2025). However, despite the long-term exposure of the PC viaduct to an urban environment, no significant corrosion of the prestressing strands was observed by visual inspection of the failure regions of the tested beams. The effects of corrosion are therefore not considered in the experimental validation presented in this paper. Moreover, inspection activities performed after the full-scale load tests allowed the identification of the reinforcing steel layout in both the precast PC beam and cast-in-situ RC slab, which was not exhaustively reported in the original design documentation.

2.3. Material characterization

The actual material mechanical properties of the PC beams have been largely investigated with both non-destructive and destructive experimental tests (Anghileri et al. 2023). The concrete compressive and tensile strength and the elastic modulus have been estimated by means of laboratory tests carried out on several cylindrical specimens extracted from the dismantled PC beams. The material properties of both reinforcing steel bars and prestressing steel strands have also been estimated with laboratory tensile strength tests. Table 1 shows the sample mean and coefficient of variation (CoV) of material mechanical properties based on the outcomes of laboratory tests. In the numerical analysis, the elastic modulus of both reinforcing and prestressing steel is assumed to be 200 GPa

TABLE 1.
PC bridge deck beams: Sample mean and coefficient of variation (CoV) of material mechanical properties based on the outcomes of laboratory tests carried out on *n* samples.

Material	*Material properties*	*n*	*Sample mean*	*CoV*
Precast concrete beam	Compressive concrete strength, f_c	25	32.3 MPa	0.14
	Tensile concrete strength, f_{ct}	9	3.4 MPa	0.14
	Concrete elastic modulus, E_c	8	27.3 GPa	0.08
Cast-in-situ concrete slab	Compressive concrete strength, f_c	5	21.9 MPa	0.22
	Tensile concrete strength, f_{ct}	2	21.4 MPa	0.38
Prestressing steel strands	Steel yielding strength, f_{py}	8	1522 MPa	0.05
	Steel ultimate strength, f_{pu}	8	1763 MPa	0.03
Reinforcing steel bars	Steel yielding strength, f_{sy}	10	449 MPa	0.06
	Steel ultimate strength, f_{su}	10	685 MPa	0.06

and 195 GPa, respectively. Due to its significant role in the performance assessment, the residual prestressing stress σ_p was estimated using the strand cutting method based on the measurement of the strain on a cut prestressing strand (Savino, Tondolo et al. 2023). The initial prestressing stress, net of instantaneous and estimated long-term losses, as reported in the original technical design documentation, was σ_{pd}=836 MPa. However, the assessment of the residual prestressing stress after a lifetime of 50 years, based on the strand cutting method, led to about σ_p=582 MPa. This result may be attributed to higher instantaneous and/or long-term prestressing losses, as well as steel corrosion effects. The large number of experimental outcomes also allowed a probabilistic analysis based on statistical tests and regression analysis for the random variables associated with the material mechanical properties (Anghileri & Biondini 2025b). Moreover, the role of involved uncertainties and the effects of new data obtained from experimental tests have been investigated in Anghileri & Biondini (2025a) through Bayesian model updating.

