

Editorial

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Anil K. Agrawal, Dist. M. (ASCE), Ph.D., P.E.

Editor-in-chief, International Journal of Bridge Engineering, Management and Research, Professor and Chair of Civil Engineering, The City College of New York, New York, NY, 10031

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It is my pleasure to publish the October issue (4th issue) of Vol. 3 of the International Journal of Bridge Engineering, Management and Research. In this issue, we are pleased to bring to you six papers in innovative areas of bridge engineering.

The design of reinforced concrete highway barriers is a safety-critical process that requires strict compliance with regulatory provisions such as the AASHTO-LRFD bridge design guidelines. Current engineering practice relies heavily on manual, iterative, and heuristic calculations to satisfy complex nonlinear material and mechanics constraints. Although large language models (LLMs) demonstrate strong generative capabilities, their direct application to structural engineering remains limited by hallucination risks and insufficient physical grounding. To address these challenges, the paper entitled “**A Lightweight Multi-Agent Framework for Automated Concrete Barrier Design**” proposes a novel “generation–evaluation–optimization” closed-loop framework for automated concrete barrier design using the multi-agent orchestration capabilities of AutoGen. Experimental results demonstrate that the proposed agentic framework achieves over 98% design accuracy, significantly outperforming standalone general-purpose LLMs. More importantly, the study reveals that design performance is not necessarily correlated with model scale, where an 8B-parameter lightweight model could outperform unconstrained 631B-parameter flagship models. This finding highlights the potential to substantially reduce computational costs while improving the accessibility of AI-assisted engineering tools for industry applications.

Tunnels are inherently resilient structures whose service life commonly extends well beyond the intended design period, resulting in a substantial number of tunnels that have been in operation for over five decades and more. From this perspective, addressing tunnel aging requires maintaining functionality and ensuring continued safe and reliable

operation. This involves a series of coordinated efforts in maintenance, risk management, and refurbishment to guarantee consistent service over extended lifespans. The paper entitled “**Refurbishment of Existing Tunnels: Common Practice and Future Perspectives**” aims to provide an up-to-date overview of the international framework for the management and refurbishment of existing tunnels. Relevant guidelines and standards adopted worldwide are examined and compared, highlighting common principles and distinctive features. Established techniques for maintenance, structural reinforcement, rehabilitation, and upgrading are then deeply reviewed, providing practical examples and illustrative images from construction sites. The paper concludes by discussing selected innovative approaches and emerging technologies shaping the future of tunnel management and refurbishment.

The paper entitled “**Behaviour of Reinforced Concrete-Filled Steel Tube Columns with External Socket Connections Under Seismic Loading**” evaluates a new external socket connection for reinforced concrete-filled steel tube (RCFST) columns intended for seismic applications. The connection incorporates a steel-tube discontinuity at the column-to-socket interface, where the main tube is intentionally terminated, and a thinner embedded sleeve is used to reduce moment transfer into adjoining elements and facilitate plastic-hinge formation away from the connection. Two full-scale specimens were tested under reversed cyclic loading to assess global behaviour, failure modes, strain demands, displacement capacity, and energy dissipation. The specimens had identical column geometry, while socket height, column concrete strength, and grout strength were varied based on analytical evaluations to intentionally induce different failure modes and examine their influence on seismic performance, although the analytical model used to guide these selections is not presented in this paper. One specimen exhibited flexural failure with hinge formation on the column outside the external socket, whereas the other developed shear failure on the column within the embedded region. Both tests confirmed that the connection components and

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footing or cap beam remained elastic, demonstrating that the proposed connection can maintain these adjoining components as capacity-protected elements. The results provide insight into the role of socket height and material strength combinations in governing failure mode and support the feasibility of achieving desirable flexural behaviour comparable to conventional cast-in-place columns.

Rollover stability is a critical concern for prestressed concrete bridge girders, particularly during handling, transportation, and erection when lateral restraint is limited. The growing use of long-span, slender sections has increased their susceptibility to rollover instability, especially under the combined influence of geometric imperfections, thermal gradients, eccentric loading, and inclination and deformation of bearing surfaces. Although these effects are well recognized in practice, current codes and design guidelines offer limited guidance, and many analytical and numerical models use simplifying assumptions that incompletely capture important nonlinear behaviors. Furthermore, experimental investigations remain scarce due to the complexity and cost of full-scale testing. The paper entitled **“Synthesis of Research on the Rollover Stability of Prestressed Concrete Girders Supported on Elastomeric Bearings”** presents a comprehensive review of rollover stability of prestressed concrete bridge girders supported on elastomeric bearings. The theoretical foundations and governing mechanisms are first summarized, including a discussion of the effects of geometric imperfections, temperature variations, wind, support conditions, and bearing rotational stiffness. Existing analytical, numerical, and experimental studies are critically examined, and key knowledge gaps are identified to guide future research and code development.

Chloride-induced corrosion of reinforcing steel in bridge decks, substructure elements, and marine infrastructure progressively deteriorates the bond at the steel–concrete interface, with consequences extending beyond local anchorage to the full member response: tension stiffening is reduced, crack spacing increases, and load-carrying capacity declines. The paper entitled **“Corrosion-Induced Bond Deterioration in Bridge Deck Concrete: Tension Stiffening, Crack Spacing, and Residual Bond Capacity in Fly Ash and Normal Portland Cement Systems”** reports findings from a direct tension test programme on thirty specimens, a legacy experimental dataset of enduring relevance given the persistent scarcity of direct tension data for corroded HVFA bridge deck concretes across three concrete types subjected to up to eight levels of accelerated corrosion, providing a structurally realistic characterisation of bond deterioration that complements the pullout test programme reported in the companion study. The results are interpreted within a six-stage cracking model and correlated with the Dickson Bridge field survey data, validating the laboratory results against field conditions. The data provided herein constitute a rare, structurally realistic specimen-scale dataset for three cementitious systems and support calibration of tension-stiffening models for corroded reinforced concrete members.

Resonances of rail bridges due to the passage of trains have been mainly investigated for single-span bridges. For multi-span bridges, it is important to determine whether

stronger resonance amplifications than those of a single-span bridge need to be considered. In the paper entitled **“Modal Analysis, Health Monitoring, and Train-Induced Resonance of Multi-Span Bridges”**, measurements of eight multi-span structures are presented with their natural frequencies and mode shapes: wooden floor beams, a concrete beam, a steel truss foot bridge, three road bridges, and two rail bridges. Typical effects such as multiple fundamental frequencies due to parallel beams and multiple spans, close eigenfrequencies of different mode shapes, clusters for simply supported or weakly coupled spans, regular separated frequencies for continuous beams and bridges are shown. The mode shapes are generally global, but sometimes more local mode shapes exist; for example, a longer mid-span is dominant in the first eigen mode. The consequences of multi-span bridges for train passages are discussed based on a frequency-domain theory where three spectra determine the resonance excitation, the axle-sequence spectrum of the train, the transfer function of the bridge, and the modal force spectrum. The method is applied to the two railway bridge examples: these are treated as single-span bridges at first and then the examples are extended to two-, three-, and multi-span bridges.

With this editorial note, it is also my pleasure to invite you to submit your papers addressing research with new and substantial contributions in bridge engineering to the International Journal of Bridge Engineering, Management and Research. The journal is committed to prompt peer review process and online publication of the paper within four weeks of acceptance. We also are committed to completing our peer review process within 90 days of paper submission. All accepted articles are generally published within three months of acceptance. You are invited to submit your papers to the next issue of the journal as soon as possible.

A Lightweight Multi-Agent Framework for Automated Concrete Barrier Design

Wanting Wang^{1,†}; Xiye Ma^{1,†}; Yuyang He¹ and Ran Cao^{2,*}

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Abstract: The design of reinforced concrete (RC) highway barriers is a safety-critical engineering task that requires strict compliance with regulatory provisions, such as the AASHTO LRFD Bridge Design Specifications. Current engineering practices rely largely on manual, iterative, and experience-driven procedures to satisfy complex material, geometric, and mechanical constraints. Although standalone general-purpose Large Language Models (LLMs) exhibit strong capabilities in knowledge representation and text generation, their direct application to structural engineering design remains challenging due to hallucination risks, numerical reasoning errors, and insufficient integration with physics-based analysis. To address these limitations, this study proposes a novel “generation-validation-modification” closed-loop framework for AI-assisted automated RC barrier design based on the multi-agent orchestration capability of AutoGen. Unlike standalone LLMs that directly generate design parameters from user prompts, the proposed framework integrates specialized agents for parameter generation, external mechanics-based calculation, target-interval evaluation, deviation diagnosis, and rule-based design modification. The performance of the proposed multi-agent framework (MAF) was evaluated using three barrier testing levels (TL-3, TL-4, and TL-5), including sixty RC barrier designs with different geometric configurations. Three DeepSeek models with different parameter scales, including DeepSeek (DS)-8B, DS-32B, and DS-671B, were investigated. All design results were evaluated according to Section 13 of the AASHTO LRFD Bridge Design Specifications, 10th Edition (2024). The results demonstrate that the proposed agentic framework (MAF-DS-8B) achieves a target-interval compliance rate of 98.3%, whereas the best-performing standalone LLM, DS-32B, achieves only 11.7%. These results highlight the potential of multi-agent architectures to improve the reliability, interpretability, and accessibility of AI-assisted engineering design systems for practical applications. The source code, agent prompts, and representative testing cases are publicly available at the project GitHub repository: <https://github.com/MXY820/barrier-design>.

Author keywords: Structural engineering; multi-agent systems; large language models; concrete barrier design; AutoGen; design automation

Introduction

Reinforced concrete barriers constitute a critical component of highway infrastructure, serving as the primary line of defense against vehicle encroachment and ensuring the safety of motorists, pedestrians, and adjacent structures. The design of these safety-critical elements is governed by stringent performance requirements, most notably the American Association of State Highway and Transportation Officials Load and Resistance Factor Design (AASHTO-LRFD) Bridge Design Specifications, Section 13, which

mandates rigorous evaluation of ultimate transverse resistance under vehicular impact loading.¹ Current practice in barrier design typically involves engineers performing yield-line analysis calculations either by hand or through bespoke spreadsheet implementations of AASHTO-LRFD provisions. This methodology requires not only familiarity with complex limit-state formulations but also deep expertise in reinforced concrete designs, material nonlinearity, and failure mode identification. Consequently, the design process is highly iterative and trial-and-error driven: engineers must propose initial geometries and reinforcement layouts, evaluate the resulting ultimate resistance capacity (R_u), compare it against required transverse impact forces (F_i) for specified test levels, and manually adjust parameters until compliance is achieved. This manual loop highlights a compelling need for computational methodologies capable of intelligently navigating the barrier design space while preserving the physical rigor mandated by governing specifications.

The past decade has witnessed transformative advances in artificial intelligence, with Large Language Models (LLMs)

*Corresponding Author: Ran Cao. Email: rcao@hnu.edu.cn

¹Research Assistant, College of Civil Engineering, Hunan University, Changsha, 410082, China

²Associate Professor, College of Civil Engineering, Hunan University, Changsha, 410082, China

[†]Equal contribution

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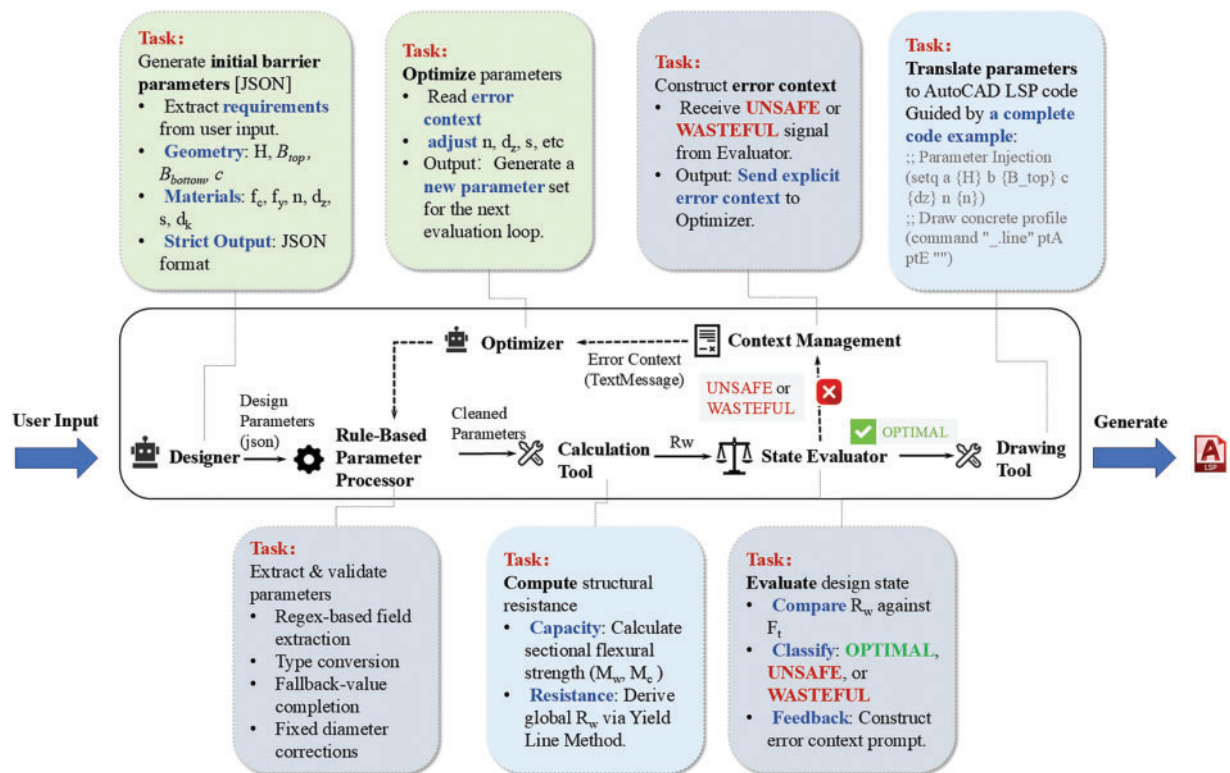


Figure 1. Proposed framework of the intelligent barrier design

emerging as particularly versatile tools capable of processing natural language specifications and generating structured outputs across diverse domains.²⁻⁵ In engineering contexts, researchers have begun exploring the application of LLMs to tasks ranging from code generation and technical documentation synthesis to conceptual design exploration and parametric modeling.⁶⁻⁸ In the specific domain of the architecture, engineering, and construction (AEC) industry, preliminary investigations have demonstrated the potential for LLMs to assist in automated code compliance checking, preliminary sizing of structural elements, and conceptual structural analysis.⁹⁻¹⁵ However, these applications remain largely exploratory, and their translation to safety-critical design tasks, where analytical errors carry profound consequences, has been appropriately cautious.

The direct application of LLMs to structural engineering design tasks encounters several fundamental obstacles that preclude their deployment as autonomous design agents. Foremost among these is the challenge of physical grounding; trained primarily on disembodied text, LLMs lack an intrinsic understanding of mechanical principles, material behavior, and spatial-geometric constraints.^{16,17} Moreover, the stochastic nature of LLM generation, while highly advantageous for creative ideation, presents unacceptable hallucination risks and reliability concerns for safety-critical infrastructure.¹⁸ To bridge this gap, recent pioneering studies in broader engineering disciplines have demonstrated that integrating domain-specific physical principles with AI agents can successfully guide generative models through

complex engineering spaces, such as automating advanced alloy discovery.¹⁹

Meanwhile, a significant advancement in AI system architecture has been the development of multi-agent collaborative frameworks,²⁰⁻²⁵ exemplified by platforms such as Microsoft's AutoGen,²⁶ which enables the orchestration of multiple specialized LLM instances collaborating toward shared objectives. Unlike monolithic LLM deployments, where a single model bears the sole responsibility for both reasoning and output generation, agentic frameworks distribute cognitive labor across distinct, specialized roles, such as planners, critics, executors, and optimizers, that communicate through structured conversational protocols.^{21,23} This multi-agent architectural paradigm has demonstrated particular efficacy in complex problem-solving domains requiring iterative refinement, robust error detection, and strict constraint satisfaction.^{20,22,26} This offers a highly promising pathway to overcome the limitations of single-agent systems in structural engineering design.

By leveraging the agentic orchestration capabilities of the AutoGen framework, this study proposes a "generation-validation-modification" closed-loop architecture to automate the design of reinforced concrete barriers. As illustrated in Fig. 1, the workflow is organized into five coupled modules that transform unstructured user intent into validated engineering outputs through iterative validation and rule-based modification.

Table 1. Design parameter definitions

Symbol	Description	Unit
grade	Barrier performance level (e.g., TL-3, TL-4)	–
H	Total height of the barrier	mm
B_{top}	Top width of the single-slope section	mm
B_{bottom}	Bottom width of the single-slope section	mm
c	Concrete cover thickness	mm
f_c	Concrete compressive strength	MPa
f_{y1}	Yield strength of stirrups	MPa
f_{y2}	Yield strength of longitudinal bars	MPa
s	Spacing of stirrups	mm
n	Total number of longitudinal bars	–
d_z	Diameter of longitudinal bars	mm
d_k	Diameter of stirrups	mm

Multi-Agent Design Framework for Concrete Barriers

Rule-based parameter processor

The workflow begins with user-provided natural language specifications, including safety requirements, test level, and geometric constraints. A Designer Agent interprets these inputs and produces an initial design proposal in the form of a structured JSON parameter set, determining key geometric variables (H , B_{top} , B_{bottom}) and material variables (f_c , f_y , n , d_z , s , d_k).

Because LLM-generated outputs are inherently stochastic, the proposed parameter set is subsequently processed by a Rule-based Parameter Processor, which performs deterministic field extraction, type conversion, and rule-based validation on the raw model response. Specifically, predefined parameter fields are extracted using regular-expression-based mapping, converted to their required data types, and stored in a fixed parameter dictionary. Rather than relying on a formal JSON Schema validator, the current implementation defines the expected fields, data types, and allowable ranges through this predefined dictionary, as summarized in Appendix Table A2.

It is important to distinguish these deterministic pre-processing operations from the design-level parameter adjustment performed later by the Optimizer Agent. The Rule-based Parameter Processor only standardizes and sanitizes the LLM output to ensure compatibility with downstream computation, whereas the Optimizer Agent modifies the design variables based on mechanics evaluation and code compliance. The resulting validated parameter dictionary is then passed to the external mechanics calculation tool for structural analysis. Table 1 summarizes the definitions, symbols, and units of the abbreviated design parameters used throughout the framework.

Mechanics calculation and state evaluation

Structural performance is evaluated based on the ultimate transverse resistance R_w , which is computed using yield line theory (Eqs. (1) and (2)) in accordance with the AASHTO LRFD Bridge Design Specifications:¹

$$R_w = \left(\frac{2}{2L_c - L_t} \right) \left(M_w + \frac{M_c L_c^2}{H} \right) \quad (1)$$

$$L_c = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2} \right)^2 + H \left(\frac{M_w}{M_c} \right)} \quad (2)$$

where H is the height of wall (m),

L_c is the critical length of yield line failure pattern (m),

L_t is the longitudinal length of distribution of impact force (m),

R_w is the total transverse resistance of the railing (kN),

M_c is the flexural resistance of cantilevered walls about an axis parallel to the longitudinal axis of the bridge (kN), and

M_w is the flexural resistance of the wall about its vertical axis (kN · m).

The computed resistance R_w is compared against the required impact force F_t . The design state is classified as:

- **UNSAFE:** $R_w < F_t$
- **WASTEFUL:** $R_w > \beta F_t$ (e.g., $\beta = 1.6$)
- **OPTIMAL:** $\alpha F_t < R_w < \beta F_t$, within a prescribed tolerance (e.g., $\alpha = 1.4$, $\beta = 1.6$)

This classification provides quantitative feedback for iterative modification and does not by itself establish global engineering optimality or comprehensive compliance with every AASHTO LRFD provision.

Context management and closed-loop modification

For nonoptimal states (unsafe or wasteful), the system constructs a structured error context based on the deviation

between R_w and the nearest target bound. This context is passed to an Optimizer Agent, which performs targeted parameter updates before recalculation.

The agent applies rule-based and mechanics-informed adjustments, such as:

- Increasing reinforcement or section dimensions for below-target designs.
- Reducing reinforcement or section dimensions for above-target designs.

The updated parameters are revalidated and re-evaluated, forming a closed-loop process. The iteration terminates when the calculated resistance falls within the target interval, or when the predefined maximum number of cycles is reached, in which case, the best available solution is returned.

Automated drafting and output generation

Once a within-target design is obtained, the system generates a parametric AutoLISP script²⁷ that encodes the full geometry and reinforcement layout. This script can be directly executed in a CAD environment to produce standardized 2D drawings.

The generated geometry not only supports engineering documentation but also provides structured data (e.g., nodal coordinates) for downstream applications such as numerical simulation and 3D modeling. A detailed example of the design process is shown in Fig. 2.

Experimental Design

To comparatively evaluate the target-interval adherence of the proposed multi-agent collaborative framework (MAF) and standalone LLM configurations, a systematic experiment was conducted. This section describes the selected models, structural design scenarios, controlled repeatability experiment, and performance metrics.

Model selection and configuration

The experimental matrix evaluates three distinct LLMs representing various scales of computational capacity: DeepSeek-8B, DeepSeek-32B, and DeepSeek-671B.^{28,29} The evaluated DeepSeek models were accessed via the SiliconCloud API platform.³⁰ These standalone models serve as baselines for comparison with MAF-DS-8B and MAF-DS-32B.

The comparison evaluates performance differences associated with integrating the selected models into the complete multi-agent workflow; it is not a component-wise ablation study.

Concrete barrier design scenarios

To ensure a comprehensive assessment across diverse impact demands, three testing levels of the barriers were selected in accordance with MASH (2016) and AASHTO LRFD (2024): TL-3, TL-4, and TL-5.

This study focuses on the single-slope barrier profile, a standard geometry in contemporary highway infrastructure.

For each test level, 20 unique design cases were generated by varying the barrier height within specified ranges (see Table 2).

Controlled repeatability experiment

To quantify uncertainty and repeatability independently of changes in barrier geometry, an additional controlled experiment was conducted for a TL-4 single-slope concrete barrier with a fixed height of 860.30 mm. The transverse design force was $F_t = 240.19$ kN, giving a study-defined target interval of 336.27–384.31 kN. Standalone DeepSeek-8B and MAF-DS-8B were each independently executed 50 times using the same external design task and target-interval criterion. Invalid outputs were retained in the total number of trials and counted as noncompliant. Wilson³¹ 95% confidence intervals were calculated for compliance rates.

Target design criteria

As detailed in Appendix Table A1, a review of nine actual design drawings from NCHRP RR 1109³² gives an average nominal-resistance-to-design-force ratio of 1.49. Allowing a practical fluctuation of approximately 10%, this study adopts $1.4F_t$ to $1.6F_t$ as a study-defined heuristic target interval for comparative evaluation. The interval is not a universally prescribed optimum in the AASHTO LRFD specifications and should not be interpreted as proof of minimum cost, minimum material use, or global engineering optimality. The framework iteratively modifies barrier parameters to move the calculated resistance toward this target interval.

Evaluation metrics

The target-interval compliance rate is defined as the proportion of experimental trials in which the calculated ultimate transverse resistance $R_{w,i}$ falls within the study-defined target interval (Eq. (3)):

$$\text{ComplianceRate} = \frac{1}{N} \sum_{i=1}^N I(L \leq R_{w,i} \leq U) \times 100\% \quad (3)$$

where $R_{w,i}$ represents the calculated ultimate transverse resistance in the i th trial; L and U denote the lower and upper bounds of the study-defined target interval, respectively; N is the total number of experimental trials; and I is an indicator function that equals 1 when the calculated resistance falls within the target interval and 0 otherwise.

To quantitatively characterize the extent to which the calculated resistance deviates from the study-defined target interval, an interval-violation metric is introduced. For the i th trial, the interval violation, e_i , is defined as:

$$e_i = \begin{cases} 0, & L \leq R_{w,i} \leq U \\ L - R_{w,i}, & R_{w,i} < L \\ R_{w,i} - U, & R_{w,i} > U \end{cases} \quad (4)$$

In essence, the interval violation e_i represents the shortest distance between the calculated resistance $R_{w,i}$ and the study-defined target interval $[L, U]$. If $R_{w,i}$ lies within the target

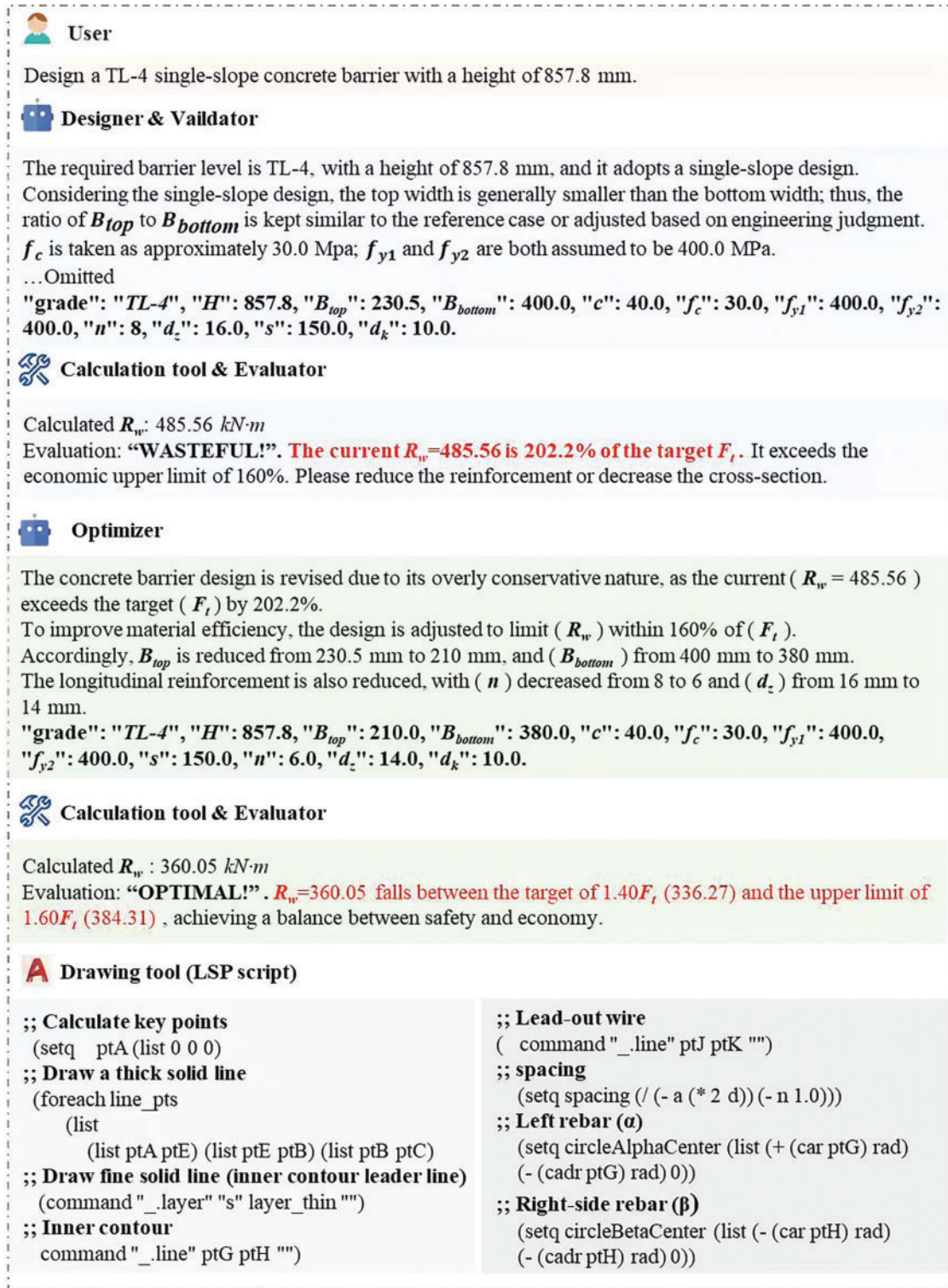


Figure 2. Example of the automated design process using the multi-agent collaboration

interval, e_i is equal to zero. If $R_{w,i}$ falls below the lower bound L , e_i is calculated as the distance ($L - R_{w,i}$); if $R_{w,i}$ exceeds the upper bound U , e_i is calculated as the distance ($R_{w,i} - U$). Therefore, a larger value of e_i indicates a greater deviation of the calculated resistance from the target interval.

To assess the overall deviation of the generated design values, the mean-squared error (MSE) over the entire dataset is computed as:

$$MSE_{interval} = \frac{1}{N} \sum_{i=1}^N (e_i)^2 \quad (5)$$

Table 2. Key parameters for the experimental design

	Parameters
Large Language Model	DeepSeek-8B, DeepSeek-32B, DeepSeek-671B
Barrier design levels	TL-3, TL-4, and TL-5
Number of design cases for each testing level	20
Barrier shape	Single-slope
Barrier height	TL-3: 685.80 mm–780.80 mm TL-4: 812.80 mm–907.80 mm TL-5: 1066.80 mm–1161.80 mm
Controlled repeatability condition	TL-4; $H = 860.30$ mm; $F_t = 240.19$ kN
Repeated runs per configuration	50 (standalone DeepSeek-8B and MAF-DS-8B)

Results and Discussion

Comparative performance of standalone LLMs

The performance of standalone DeepSeek (DS) models in the direct design of TL-4 concrete barriers is illustrated in Fig. 3. The generated resistance values frequently deviate from the study-defined target interval of $(1.4F_t-1.6F_t)$.

As shown in Fig. 3a, the DS-8B produces large output variability and several extreme outliers, including both below-target and markedly above-target resistance values. DS-32B and DS-671B show smaller extreme deviations in some cases, but most outputs still fall outside the target interval. These observations indicate limited target-interval adherence under the evaluated standalone settings.

Quantitatively, the overall target-interval compliance rates for DS-8B, DS-32B, and DS-671B are merely 6.7%, 11.7%, and 8.3%, respectively (Table 3). Fig. 4 presents representative unsuccessful outputs from the standalone models. Under the selected task and prompting conditions, increasing model scale alone did not consistently improve target-interval adherence. Results for TL-3 and TL-5 are provided in the Appendix.

Efficacy of the multi-agent collaborative framework

As shown in Fig. 5 and Table 3, the proposed MAF configurations achieve substantially higher target-interval compliance rates than the corresponding standalone LLM configurations in the original multi-condition evaluation. The overall compliance rates are 98.3% for MAF-DS-8B and 88.3% for MAF-DS-32B, compared with 6.7%, 11.7%, and 8.3% for standalone DS-8B, DS-32B, and DS-671B, respectively. These results support improved target-interval adherence, but do not by themselves demonstrate global engineering optimality.

Fig. 7b and Table 6 summarize the mean-squared interval violation, which measures the magnitude of resistance violations outside the target interval rather than conventional prediction error. The lower values obtained by the MAF

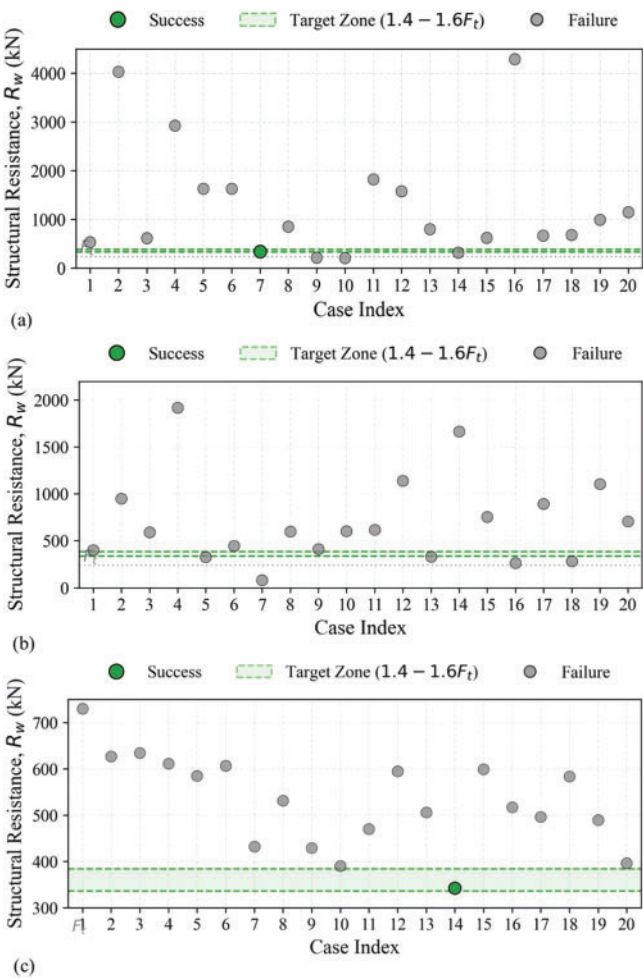


Figure 3. Target-interval compliance results for TL-4 barriers using general large language models: (a) DS-8B; (b) DS-32B; (c) DS-671B

configurations are associated with the complete closed-loop workflow integrating parameter generation, external mechanics calculation, target-interval evaluation, error feedback, and rule-based modification.

Under the evaluated settings, MAF-DS-8B achieved a higher target-interval compliance rate than MAF-DS-32B

Table 3. Comparison of target-interval compliance rates across different protection levels

Experimental group	TL-3	TL-4	TL-5	Compliance rate
DeepSeek 8B	0%	5%	15%	6.7%
DeepSeek 32B	10%	0%	25%	11.7%
DeepSeek 671B	5%	5%	15%	8.3%
MAF-DS-8B	100%	100%	98.3%	98.3%
MAF-DS-32B	80%	90%	95%	88.3%

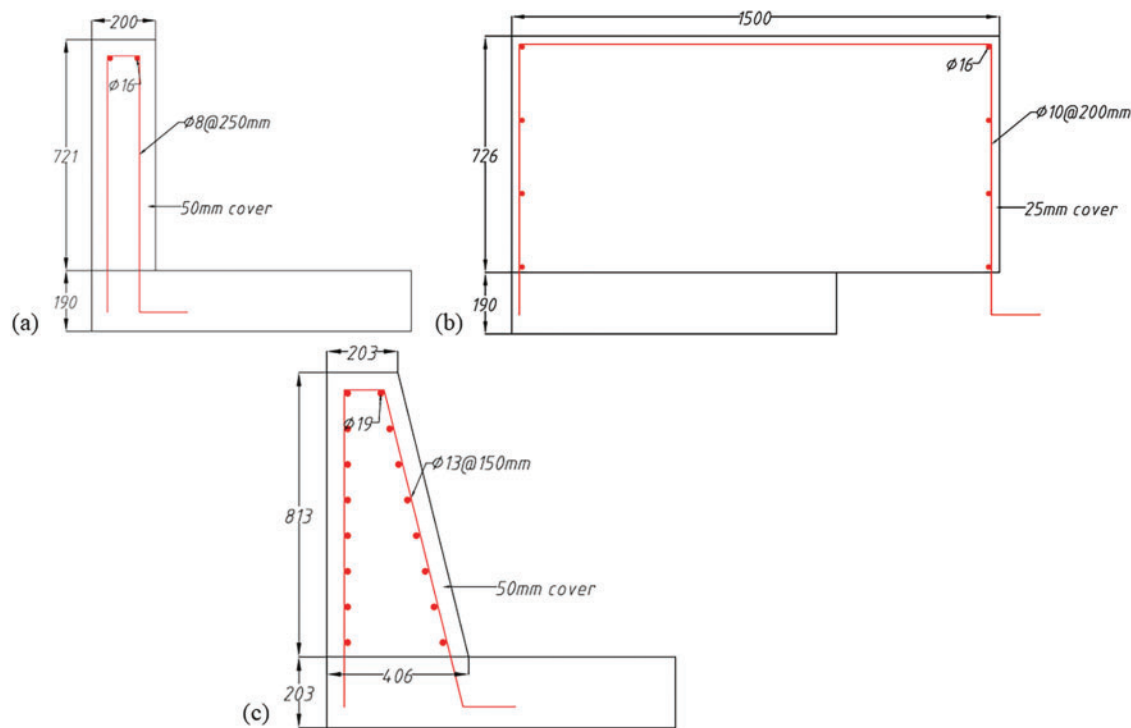


Figure 4. Representative standalone-LLM outputs: (a) below-target design; (b) geometrically inconsistent design; (c) above-target design

and the standalone DeepSeek-671B configuration. This task-specific observation suggests that structured agent collaboration may enable a lightweight model to achieve competitive performance.

To provide uncertainty estimates and a controlled baseline comparison, we independently executed standalone DeepSeek-8B and MAF-DS-8B fifty times each under a fixed TL-4 condition. The statistical results are summarized in Table 4. As shown in the table, when using the same foundation model (DS-8B), MAF produced substantially more stable and compliant design results than the standalone LLM.

The resistance values in Table 5 were independently recalculated using a separate verification script based on Eqs. (1) and (2). The checks confirm numerical consistency and adherence to the study-defined resistance interval.

Fig. 6a–c and Table 5 present representative design examples generated by MAF-DS-8B. Manual evaluation confirms that these designs satisfy the relevant code requirements while exhibiting reasonable structural configurations that

closely resemble the design drawings produced by professional engineers.

Limitations

The present study has several limitations that highlight key directions for future research. First, while evaluation metrics such as the target-interval compliance rate and deviation magnitude assess adherence to prescribed design drawing specifications, they do not directly quantify global engineering optimality, such as material consumption or construction cost. Second, the current comparison between standalone LLMs and the proposed Multi-Agent Framework (MAF) workflow represents a system-level baseline evaluation rather than a component-wise ablation study. Consequently, future work will separately isolate and evaluate the individual effects of deterministic validation, external mechanics calculations, state evaluation, and rule-based modifications. Third, more detailed statistical and reliability

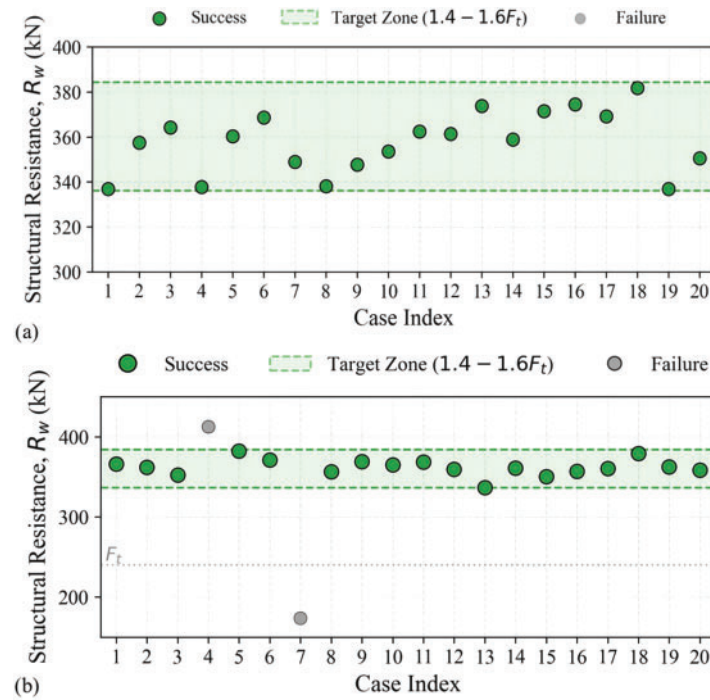


Figure 5. Target-interval compliance results for TL-4 barriers using the multi-agent framework: (a) MAF-DS-8B; (b) MAF-DS-32B

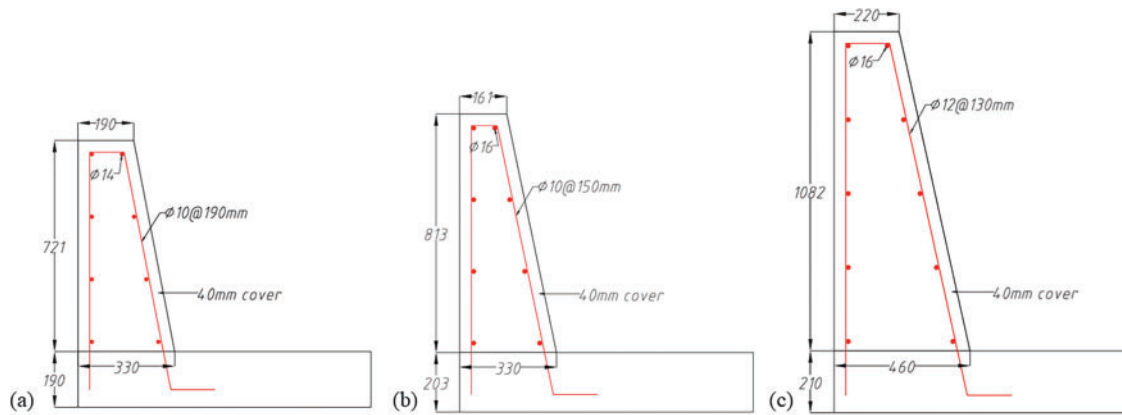


Figure 6. Representative concrete barrier designs generated by the multi-agent framework: (a) TL-3; (b) TL-4; (c) TL-5

Table 4. Statistical comparison between standalone DeepSeek-8B and MAF-DS-8B under a fixed TL-4 design condition

Metric	Standalone DeepSeek-8B	MAF-DS-8B
Number of trials	50	50
Within-target outputs	2	49
Out-of-target outputs	48	1
Target-interval compliance rate	4.0%	98.0%
Wilson 95% CI	1.1%–13.5%	89.5%–99.6%
Mean	1499.15 kN	360.98 kN
Sample variance	1621.82 kN	14.11 kN
MSE_{interval}	$3.82 \times 10^6 \text{ kN}^2$	9.35 kN^2

analyses are needed to rigorously evaluate agent performance and account for operational uncertainties. Finally,

our model size comparisons were limited to the DeepSeek series; evaluating a broader spectrum of foundation models

Table 5. Design parameters and verification results for the representative barriers shown in Fig. 6

Parameter	Unit	TL-3	TL-4	TL-5
F_t	kN	240.19	240.19	551.55
f_c	MPa	30	30	30
Reinforcement yield strength	MPa	420	400	400
Calculated R_w	kN	348.81	366.31	840.56
Target interval check (manual)	-	Yes	Yes	Yes

Table 6. Comparison of mean-squared interval violation for concrete barriers using different LLM configurations (kN²)

Experimental group	TL-3 (10 ⁴)	TL-4 (10 ⁴)	TL-5 (10 ⁴)	Average MSE (10 ⁴)
DeepSeek 671B	2.91	3.01	8.65	4.86
DeepSeek 32B	25.31	30.89	9.35	21.85
DeepSeek 8B	355.44	216.63	423.83	331.97
MAF-DS-32B	0.19	0.14	0.12	0.15
MAF-DS-8B	0	0	0	0

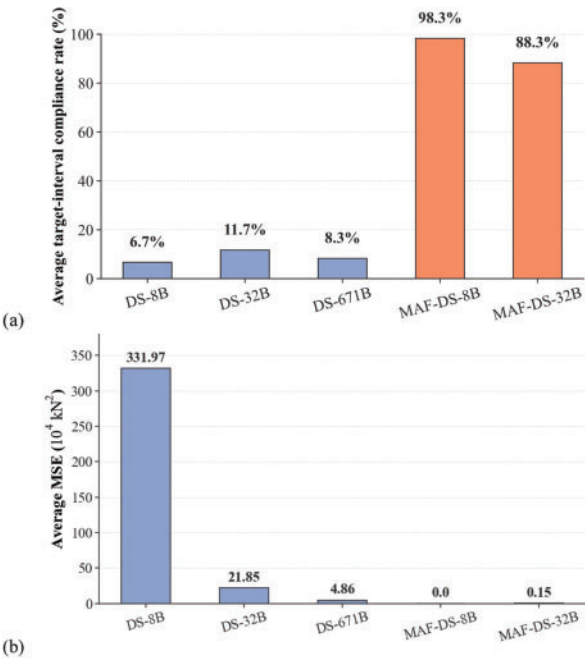


Figure 7. Quantitative comparison of the generated design results using different models: (a) average target-interval compliance rate; (b) average MSE

using the proposed benchmark remains an important subject for future investigation.

Conclusion

This paper presented a novel multi-agent collaborative framework tailored for the automated design of concrete

barriers. By structuring the design process into a modular agentic architecture integrated with deterministic validation layers and rule-based modification routines, the framework successfully bridges the gap between generative artificial intelligence and safety-critical structural engineering.

The empirical findings of this study yield several critical insights and contributions to the field of AI-assisted engineering:

- The proposed agentic methodology achieved target-interval compliance rates exceeding 98%. This represents a transformative advancement over standalone, general-purpose LLMs, which inherently lack the domain-specific constraints required for structural engineering tasks.
- Operating within the proposed framework: An 8B-parameter lightweight foundation model consistently outperformed unconstrained 671B-parameter flagship models. This demonstrates that specialized, constrained architectures can achieve superior localized efficacy for specific design routine tasks.
- By successfully orchestrating physical guardrails, structured error feedback, and heuristic parameter refinement, this work establishes a replicable template for integrating AI into highly regulated, code-compliant structural design domains.

While the current framework demonstrates robust capabilities in generating code-compliant structural designs, future research will focus on extending this agentic pipeline toward end-to-end engineering workflows. Specifically, the design outputs produced by the current system will serve as inputs to a downstream agentic framework that automatically generates high-fidelity simulation models in commercial finite

element analysis (FEA) platforms, such as ANSYS and LS-DYNA. This extension will establish a seamless connection between conceptual design generation and physics-based validation, advancing toward fully autonomous, closed-loop structural engineering systems.