2.4. Full-scale load test setup

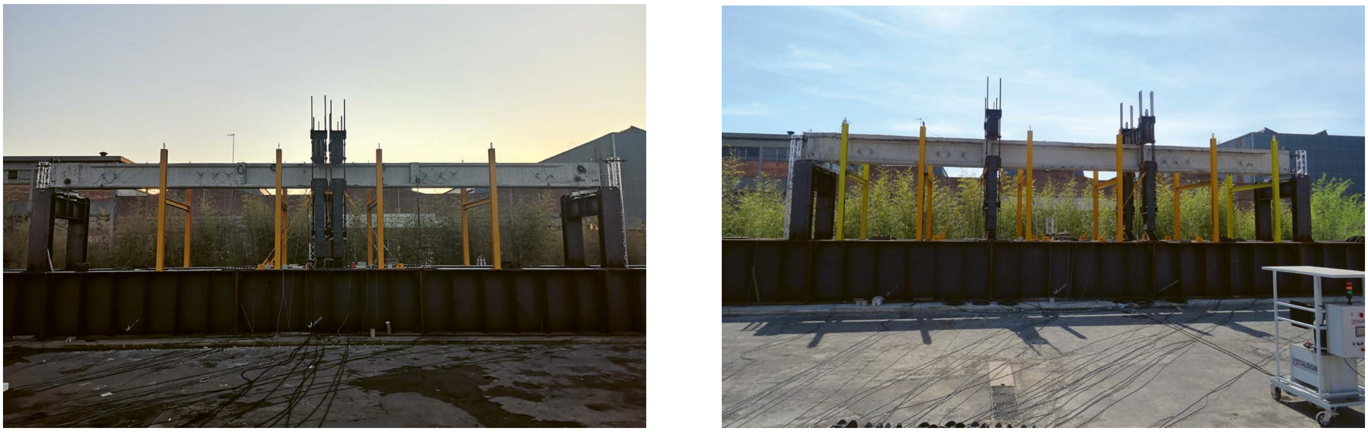
The residual structural capacity of the PC bridge deck beams is investigated with full-scale load tests. The PC beams were tested with a steel reaction framework (Figure 2) under simple supports with a span length of about 19.00 m and loaded up to collapse. The applied load was transferred by two transverse steel beams to the PC beam. The experimental test setup was based on several sensors, including load cells, transducers, and displacement potentiometers installed to record the applied load, bending and shear strains, strand slips, support settlements, and vertical deflection (Tondolo et al. 2021, 2022). The reaction steel frame was designed to allow for a variable distance between the applied forces and to reproduce the in-service span of the PC bridge deck beams (Savino, Quattrone et al. 2023). Multiple full-scale load tests with different values of the shear span ratio $\alpha=a/l$ (i.e., a=shear span; l=half beam span) were conducted to study both bending and shear failures (Tondolo et al. 2025). In this paper, the experimental results of tests associated with shear span ratio $\alpha\approx$1.00, 0.68, 0.47, and 0.32 are considered to investigate the flexural and shear behavior of the PC beams.

3. FINITE ELEMENT MODELING OF PC BRIDGE DECK BEAMS

3.1. Finite element modeling of concrete structures

Finite element formulations for the nonlinear analysis of RC/PC structures have been proposed in the literature characterized by distinctive features such as the finite element discretization level, simplicity and robustness of the formulation, capability of properly describing the real structural behavior, accuracy of the solution process, and computational cost (Malerba 1998). The RC/PC beam finite element (BFE) formulation provides an effective trade-off among the above-mentioned factors assuming the linearity of the cross-sectional strain field and accounting for the nonlinear constitutive laws of the materials, i.e., concrete, reinforcing steel, and prestressing steel. The BFE formulation neglects shear failures and bond-slip of steel.

To account for shear effects and local stress-diffusion phenomena, the Modified Compression Field Theory (MCFT) is adopted in this paper for the nonlinear plane-stress analysis of RC/PC structures. The MCFT is formulated based on a smeared rotating crack approach (i.e., cracks change orientation according to the direction of principal strains) and considers the cracked RC medium as an orthotropic material with its own constitutive laws. The critical crack direction is assumed to be normal to the principal tensile strain direction. Equilibrium, compatibility, and constitutive laws are formulated in terms of average stresses and average strains, and the directions of principal stresses and strains are considered coincident (Vecchio & Collins 1986). Among the various formulations proposed in the literature to account for shear effects of RC/PC structures, the MCFT was selected based on multiple factors, including the robustness of the formulation and the accuracy of the solution process (Vecchio 2001). The use of a specific type of finite element modeling approach (BFE or MCFT) should be guided by the expected governing behavior and failure mechanism. While both BFE- and MCFT-based models generally provide accurate results under flexure-dominated failures, significant deviations may arise when the structural response is governed by shear-dominat-



(a) (b)

Figure 2. Full-scale load tests: (a) Three-point bending test of PC beam without RC slab; (b) Four-point bending test of PC beam with RC slab.