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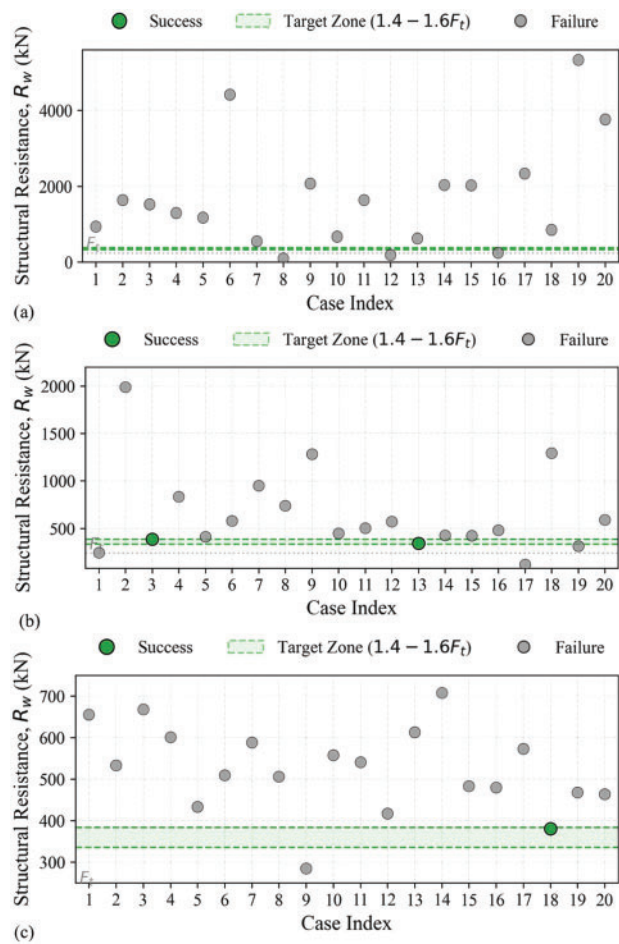


Figure A1. Target-interval compliance results of TL-3 barriers using general large language models: (a) DS-8B; (b) DS-32B; (c) DS-671B

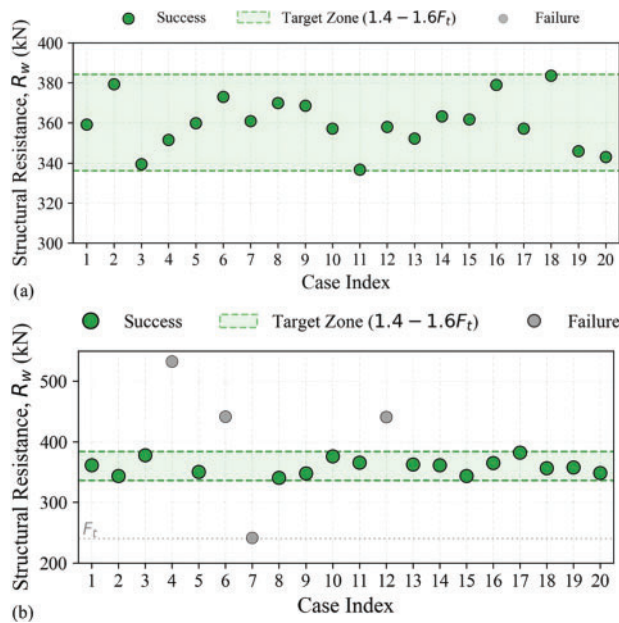


Figure A2. Target-interval compliance results of TL-3 barriers using multi-agent framework: (a) MAF-DS-8B; (b) MAF-DS-32B

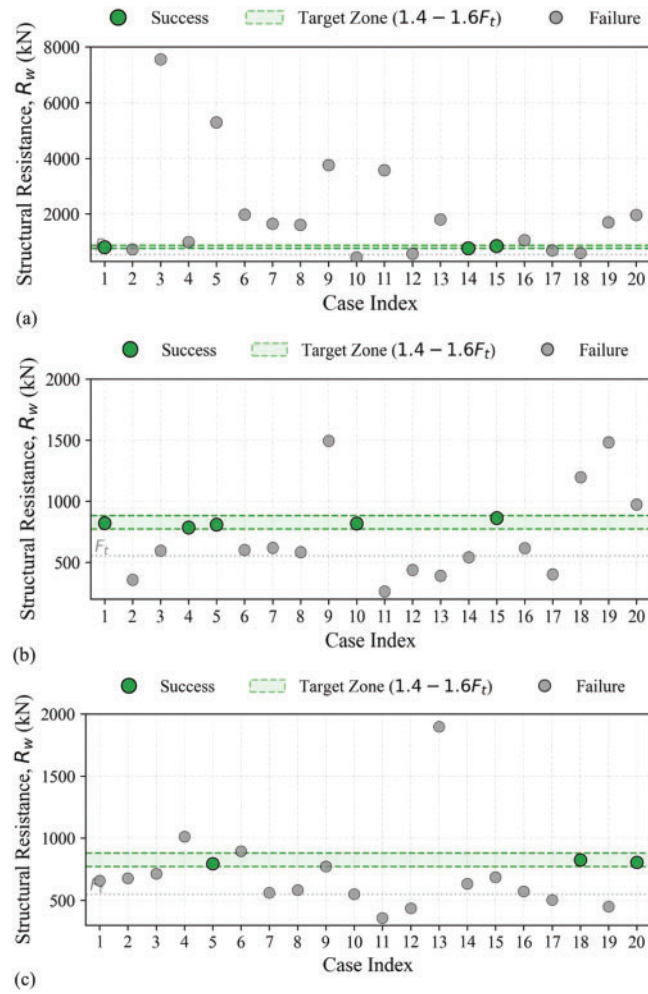


Figure A3. Target-interval compliance results of TL-5 barriers using general large language models: (a) DS-8B; (b) DS-32B; (c) DS-671B

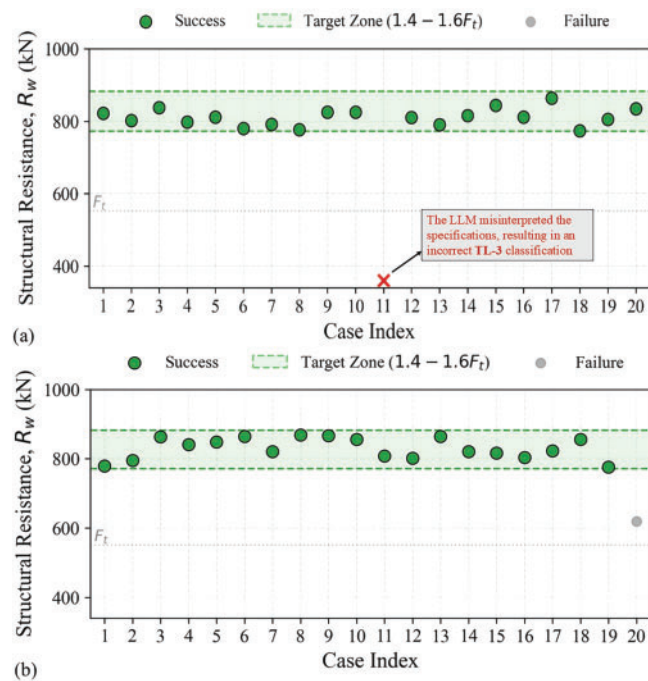


Figure A4. Target-interval compliance results of TL-5 barriers using multi-agent framework: (a) MAF-DS-8B; (b) MAF-DS-32B

Table A1. Design resistance and force ratios for NCHRP 1109 TL-series bridge rail design

Case	Barrier type	R_w (kips)	F_t (kips)	R_w/F_t
1	TL-5 Concrete Barrier	397	162	2.45
2	TL-5 Concrete Barrier	258	162	1.59
3	TL-4 Steel Post-and-Beam Railing	79.04	54	1.46
4	TL-4 Railing	119.64	68	1.76
5	TL-4 Steel Post-and-Beam Railing	79.04	54	1.46
6	TL-4 Railing	119.64	68	1.76
7	TL-3 Curb-Mounted Steel Post and Aluminum Rails	57.98	54	1.07
8	TL-3 Curb-Mounted Steel Post and Aluminum Rails	110.51	70	1.58
9	TL-3 Curb-Mounted Steel Post and Aluminum Rails	86.07	70	1.23
Average		—	—	1.49

Table A2. Parameter fields, fallback values, and deterministic correction rules used in the implemented workflow

Parameter	Data type	Unit	Fallback value or programmed action
Grade	String	-	TL-3
H	Float	mm	813.0
B_{top}	Float	mm	230.0
B_{bottom}	Float	mm	400.0
c	Float	mm	38.0
f_c	Float	MPa	28.0
f_{y1}	Float	MPa	400.0
f_{y2}	Float	MPa	400.0
s	Float	mm	150.0
n	Integer	-	8
d_z	Float	mm	d_z to 16 mm
d_k	Float	mm	d_k to 10 mm

Note: The listed fallback values are programmatic completion values used when a field cannot be extracted; they are not presented as universally recommended design values. The final two rows report the fixed correction rules applied to anomalous reinforcement diameters.

Refurbishment of Existing Tunnels: Common Practice and Future Perspectives

S. De Feudis*; A. Insana and M. Barla

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Abstract: Tunnels are inherently resilient structures whose service life commonly extends well beyond the intended design period, resulting in a substantial number of tunnels that have been in operation for over five decades and more. From this perspective, addressing tunnel aging requires maintaining functionality and ensuring continued safe and reliable operation. This involves a series of coordinated efforts in maintenance, risk management, and refurbishment to guarantee consistent service over extended lifespans. This article has the aim to provide an up-to-date overview of the international framework for the management and refurbishment of existing tunnels. Relevant guidelines and standards adopted worldwide are examined and compared, highlighting common principles and distinctive features. Established techniques for maintenance, structural reinforcement, rehabilitation, and upgrading are then deeply reviewed, providing practical examples and illustrative images from construction sites. The article concludes by discussing selected innovative approaches and emerging technologies shaping the future of tunnel management and refurbishment.

Author keywords: Maintenance; tunnelling; tunnel engineering; tunnel refurbishment; asset management

Introduction

As inherently resilient geotechnical structures, tunnels often remain serviceable well beyond their intended design life. Therefore, an increasing proportion of the global tunnel network now consists of aging assets that have been in continuous operation for more than five decades.¹ The literature indeed abounds with case studies documenting operational tunnels that have surpassed one hundred years of service.

Among the European countries with the highest tunnel density per unit of national surface area (Fig. 1), Switzerland, Italy, and Austria stand out as remarkable examples. In Switzerland, *Rhaetian Railways AG* operates a railway network of 384 km. Because of the Alpine terrain, this tallies around 77 km of tunnels, 59 km of which, corresponding to 115 tunnels, were realized between 1901 and 1914.^{2,3} Similarly, in Italy, *Rete Ferroviaria Italiana S.p.A.* manages about 24,500 km of railway lines, including 1,460 tunnels, two-thirds built before 1930.⁴ Likewise, *Autostrade per l'Italia S.p.A.*, operates over 2,900 km of motorways comprising 595 tunnels, corresponding to around 365 km, most realized during the 1960–1980 economic boom.^{5,6} Regarding Austria, instead, *ÖBB-Infrastruktur AG* operates and maintains 246 tunnels, with an overall length of 250 km, nearly 150 of which

are more than 100 years old and 35 are over 150 years old.⁷ Additional evidence of aging underground infrastructure from the U.S.A., Taiwan, the U.K., Germany, and Canada can be found in Zeidler et al.,⁸ Chiu et al.,⁹ Burgess et al.,¹⁰ Atkinson et al.,¹¹ Vetter et al.,¹² and Motallebi et al.,¹³ respectively.

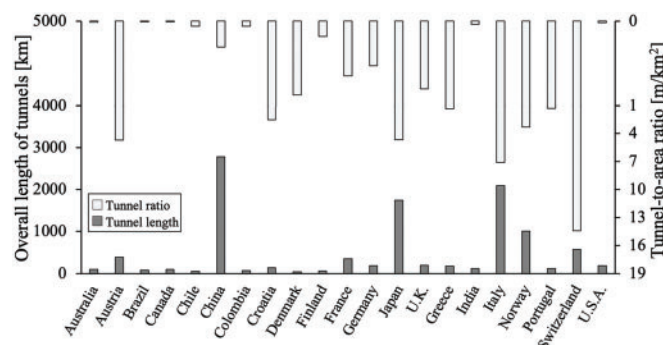


Figure 1. Motorway and railway tunnel length and tunnel-to-area ratio for some countries worldwide (modified from De Feudis et al.,¹⁴)

An additional noteworthy aspect emerges when examining long-term trends: the expansion of motorway and railway networks appears to have undergone a pronounced deceleration over recent decades.¹⁵ This is a symptom that attention is edging toward the refurbishment of the existing networks rather than expansion and development. As noted above, this shift is largely driven by the progressive aging of tunnel assets, which increasingly poses risks to owners, concessionaires, and users. For owners, aging infrastructure

*Corresponding Author: S. De Feudis.

Email: simone.defeudis@polito.it

Department of Structural, Geotechnical and Building Engineering, Politecnico di Torino, Turin, Italy

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threatens long-term asset and investment value; for concessionaires, it affects revenue stability and growth; and for users, it compromises safety and operational efficiency.

As aging tunnels continue to experience cumulative degradation, increasingly stringent safety requirements, coupled with evolving performance demands, make their management ever more critical. This highlights the paramount importance of regular and systematic inspection routines, reliable condition assessments, and the prioritization of refurbishment interventions. In response, research and practice are progressively converging toward innovative and longer-lasting refurbishment methods, effective management strategies, and emerging technologies to support the long-term resilience of tunnel infrastructure.

In this framework, this article focuses on the management and refurbishment of existing tunnel infrastructure. It offers a state-of-the-art review of the most established strategies and solutions for addressing tunnel aging and deterioration, while highlighting recent methodological advances and technological innovations. Through a synthesis of current knowledge and emerging trends, the article aims to inform effective management and refurbishment practices to ensure the long-term functionality and safety of tunnel assets.

Risk Management and Safety Assessment

Tunnel management presents many operational, risk minimization, and refurbishment tasks whose fulfilment allows the infrastructure to serve safely and consistently. It includes time-scheduling and resource allocation for inspections and investigation campaigns aimed at verifying the state of preservation of the structure and planning refurbishment works that influence tunnel serviceability. Tunnel management also involves the pre-codification of effective responses to incidents and emergencies, ideally supported by early detection through monitoring systems. Therefore, several countries and international organizations have developed guidelines for tunnel management and risk assessment.

For instance, following the ceiling panel fall in the Sasago tunnel (Figs. 2a and 2b) in 2015, the Japanese *Ministry of Land, Infrastructure, Transport and Tourism* (MLIT) issued the “Guideline for Periodical Inspection of Road Tunnels”,¹⁶ instructing standardized close-visual inspections of all road tunnels at least every 5 years. The inspection outcomes, involving not only structural components but also portals, joints, and mechanical and electrical facilities, are used to

classify tunnels on a uniform scale, with 4 distinct stages ranging from I to IV, that determine the need for monitoring and/or refurbishment. Tunnels receiving lower ratings have a shortened inspection interval, from 5 to 2 years.

In 2015, the Chinese *Ministry of Transport* (MOT) published the “Technical Specifications for Maintenance of Highway Tunnels”,¹⁷ standardizing management practices according to tunnel class, defined by length, traffic volume, and strategic importance. Such a procedure evaluates tunnel safety through three components, i.e., structural condition, mechanical and electrical equipment, and additional facilities, each assessed via routine and periodic inspections or overhauls at frequencies determined by tunnel importance and state of preservation. Defects are graded from 1 to 4, and their severity and location contribute to a weighted overall technical condition score through which the tunnel can be classified into categories ranging from 1 to 5. This category determines the inspection frequencies, varying from 1 to 3 months for routine inspections and from 1 to 3 years for periodic inspections, and guides the choice of the refurbishment approach. In this regard, daily patrols to verify normal operating conditions are mandatory, and tunnel cleaning is considered ordinary maintenance activity with a prescribed frequency based on tunnel class and category. Emergency and special inspections are also envisaged, the former for structure, which suffered from natural disasters, traffic accidents, or other abnormalities and the latter when extensive degradation requires further investigation.

In the same year, a decade after the Ted Williams Tunnel accident (Figs. 2c and 2d), the U.S.A. *Federal Highway Administration* (FHWA) released the “Tunnel Operations, Maintenance, Inspection, and Evaluation (TOMIE) Manual”¹⁸ to regulate road-tunnel management. The process starts with a review of as-built documents, previous testing procedures, refurbishment records, and incident histories, as required by the *Specifications for the National Tunnel Inventory*.¹⁹ An initial inspection, to be carried out before opening to traffic or within 2 years from *National Tunnel Inspection Standards*²⁰ issuing, establishes baseline tunnel conditions and highlights areas needing attention during the 2-year (extendable to 4 years with proper justifications) routine inspection cycle. The TOMIE Manual also instructs about in-depth (with level and frequency to be established on purpose based on the needs of the tunnel facility and routine inspection findings), damage, and special inspections when safety concerns arise or extraordinary events occur. Inspection results are used to rate tunnel condition, guide

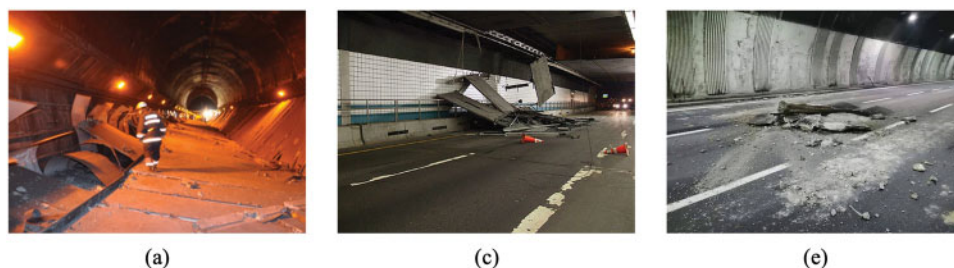


Figure 2. (Continued)



Figure 2. Tunnel local collapses that drove the development of modern tunnel guidelines: (a) Sasago tunnel ceiling panel fall and (b) subsequent refurbishment^{16,20}; (c) Ted Williams tunnel ceiling panel fall and (d) vehicle with a fatally injured occupant beneath²²; (e) Berté tunnel concrete vault collapse and (f) resulting cavity in the lining²³

preventive or corrective maintenance, and, eventually, plan structural reinforcement or rehabilitation. The TOMIE manual recommends that 70–80% of maintenance activities be preventive, reserving corrective actions for unforeseen events.

Concerning Europe, the *European Parliament and Council of the European Union* issued the “Directive 2004/54/EC”,²⁴ which establishes minimum safety requirements for tunnels in the Trans-European Road Network. Though it provides only limited guidance on inspection, whose interval should never exceed 6 years, and refurbishment, it sets general structural, equipment, and safety-system principles that newly built tunnels must satisfy and to which existing ones must be adapted within 10–15 years since its issuance. Safety measures must be supported by risk-assessment analyses carried out by qualified independent bodies, considering factors such as tunnel geometry, traffic volume and composition, and heavy vehicle proportion.

Following the fall of nearly 2000 kg of concrete from the Berté tunnel vault in December 2019 (Figs. 2e and 2f) and the approaching deadline for compliance with the “Directive 2004/54/EC,” the Italian *Consiglio Superiore dei Lavori Pubblici* (CSLLPP) released the “Linee Guida per la Classificazione e Gestione del Rischio, la Valutazione della Sicurezza ed il Monitoraggio delle Gallerie Esistenti”.²⁵ These establish a multilevel, preventive tunnel management procedure to anticipate damage that could compromise structural safety or network serviceability. The 6 analysis levels are structured hierarchically and progressively increase the degree of knowledge and analytical refinement, moving from inventory and condition assessment to quantitative safety verification and network-scale evaluation.

Level 0 — Census and data acquisition provides the knowledge baseline. It involves the systematic collection of geometric, structural, geotechnical, hydrogeological, traffic, and historical data, including past inspections, interventions, and significant events. Its objective is to identify potential risk factors and define critical areas requiring targeted investigation.

Level 1 — Defectiveness sheets completion translates the collected information into a standardized condition assessment. The tunnel is subdivided into inspectional sectors and observed defects are classified according to location, extent, magnitude, and gravity, the latter being a qualitative index to reflect the likelihood of present or future safety

implications. The result is a structured representation of the state of preservation of the tunnel, forming the basis for risk classification.

Level 2 — Attention Class (AC) assessment provides an expeditious risk-based prioritization at the national scale. Based on hazard, vulnerability, and exposure factors derived from previous levels, diverse risk domains are evaluated: i) global stability of the structure, with emphasis on ground-tunnel interaction; ii) local stability of the structure against block detachments; iii) road pavement conditions; iv) landslide mechanisms; v) seismic events; and vi) flooding phenomena. For each inspectional sector, ACs (High, Mid-high, Mid-low, Low) are assigned and combined into an overall AC to determine inspection frequencies (ranging from 2 to 12 months and from 1 to 6 years for routine and in-depth inspections), AC update and maintenance activities.

Level 3 — Preliminary safety assessment consists of simplified analytical evaluations aimed at verifying the consistency of the assigned ACs and investigating possible global or local instabilities, landslide mechanisms, or flooding phenomena. It is mandatory for mid-high AC tunnels and recommended for lower classes.

Level 4 — Accurate safety assessment involves advanced investigations and 2D/3D numerical modelling to quantify safety margins and assist decision-making on management, monitoring, and refurbishment of tunnels. It is triggered when Level 3 indicates insufficient safety margins or when Level 2 assigns a high AC.

Level 5 — Network resilience assessment applies to tunnels and underground infrastructures of national and international strategic relevance and evaluates the socio-economic and traffic consequences of potential service disruptions, extending the analysis from structural safety to system-level functionality.

A flow chart illustrating the hierarchical structure, sequential progression, and interdependencies among the 6 analysis levels defined by the Italian guidelines is illustrated in Fig. 3. Collectively, these guidelines, whose logical framework is systematically compared in Table 1, illustrate the principles of tunnel management and risk assessment, providing the background upon which targeted refurbishment strategies can be developed. The following section focuses on the most widely employed refurbishment interventions for existing tunnels in the currently established practice.