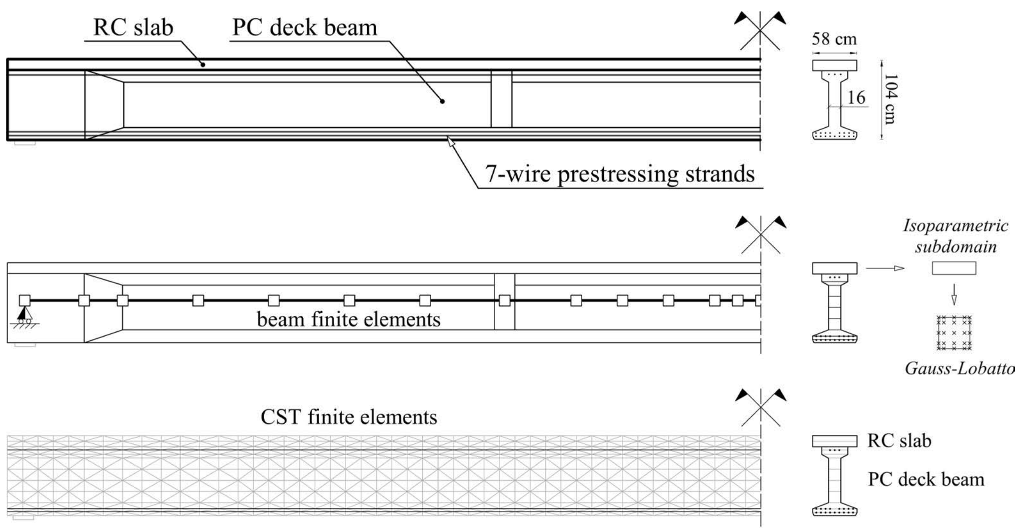


Figure 3. PC bridge deck beams: Longitudinal profile, structural modeling, and element discretization based on BFE model with isoparametric discretization and Gauss-Lobatto integration scheme and MCFT model with CST finite elements.

ed failures, which the BFE formulation is unable to capture. A proper model selection should also account for additional aspects, including importance of stress-diffusion effects near disturbed regions, required accuracy of the results at both local and system levels, capability to represent structures composed of multiple layers and/or realized at different stages, and balance between accuracy and computational costs.

3.2. BFE- and MCFT-based modeling of PC bridge deck beams

In this paper, both BFE- and MCFT-based formulations are validated based on the results of multiple full-scale load tests on PC bridge deck beams. In BFE modeling, the structure is discretized into beam finite elements, the member cross-section is subdivided into four-node isoparametric subdomains, and numerical integration is performed using a Gauss-Lobatto quadrature rule (Bontempi et al. 1995). The BFE model of the PC bridge deck beams is based on a discretization with 13 elements for half of the beam (Figure 3), with one outer element with rectangular cross-section at the support region, one adjacent element with linearly varying width of the beam web, and eleven inner elements with I-shaped cross-section. The beam cross-section is subdivided into nine quadrilateral isoparametric subdomains. Numerical integration is based on an 8×8 Gauss-Lobatto integration scheme. Moreover, eight sampling cross-sections are considered for each beam finite element.

In MCFT-based modeling, the structure is discretized into bi-dimensional constant strain triangle (CST) finite elements with smeared reinforcement representing the stirrups. The structural modeling is complemented by truss elements, attached to the concrete mesh, to reproduce discrete steel reinforcement and prestressing strands. The MCFT plane-stress finite element model of the PC beams is based on a discretization with 1128 CST finite elements for half of the beam under four-point loading (Figure 3). Two additional models based on a mesh with 1880 and 2256 CST finite elements are also considered for numerical analysis of non-symmetric load tests under three-point bending of a PC beam with or without the top slab, respectively. The stirrups are modeled as smeared reinforcement over the beam volume. Longitudinal

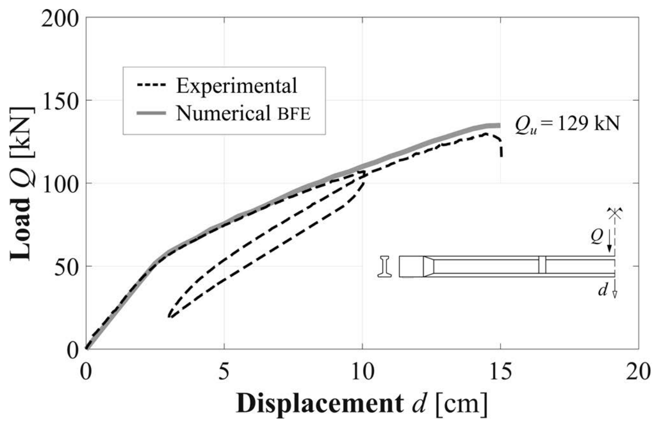


Figure 4. Experimental vs numerical (BFE) results in terms of load Q versus midspan displacement d of PC beam without RC slab under three-point bending test with shear span ratio $\alpha \approx 1.00$.

bars and strands are modeled as truss elements built over the CST mesh. Separate nodal points are considered at the beam-slab interface with vertical and longitudinal links to model the beam-to-slab interaction (Anghileri & Biondini 2022, 2023). The model discretization and numerical integration were selected to achieve an optimal trade-off between accuracy of results and computational cost.

In the BFE model, the constitutive stress-strain relationships of concrete are based on the Saenz's model and an elastic-plastic law for concrete in compression and tension, respectively. In the MCFT-based model, for concrete in compression the Hognestad parabola is selected with compressive strength related to transversal principal strain to account for cracking effects. For concrete in tension, the behavior is linear up to cracking with a post-cracking softening branch accounting for the tension stiffening effect. These models were calibrated based on the results of laboratory tests, including eight uniaxial compressive tests on cylindrical concrete samples carried out to obtain complete stress-strain curves (Anghileri & Biondini 2025b). A bilinear hardening constitutive law is assumed for longitudinal reinforcing steel, transversal stirrups, and prestressing steel in both BFE- and MCFT-based models. Material mechanical properties and the residual prestressing level are based on the mean value of the outcomes of experimental laboratory tests (Table 1).

4. EXPERIMENTAL VALIDATION

4.1. Nonlinear structural analysis

The nonlinear structural analysis of the PC bridge deck beams was performed to validate the finite element formulations against the results of multiple full-scale load tests within the BRIDGE|50 research project. The nonlinear analyses were also used to investigate the role of the RC slab in the structural behavior of the PC beam and the transition from flexural to shear failure mechanisms. The experimental load protocol consisted of an initial loading phase up to concrete cracking, a pause under load to assess the concrete cracking pattern, followed by unloading, and a final reloading phase up to beam collapse. The nonlinear finite element analyses were carried out under monotonic loading. The BFE model was adopted to validate the numerical predictions against the experimental results of the full-scale three-point bending load test ($\alpha \approx 1.00$) of the PC bridge deck beam without the top RC slab. The MCFT-based model was used to account for shear effects associated with four-point bending tests ($\alpha \approx 0.68$, 0.47, 0.32) and the beam-slab interaction.