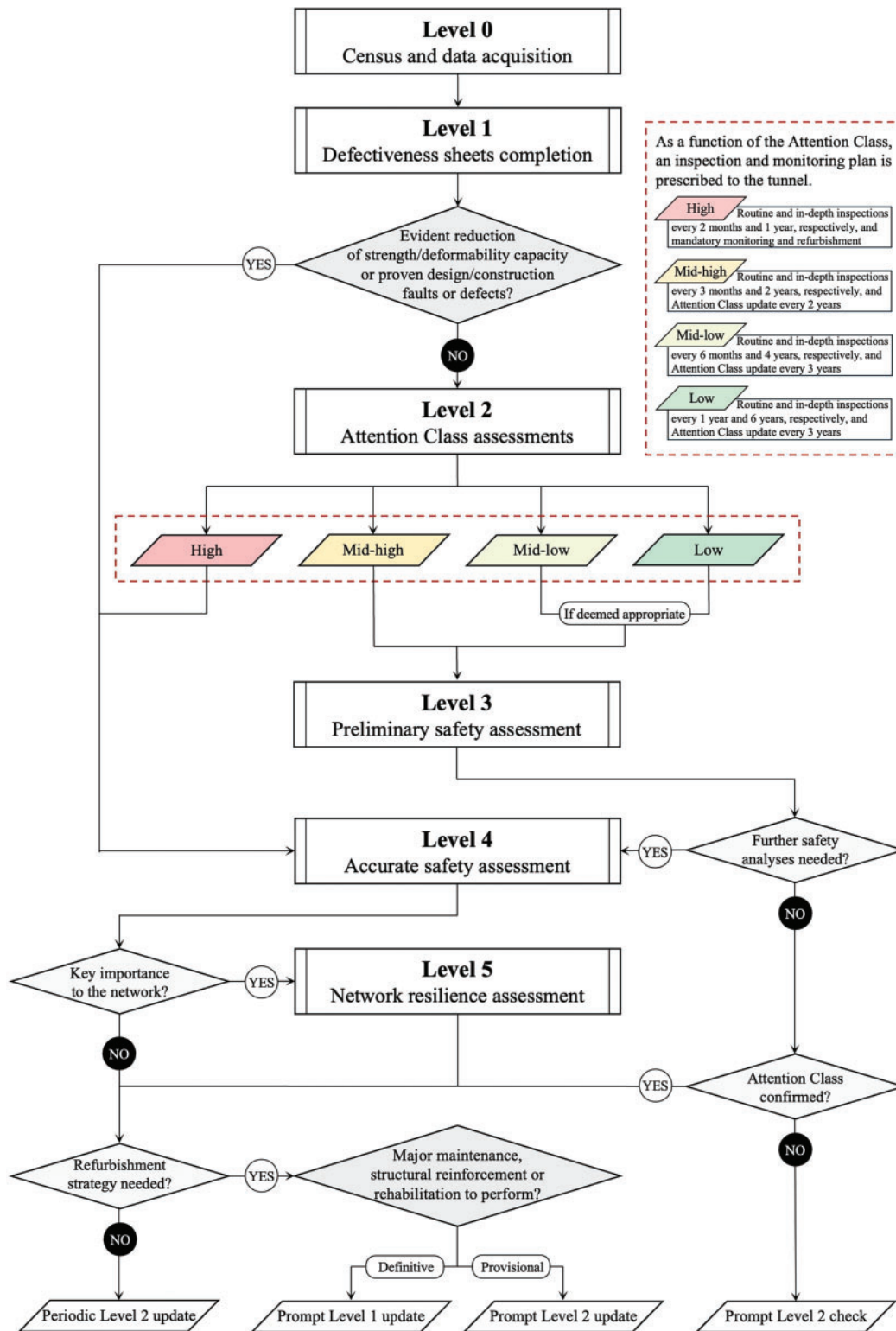


Figure 3. Multilevel procedure of the Italian guidelines and relationship between the different analysis levels (modified and implemented from CSLPP²⁵)

Tunnel Refurbishment

Despite their resilience, tunnels unavoidably undergo performance degradation due to progressive aging and deterioration. Hence, refurbishment, intended as the set of actions undertaken to keep safety and serviceability above

the minimum acceptable thresholds throughout operation, becomes vital. As schematically illustrated in Fig. 4, the performance level of a structure declines steadily if no interventions are undertaken, eventually dropping below acceptable limits. In contrast, timely maintenance, structural reinforcement, rehabilitation, or upgrading works introduce

Table 1. Comparison among international tunnel management guidelines

	Inspection frequency	Evaluation indicators	Classification systems	Notes
MLIT (JPN)	2–5 years based on tunnel preservation	Structural and nonstructural elements	I–IV, with IV denoting the best preservation	–
MOT (CHN)	1–3 months and 1–3 years for routine and periodic inspections based on tunnel importance and preservation	Structural, nonstructural elements, road pavement and traffic signs and marking	1–5, with 5 denoting the worst preservation	Emergency and special inspections are envisaged to deal with abnormalities
FHWA (USA)	2–4 years for routine inspections with in-depth inspections scheduled as required	Structural and nonstructural elements	No clear classification system, but prioritization of repair works	Damage and special inspections are envisaged to deal with abnormalities
CSLLPP (ITA)	3–12 months and 1–6 years for routine and in-depth inspections based on tunnel preservation	Structural, nonstructural elements, road pavement, and traffic signs and marking	ACs (High, Mid-high, Mid-low, Low), with High denoting the worst preservation	Special inspections are envisaged to deal with abnormalities

step-wise improvements that restore, enhance, or adapt the tunnel's performance to current or evolving functional requirements (De Feudis et al.),^{1,26,14}

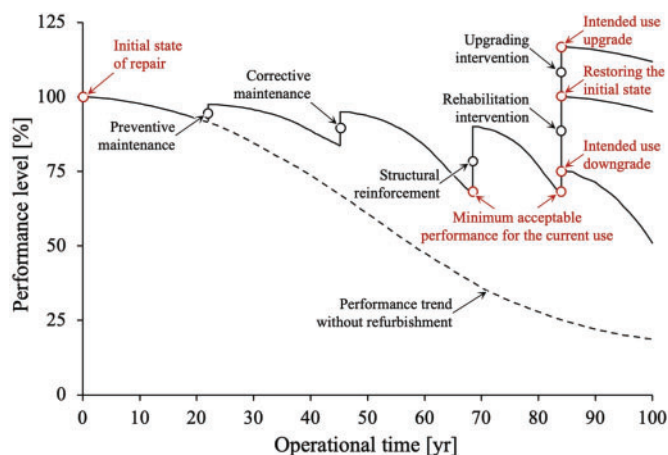


Figure 4. Life cycle performance of constructions (modified from ISO 15686-7:2006²⁷)

When refurbishment is not feasible or justified, repurposing may offer an alternative, whereby tunnels are reassigned to functions with lower performance demands, such as cultural or exhibition spaces,^{28–30} underground storage and logistics facilities^{31,32} or for more cutting-edge use.^{33–36}

Maintenance works

Maintenance work is intended as minor repairs aimed at guaranteeing standard safety levels during service, referred to as minor maintenance, or slightly extending the structure's lifetime, referred to as major maintenance, awaiting longer-term interventions to be designed and realized. Therefore, maintenance does not only imply repair works dealing with

tunnel structural components but also not structural ones, such as sidewalks and manholes, fire extinguishing systems, mechanical, electrical, and hydraulic equipment, safety systems, and much more. In the following, attention is focused solely on tunnel lining maintenance.

Maintenance works are, by definition, quick-to-apply, low-durability, and nearly noninvasive interventions that allow controlling or slowing down the evolution of defects in the short term. However, these generally do not provide reliable, long-lasting solutions but rather options to delay more laborious and onerous works depending on the current necessities.

Securing the intrados

When inspections reveal superficial deteriorations related to the materials and/or affecting the structural elements and the tunnel geometry (in particular, cracks and workmanship defects) with tolerable thicknesses, nets are arranged onto the tunnel intrados to prevent the detachment of thin concrete slabs. These coarse nets, made up of electro-welded steel wire or monolithic fiber-reinforced polymer meshes, are commonly coupled with a finer one, thus making small concrete fragments remain in place and fixed through mechanical or chemical wall plugs. Such maintenance work is versatile and prone to being coupled with most of the others discussed below.

Agresti et al.³⁷ documented electro-welded steel wire meshes used to temporarily cope with superficial, i.e., up to 5 cm deep, defects (mostly cracks, shallow voids of the lining and shotcrete deteriorations) in the innermost portion of the Scampitella tunnel (Avellino, Italy). Fig. 5a illustrates an analogous installation in the Citerna tunnel (Florence, Italy). In the U.K., instead, an innovative and novel variant was developed: the *Ram-Arch*[®] system. This consists of modular V-shaped corrugated elements that enhance self-support

against local detachments. Among others, as depicted in Fig. 5b, *Ram-Arch*[®] was utilized to maintain the Whiteball tunnel in Somerset.¹¹



Figure 5. Installation of (a) protective steel nets in the Citerna tunnel and (b) *Ram-Arch*[®] system in the Whiteball tunnel³⁸

Crack stitching

Despite being affected by cracks, countless tunnels are fundamentally unloaded, thus supporting only their weight. Such cracks are not a consequence of the stress-strain state in the structure but rather shrinkage, which occurs due to aging and depends on manufacturing, casting, and curing parameters. For instance, this is a recurrent situation with shallow motorway tunnels in Italy,³⁷ unless very specific issues arise.³⁹ In such a condition, crack stitching is a valid option to stop cracks from enlarging and seal such a preferential water path. This involves drilling micro-holes across cracks, which should be reinforced with steel rebars and sealed with resins. Preliminarily, the crack trace should be sealed to avoid pressurized resins pouring out.

Fig. 6a shows crack stitching works in the Croci di Calenzano tunnel (Florence, Italy). Specifically, Fig. 6b illustrates a close view of a stitched crack where its sealed trace (in grey) and stitching points (micro-holes nearby red dots) are visible.

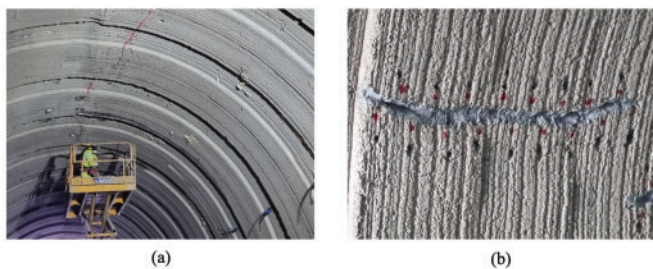


Figure 6. (a) Crack stitching operations in Croci di Calenzano tunnel and (b) close view of a stitched crack (courtesy of Lapo Baccolini)

Cavity filling

It is not uncommon that tunnel inspection outcomes reveal anomalies in the lining thickness^{40,41} or even voids^{42,43} and karsts⁴⁴ affecting the surrounding ground. To address this issue, the cavities are typically filled with high-performance concrete in the first case and with lightweight mortar or foam in the second. Whether high-performance concrete ensures structural continuity within the tunnel lining, lightweight

fillers achieve the double aim of avoiding excessive additional loads to the existing structure (due to the material weight and ground fragments falling from above) and allowing stress redistribution along the structure. Cavity filling may be coupled to structural reinforcements, which act as additional support to sustain the overload in the case of highly deteriorated concrete.

In Fig. 7, different applications in terms of material and technique, but with even planned durability, are shown. Fig. 7a is an image taken from ground void filling operations behind the Poggio tunnel (Genoa, Italy) through lightweight mortar. Fig. 7b, instead, depicts a lining anomaly being filled with sprayed concrete in the Olimpia tunnel (Alessandria, Italy). Indeed, despite the rehabilitation intervention planned in this tunnel,¹⁵ spot interventions were necessary to ensure worker safety.

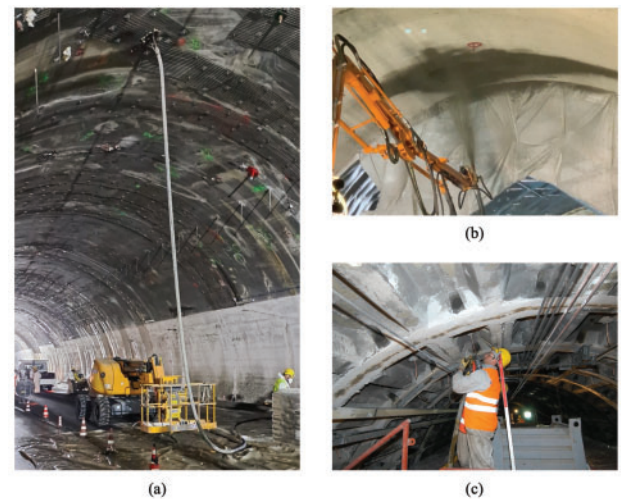


Figure 7. Filling operations of (a) voids behind the Poggio tunnel lining (courtesy of Alessandro Tonghini), (b) an anomaly within the lining of Olimpia tunnel, and (c) the gap between the Rome tunnel Metro Line A and the ground⁴⁵

Depending on the application and the design, cavity-filling interventions may also be longer-lasting. For instance, expanding foam was pumped in the gap between the tunnel lining of the Metro Line A in Rome and the surrounding ground, as visible in Fig. 6c.⁴⁵ The aim was to avoid water infiltrations, which induced several disruptions. Such maintenance work, realized in 2005, results still effective nowadays.

Corrosion protection or substitution of steel reinforcements

Steel rebars allow reinforced concrete tunnel linings to absorb tensile stresses. Therefore, their corrosion due to concrete cover deterioration may result in abrupt strength losses. Depending on the corrosion level, steel reinforcements can be either coated with an anti-corrosive agent or even replaced. In both instances, in the area to be treated, steel has to be entirely exposed through hydro-demolition of the concrete cover. Finally, this is commonly rebuilt through shotcrete.

Fig. 8 illustrates the rebar corrosion protection in the Monte Baldo tunnel (Treviso, Italy).



Figure 8. Rebar corrosion protection intervention in the Monte Baldo tunnel: (a) exposed rebars treated with protective coating and (b) concrete cover reconstruction

Groundwater drainage

In the absence of waterproofing systems, coping with groundwater infiltration is paramount. Indeed, water induces significant deterioration over time of concrete, especially in correspondence with the construction joints that act as preferential seepage pathways. Hence, when water incomes, but even concretions, freezing traces, and efflorescence are encountered during tunnel inspections, a drainage system is usually realized. To this end, polyvinyl chloride (PVC) drainage tubes are usually installed along the tunnel vault to convey groundwater inside the tunnel. Interaction with the concrete lining is avoided by employing half-fenestrated tubes able to collect groundwater only from behind it. Water is then funnelled into collectors through corrugated metal sheets that prevent water from falling onto the carriageway or tracks. As for nets, such maintenance work is also prone to being coupled with others.

Fig. 9a shows the drainage solutions with PVC drains and corrugated metal sheet collectors implemented in the San Fermo tunnel (Como, Italy).

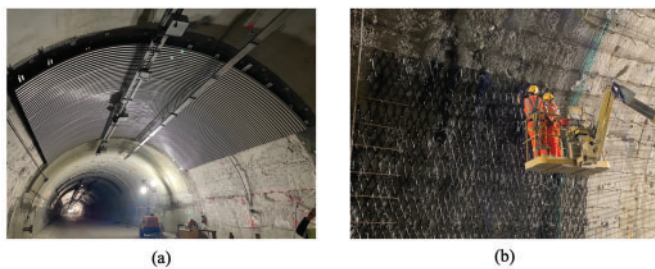


Figure 9. Installation of drainage systems with (a) fenestrated tubes and metal collectors in the San Fermo tunnel and (b) *Dolenco Drains*® in the Olympia tunnel

Alternatively, plastic modular drainage meshes could be arranged on the intrados by simply nailing them onto the tunnel wall, thus ensuring high installation rates. To protect them from fire and physical damage, these are commonly encapsulated into shotcrete layers. Hence, this kind

of drainage solution could be combined with thin surface scarification of the existing lining for clearance reasons. However, in contrast with the previous drainage methodology, this would allow groundwater to seep through the existing lining, thus not slowing down its aging process. Such meshes may also fit as drainage systems to be installed along with rehabilitation interventions.

Fig. 9b depicts the installation of the *Dolenco Drains*® in the Olympia tunnel. Such a drainage mesh was chosen to handle water infiltrations and relieve the related overpressures from behind the newly built lining.

Waterproofing the intrados

The philosophy is twofold for managing groundwater infiltrations: handling them with drainage or waterproofing systems. The latter option is usually preferred in cold climates where icing risk in tunnels is real. To this end, the *Norwegian Public Roads Administration* (NPRA) developed different water and frost protection interventions depending on the annual average daily traffic of tunnels.⁴⁶ While structural rehabilitation solutions concern only high-traffic tunnels, for low-traffic ones, shorter-term solutions are used, such as the Miljøhvelv inner lining. This consists of an aluminum cassette, mineral wool mats, and an overlay of high-quality PVC foil.

Conversely, considering the proper tunnel clearance, waterproofing envelopes made of polyethylene (PE) or foam layers encapsulated within shotcrete or even waterproof sprayed concrete⁴⁷ can be employed. In case of isolated water dripping at construction joints, these may be sealed with resins.⁴⁸

Maintenance works for masonry tunnels

The techniques used for maintaining masonry tunnels are borrowed from the masonry buildings field.⁴⁹ Hence, analogously to above-ground structures, the bed joint remake with either high-performance mortar (Fig. 10a) or steel or composite tendons (Fig. 10b) represents a viable solution for enhancing the bearing capacity of the structure. To manage groundwater infiltrations, instead, hydrophobic resins may be sprayed on the intrados or injected behind the extrados.



Figure 10. Bed joint remake of a motorway masonry tunnel in Italy through (a) high-performance mortar and (b) steel tendons (courtesy of Lapo Baccolini)

Structural reinforcements

Structural reinforcement refers to major repairs aimed at improving bearing capacity, stiffness, ductility, or a combination of all the preceding, fairly extending the structure's lifetime. This approach is usually opted for if rehabilitation is technically and/or economically unfeasible, but high safety levels and/or new standards are to be met as soon as possible. Hence, structural reinforcement shapes up to be a bargain between maintenance and structural rehabilitation, thus providing reliable but not definitive solutions to secure existing tunnels.

Lining bolting

Tunnel lining bolting, through steel self-drilling or *Swellex*[®] anchors or fibreglass dowels, is usually adopted when tunnel lining is affected by deep deteriorations such as triggering potential block detachments. These can happen due to the aging of the lining materials, honeycombing, flaking, and exfoliation, as well as interconnected crack patterns shaping unstable wedges. The same technique may also be used to underpin tunnel abutments to control time-delayed convergence.

Fig. 11 illustrates two installations in Italian motorway tunnels. If Fig. 11a shows drilling operations in the Della Maddalena tunnel (Genoa, Italy), Fig. 11b presents a picture of a completed intervention in the Monte Baldo tunnel. In the latter case, this consists of the anchorage of the lining through *Swellex*[®] anchors and electro-welded steel wire meshes for finer concrete fragments.

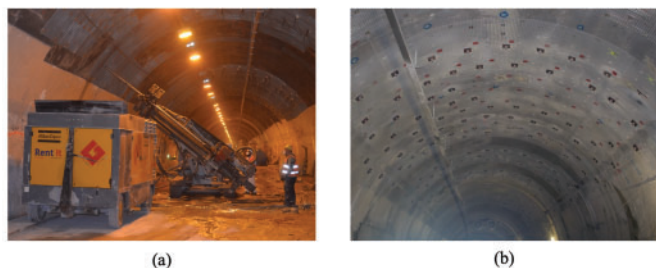


Figure 11. (a) Drilling operations in the Della Maddalena tunnel and (b) a completed lining bolting intervention in the Monte Baldo tunnel

Steel rib reinforcement

As an alternative to bolting, steel ribs are also capable of providing additional reinforcement to tunnel structures to cope with local detachments or even significant strength losses. Based on the aging conditions of the lining, these may be either fixed on the tunnel wall through high-performance plugs or anchored to the surrounding ground through bolts. However, in critical situations, ground-footed solutions may be required. The latter should be preferred in the case of shallow tunnels where local lining detachments in soil masses may also induce sinkholes. Moreover, based on the relative stiffness with respect to the existing tunnel structure, size of the intervention along the tunnel development and interposition of waterproofing sheeting, ground-footed steel

arching may also be intended and designed as a structural rehabilitation work.

Fig. 12 illustrates two examples from the Monte Baldo tunnel and the Pozzolatico tunnel (Florence, Italy), where dowelled (Fig. 12a) and ground-footed (Fig. 12b) steel ribs were respectively installed to address different degrees of deterioration of the tunnel lining.



Figure 12. Installation of (a) dowelled steel ribs in the Monte Baldo tunnel and (b) ground-footed steel ribs in the Pozzolatico tunnel

Invert arch construction or reinforcement

Not infrequently, it may happen to chance upon tunnels with unlined inverts. Invert arches and base slabs react to lateral thrusts, which, otherwise, would provoke the lining to accommodate horizontal convergence and coherently develop intrados and extrados cracks at the springlines and the crown, respectively. The same, depending on the deterioration level and with different orders of magnitude, may happen when the invert is lined but the structural element is highly damaged. In such circumstances, a horizontal slab may be built and, eventually, underpinned⁴ to strengthen the bearing system. In the presence of a structural element at the invert, the intervention is usually built above it and connected through metal pins.

Fig. 13 illustrates the works for the reinforcement of the invert arch carried out in the Bellosguardo tunnel (Florence, Italy).



Figure 13. Invert arch reinforcement works for the Bellosguardo tunnel (courtesy of Lapo Baccolini)

Fiber-reinforced material strengthening

Structural reinforcement is also achievable with Fiber-Reinforced Cementitious Matrices (FRCMs) or Ultra-High Tensile Strength Steel Fabrics (UHTSSFs). These represent compelling but flexible solutions able to increase the bearing

capacity of the tunnel lining (specifically tensile strength capacity) while preserving the clearance. To this end, however, the tunnel intrados should be roughened to guarantee the proper adhesion. Such high-strength materials may also be employed for systematic reinforcement of plain concrete linings against unexpected loads or seismic conditions. To this end, these would be arranged along the entire tunnel profile with regular strip distances and not only nearby defects.⁵⁰

Fig. 14 shows the application of UHTSSFs in tunnels belonging to the Bologna-Florence stretch of the A1 motorway for corrective (Fig. 14a) and preventive (Fig. 14b) structural reinforcement of the lining.

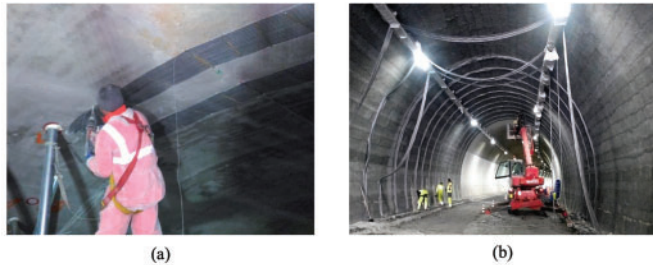


Figure 14. Installation of UHTSSFs in Bologna-Florence motorway tunnels for (a) corrective and (b) preventive structural reinforcement⁵⁰

Rehabilitation interventions

Rehabilitation means major repairs planned to largely extend the designed tunnel service life with long-term interventions whose goal is the integral or partial replacement of single structural components or even the entire bearing system. Moreover, these may also be aimed at fulfilling new requirements due to recent in-force legislation and standards, for instance, about electrification lines, fire protection, safety equipment, etc.

When facing severe aging conditions spread along entire tunnel structures, cost-benefit analyses, as well as technical considerations, often prove that rehabilitation interventions represent the best alternative compared to numerous local repair works of a limited lifespan. As a function of the deterioration level, existing tunnel linings can be replaced to various extents, thus involving them whole or only partially. Partial replacement is preferred from the economic and safety points of view. Indeed, apart from the obvious economic benefits, limiting demolition thickness to the needed extent would allow the residual lining portion to join fibreglass dowels, usually employed as pre-consolidation, in acting as temporary support during demolition.⁵ However, less invasive solutions affecting only the lining cortex, depending on the regular implementation along the tunnel development and the presence/absence of waterproofing sheeting, may also be utilized for structural reinforcement. The following briefly presents case histories from around the world.

Utilizing precast concrete elements for tunnel lining rehabilitation saves time, offers high-quality, durable

components, and requires less maintenance compared to cast-in-place concrete.⁵¹ Grossauer et al.² and Modetta et al.⁵² reported on the development, testing, and implementation of the *Standard Tunnel Construction* method. This approach was developed in response to the urgent need to renovate centenarian Rhaetian Railway tunnels with longer-lasting and more reliable interventions than the *Gentle Repair* approach. The *Standard Tunnel Construction* method envisages the rehabilitation of existing tunnels with precast elements under continued operation through a mobile protective shield. Supporting the lining with hydraulic cylinders, this protects workers and passing trains from falling masonry. After being tested with positive results, the *Standard Tunnel Construction* method was implemented in the Glatscheras tunnel, as shown in Fig. 15a, and the Sasslatsch tunnel I (Grisons, Switzerland). Fortunato and Barbieri³ reported that this approach was also used for enlarging the Mistail tunnel (Grisons, Switzerland). For all three, waterproofing was entrusted only to segment gaskets.



Figure 15. (a) Precast segment installation in the Glatscheras tunnel⁵³ and (b) precast predalle positioning in the Colle Marino tunnel⁶

The thicker the lining depth to rehabilitate, the heavier the precast segments to handle through the tunnel safely. Contingent upon the machinery and the available underground space, this might represent a limit. For instance, Grossauer et al.² identified a limit of 5.2 t per segment during testing. To cope with this issue, precast arched predalles can be employed as disposable shuttering. Such structural elements have experienced widespread use in recent years in Italy. For instance, Agresti et al.³⁷ and Mazzola et al.⁵ reported on the structural rehabilitation of, respectively, the Scampitella tunnel and the Colle Marino tunnel (Pescara, Italy) through arched predalles. In the former tunnel, three types of interventions, differing in terms of lining thickness and portions to rehabilitate, were designed based on the deterioration level. In the latter, instead, the complete replacement of the lining was planned for the entire tunnel length. Both tunnels were provided with waterproofing sheeting, as shown in Fig. 15b.