4.2. Three-point bending tests: Flexural failure

The testing program is currently ongoing. The first four load tests have been carried out with shear span ratio $\alpha \approx 1.00$ (i.e., three-point loading) to favor a pure bending failure. Moreover, to study the behavior of the PC beam alone, one beam was tested under three-point loading up to failure after the removal of the top RC slab. Figure 4 compares BFE-based numerical results with the experimental outcomes in terms of load Q versus midspan displacement d, net of self-weight and prestressing, for the PC beam without top RC slab under three-point loading ($\alpha \approx 1.00$). The experimental results included a preliminary loading phase up to Q=107 kN, a pause under load for assessment of the cracking pattern and dynamic testing, a subsequent unloading, and final reloading up to collapse (Q=129 kN). The tested beam exhibited structural failure that began with crushing in compression of the top RC slab at the critical region (midspan) and then propagated through the entire depth of the beam. After the full-scale load test, the PC beam was placed on supporting New Jersey barriers in the testing site to investigate concrete crack pattern and failure mechanism (Figure 5a). The close correspondence between numerical and experimental results validates the finite element formulation, the modeling strategies, and the results of the diagnostic activities. The structural response of the beam is also reproduced with high accuracy by using the MCFT-based structural model (Anghileri & Biondini 2025b).

Key factors to be investigated for the PC deck beams include the influence of the construction phases of the viaduct and the actual degree of collaboration between PC I-beams and top RC slab. In fact, the RC slab was cast with the PC deck beams already assembled and in place under the effects of beam self-weight and prestressing action. Therefore, considering the type of structural failure observed in the three-point bending tests of PC beams with RC slab (i.e., crushing of top slab and lack of connection between beam and slab at midspan at incipient collapse), the possible lack of interaction between beam and slab was considered using separate MCFT-based finite element models connected at the interface by means of links. In the longitudinal direction, links are considered rigid up to concrete cracking and elastic after cracking, with stiffness estimated to best fit the experimental results (Anghileri & Biondini 2025b). Figure 6 compares numerical and experimental results in terms of load Q versus midspan displacement d, net of self-weight and prestressing, for a PC beam with top RC slab tested under three-point loading. The PC beam with RC slab has been tested with a

(a) (b)

Figure 5. Failure mechanism after the full-scale load test based on three-point loading with shear span ratio $\alpha \approx 1.00$ of PC bridge deck beam (a) without top RC slab and (b) with top RC slab.

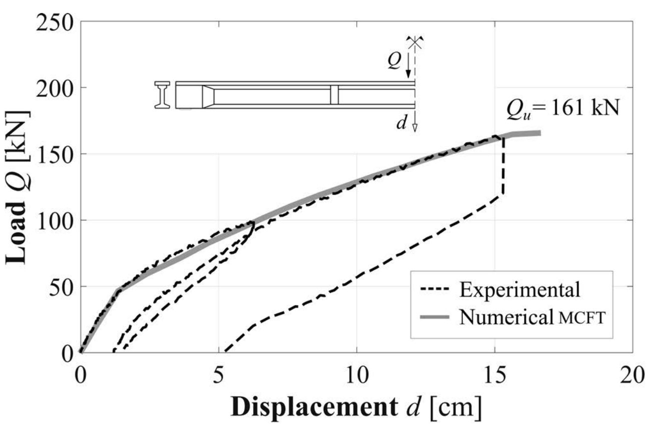


Figure 6. Experimental vs numerical (MCFT) results in terms of load Q versus midspan displacement d of PC beam with RC slab under three-point bending test with shear span ratio $\alpha \approx 1.00$.

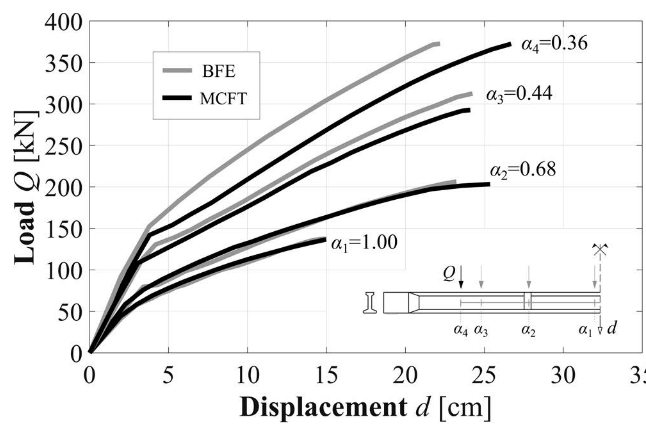


Figure 7. Load Q versus midspan displacement d of PC deck beam without RC slab: Comparison of BFE- and MCFT-based nonlinear analysis results for different values of the shear span ratio $\alpha = a/l$.

three-point bending test ($\alpha \approx 1.00$) using a first loading phase up to Q=100 kN, a subsequent unloading, and final load increase up to the collapse load Q=161 kN. The tested beam exhibited a structural failure associated with full crushing in compression of the top RC slab at midspan (Figure 5b).

The comparison of numerical and experimental results shows good agreement considering the pure bending failure of the tested beam. However, significant deviations may occur in the BFE model under four-point loading with reduced shear span ratios because of possible shear-dominant failures that the BFE formulation is unable to capture. Figure 7 compares the numerical results of the PC bridge deck beam without the top RC slab obtained using BFE- and MCFT-based models, in terms of load Q versus midspan displacement d, under four-point bending with different locations of the point loads. The results show that the reduction of the shear span ratio α leads to progressively larger deviations between the BFE and MCFT models, with a transition from flexural- to shear-dominated behavior and failure mechanisms. To this purpose, the MCFT-based model is used to investigate four-point bending tests.

4.3. Four-point bending tests: From flexural to shear failure

Additional tests have been performed under four-point loading with multiple shear span ratios to investigate the flexural-shear interaction and shear-dominant failure mechanisms. Figure 8 and Figure 9 show the numerical (MCFT) versus experimental comparison, in terms of load Q versus midspan displacement d, net of self-weight and prestressing, for the PC bridge deck with top RC slab tested under four-point loading with shear span ratio $\alpha \approx 0.68$ and $\alpha \approx 0.47$, respectively. The PC beam tested with span ratio $\alpha \approx 0.68$ was subjected to a preliminary loading phase up to Q=134 kN, a subsequent unloading and final reloading up to the collapse load Q=254 kN. The PC beam tested with span ratio $\alpha \approx 0.47$ has been associated with a single application of the load up to Q=363 kN. The failure of these PC beams occurred with full crushing in compression of the top slab in the critical region close to the applied load (Figure 10). It is worth noting that when passing from three-point to four-point bending tests, the slip between the precast beam and the top slab became less important, and the structural response tended

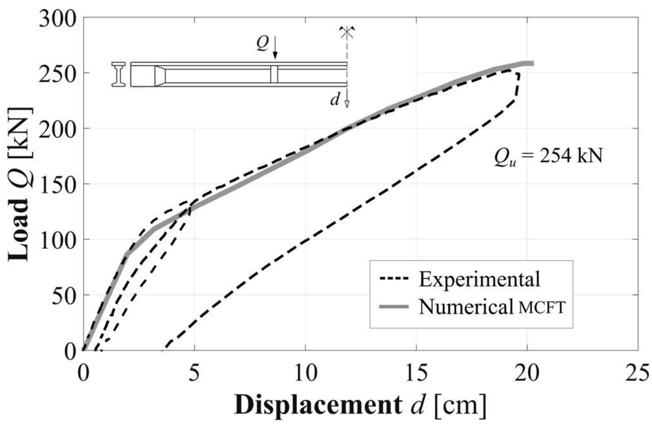


Figure 8. Experimental vs numerical (MCFT) results in terms of load Q versus midspan displacement d of PC beam with RC slab under four-point bending test with shear span ratio α≈0.68.