Alternatively, the rehabilitated tunnel lining can be cast in place. That is the case of the T3 tunnel (Bern, Switzerland), where partial structural rehabilitation took place.⁵⁴ The project envisaged equipping the structure with a ground-water collection system and replacing a 20 cm thick layer of the existing lining with an inner shell, as depicted in Fig. 16a. Furthermore, during the works, sprayed liquid and conventional synthetic waterproofing were tested. Despite the

former being initially preferred, the latter guaranteed better results and faster installation. A similar rehabilitation work, but using sprayed concrete, was experienced by the Manfreida tunnel (Genoa, Italy), as reported by Panebianco⁶ and illustrated in Fig. 16b. The same concept was also revealed effective for masonry tunnels, such as the Brunel's Thames tunnel (London, U.K.), which was waterproofed and relined to accomplish new clearance requirements.⁵⁵



Figure 16. (a) Modified shuttering used for T3 tunnel rehabilitation⁵⁴ and (b) shotcreting of the Manfreida tunnel⁶

Special mention, in terms of material choice, technique adopted, and/or underlying motivations, goes to Broch et al.,⁴⁶ Burgess et al.,¹⁰ Warren and Tromans,⁴³ and Bowers and Moss.⁵⁶ As stated above, Broch et al.⁴⁶ illustrated the NPRA solutions to cope with the icing risk in existing tunnels. For high-traffic tunnels, options envisaging the interposition of an insulation membrane behind a cast-in-place or precast concrete inner lining were implemented, as shown in Fig. 17a. Thereafter, the *Ekeberg* solution came. Combining precast elements at the springlines and sprayed concrete along the crown allows for cutting the construction time, thus costs, and withstanding high vehicle loads due to impacts at the bottom of the vault. At the time of the reference, 9 Norwegian tunnels with an overall length of about 13 km underwent the same structural rehabilitation work just outlined.

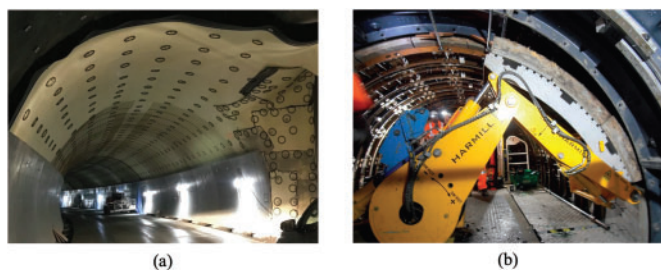


Figure 17. (a) Insulation of the Porsgrunn tunnel⁵⁷ and (b) train-mounted segment erector used in the Jubilee line⁵⁶

Burgess et al.¹⁰ and Bowers and Moss⁵⁶ discussed two similar applications to the London Underground, Northern and Jubilee lines, respectively. Both rehabilitation plans envisaged the existing lining replacement through metal segments. However, whether in the former, duplex stainless steel was selected to cope with corrosion from acids; in the latter, bolted spheroidal graphite iron segments were

used to deal with difficult geological conditions posed by shrinkable clays. Heating and ventilation due to train service accelerated clay desiccation, thus provoking a reduction of ground support to the original concrete ring over time. For both, most of the work was performed during night shifts, but for the Jubilee line, the original plan was sped up thanks to the use of two train-mounted segment erectors, one of which is shown in Fig. 17b.

Also, Warren and Tromans⁴³ chronicled the rehabilitation of two British tunnels. Higham and Strood (Rochester, U.K.) are centenarian Victorian tunnels built among 1819 and 1824 and rehabilitated in 2004 after several flooding events following sustained periods of heavy rainfall and chalk falls. The rehabilitation projects envisaged, among other things, lining the unlined sections (62% and 25% of Higham and Strood tunnels, respectively), brickwork repairs where required, and the installation of a drainage system. These were carried out through protective canopies, marking the operation time. Indeed, the new bearing system foundations were cast ahead of the canopy, the tunnel wall secured from beneath it and the concrete lining built behind it.

As for the Higham and Strood tunnels, the Bricchetto tunnel (Asti, Italy) is a centenarian railway tunnel that only underwent structural rehabilitation in 2014.⁴ This was a masonry tunnel with an unlined invert whose rehabilitation plan envisaged the construction of an underpinned "U"-shaped invert arch acting as a strut against horizontal convergence. For this purpose, a precise staggered pattern of masonry demolition and concrete casting was followed to ensure safety and avoid local instabilities. Then, as depicted in Fig. 18a, the existing vault was partially hydro-demolished, shaping accommodations for steel ribs made monolithic with the masonry lining through shotcrete.

Despite being less common, rehabilitation interventions may only affect invert arches or base slabs. This is the case of the Mont-Blanc tunnel, where the base slab was renewed over a 555 m section in the center of the tunnel.⁵⁸ Structural rehabilitation works were only handled during night shifts using custom-designed ready-for-use lifting gantries able to deal rapidly with slab loading, transportation, and unloading (Fig. 18b). To expedite the process, precast concrete male/female shear key slabs were used, thus ensuring easy-to-make and safe connections between adjacent ones. Such rehabilitation may also be carried out when only roadbed or track lowering is pursued.

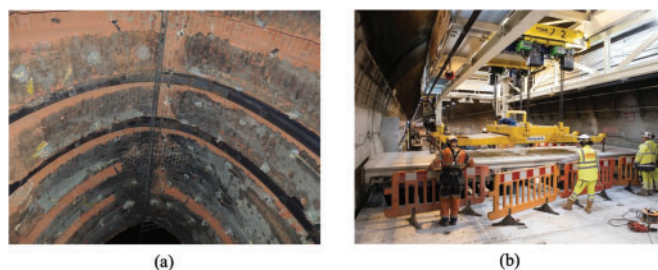


Figure 18. (a) Steel ribs accommodation in the Bricchetto tunnel⁵⁹ and (b) slab installation in the Mont-Blanc tunnel⁵⁸

Upgrading interventions

Upgrading involves major repairs intended for either structural rehabilitation or in-force legislation meeting or the fulfilment of new requirements due to upgrades in the intended use of tunnels. This proves necessary whenever a network capacity improvement is needed (e.g., the ever-growing demand for transportation leads to the quick obsolescence of tunnels) and new bore excavation is unfeasible for a list of reasons (economic, technical, increasing environmental sensitivity, etc.).

Upgrading interventions always involve enlarging the original size of underground structures. To this end, different techniques can be adopted depending on the ground typology, the need to keep service, and the standardization of the machinery to employ.^{11,59} From the perspective of closing the traffic lines, tunnels may be conventionally or mechanically re-bored. However, this rarely fits with the will of owners and/or concessionaires. Indeed, stopping major traffic lines affects the economy and society of the served area. Hence, several approaches to allow tunnel enlargement during continuous traffic have been developed. With this aim, traffic is always protected by purpose-built movable steel shields or the existing linings. In the following, case histories from all over the world will be briefly illustrated.

Being the eldest ones, thereby designed for traffic levels of yesteryear, masonry tunnels usually undergo upgrading. Among others, Gall et al.⁵⁵ and Zeidler et al.⁸ made some pioneering work in this sense regarding the Berry Street tunnel (Pittsburgh, Pennsylvania) and the Bergen tunnels (Jersey City, New Jersey). After almost 40 years of closure, the former was reopened to host a new busway connection. After rock bolting and shotcreting where needed, the existing horse-shoe brick-lined tunnel was enlarged into a circular concrete-lined one, passing from around 8 m width to 12 m diameter (see Fig. 19a). Compliant with the technology of the time, the Berry Street tunnel was provided with waterproofing and ventilation systems. The latter, analogously, were upgraded to improve the clearance gauge and safety, threatened by water leakages and ice accumulation during winter.



Figure 19. (a) Berry Street tunnel enlargement⁵⁵ and (b) concrete-filled section of the Grazia Bassa tunnel⁶¹

Years later, the same fate met the Grazia Bassa tunnel (Genoa, Italy), which, as for the two case histories above, was enlarged for transportation purposes through conventional excavation technique.⁶¹ In this case, pre-consolidation works were performed along the entire tunnel development since

the tunnel intercepted a weak rock formation. For safety reasons, at the most critical points, the Grazia Bassa tunnel was even filled with lightweight concrete and then dug from scratch (see Fig. 19b). This approach, but with a different technique, was adopted for the narrower Farnworth tunnel (Manchester, U.K.) tube.⁶² Indeed, after filling the bore with 7500 m of foamed concrete, this was re-bored with a Tunnel Boring Machine (TBM).

Tunnel closure was allowed in Berry Street, Bergen, and Grazia Bassa tunnels. As previously remarked, this is rarely possible except for critical cases (e.g., the Grazia Bassa and Farnworth tunnels extended close to other railway and motorway facilities) and already disused tunnels (e.g., the Berry Street tunnel remained unused for several years). Most often, during enlargement works, full, at most reduced, traffic capacity must be ensured and, consequently, the necessary safety level for customers. To overcome this specific challenge, a mechanized approach is usually preferred, thus employing Tunnel Enlargement Machines (TEMs). Despite technology differing based on the specific applications, these generally consist of a front shield preventing the existing lining from breaking, one or more central units containing the equipment for demolition, pre-consolidation and relining and a rear unit transporting supply devices, such as hydraulic, power, and ventilation units.

For instance, TEMs were successfully used to upgrade, thus meeting modern requirements for rail traffic, the Gaintxurizqueta, Loyola and Capuchinos tunnels in San Sebastian, Spain⁶³ and the Kuckuckslay tunnel in Cologne, Germany.¹² Whether, in the former cases, demolition and excavation were carried out through hydraulic breakers, in the latter, explosives were used according to the *Tunnel-in-Tunnel* method. The TEMs employed are shown in Fig. 20.



Figure 20. TEMs used for the (a) Gaintxurizqueta, Loyola, and Capuchinos tunnels⁶³ and (b) Kuckuckslay tunnel¹²

In the field of concrete-lined tunnels, the examples are fewer but equally virtuous and, probably, more peculiar. The Shikishima tunnel (Hokkaido, Japan) underwent enlargement operations to double the cross-sectional area, thus accommodating a larger volume of vehicles.⁶⁴ The upgrading plan envisaged the portals to be rebuilt through the cut-and-cover technique and the central portion to be repoured on site with a longitudinally movable and transversally expandable shield. Such features allowed traffic regulation so that this would have been folded while working and unfolded during work shifts, thereby permitting halved and full traffic capacity, respectively. During competent rock excavation

through explosives, traffic was temporarily prohibited. The works lasted 5 months, reaching an average advancement rate of 1 m per day.

To deal with softer grounds, Maeda and Takatsuji⁶⁵ and Honda et al.⁶⁶ patented the so-called *Enlargement Shield Tunnelling* method. This envisages enlarging tunnel cross-sections by means of two main machines: the circumferential shield machine and the enlargement shield machine (see Fig. 21a). After stabilizing the surrounding ground through chemical grouting, the starting base for the circumferential shield machine is excavated. To this end, a square dugout is realized directly beneath the tunnel. Starting from here, the machine is driven through guiding rods carved in the existing lining to widen the excavation behind it and install concave segments. Starting from there, the first existing ring is demolished, and, in the gained over space, the enlargement shield machine is assembled. This has an annular structure, which allows over-excavating behind the existing tunnel structure and installing a new lining. Water inflows, if any, are handled by pneumatically applying pressure on the excavation boundary. Such upgrading work was applied in the Minamisenju utility tunnel (Tokyo, Japan) and other tunnels around Japan.⁶⁶

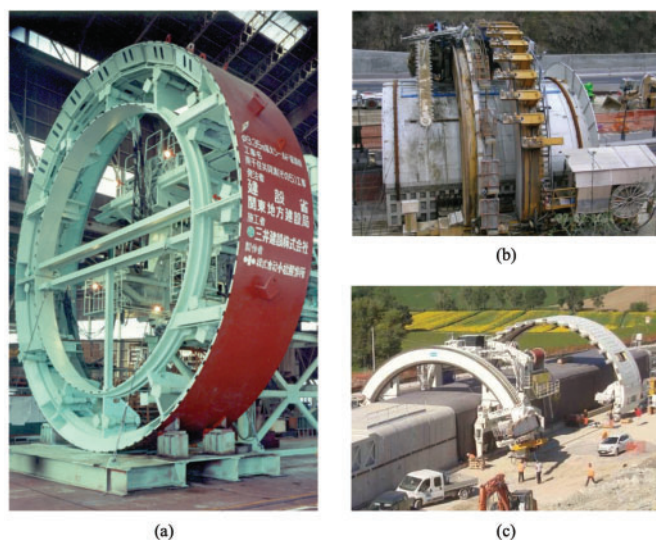


Figure 21. (a) Enlargement shield machine used for the Minamisenju tunnel,⁶⁶ (b) multipurpose excavation machine used for the Nazzano tunnel,⁶⁷ and (c) mechanical pre-cutting and segment erection machines used for the Montedonimi tunnel⁶⁸

The main drawback of the above technology is the impossibility of traffic transit during enlargements. In this sense, the Italian experience led by Lunardi et al.^{67,67}) has been pioneering worldwide with the enlargement of the Nazzano (Rome, Italy) and the Montedonimi (Ancona, Italy) tunnels. Based on the application (the first numbers refer to Nazzano and the second ones to Montedonimi), after surrounding ground improvement through precut technology and, where needed, jet-grouting for a length of 4.5–6 m, a cyclic alternation of excavation rounds of 1–2 m and precast segmental

arch erections was carried out until an advancement of 3–4 m was reached. The experience gained during the Nazzano tunnel enlargement was key to improving the machinery. Indeed, while, in the former case, a multipurpose machine (see Fig. 21b) able to either precut or erect segments was used, in the latter, two different machines (see Fig. 21c) were built. This aspect was revealed to be paramount in terms of downtime alternation for maintenance purposes.

Comparative Technical Assessment and Selection Criteria for Refurbishment Interventions

Whereas, in most maintenance works, a direct correspondence can be established between an identified defect and the corresponding repair works, the same relationship does usually not apply to structural reinforcement or rehabilitation and upgrading interventions. In such cases, the most appropriate intervention results from the combination of multiple technical, operational, and contextual factors, none of which is individually decisive.

Tunnel dimension and geometry influence the feasibility of certain types of interventions, as restricted cross-sections limit the size and weight of elements that can be handled and installed. Clearance gauge requirements may further impose stricter limits on the inward encroachment of additional lining thickness, in some cases ruling out otherwise structurally advantageous solutions. Lining typologies also directly affect the material compatibility, demolition techniques, and structural interaction between existing and new bearing systems.

The current stress state of the existing lining becomes critical whenever demolition induces stress redistribution across the structural elements, as inadequate phasing may trigger local instabilities. This aspect may be less demanding in case of integral demolition, provided that surrounding ground stability is appropriately ensured through pre-consolidation. Consequently, the hydrogeological context also plays a fundamental role: ground characteristics influence the stress regime of the lining, while water inflows or persistent infiltrations highlight the need for a waterproofing system setup.

Finally, traffic disruption constraints are often the most binding operational parameters. The requirement to maintain traffic, even at reduced capacity, or to complete refurbishment within limited time windows tends to favor faster and ease-to-deploy solutions, whereas full tunnel closure can allow a broader range of options to be considered. The spatial extent of interventions may further condition the overall technical and economic viability.

Table 2 provides a comparative overview of the principal refurbishment interventions discussed in the previous sections, outlining their typical fields of applicability based on specific technical and operational criteria.

Table 2 condenses the information and analysis developed throughout Subsections 3.1 to 3.4 into a comparative decision framework. Rather than ranking interventions hierarchically, the table highlights their compatibility with

Table 2. Field of applicability for refurbishment interventions (modified and implemented from Panebianco⁶)

Refurbishment intervention	Tunnel geometry and dimension		Clearance gauge requirements		Lining typology and materials			Stress regime of the lining		Geological context		Hydrological context		Traffic disruption		Frequency or extent	
	Na	Wi	Y	N	C	M	RC	H	L	R	S	D	W	T	P	Di	Lo
Maintenance works																	
Securing the intrados	+	+	+	+	+	○	+	+	+	+	+	+	+	+	+	+	+
Crack stitching	+	+	+	+	+	–	–	+	+	+	+	+	–	+	+	○	+
Cavity filling	+	+	+	+	+	–	+	○	+	+	○	+	○	+	○	○	+
Corrosion protection or substitution of steel rebars	+	+	+	+	–	–	+	–	+	+	+	+	○	○	+	–	+
Groundwater drainage	+	+	○	+	+	+	+	+	+	+	+		+	+	+	+	+
Waterproofing the intrados	+	+	+	+	+	+	+	+	+	+	+		+	+	+	+	+
Masonry interventions	+	+	+	+		+		○	+	+	+	+	–	+	+	○	+
Structural reinforcement																	
Lining bolting	○	+	+	+	+		○	○	+	+	+	+	○	+	○	○	+
Steel rib reinforcement	○	+	–	+	+	+	+	+	+	+	+	+	○	+	–	+	○
Invert arch construction or reinforcement	+	+	○	+	+	+	+	○	+	+	+	+	+	+	○	+	○
Fibre-reinforced material strengthening	+	+	+	+	+	○	+	+	+	+	+	+	○	+	+	+	+
Rehabilitation interventions																	
Integral replacement with cast-in-situ concrete	+	+	+	+	+	+	+	○	+	+	○	+	+	+	–	+	–
Partial replacement with precast concrete elements	○	+	+	+	+	○	+	+	+	+	+	+	+	+	–	+	○
Partial replacement with cast-in-situ/sprayed concrete	+	+	+	+	+	○	+	+	+	+	+	+	+	+	–	+	○
Partial replacement with metal segments/plates	+	+	○	+	+	○	+	+	+	+	+	+	○	+	–	+	○
Upgrading interventions																	
Tunnel enlargement via conventional re-boring	+	+	+	+	+	+	○	○	+	+	○	+	+	+	–	+	○
Tunnel enlargement through TEMs	+	○	+	+	+	+	○	+	+	○	+	+	○	+	○	+	–

Note: + = Applicable; ○ = Applicable with limitations; – = Not applicable; Na = Narrow; Wi = Wide; Y = Yes; N = No; C = Concrete; M = Masonry; RC = Reinforced Concrete; H = High; L = Low; R = Rock; S = Soil; D = Dry; W = Wet; T = Total; P = Partial; Di = Distributed; Lo = Localised.

a set of governing technical and operational parameters, including structural requirements, environmental conditions, geometric constraints, and service continuity demands.

Maintenance works generally prove suitable for localized defects regardless of tunnel dimension and geometry, yet their applicability diminishes when deterioration becomes widespread, as maintenance interventions are inherently limited in scope, structural impact, and frequency of application. Conversely, rehabilitation and upgrading interventions become more and more technically and economically justified when deterioration is widespread, extends over notable tunnel length, and performance deficiencies affect the structure globally. This confirms the progressive logic introduced at the beginning of Section 3: as the performance level declines and deterioration becomes systemic, increasingly invasive and structurally transformative measures are required.

The hydrogeological context also exerts a decisive influence. In wet conditions, drainage and waterproofing systems

gain particular relevance, while certain structural reinforcement techniques may prove challenging to execute or require further preventive measures to ensure long-term durability. For instance, reinforcing tunnel lining through bolting involves drilling operations that can prove challenging under significant groundwater pressure, while even the mere presence of water inflows or surface humidity may compromise the adhesion of composite materials to the tunnel intrados. In competent rock masses, where lining deterioration is typically non-evolutionary or advances slowly and remains largely independent of the stress state, a broader range of strategies remains technically viable. In soils associated with a wider range of stress regime of the linings based on the overburden, greater caution is required during both design and execution: steel ribs in shallow tunnels, for instance, must be designed not only to reinforce the existing structure but also to sustain potential sinkhole formation, while full lining replacement should generally be preceded by extensive pre-consolidation.

Stress regimes of tunnels may be critical if partial demolition of the lining triggers stress redistribution phenomena potentially leading to local collapses, unless adequate pre-consolidation is ensured and a proper staged demolition sequence is designed. With regard to lining material, plain concrete linings are compatible with the broadest range of refurbishment options. Reinforced concrete linings introduce constraints during demolition, where hydro-demolition is mandatory in place of milling. Masonry linings, on the other hand, demand careful preliminary assessment of their overall condition, as, for instance, drilling operations or any intervention that perturbs the static equilibrium of the structure may pose significant risks.

The need to maintain traffic, even at reduced capacity (with partial closures or night shifts), strongly favors pre-cast, sprayed, or mechanized solutions over conventional approaches. On the contrary, where full closure is permissible, a broader range of technically optimal but operationally disruptive interventions becomes feasible. Tunnel geometry and available clearance also directly affect feasibility, limiting the use of thick inner linings in narrow bores.

While the selection of a refurbishment strategy is ruled by the contextual and operational parameters discussed above, its effectiveness ultimately depends on the type and severity of the defect being addressed, as the same technique may prove decisive in one scenario and entirely inadequate in another. Table 3 provides a defect-oriented complement to the applicability-based framework of Table 2, mapping the principal refurbishment interventions against the most typical deterioration typologies encountered in existing tunnel linings.

Maintenance is inherently suited to structurally shallow and localized defects, with each technique targeting a specific and well-defined deterioration mechanism. Securing the intrados provides immediate containment against light detachments and shallow lining deterioration, without any structural ambition. Crack stitching effectively arrests sporadic cracking (e.g., shrinkage-induced) and seals preferential water paths, though its effectiveness diminishes when cracking is persistent, as the underlying structural cause remains unaddressed. Cavity filling targets lining anomalies and ground voids, with effectiveness in the latter case depending on cavity geometry and extent. Drainage systems prove notably effective against active dripping, where pressure relief is paramount, whereas intrados waterproofing better suits damp conditions with diffuse infiltrations, avoiding the development of ice in cold climates. Corrosion protection or substitution of steel rebars applies exclusively to reinforced concrete linings (being irrelevant for both masonry and plain concrete tunnels, as shown in Table 2). Masonry interventions (confined to their respective lining typology as shown in Table 2) are principally effective against localized deterioration of and/or cracking along mortar beds, exfoliation, and chemical degradation in masonry tunnels.

Structural reinforcement interventions address defects of greater severity and structural implication. Lining bolting is effective against detachment-driven instabilities to provide

active or passive restraint. Additionally, steel rib reinforcement also addresses deterioration-induced defects across a range of severities and, when ground-footed, provides an effective response to significant local strength losses in the lining. Notably, both lining bolting and steel rib reinforcement are frequently implemented in combination with maintenance works (e.g., intrados wire meshes, drainage systems, or ground cavity filler) to enhance the overall effectiveness of the repair and to address multiple concurrent defects that commonly coexist in deteriorated tunnel linings. Invert arch construction or reinforcement is specifically suited to structural deficiencies at the base of the structure, while fiber-reinforced material strengthening proves effective for cracking and shallow deterioration requiring tensile capacity enhancement but may become ineffective as the severity of defects increases.

Rehabilitation and upgrading prove consistently effective across a wide range of defect categories, including also shallow and localized ones, yet the associated technical and economic effort renders them disproportionate and unjustifiable outside the most severe deterioration scenarios. The same applies to enlargement techniques through both conventional re boring and TEMs, even if their implementation is almost exclusively driven by capacity or clearance requirements, being unrelated to deterioration-driven defects.

Overall, Tables 2 and 3 together offer a tool for the preliminary comparative assessment of refurbishment interventions, the former from an applicability-based standpoint, the latter from a defect-oriented perspective, to be complemented by detailed structural, cost-benefit, and risk assessments in the subsequent design stages.

Innovative Techniques for Tunnel Refurbishment

In recent years, Building Information Modelling (BIM) and Artificial Intelligence (AI) have gained huge momentum in the field of tunnel refurbishment. On the one hand, BIM represents an innovative process for managing and storing different kinds of data on physical assets throughout their life cycle. On the other hand, AI is a powerful and versatile tool that takes advantage of the brute computational power of modern computers to carry out detection, classification, or even decision-making activities.

BIM technology solves problems related to fragmentation and decentralization of data, thus allowing for a collaborative and efficient workflow that addresses not only design and construction stages but also operation, maintenance, and possibly demolition.⁶⁹ Zhou et al.,⁷⁰ for instance, considered the Shigu tunnel case history (Baoji, China) whose design envisaged the generation of a 3D BIM model including geometrical, design, operational data, and other information. This was particularly beneficial during the operational stage to simulate emergency-saving cases and organize ventilation and personnel evacuation. Yin et al.⁷¹ developed a BIM-based framework for the operation and maintenance of utility tunnels. Data stored in a BIM

Table 3. Refurbishment interventions for most of the typical defects in existing tunnels

Refurbishment intervention	Deterioration of the lining		Water-related deteriorations		Voids or anomalies		Local detachments		Intense cracking		Steel rebar corrosion
	D	S	Dr	Da	Gr	Li	H	L	Pe	Sp	
Maintenance works											
Securing the intrados	–	+	–	–	–	–	–	+	–	–	–
Crack stitching	–	–	–	–	–	–	–	–	○	+	–
Cavity filling	–	–	–	○	+	+	–	–	–	–	–
Corrosion protection or substitution of steel rebars	–	–	–	–	–	–	–	–	–	–	+
Groundwater drainage	○	–	+	+	–	–	–	–	–	–	–
Waterproofing the intrados	–	–	○	+	–	–	–	–	–	–	–
Masonry interventions	○	+	–	–	–	–	–	+	○	+	–
Structural reinforcement											
Lining bolting	○	–	+	+	+	+	+	–	–	–	–
Steel rib reinforcement	+	+	+	+	+	+	+	+	○	–	–
Invert arch construction or reinforcement	+	+	–	–	–	–	–	–	–	–	–
Fiber-reinforced material strengthening	–	+	–	–	–	–	–	+	○	+	–
Rehabilitation interventions											
Integral replacement with cast-in-situ concrete	+	+	+	+	+	+	+	+	+	+	+
Partial replacement with precast concrete elements	○	+	+	+	○	+	+	+	+	+	+
Partial replacement with cast-in-situ/sprayed concrete	○	+	+	+	○	+	+	+	+	+	+
Partial replacement with metal segments/plates	–	+	+	+	○	+	+	+	+	+	+
Upgrading interventions											
Tunnel enlargement via conventional re boring	+	+	+	+	○	○	+	+	+	+	+
Tunnel enlargement through TEMs	+	+	+	+	○	○	+	+	+	+	+

Note: + = Effective; +* = Effective in combination with proper maintenance works; ○ = Conditionally effective; – = Ineffective D = Deep; S = Shallow; Dr = Dripping; Da = Damp; Gr = Ground; Li = Lining; H = Heavy; L = Light; Pe = Persistent; Sp = Sporadic.

model was shared in real-time with maintenance and management personnel, thus allowing for more cost-effective management decisions. In Italy, Ciccone et al.⁷² dealt with the digitalization of the existing Cancellone-Benevento railway line, including the informatization of six different tunnels. This was paramount for supporting the decision-making process and assisting management, operation, and maintenance of the assets. However, the BIM potential does not stop here: Alsahly et al.,⁷³ indeed, developed a BIM-to-FEM

approach that allows producing a Finite Element Method (FEM) model starting from a cross-sectional view of the BIM leading to the development of a tunnel digital twin. Despite being applied for predicting the surface subsidence during mechanized tunnelling at the design stage, this may also be revealed as an interesting and effective feature to examine the evolution of the behaviour of tunnels in time to schedule refurbishment activities.

Conversely to BIM, AI addresses a larger variety of issues depending on the versatility and complexity of the algorithm implemented. For instance, Ahmed et al.⁷⁴ tested diverse models to forecast the deterioration of existing tunnels starting from geometrical, construction, and operational information, reaching accuracies of 80%. Schneider et al.⁷⁵ and Panella et al.,⁷⁶ instead, tested two BIM-compatible systems for automatic defect detection. In both cases, an overall time saving of around 30–60% was experienced, with a noteworthy time and personnel reduction during surveys of 90% and 50%, respectively. Focusing only on cracks, Protopadakis et al.⁷⁷ and O'Brien et al.⁷⁸ used Deep Learning (DL) and Convolutional Neural Networks (CNNs) to automatically spot their presence on the tunnel lining. Furthermore, in the latter case, the detected cracks were also classified among four categories (vertical, horizontal, diagonal, and complex), achieving accuracies of 98.4% and 92.3% for the detection and classification tasks, respectively.

The growing trend towards BIM and AI usage in tunnel refurbishment is a symptom that the current modus operandi, made up of human-based visual inspections and manual defect detection, will soon be replaced by automated systems. Besides being a time- and resources-consuming operation, the current process is also prone to inherent limitations such as human subjectivity or lack of repeatability. To achieve an objective and consistent outcome, Protopadakis et al.⁷⁷ and Foria et al.⁷⁹ developed and tested the ROBO-SPECT (Fig. 22a) and ARCHITA (Fig. 22b) robotic platforms for automatic inspection of existing tunnels. Whereas the former appears as an intelligent device able to detect and localize cracks in real-time through AI, the latter is a multi-dimensional mobile mapping system consisting of linked and integrated equipment (linear and thermal cameras, laser scanner sensors, georadar, etc.) aimed at retrieving as much data as possible to post-process at a later time. This approach proved considerable advantages in terms of time and cost reduction and improved resource management. More recently, Chen et al.⁸⁰ proposed an autonomous mobile robotic system for in-process tunnel lining deformation detection during construction, integrating high-precision laser and compensation algorithms to achieve high accuracy even under complex site conditions.



Figure 22. (a) ROBO-SPECT⁷⁷ and (b) ARCHITA⁸¹ robotic platforms for tunnel inspection

In parallel, drone-based inspection strategies are gaining relevance. Among others, Bendris and Cayero Becerra⁸² highlighted the potential of aerial platforms equipped with vision- and thermal-based sensors for rapid, noncontact

tunnel assessments, particularly in hard-to-access areas, thus expanding the operational flexibility of automated inspection technologies.

Analogous to inspection methodologies, BIM and AI are also influencing the way of conceiving refurbishment, which is currently intended as either corrective or preventive. Whether the corrective approach consists of restoring the operability of a system once a defect or failure has occurred, the preventive one aims at preserving its service life by anticipating any impending problems. However, the current race for informatization and automation is paving the way to predictive refurbishment. In line with recent advancements in AI- and BIM-based real-time monitoring frameworks (Afrazi et al.⁸³ among others), this approach relies on continuous data acquisition, sensor integration, and data-driven analytics to support proactive decision-making. It enables a wiser management of material and personnel resources, as refurbishment activities are deferred in time until they are actually required, thereby optimizing intervention procedures. To this end, incipient faults should be detected, identified and tracked until an acceptable and a priori established level of degradation and then the refurbishment strategy activated.

Yuan et al.⁸⁴ differ between two types of predictive refurbishment, namely, condition-based maintenance (CBM) and prognostics and health management (PHM). CBM utilizes data-driven or physics-based reliability models to predict the residual service life and the extent of refurbishment to perform. PHM, instead, uses high-frequency monitoring systems for real-time performance diagnostics. Regardless of the type, the predictive approach aims at evaluating the residual useful life of a system or a part of it. For instance, Han et al.,⁸⁵ considering a physics-based CBM approach based on tunnel lining aging due to carbonation penetration and chloride erosion, evaluated a probabilistic-based maintenance strategy that accounted for different failure modes. Farahani et al.,⁸⁶ instead, proved through small-scale laboratory testing how existing tunnels may extremely benefit from the PHM approach.

Recent and more operation-oriented applications have been proposed by Bellini Machado and Massao Futai,⁸⁷ Vollmann et al.⁸⁸ and Khan.⁸⁹ From a degradation modelling perspective, Bellini Machado and Massao Futai⁸⁷ developed an inspection digital twin for a railway tunnel in Northern Brazil, coupled with serviceability indices to quantify structural deterioration and support predictive refurbishment planning. These indices are derived from visual inspection data transformed into service-level indicators, which are subsequently used to calibrate stochastic degradation models aimed at estimating target refurbishment times and optimizing management during service. Vollman et al.⁸⁸ developed a BIM-based methodology supporting predictive operation and refurbishment for German road tunnels. This approach couples static inspection logs and dynamic real-time monitoring data, pays particular attention to data interoperability, and applies AI-based defect recognition to support lifecycle-oriented decision-making. Khan,⁸⁹ instead, advanced a data-driven digital-twin framework integrating BIM, monitoring systems, and facility management

databases, using AI algorithms to predict component-level asset states and generate refurbishment actions.

Alongside digitalization and automation, an emerging research frontier in the sustainable refurbishment of existing tunnels is energy retrofitting,^{26,89} a concept that broadens the traditional role of underground transport infrastructure by enabling tunnels to function not only as mobility assets but also as systems capable of generating, storing and, where feasible, efficiently utilizing renewable geothermal energy. Within this perspective, tunnels are reconceptualized as thermo-active infrastructures, i.e., energy geostructures,^{91,92} able to harness ground-trapped thermal energy that would otherwise remain unexploited and, potentially, interacting with surface thermal energy grids.^{93,94}

Energy retrofitting entails instrumenting existing tunnels for heat exchange, either for direct exploitation or for cyclic storage. Energy retrofitting is conceived to be implemented along with scheduled rehabilitation and upgrading interventions or functional repurposing activities, thereby minimizing additional construction impacts and costs. In this context, De Feudis⁹⁰ introduced a comprehensive and systematic framework for transforming existing tunnels into energy geostructures through the installation of geothermal piping. The framework encompasses 6 different energy retrofitting solutions, specifically conceived to fit several decay contexts and diverse levels of refurbishment necessity, spanning from minimally invasive options for tunnels requiring limited intervention to highly invasive configurations fully integrated within major structural repair works. In addition, De Feudis⁹⁰ assessed the economic attractiveness of such an approach through an LCOE analysis, demonstrating high profitability with representative LCOE values of around 100 €/MWh_t. In this regard, the Olimpia tunnel in Italy,¹⁵ illustrated in Fig. 23, represents the first documented worldwide example of an existing tunnel instrumented for geothermal heat exploitation while simultaneously undergoing structural rehabilitation, marking a significant milestone in the advancement of sustainable underground infrastructure refurbishment.

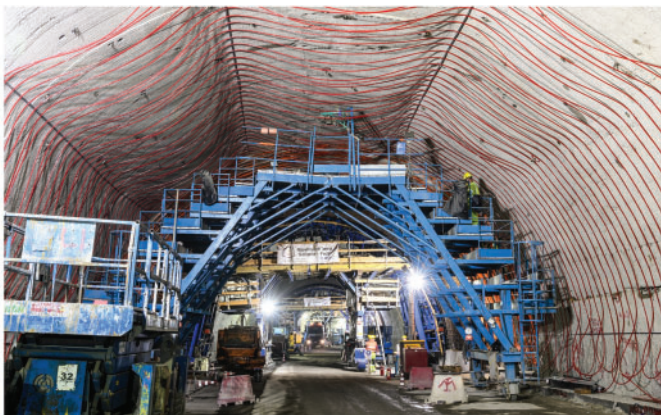


Figure 23. Energy retrofitting of the Olimpia tunnel

Conclusions

This paper has offered a state-of-the-art review of current practices, regulatory frameworks, and technological developments in the refurbishment of existing tunnels, with particular focus on established maintenance, structural reinforcement, rehabilitation and upgrading techniques, as well as emerging digital tools supporting lifecycle management of the assets.

Such a review is based on the authors' experience in the field, published literature, technical guidelines, and documented case studies, which, while offering a broad international perspective, may not fully capture unpublished operational data and performance records or region-specific practices. Owing to the authors' background, a non-negligible proportion of the detailed case studies refers to Italian applications. This may introduce a degree of geographical imbalance; however, it also represents a strength, as several of these examples derive from direct technical involvement and first-hand knowledge rather than exclusively from secondary sources. In addition, emerging technologies such as BIM-integrated asset management, AI-supported inspection, and energy retrofitting are, in several cases, at varying stages of implementation, with long-term validation data still limited. Within these boundaries, and based on the synthesis of international experiences and methodological advancements, the main conclusions can be drawn as follows.

- Tunnel refurbishment worldwide is converging toward structured, risk-based methodologies, where inspection frequency, defect classification, and refurbishment prioritization follow codified practices aimed at ensuring consistent performance, safety, and regulatory compliance over service lives.
- Maintenance works, although essential for stabilizing degradation and preserving short-term safety, generally constitute temporary measures that require long-term planning of structural reinforcement or rehabilitation to achieve sustainable performance recovery.
- Structural reinforcement offers substantial improvements in load-bearing capacity while maintaining service continuation, thus serving as an effective intermediate strategy between local maintenance actions and full-scale rehabilitation.
- Rehabilitation interventions enable comprehensive and durable renewal of aging tunnel structures and their adoption is increasingly guided by cost-benefit analyses, operational constraints, and the opportunity to integrate modern waterproofing, safety, and functional upgrades.
- Upgrading interventions reflect noteworthy technological progress, especially through mechanized enlargement and shield-based methods that permit complex works under traffic, thereby limiting disruptions in increasingly capacity-constrained transport networks.

- BIM and AI are increasingly becoming key for integrating asset information, improving inspection objectivity, and supporting decision-making; when combined with robotic inspection platforms, they offer large gains in survey efficiency, defect detection accuracy, and worker safety. Future research on BIM should concentrate on the development of integrated, all-in-one BIM infrastructures capable of managing tunnel assets throughout the entire lifecycle, from design and construction to management, including the implementation of BIM-to-FEM interoperability to design refurbishment interventions. This BIM-to-FEM integration represents a promising research direction that could be extended beyond tunnels to bridges, viaducts, and other infrastructures, linking digital asset information with structural analysis, condition assessment, and refurbishment decision-making. With regard to AI, the next logical step would be the development and validation of fully autonomous robotic inspection systems capable of navigating tunnel environments, performing real-time defect detection, and directly updating BIM models with minimal human intervention.
- The transition toward predictive refurbishment, empowered by monitoring systems, data analytics, and model-based performance forecasting, provides a promising opportunity for optimizing resource allocation and extending tunnel service life. Further investigations should aim at integrating long-term monitoring data with probabilistic deterioration models, defining threshold-based intervention criteria linked to measurable performance indicators, and validating them in diverse geological and structural contexts.
- Energy retrofitting is emerging as a viable complement to conventional refurbishment interventions, enabling existing tunnels to be converted into energy geostructures while undergoing rehabilitation, upgrading, or repurposing, thereby enhancing the sustainability and life-cycle performance of tunnel assets. As the Olimpia tunnel currently represents the only documented worldwide application, future research should prioritize replicating and adapting this framework across diverse geographical, geological, climatic, and regulatory contexts, alongside developing novel solutions to support a range of refurbishment scenarios, as well as on the integration of such multi-purpose tunnel assets within dense urban environments, where it can function as both thermal energy exploitation and storage facilities. Particular attention shall be devoted to their effective connection with district heating and cooling networks in order to quantify and maximize their potential contribution to reducing urban carbon emissions.

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A Synthesis of Research on the Rollover Stability of Prestressed Concrete Girders Supported on Elastomeric Bearings

Pegah Reza Ahrabi¹; Matthew Whelan^{2*}; Nicole Braxtan³ and Shenen Chen⁴

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Abstract: Rollover stability is a critical concern for prestressed concrete bridge girders, particularly during handling, transportation, and erection when lateral restraint is limited. The growing use of long-span, slender sections has increased their susceptibility to rollover instability, especially under the combined influence of geometric imperfections, thermal gradients, eccentric loading, and inclination and deformation of bearing surfaces. Although these effects are well recognized in practice, current codes and design guidelines reviewed in this study (including both U.S. and European provisions) offer limited guidance, and many analytical and numerical models use simplifying assumptions that incompletely capture important nonlinear behaviors. Furthermore, experimental investigations remain scarce due to the complexity and cost of full-scale testing. This article presents a comprehensive review of rollover stability of prestressed concrete bridge girders supported on elastomeric bearings. The theoretical foundations and governing mechanisms are first summarized, including a discussion of the effects of geometric imperfections, temperature variations, wind, support conditions, and bearing rotational stiffness. Existing analytical, numerical, and experimental studies are critically examined, and key knowledge gaps are identified to guide future research and code development.

Author keywords: Prestressed concrete girders; rollover stability; elastomeric bearings; lateral stability; bridge construction

Introduction

Lateral instability has resulted in the catastrophic collapse of numerous bridges during construction. Most lateral instabilities occur during transportation and construction, when girders are more vulnerable to overturning due to limited intermediate lateral support. In 2000, eight girders of the Souvenir Boulevard Bridge in Laval, Canada,¹ slid off their bearings and tilted, with four of the girders undergoing collapse. The investigation indicated that the sliding pot bearings did not provide sufficient resistance to rotation about the longitudinal axis of the girder, which allowed them

to twist and lose stability. In 2005, during the construction of a precast prestressed concrete bridge in Pennsylvania,² several girders collapsed due to inadequate lateral restraint combined with additional sweep induced by non-uniform thermal effects. The absence of sufficient temporary bracing increased the vulnerability of the girders to rollover during erection. A similar event occurred during the erection of the Red Mountain Freeway in Arizona,³ where nine girders collapsed. The investigation concluded that the collapse was initiated by the lateral instability of a single girder, which triggered progressive failure of adjacent girders. Contributing factors included bearing eccentricity, initial geometric sweep, creep-induced sweep, thermal sweep, and bearing inclination in both the transverse and longitudinal directions. These failures have drawn increased attention to the lateral stability of prestressed concrete girders during construction stages.

Any slender girder supported with insufficient rotational restraint may experience rigid-body rotation and overturning when destabilizing moments exceed the rotational resistance provided by the supports. Therefore, rollover instability may occur in concrete or steel girders under insufficient support and loading conditions. However, rollover failure is more likely to govern the lateral stability of prestressed concrete girders during transportation and erection. This is because, in contrast to steel girders, prestressed concrete girders have relatively high torsional and minor axis flexural stiffness, producing lateral-torsional buckling

*Corresponding Author: Matthew Whelan.

Email: M.Whehan@charlotte.edu

¹Graduate Research Assistant, Department of Civil and Environmental Engineering, University of North Carolina at Charlotte, 9201 University City Blvd., Charlotte, NC, 28223-0001

²Associate Professor, Department of Civil and Environmental Engineering, University of North Carolina at Charlotte, 9201 University City Blvd., Charlotte, NC, 28223-0001

³Associate Professor, Department of Civil and Environmental Engineering, University of North Carolina at Charlotte, 9201 University City Blvd., Charlotte, NC, 28223-0001

⁴Professor, Department of Civil and Environmental Engineering, University of North Carolina at Charlotte, 9201 University City Blvd., Charlotte, NC, 28223-0001

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resistance greater than the resistance to rollover stability for the uncracked sections.⁴ Rollover instability is often mistaken for lateral torsional buckling; however they are distinct failure modes. Lateral torsional buckling occurs under flexure when the girder bends laterally and twists at the same time, typically due to superimposed loads. In contrast, rollover instability happens when the support fails to provide sufficient restraint against rotation about a roll axis parallel to the girder's longitudinal axis. The overturning is due to torsional moments that arise from lateral loads or, in the presence of geometric imperfections, material nonlinearity, or support rotation, torsional moments that arise from eccentric vertical loads, including self-weight. Engineers often rely on simplified assumptions and equations that overlook the combined effects of sweep, solar radiation, the nonlinear rotational resistance produced by the bearings, and other destabilizing factors common in real-world conditions. Experimental studies, especially full-scale investigations that reflect realistic transport or erection conditions, remain limited. With advancements in computational tools, researchers have developed more detailed analytical and numerical models to study rollover mechanisms and predict girder behavior more accurately. However, questions remain about how well these models represent field conditions and practical design cases.

This article presents a state-of-the-art review of the rollover stability of prestressed concrete bridge girders supported on elastomeric bearings. The review included publications from 1989 through 2026, beginning with the foundational study by Mast.⁴ No explicit exclusion criteria were applied, allowing relevant studies on the rollover stability of prestressed concrete girders and elastomeric bearing behavior to be considered. The reviewed literature includes peer-reviewed journal articles, conference papers, technical reports, theses, and design codes and guidelines. The goal is to clarify the key mechanisms that contribute to rollover instability, identify the significance of influential factors, review current design specifications and guidance, and synthesize key findings and developments from existing experimental, analytical, and numerical studies. Assumptions and limitations of existing analytical approaches are also identified. The paper concludes with a comparative and synthesis table and summary of knowledge gaps and research

needs to guide future work aimed at improving design for safety and construction efficiency.

Theory and Fundamental Concepts

Geometric imperfections in a girder often cause a lateral shift of its center of mass, creating an eccentricity of gravity loads relative to the girder's roll axis (Fig. 1a). This eccentricity generates an overturning moment that initiates rotation of the girder about the roll axis. Fig. 1b illustrates the equilibrium of a girder with such imperfections supported on an elastomeric bearing. As the girder begins to rotate on the supports, a portion of its self-weight acts about the weak axis, producing additional minor axis bending. This deformation increases the roll angle, further amplifying the overturning moment and consequently the deflection, a form of geometric nonlinear behavior. The presence of wind loading also contributes overturning forces and corresponding moments, leading to greater eccentricity and lateral deflections. By equilibrium, the overturning moment is given by:

$$M_a = W [(z + y_r) \theta + e_i + e_{brg} + z_{wind}] + M_{OT} \quad (1)$$

where W is the girder self-weight; z , z_{wind} , e_i , and e_{brg} are the lateral deflection due to girder self-weight, lateral deflection due to wind loading, and initial sweep and bearing eccentricity of the girder, respectively, all taken at the center of gravity of the deflected girder; y_r is the vertical distance from the roll axis to the center of gravity; θ is the roll angle of the girder measured from the vertical axis; and M_{OT} is the overturning moment due to the applied lateral load, taken about the roll center. The roll center is typically assumed to be located at the mid-depth of the elastomeric bearing. The elastomeric bearings resist overturning moments through their rotational stiffness, k_θ . The resisting moment provided by the bearings is

$$M_r = k_\theta (\theta - \alpha) \quad (2)$$

where α is the superelevation or tilt angle of the support. When the overturning moment exceeds the resisting moment provided by the bearings, equilibrium is no longer satisfied, leading to rollover instability. Notably, the rotational stiffness of an elastomeric bearing is nonlinear and, furthermore,

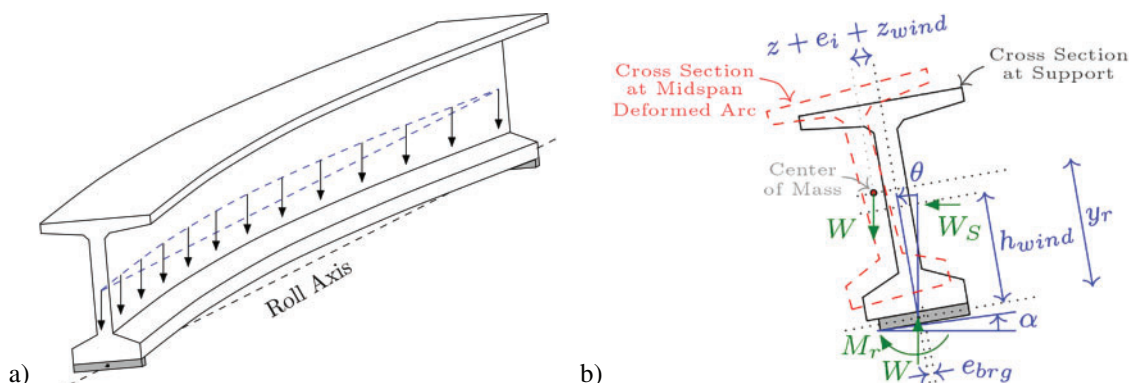


Figure 1. Equilibrium of girder on elastic support: a) Torsion produced by eccentric vertical loads; b) Free body diagram

as the girder rotates, the contact area between the girder and the bearing surface decreases as a result of partial uplift of the girder. This reduces the bearing rotational stiffness and, consequently, the resisting moment provided by the bearing against further girder rotation, leading to partial uplift of the girder. Therefore, changes in bearing rotational stiffness directly affect the resisting moment and, consequently, the capacity of the girder against rollover instability.