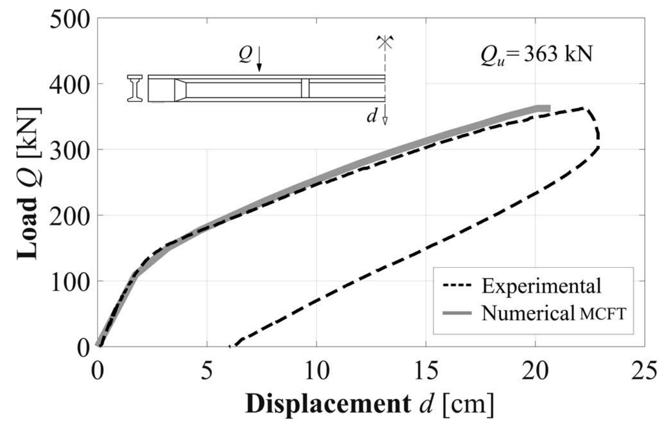


Figure 9. Experimental vs numerical (MCFT) results in terms of load Q versus midspan displacement d of PC beam with RC slab under four-point bending test with shear span ratio α≈0.47.

(a)

(b)

Figure 10. Failure mechanism of the PC bridge deck beams with top RC slab under four-point loading with shear span ratio (a) α≈0.68 and (b) α≈0.47.

toward that of a monolithic beam-to-slab connection. Good agreement between numerical and experimental results has also been achieved considering different damage scenarios, associated with concrete cover removal and both reinforcing and prestressing steel cuts, for the validation of finite element formulations combined with damage modeling strategies (Anghileri & Biondini 2025b). Moreover, the large amount of data and experimental outcomes allowed the extension of the validation process on a statistical basis for a probabilistic description of the structural response of the investigated PC bridge deck beams (Anghileri & Biondini 2025a).

To investigate a shear-dominant failure mechanism, a full-scale load test with a three-point bending scheme and shear span ratio α≈0.32 was performed on a PC beam with top RC slab. The experimental load protocol consisted of a single loading phase up to the collapse load Q=723 kN. Figure 11 compares the experimental outcomes with the MCFT-based numerical results, net of self-weight and prestressing, in terms of applied load Q versus midspan displacement d. The tested PC bridge deck beam exhibited a structural failure that initiated with the formation of an inclined shear-dominant concrete crack developed from the region around the bearing support to the point of application of the load (Figure 12). The close agreement between numerical results and experimental outcomes validates the nonlinear finite element analysis. These results are complemented by numerical analyses aimed at further investigating the residual structural behavior of the tested PC beams under shear failure and at supporting appropriate planning of the ongoing full-scale load tests. As an example, Figure 13 shows the MCFT-based numerical results of three-point bending tests on a PC beam without top RC slab, in terms of load Q versus midspan displacement d, for different shear span ratios. In addition, Figure 14 shows the collapse load Q_u versus shear span ratio α with an indication of the estimated crack pattern at collapse.

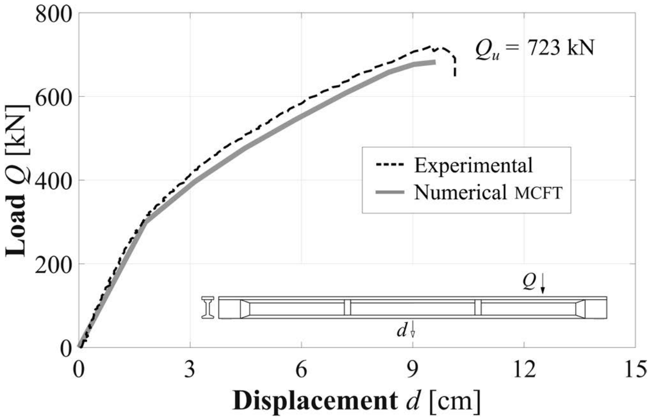


Figure 11. Experimental vs numerical (MCFT) results in terms of load Q versus midspan displacement d of PC beam with RC slab under three-point bending test with shear span ratio $\alpha \approx 0.32$.