To evaluate girder stability, Mast⁴ introduced two analytical methods for calculating factors of safety, one against cracking and one against rollover, for girders supported on elastic bearings. These definitions later formed the basis of the PCI Bridge Design Manual provisions⁵ for assessing girder stability during handling and transportation. The factor of safety is defined as the ratio of resisting moment over the applied overturning moment:

$$FS = \frac{M_r}{M_a} = \frac{K_\theta (\theta - \alpha)}{W[(z + y)\theta + z_{wind} + e_i] + M_{OT}} \quad (3)$$

where K_θ is the rotational stiffness of the support, which is assumed to behave linearly elastically, and α is the support inclination. At a certain rotation, the tensile stress at the top flange exceeds the concrete modulus of rupture, and cracking initiates. The corresponding tilt angle associated with the onset of cracking, θ_{cr} , can be calculated from the ratio M_{cr}/M_g , where M_{cr} is the lateral moment required to initiate cracking, considering the existing stress developed by the combined effects of prestressing, self-weight, and wind loads, and M_g is the moment generated by the girder self-weight.⁶ By substituting into Eq. 4, the factor of safety against cracking, FS_{cr} , can be determined:

$$FS_{cr} = \frac{K_\theta (\theta_{cr} - \alpha)}{W((z + y_r)\theta_{cr} + e_i + z_{wind}) + W_S h_{wind}} \quad (4)$$

where W_S is the wind load on the girder and h_{wind} is the vertical distance from the roll axis to the mid-depth of the girder.

Cracking reduces the lateral stiffness of the girder, leading to larger lateral deflections under the same loading conditions. To account for this reduction in stiffness, Mast⁷ proposed an empirical factor, $(1 + 2.5\theta)$, to estimate the cracked moment of inertia from the uncracked moment of inertia. This factor is applied to the lateral displacement terms z and z_{wind} to estimate the deformation of the cracked section. By incorporating this amplification into the stability formulation, the factor of safety against rollover can be expressed as

$$FS_r = \frac{K_\theta (\theta_r - \alpha)}{W[(z_0\theta_r + z_{wind})(1 + 2.5\theta_r) + y_r\theta_r + e_i] + W_S h_{wind}} \quad (5)$$

where θ_r is the critical rollover rotation angle corresponding to the onset of uplift. Mast defined θ_r by establishing equilibrium between the resisting and overturning moments, with the bearing reaction acting at the kern point. The Recommended Practice for Lateral Stability of Precast, Prestressed Concrete Bridge Girders⁶ extended this formulation by incorporating the effects of lateral loads into the moment

equilibrium equation. According to the PCI Recommended Practice, lift-off is assumed to occur when the resultant bearing reaction reaches the kern point of the bearing. The corresponding rotation at which lift-off initiates is given by

$$\theta_r = \frac{W(z_{max} - e_{brg} - h_r\alpha) - (WS)[h_r + (z_{max} - e_{brg})\alpha]}{K_\theta} + \alpha \quad (6)$$

where e_{brg} is the bearing eccentricity; z_{max} is the maximum kern distance of the bearing, calculated as

$$z_{max} = \frac{W_{brg}}{6} \quad (7)$$

where W_{brg} is the bearing width in the direction of rotation. The Recommended Practice for Lateral Stability of Precast, Prestressed Concrete Bridge Girders,⁶ Section 3.4.5, and the PCI Bridge Design Manual,⁸ Section 8.10, specify minimum factors of safety of 1.0 against cracking, and 1.5 against rollover.

These formulations assume that the girder remains fully in contact with the bearing. However, as discussed earlier, increasing eccentricity and rotation progressively reduce the stability of the system, causing partial lift-off of the girder from the bearing surface. Consequently, full contact between the girder and the bearing in most cases will not be maintained until the instability limit. The assumption that the rotational stiffness of the bearing remains linearly elastic with no limit, which is adopted by the rollover factor of safety computed by Eq. 5, may overestimate the rollover capacity of seated girders because, at large rotations, the bearing can no longer deform uniformly, resulting in partial separation at the girder-bearing interface.⁹

Eamon et al.⁹ proposed a tipping factor of safety in which the girder is assumed to overturn about an effective toe point rather than the bearing roll center. In this formulation, the resisting moment is calculated from the girder self-weight acting through an eccentricity measured relative to the effective toe point. The overturning moment includes contributions from wind loading, fascia jack forces, and applied lateral loads. However, a key limitation of this approach is that the location of the effective toe point depends on the girder properties, loading conditions, and bearing behavior. Consequently, the location of the rotation axis is not fixed and may vary throughout the loading process. The corresponding factor of safety is defined as

$$FS_{tip} = \frac{M_r}{M_{OT}} = \frac{W \cdot e}{M_w + M_b + M_p} \quad (8)$$

where M_w is the overturning moment induced by lateral wind loads and calculated about the effective toe point; M_b is the overturning moment generated by fascia jack forces; and M_p is the overturning moment resulting from an applied lateral load at midspan.

Rollover stability in prestressed concrete girders is influenced by a combination of factors. While a single factor may not cause failure on its own, the cumulative effect of multiple factors can significantly reduce the girder's stability. Geometric imperfections are one of the main contributors

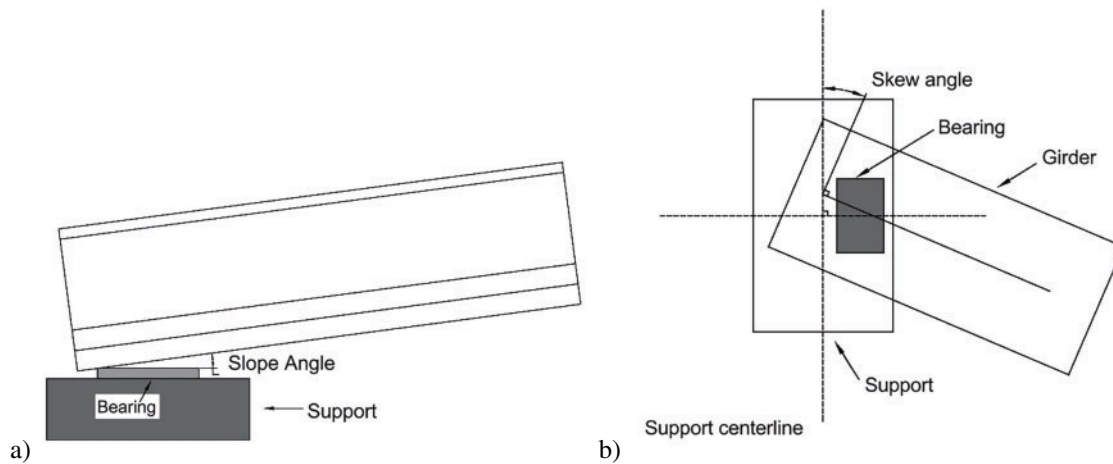


Figure 2. Definition of: a) slope angle; b) skew angle

to this failure mode. Sweep, an unintended horizontal curvature of the girder, is the most critical factor and often results from inaccuracies in fabrication, misalignment of strands, support misalignment, or temperature gradients from solar radiation.^{10,11} Other imperfections, such as camber, slope angle, and skew angle, also increase rollover risk. The slope angle is the vertical angle between the girder centerline at the support and a horizontal line aligned with the bearing surface (Fig. 2a). This slope may arise from camber due to eccentric prestressing forces, construction imperfections, or the overall grade of the bridge.¹¹ The skew angle refers to the alignment of the girder longitudinal axis relative to the principal axes of the bearing (Fig. 2b), resulting in nonuniform pressure distribution. Both the slope and skew angles can reduce the effective rotational stiffness of elastomeric bearings, which lowers their ability to resist overturning moments.¹² In addition, the effective prestressing force, along with time-dependent effects such as creep and shrinkage, induces additional deformations in the girder that evolve over time⁶ and may amplify the geometric imperfections that are responsible for rollover instability. Temperature differences across the girder cross-section, caused by solar exposure or heat from curing, can lead to uneven thermal expansion and induce bowing or twisting of the girder. When combined with existing geometric imperfections or flexible supports, these thermal effects can further reduce rollover stability, particularly during construction operations and stages where the girder has no temporary or permanent bracing.¹³

Code Provisions

To improve rollover stability, design codes provide several limitations and recommendations.

The PCI Bridge Design Manual,⁸ Section 3.6.2.1, requires bearing surfaces to be checked for horizontal and vertical alignment, flatness, levelness, and slope, with a flatness tolerance of 1.6 mm (1/16 in). Section 10.4.1.2 specifies a limit of 0.29° (0.005 rad) for unintended bearing surface slope introduced by construction tolerances. Section 3.7.4

further recommends the installation of temporary steel or timber diaphragms immediately after erection and prior to final connection. These diaphragms are intended to prevent girder instability caused by wind, seismic effects, or thermally induced sweep.

Table 1 summarizes the permissible sweep tolerances specified in various codes and guidelines. The permitted sweep for bridge girders varies significantly across these codes, with the most stringent requirement being imposed by PCI.⁸ The PCI Tolerance Manual for Precast and Prestressed Concrete Construction¹⁴ and the Manual for Quality Control for Plants and Production of Structural Concrete¹⁵ limit allowable sweep and lateral displacement to 3.2 mm per 3 m (1/8 in per 10 ft) of girder length, variation in strand placement and prestressing force eccentricity to 6.4 mm (1/4 in), and camber variation from the design camber to 3.2 mm per 3 m (1/8 in per 10 ft) of girder length, with a maximum of 12.7 mm (1/2 in) for girders up to 24.4 m (80 ft) long and 25.4 mm (1 in) for girders longer than 24.4 m (80 ft). These limits remain applicable and are referenced in the newly released updated guidelines for lateral instability of prestressed concrete bridge girders.⁶ In addition, Section 12.1 of the PCI Tolerance Manual for Precast and Prestressed Concrete Construction¹⁴ limits bearing eccentricity, defined as the lateral offset between the girder bottom centerline and the elastomeric bearing pad centerline, to 25.4 mm (1 in).

The AASHTO bridge design and construction specifications provide additional guidance related to sweep and support conditions. The AASHTO LRFD Bridge Construction Specifications¹⁶ require contractors to ensure the safety of precast members during all stages of construction and to provide adequate temporary bracing. Likewise, the AASHTO LRFD Bridge Design Specifications¹⁷ state in Section 5.12.3.2.1 that the contractor is responsible for the safe shipping, erection, and temporary bracing of precast girders. In addition, Section 14.8.2 limits the maximum slope between the girder and the bearing surface to 0.57° (0.01 radians). If this limit is exceeded, the use of a tapered plate is required to provide a level bearing surface.

Table 1. Sweep permitted by design codes and guidelines

Code or guideline	Sweep tolerance (e_i)
PCI ⁸	3.2 mm per 3 m (1/8 in per 10 ft) $= L/960$
EN 15050:2008 ¹⁸	$L/500$
Spanish Code EHE-08	$L/750$

Bearing Behavior and Roll Stiffness

Mechanical behavior of elastomeric bearings

Bearings are used in bridge construction to support girders and transfer vertical loads from the girders to the substructure while permitting horizontal displacements caused by thermal expansion, construction, elastic shortening, creep, and shrinkage.¹⁹ Several types of bearings are used in bridge structures, including elastomeric bearings, pot bearings, rocker bearings, cylindrical bearings, spherical bearings, and roller bearings. Each type of bearing provides different levels of translational and rotational stiffness while allowing for required seasonal thermal expansion and contraction of the superstructure.¹¹ Elastomeric bearings are produced as plain elastomeric pads, fiberglass-reinforced elastomeric pads, steel-reinforced elastomeric pads, and cotton duck-reinforced elastomeric pads. Among these, steel-reinforced elastomeric pads are widely used to support bridge girders in the United States because they are easy to use, durable, and economical.^{20,21}

Steel-reinforced elastomeric bearing pads consist of several layers of neoprene rubber bonded to embedded steel shims. Neoprene provides high resistance to heat, flames, and weathering and has reliable adhesion to metal.²² Elastomers, including neoprene, are nearly incompressible, with a Poisson's ratio greater than 0.49. Consequently, when subjected to compression, they expand laterally to maintain their volume. In steel-reinforced elastomeric bearings, the steel shims are sufficiently stiff to restrain the lateral movement of the elastomer at the steel–elastomer interfaces.^{23–26} The steel shim plates reduce the lateral bulging of the elastomer layers by restraining their expansion (Fig. 3). As a result, when the pad is compressed, the individual elastomer layers bulge toward the edges of the pad. This restraint

provided by the steel shims significantly increases the compressive stiffness of the bearing compared to an unreinforced plain elastomeric pad having the same elastomer thickness and volume. At high levels of compression, the stiffness becomes nonlinear because bulging moves a portion of the elastomer outside the main load path. Consequently, the effective thickness of the load-carrying elastomer layers decreases, which increases the compressive stiffness of the pad.²⁶ For a constant total elastomer thickness, increasing the number of steel shim plates increases the axial stiffness because the thickness of the individual elastomer layers decreases. However, the increase in shear stiffness resulting from thinner individual elastomer layers reduces the ability of the bearing to accommodate thermal expansion and contraction of the girder with minimal restraint.²⁷

Steel-reinforced elastomeric bearings are fabricated in fully bonded, fully unbonded, and single-side bonded configurations. In fully bonded bearings, steel end plates are vulcanized to the top and bottom surfaces of the elastomeric bearing and are subsequently welded or bolted to the superstructure and substructure. In fully unbonded bearings, the end plates are omitted, and the bearing is placed directly between the supporting elements, where forces are transferred through direct bearing and friction at the contact surfaces. In a single-side bonded configuration, only the top surface of the bearing is connected to the superstructure, while the bottom surface remains unbonded and rests directly on the substructure.^{28,29}

These bearings exhibit different responses under axial loads and bending moments. The girder self-weight and applied axial loads generate compressive strains within the bearing. When rotation is introduced, tensile strains develop in the bearing layers and counteract the compressive strains. As a result, one side of the bearing tends to move upward, leading to uplift. According to NCHRP 596, the bearing and girder surfaces remain in full contact during uplift despite the upward movement of one edge.³⁰ In unbonded bearings, however, sufficiently large rotation may cause partial separation between the bearing and the supporting surface, resulting in lift-off. The partial loss of contact between the bearing and the supporting surface during lift-off reduces the effective contact area,^{28,30–34} which alters the distribution of stresses within the bearing and can significantly affect its rotational stiffness.

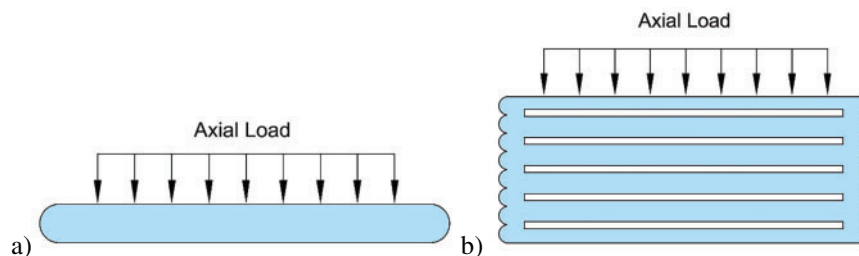


Figure 3. Behavior of elastomeric bearings in compression: a) unreinforced pad; b) effect of steel reinforcement in restraining bulging of pad

As discussed previously, compressive loading causes the elastomer to bulge outward. When rotation is applied, compression decreases on one side of the bearing and increases on the other. Due to the near-incompressibility of the elastomer, material is displaced laterally from the highly compressed side toward the less compressed side, partially filling the space created by the upward movement of the tension-side edge. Consequently, the bearing may remain fully compressed even as one edge moves upward. Lift-off initiates only when the applied rotation becomes large enough to reduce the contact stress at the bearing edge to zero, resulting in a loss of contact between the bearing and the supporting surface. Prior to the 2010 edition of the AASHTO LRFD Specifications,¹⁶ lift-off was not permitted in the design of elastomeric bearings. Following the findings of Stanton et al.,³⁰ which showed that partial lift-off does not adversely affect bearing performance when properly considered in design, the current specifications permit partial lift-off. The current AASHTO LRFD Specifications¹⁷ provide two approaches for evaluating bearing stability, methods A and B. Method A limits the overall bearing thickness to one-third of the minimum plan dimension of a rectangular bearing or one-fourth of the diameter of a circular bearing. Method B uses more detailed explicit stability equations. In general, bearings that satisfy the Method A requirements will also satisfy the more rigorous Method B stability checks.⁸

Analytical and experimental studies on bearing behavior

An accurate representation of bearing rotational behavior is essential for evaluating girder rollover stability. In particular, changes in bearing contact conditions during the onset of uplift can reduce the effective rotational stiffness and, consequently, the resisting moment provided by the bearing against further girder rotation. During girder placement, the self-weight of the girder is transferred to the bearings. Before the installation of lateral bracing and the deck, the girders are relatively unstable, and flexible bearing pads allow the girders to rotate about their longitudinal axis, as discussed earlier. The importance of bearing rotational stiffness and lift-off was first highlighted by Mast,⁷ who noted the lack of experimental studies on bearing behavior, its impact on beam stability, and the effect of loss of contact during rotational lift-off.³⁵ Also, Mast⁷ investigated the roll stiffness of plain and laminated elastomeric bearing pads by comparing the resisting moments developed by plain and laminated bearings for a 41 m PCI BT-72 girder. The study showed that laminated bearings provided adequate resistance to girder overturning, whereas the girder was unstable when supported on plain pads.

Stanton et al.³⁰ developed analytical equations for predicting lift-off initiation in unbonded infinite strip elastomeric bearings, assuming an incompressible elastomer and inextensible reinforcement. The study showed that lift-off occurs when the applied rotation reaches three times the

characteristic rotation, or

$$\theta_0 = 3\theta_c = -\frac{3\varepsilon_a}{S} \quad (9)$$

where ε_a is the axial strain (negative in compression); S is the bearing shape factor; and θ_c is the characteristic rotation, corresponding to the onset of upward movement on the tension side of the bearing. Although the tension-side edge reaches its original elevation at θ_c , the bearing remains in compression, and full contact is maintained. To account for the reduction in contact area after lift-off, the authors introduced the parameter η , defined as the ratio of the effective contact width to the original bearing width. As the applied rotation increases beyond the lift-off rotation, the value of η progressively decreases, reflecting continued reduction of effective contact area. The findings formed the basis for design recommendations and stability procedures that were subsequently incorporated into the AASHTO LRFD Specifications¹⁶ for elastomeric bearings.

Green et al.³⁶ numerically studied the effects of skew angle on girder uplift at the supports. They found that increasing the skew angle reduced uplift at the girder ends and limited the loss of contact between the girder and the elastomeric bearings. However, increasing the skew angle also increased girder deflection and tensile stresses, indicating that higher bearing stiffness was required to control deflections and cracking stress. Likewise, Consolazio and Hamilton¹¹ carried out a parametric study to investigate the influence of skew angle, bearing slope, and bearing pad creep on the roll stiffness of bearings and the stability of girders. A series of tests were performed on different elastomeric bearing pads subjected to varying compressive stresses. Low to intermediate stresses represented typical dead load conditions where full and uniform contact developed across the entire bearing pad area, whereas higher stresses represented localized stress concentrations at the pad corners when skew and support slope reduced the initial contact area between the girder and the bearing. It was found that skew-slope interaction significantly reduced roll stiffness, whereas short-term bearing creep had a relatively minor effect. In a later full-scale experimental and numerical study conducted by the same authors,¹² the influence of skew angle and slope angle on the bearing roll stiffness was evaluated. The results confirmed that combining skew angle with bearing slope caused a noticeable reduction in bearing pad roll stiffness.

Harper and Consolazio²⁶ presented a numerical approach to estimate the axial stiffness of bearing pads. The method was validated through axial compression tests conducted on two standard types of elastomeric bearings used in Florida. In addition, the authors proposed a simplified grillage method to calculate the roll stiffness of steel-reinforced elastomeric bearings. In this approach, the bearing pad was divided into rectangular regions, and a compression-only spring was assigned to each region to approximate the non-linear moment-rotation behavior of bearings developed as a result of lift-off and loss of contact, as shown in Fig. 4a. The authors noted that the compressive stiffness distribution becomes nonuniform under compression due to rubber bulging, such that the stiffness varied with the distance from

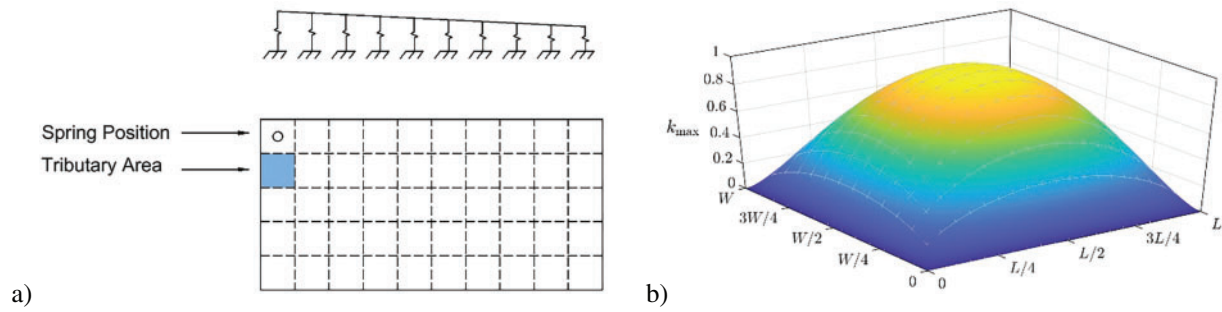


Figure 4. Simplified modeling of elastomeric bearings: a) grillage model introduced by Harper and Consolazio;²⁶ b) distribution of compressive axial stiffness in steel-reinforced elastomeric bearing predicted by bubble function

the center of the bearing. A parabolic bubble function was introduced to estimate the distribution of axial stiffness within the bearing pad. The bubble function was defined as

$$k(x, z) = k_{\max} \left[1 - \left(\frac{2x}{L} - 1 \right)^2 \right] \left[1 - \left(\frac{2z}{W} - 1 \right)^2 \right] \quad (10)$$

where L and W are the smaller and larger plan dimensions of the bearing, respectively, x and z are normalized coordinates, and $k_{\max}$ is the maximum axial stiffness at the center of the pad. The distribution of axial stiffness resulting from Eq. 11 is shown in Fig. 4b. Cardoso and Lima³⁷ numerically studied the influence of geometric imperfection and skew angle on bearing stiffness and girder rollover stability using this bubble function. This study confirmed earlier research findings indicating that the rotational stiffness of the bearing decreases with increasing sweep and skew angle.