5. CONCLUSIONS

The experimental validation and calibration of finite element models for nonlinear analysis of concrete structures based on BFE- and MCFT-based formulations have been presented. Structural modeling was developed using RC/PC beam finite elements and bi-dimensional finite elements for plane-stress analysis formulated in accordance with the MCFT and accounting for material nonlinearities associated with the constitutive laws of the materials, i.e., concrete, reinforcing steel, and prestressing steel. The calibration was based on the outcomes of laboratory tests on material mechanical properties and residual prestressing levels. The validation was based on multiple full-scale load tests on 50-year-old PC bridge deck beams under different loading conditions. The nonlinear structural analyses were used to accurately investigate the attainment of multiple limit states characterizing the structural response of the PC beams, including concrete cracking, steel yielding, and concrete crushing. The good agreement between experimental and numerical results allowed validation of the finite element formulations for PC beams with and without the top RC slab under both flexural and shear failure conditions. The role of the cast-in-situ RC slab in the structural capacity of PC bridge deck beams was also investigated considering the structural response of PC beams with and without the top RC slab under different shear span ratios. Based on the above, the main contributions of this paper include the calibration and validation of finite element formulations associated with the nonlinear analysis of concrete structures, using full-scale load tests on 50-year-old PC bridge deck beams under different loading conditions and failure modes. The formulations proposed provide a solid ground for a successful implementation in practice of life-cycle-oriented methods for design, assessment, maintenance, and management of aging RC/PC bridges. In addition, the results presented in this paper complement the experimental activities to further investigate the structural behavior of the tested PC bridge deck beams and support appropriate planning of the ongoing full-scale load tests. Future developments will be devoted to broadening and enriching the experimental results, further validating the finite element formulations under different exposure and damage conditions.

(a) (b)

Figure 12. Failure mechanism of the PC bridge deck beam with top RC slab under three-point loading with shear span ratio $\alpha \approx 0.32$.

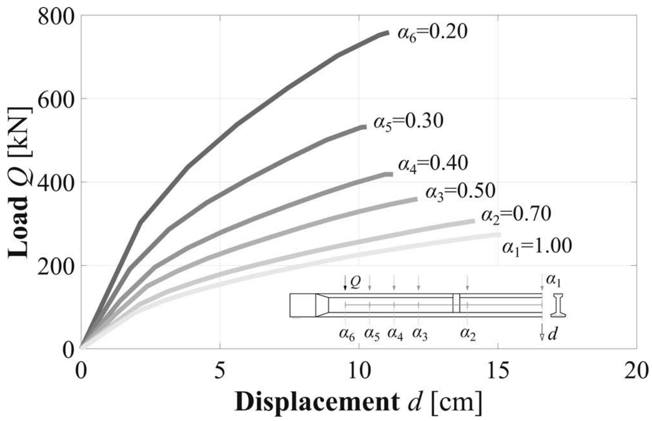


Figure 13. MCFT-based numerical results of three-point bending tests on PC beam without top RC slab: Load Q versus midspan displacement d for different values of the shear span ratio α.

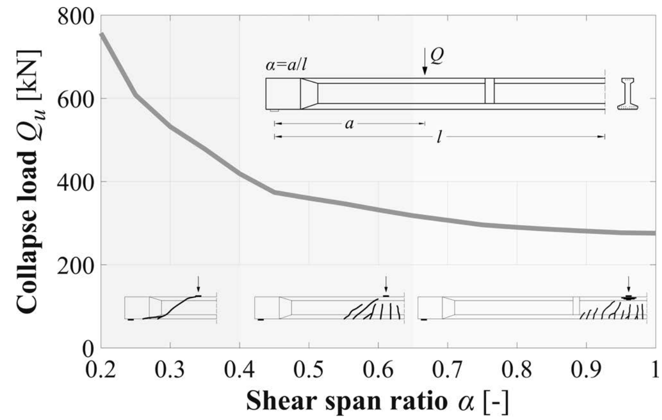


Figure 14. MCFT-based numerical results of three-point bending tests on PC beam without top RC slab: Collapse load Q_u versus shear span ratio α and crack pattern at collapse.

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