Oliveira et al.³⁸ numerically investigated the effect of bearing pad dimensions on the rollover stability of precast girders supported on bearings. The results indicated that bearing stiffness was a key parameter affecting beam stability. Concurrently, Krahel et al.³⁵ proposed a simplified analytical model to describe the nonlinear contact interaction between precast girders and steel-reinforced elastomeric bearings, including contact loss mechanisms that develop during lift-off. In this model, the interaction between the girder and the bearing was represented by a rigid beam supported on elastic springs. The girder was assumed to rotate primarily about its longitudinal axis; therefore, the bubble function was modified to vary only about this axis, and the bearing moment–rotation behavior was treated as a plane problem.³⁹ Under these assumptions, the amplitude of the bubble function was also modified to provide axial stiffness equivalent to the original formulation (Fig. 5a). Moreover, the authors proposed a new simplified approach to approximate nonlinear softening due to lift-off. It was assumed that some springs lose contact with the rigid plate during lift-off, thereby altering the peak compressive stress and the stress distribution along the pad. To approximate this behavior, the axial stress distribution along the pad length was modified by excluding the portion under the width of the bearing where lift-off has been calculated, as illustrated in Fig. 5b. Using the energy method, the following

closed-form nonlinear equation was developed to determine the moment–rotation relationship of the bearing:

$$M = \frac{E_{\text{pad}} W \theta (L^2 + 6Lx_0 + 3x_0^2) (L - x_0)^4 + 30PHL^2 x_0^2}{20HL^2 (L + 2x_0)} \quad (11)$$

where W and L denote the width and length plan dimensions of the bearing, respectively; H is the bearing thickness; E_{pad} is the effective elastic modulus of the elastomeric bearing in compression; P is the vertical load on the bearing resulting from the girder self-weight; and x_0 is the width of contact loss between the girder and the bearing. The model was validated by comparing its predictions with the experimental measurements published by Consolazio and Hamilton.¹² By incorporating the lift-off behavior, the proposed formulation provided improved predictions of bearing rotational stiffness compared to prior analytical models.

In a recent study, Costa et al.³⁹ proposed equilibrium equations to evaluate the lateral stability of precast girders supported on bearings during the erection stage. A simplified approach was proposed, wherein the nonlinear moment–rotation relationship developed by Krahel et al.³⁵ was approximated using a bilinear function, with separate solutions defined for $\theta < \theta_{\text{crit}}$ and $\theta > \theta_{\text{crit}}$ (Fig. 6). Using this bilinear relationship in combination with the energy-based approach, analytical expressions for the critical load were developed. In these expressions, geometric imperfections were coupled with the nonlinear contact interaction between the girder and the bearing to evaluate different conditions. The proposed approach was applied to evaluate the equilibrium states of a PCI BT-54 girder for different cases, including a perfect geometry, pure sweep, sweep combined with rotation, and camber without sweep, and the results were compared with the analytical studies of Oliveira⁴⁰ and Krahel et al.⁴¹ It was revealed that linear rotational spring stiffness assumptions, such as those suggested by current design codes and guidelines, result in an overestimation of the rollover capacity by neglecting the softening effect of contact nonlinearity during lift-off.

Van Engelen²⁸ developed analytical equations for predicting lift-off initiation in infinite strip, rectangular, and circular elastomeric bearings, considering the effects of elastomer

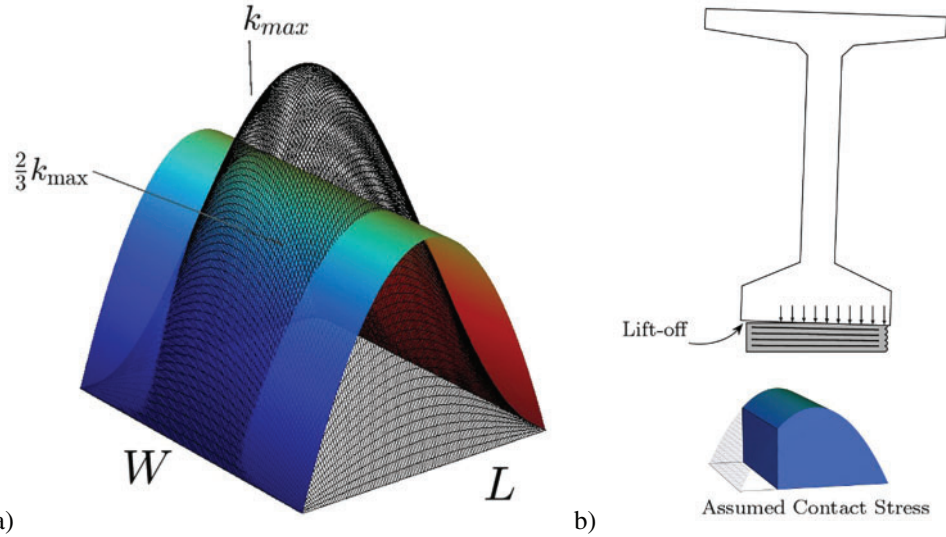


Figure 5. Assumptions adopted by Krahl et al.³⁵: a) equivalent stiffness distribution; b) stress distribution

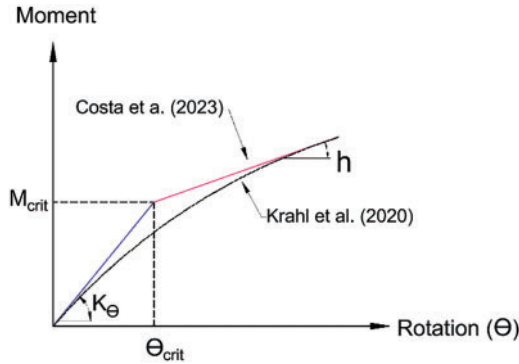


Figure 6. Bi-linear approximation of moment-rotation relationship proposed by Costa et al.³⁹

compressibility and reinforcement extensibility. For rectangular bearings, the lift-off rotation was expressed by

$$\frac{\varepsilon_c t}{\theta_0 b} = \frac{\sum_{n=1}^{\infty} \frac{1}{\xi_{rn}^2} \left(1 - \frac{1}{\cosh(\xi_{rn})}\right)}{\sum_{n=1,3,5,\dots}^{\infty} \frac{1}{\xi_{cn}^2} \left(1 - \frac{1}{\cosh(\xi_{cn})}\right)} \quad (12)$$

where

$$\xi_{cn}^2 = \lambda^2 \frac{(\rho + 1)^2}{\rho^2} + \frac{n^2 \pi^2}{4 \rho^2} \quad (13)$$

$$\xi_{rn}^2 = \lambda^2 \frac{(\rho + 1)^2}{\rho^2} + \frac{n^2 \pi^2}{\rho^2} \quad (14)$$

$$\frac{1}{K_e(e)} = \frac{1}{K} + e \frac{t}{E_f t_f} \quad (15)$$

$$\lambda^2 = \frac{12 G S^2}{K_e(2)} \quad (16)$$

where θ_0 denotes the rotation at initiation of lift-off; λ is the compressibility–extensibility parameter; a and b are the half-dimensions of the bearing plan; $\rho = \frac{b}{a}$ is the aspect ratio of the rectangular bearing; $S = \frac{ab}{t(a+b)}$ is the shape factor; K

is the bulk modulus of the elastomer, t is the thickness of a single elastomer layer, t_f is the thickness of the reinforcement layer, E_f is the effective elastic modulus of the reinforcement, and ε_c is the compressive strain. The coefficient e depends on the bearing geometry and loading conditions.

The study further investigated the post lift-off response of unbonded bearings by considering the reduction in effective contact area. Using the effective contact width, $b_i = \eta b$, analytical expressions were developed for the instantaneous bending modulus (Eq. 17), and the corresponding moment-rotation relationship (Eq. 18) after lift-off. In these equations, $(E_b I)_i$ is the instantaneous bending stiffness after lift-off, $(E_b I)_{eff}$ is the effective bending stiffness before lift-off. The results indicated that current design provisions do not accurately predict the rotational capacity of elastomeric bearings prior to lift-off or the development of tensile hydrostatic stresses. The results also showed that the bending modulus decreases after lift-off while the rotation continues to increase, resulting in a nonlinear moment-rotation response.

$$\frac{(E_b I)_i}{(E_b I)_{eff}} = \frac{\eta^3 \sum_{n=1}^{\infty} \frac{1}{n^2 \pi^2 \xi_{rni}^2} \left(1 - \frac{\tanh(\xi_{rni})}{\xi_{rni}}\right)}{\sum_{n=1}^{\infty} \frac{1}{n^2 \pi^2 \xi_{rn}^2} \left(1 - \frac{\tanh(\xi_{rn})}{\xi_{rn}}\right)} \quad (17)$$

$$M = \int \frac{(E_b I)_i}{t} d\theta \quad (18)$$

The analytical solutions developed by Van Engelen²⁸ were verified by Bai et al.³³ using finite element models of infinite strip elastomeric bearings with different numbers of layers. The study examined the influence of lift-off on the instantaneous bearing geometry and moment–rotation response under combined compression and rotation. The analytical predictions showed good agreement with the finite element results, indicating that the proposed formulations can accurately predict lift-off behavior of unbonded reinforced elastomeric bearings.

Table 2. Geometric, material, and loading properties of the elastomeric bearing used in illustrative comparison

Property	Value
Bearing length, L	508 mm
Bearing width, W	508 mm
Total bearing thickness, H	60.3 mm
Number of elastomer layers	4
Elastomer layer thickness, t	12.7 mm
Steel reinforcement thickness, t_f	3.18 mm
Elastomer shear modulus, G	0.758 MPa
Elastomer bulk modulus, K	3102.6 MPa
Reinforcement elastic modulus, E_f	200,000 MPa
Applied compressive load, P	444.8 kN

Table 3. Comparison of predicted rotation at initiation of lift-off and corresponding moment

Method	$\theta_{\text{lift-off}}$ (rad)	$M_{\text{lift-off}}$ (kN · m)
PCI ⁶	0.00152	18.83
Stanton et al. ³⁰	0.00123	20.90
Krahl et al. ³⁵	0.00187	22.65
Van Engelen ²⁸ -Infinite-strip	0.00100	23.59
Van Engelen ²⁸ -Rectangular	0.00161	27.16

Comparison of predictions for rotation at lift-off and moment–rotation resistance

This section presents a comparison of predictions for the rotation at initiation of lift-off and moment–rotation resistance obtained by existing methods for a representative unbonded steel-reinforced elastomeric bearing. The geometric and material properties of the bearing considered in this illustrative example are summarized in Table 2. The calculated rotation at the initiation of lift-off and corresponding lift-off moments determined using the methods previously reviewed are presented in Table 3. Initial inclination of the support and eccentricity of the bearing were neglected in determining equilibrium with the PCI⁶ method, since none of the other methods account for such geometric imperfections. The results indicate significant variation among the analytical methods, with predicted lift-off rotations ranging from 0.00100 rad to 0.00187 rad. Differences among the predictions can be attributed to the assumptions used in each method. As discussed earlier, the Krahl et al.³⁵ approach is based on an energy method in which lift-off is represented by a reduction in the contact length between the bearing and girder. The Stanton et al.³⁰ and Van Engelen²⁸ formulations are based on a pressure solution. In contrast, the PCI⁶ method is based on force equilibrium and determines lift-off from the location of the resultant reaction within the bearing kern point. The PCI⁶ method is found to produce a conservative estimate of the rotation and moment resistance

at the initiation of lift-off compared to the Krahl et al.³⁵ and Van Engelen²⁸ formulations for rectangular bearings, underestimating the lift-off moment by 17% and 31%, respectively.

Fig. 7 compares the moment–rotation relationships predicted by the PCI⁶ method, Krahl et al.³⁵ approach, and Van Engelen²⁸ formulations for both infinite-strip and rectangular bearing geometries. The comparison highlights the significant conservatism inherent in the PCI⁶ method, which predicted lower rotation and corresponding moment at lift-off than the more refined approaches and, more significantly, does not leverage the nonlinear resistance offered by the bearing after initiation of lift-off. Incorporating nonlinear rotational resistance into stability assessments using the Krahl or Van Engelen formulations could reduce the conservatism associated with the PCI⁶ method and potentially allow greater sweep tolerances for seated girders. However, given the apparent discrepancies between the resistance predicted by these models for this illustrative bearing example, further research should be directed toward experimental validation of the techniques before adopting a formulation for practical implementation.

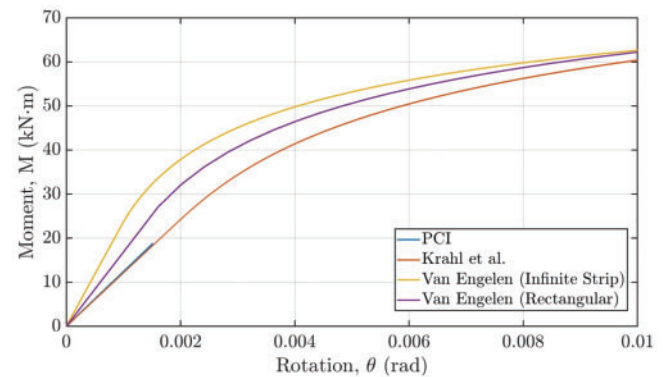


Figure 7. Comparison of predicted moment–rotation relationships for illustrative bearing example

Experimental and Analytical Studies on Girder Rollover

Rollover stability under gravity

The influence of geometric imperfections and support flexibility under gravity has been the foundation of most analytical and experimental investigations for rollover stability. The first comprehensive analytical and experimental investigation of rollover stability in prestressed concrete girders supported on elastic bearings was conducted by Mast.⁷ As previously discussed, Mast developed simplified equations to estimate factors of safety against cracking and rollover failure, assuming that the girder behaves torsionally rigid. The proposed equations account for the influence of initial sweep, bearing stiffness, and initial bearing inclination. His analytical work was validated through full-scale lateral bending tests of 45.5 m long prestressed I-girders with initial sweep supported on steel supports.⁴² Although this girder was not seated on elastomeric bearings, it is

included in this review due to the scarcity of experimental tests on rollover instability and due to important findings related to the fundamental behavior of the girders during rollover. The experimental observations confirmed that torsional deformation was minimal, supporting the assumption that torsional flexibility could be neglected, as the midspan twist remained small relative to the overall roll angle.

Building on Mast's framework, Burgoyne and Stratford⁴³ developed an analytical relationship between the critical self-weight producing rollover and the rotational stiffness of flexible supports. Their findings indicated that initial imperfections generated increased tensile stresses and promoted cracking, thereby reducing the effective minor-axis bending stiffness. The key difference between their method and Mast's lies in how imperfections were incorporated. Mast⁷ introduced initial imperfections directly into the equilibrium equations, whereas Burgoyne and Stratford assumed a geometrically perfect beam and then computed the stresses induced by initial sweep. By evaluating the resulting tensile stresses at critical locations, they concluded that the minor-axis flexural stiffness should be reduced once cracking is expected to occur.

Further developments were presented by Consolazio and Hamilton¹¹ through an extensive parametric study evaluating the influence of sweep, girder type and cross-sectional properties, span length, and bracing stiffness on the lateral stability of seated girders. It was demonstrated that buckling capacity was highly sensitive to girder cross-sectional properties and span length, and decreased rapidly as span length increased. In addition, sweep and insufficient bracing substantially reduced the rollover and buckling capacities.

Hurff and Kahn⁴⁴ conducted full-scale experimental and numerical investigations on the rollover stability of a PCI BT-54 girder with a 30.5 m span. The effects of initial sweep, initial bearing inclination, and bearing stiffness were evaluated. The girder was supported on two steel-reinforced elastomeric bearings and subjected to a concentrated vertical load at the midspan. The vertical, lateral, and rotational response of the girder was monitored at the midspan and at the bearing supports using string potentiometers and strain gauges. Surface-mounted LVDTs were used to estimate effective prestress and measure the strain distribution through the cross section at the midspan. Initial lateral displacements at the top and bottom of the girder were also measured at different locations along the span to characterize the sweep. Prior to testing, a layer of epoxy was applied at the bearing surfaces to mitigate gaps caused by initial out-of-flatness and camber-induced end slope. The test program was performed with an initial bearing inclination of 0.05 radians, which resulted in amplification of the initial sweep. Rollover was the governing instability mode exhibited by the girder during the experiment, occurring before cracking or lateral-torsional buckling, and the results indicated that geometric imperfections and bearing rotation significantly influenced rollover stability. Test measurements indicated significant shear deformation in the bearings, which reduced the vertical stiffness near the compressed edge. In addition, loss of contact occurred prior to rollover, leading to a reduction in rotational stiffness and a more nonlinear response.

The numerical model developed by the authors did not reproduce the experimental behavior, particularly at higher load levels, and instead predicted a stiffer, nearly linear response. This discrepancy was due to neglecting bearing shear deformation and idealizing the supports as linearly elastic.

Chamorro-Varilla and Aristizábal-Ochoa⁴⁵ investigated the static response of a long-span elastic girder with initial sweep and overhangs at both ends, supported on elastomeric bearings with initial inclination and subjected to a concentrated vertical load at the midspan. An analytical model was developed to predict deflections, stresses, internal forces, twist, and rigid-body rotation of the girder. Although the cross section considered in the study was doubly symmetric, the proposed approach was generalized to the static analysis of typical precast bridge girders. To evaluate the accuracy and efficiency of the method, a finite element model was developed based on the girder in the experimental study by Hurff and Kahn⁴⁴ and the results were compared. While the analytical model predicted well the overall structural response, differences were observed relative to the experimental results. These discrepancies were attributed to the omission of prestressing effects, shear deformation of the bearing, and uncertainty in the assumed bearing stiffness used in the analytical formulation.

Lee⁴⁶ proposed analytical expressions to estimate the critical weight and lateral displacement of both perfect and imperfect girders, accounting for initial sweep and elastomeric bearing stiffness. The formulation was based on the critical weight equation developed by Burgoyne and Stratford,⁴³ with modifications to incorporate the additional lateral displacement associated with rotation along the girder. To evaluate the influence of initial sweep and slenderness on lateral stability, numerical models of AASHTO BT-54, BT-63, and BT-72 girders with a 30~m span were analyzed using the Abaqus finite element software. The girders were modeled with linear beam elements, while the bearings were represented as compression-only springs connected to the girder through rigid link elements. The results indicated that initial sweep reduced both the critical weight and the lateral displacement at which instability occurs. In addition, increasing slenderness further reduced the critical weight.

Cardoso and Lima³⁷ investigated the effect of bearing stiffness, concrete strength and material nonlinearity of the girder on the lateral stability of precast beams supported on elastomeric bearings using finite element simulations conducted with ANSYS. The results showed that increased concrete strength and, consequently, beam stiffness reduced beam displacements and limited the loss of contact area between the beam and the bearings.

Krahl and colleagues conducted a series of analytical studies using the Rayleigh–Ritz method and Monte Carlo simulation. Krahl et al.⁴⁷ numerically and analytically studied the rollover instability of precast concrete girders supported on elastomers by developing a finite element model of a PCI BT54 that had been previously tested by Hurff and Kahn.⁴⁴ The nonlinear behavior of the elastomeric

bearings was represented using the grillage model with non-linear compression springs. A parametric study assessing the influences of different parameters, including sweep, bearing inclination, concrete elastic modulus, top flange width, and span length, on instability load was carried out. While concrete elastic modulus had only a minor influence, the remaining parameters significantly affected the instability load, with increased bearing skew and sweep and reduced top-flange width leading to lower capacity. Krahle et al.⁴⁸ developed closed-form equations to estimate rollover load for beams on flexible supports under various loading conditions. However, the formulation was based on linear analysis to develop closed-form solutions for elastic buckling and did not account for initial inclination of the girder as a possible geometric imperfection. Subsequently, Krahle et al.⁴⁹ extended the approach to incorporate nonlinear rollover behavior and examined the effects of sweep, camber, and support inclination. Their results demonstrated that bearing inclination was the most influential geometric imperfection, and that the most critical rollover condition occurred when initial sweep and rotation were combined with camber. Oliveira et al.³⁸ further combined the Rayleigh–Ritz method with the Minimum Potential Energy approach to estimate the rollover capacity of precast beams supported on bearing pads. A parametric study was conducted to evaluate the influence of initial sweep, initial support inclination, concrete compressive strength, bearings geometry, and geometry of the cross-section on beam stability. The results indicated that beam geometry and bearing stiffness were among the most influential parameters controlling the critical rollover load. The rollover capacity increased with higher concrete strength and larger top flange width, whereas it decreased with larger initial sweep and support inclination. Moreover, the initial cracking load was found to be close to the critical load associated with rollover instability.

Recently, Costa et al.³⁹ analytically evaluated the lateral stability of slender ultra-high-performance concrete (UHPC) beams supported on elastomeric bearings, considering geometric imperfections and the nonlinear behavior of the bearings. It was found that ordinary concrete beams were more sensitive to initial imperfections, although the more slender cross-section of UHPC girders still made them vulnerable to lateral instability despite their higher strength.

Influence of wind loading on rollover stability

Wind loading introduces additional lateral forces and overturning moments that amplify existing eccentricities and reduce available safety margins during erection. Mast⁷ evaluated wind effects and showed that wind loading increased the initial eccentricity, generating an overturning moment about the bottom edge of the bearings. Long span prestressed girders were found to be highly vulnerable to rollover under wind loading unless adequate lateral bracing was provided.

Plaut and Moen⁵⁰ investigated the lateral stability of unbraced, horizontally curved prestressed concrete girders supported on bearings under wind loading, considering both rollover and sliding behavior. A modified factor of safety incorporating wind effects was introduced, and it was shown

that PCI design equations, which were developed primarily for straight girders and small roll angles, may overestimate the factor of safety against cracking. A sliding-instability criterion based on wind force and friction was also established, demonstrating that low-friction bearings significantly increased the risk of sliding failure. Their analyses further indicated that roll angle increased with decreased bearing stiffness, increased sweep, reduced weak-axis stiffness due to cracking, and higher wind loads.

Consolazio and Hamilton¹¹ conducted a parametric study to evaluate the lateral stability of prestressed concrete girders under wind loading. The study examined the effects of girder section type, span length, skew angle, bearing slope, and initial sweep on the rollover capacity of the girders under lateral wind pressure and developed empirical equations to estimate wind capacity. The results demonstrated that wind capacity linearly decreased with span length. Later, Lee et al.⁵¹ conducted analytical and numerical studies to estimate the critical wind load, lateral displacement, and rotational angle that cause rollover. The effects of girder length and cross-sectional properties were examined, and the results showed that girder length strongly influenced critical wind load and support rotation, whereas cross-sectional properties had comparatively minor influence.

In a recent study, Cardoso and Lima⁵² numerically investigated the influence of wind loading on the lateral stability of precast concrete girders supported on elastomeric bearings. In addition, the effects of concrete strength and bearing compression stiffness were evaluated by developing a finite element model, in which the nonlinear bearing response was represented using a grillage system composed of compression-only springs. The results showed that concrete strength significantly influenced lateral stability, and that wind loading increased the overturning moment, resulting in loss of contact between the girder and the bearing, a reduction in the effective roll stiffness, and a corresponding decrease in rollover stability.

Thermal effects on rollover stability

The influence of thermal effects on rollover behavior were examined by Lee,¹³ by developing vertical and transverse thermal gradients and carrying out a 3D nonlinear finite element analysis of a PCI BT-63 girder with a 30.5 m span length. The girder remained stable under self-weight and thermal loading in the absence of initial sweep and support rotation. However, instability occurred when thermal gradients were combined with an initial sweep of 114 mm and a bearing inclination of 5°. Honing et al.⁵³ further improved the thermal-gradient model proposed by Lee to evaluate the effect of thermal sweep on the rollover stability of precast girders under lateral wind loading using FE simulations. A parametric study was carried out to assess the influence of girder type, span length, and environmental conditions. The results indicated that thermal sweep reduced wind resistance, and this reduction became more pronounced for longer spans. Hurff⁵⁴ provided additional insight through full-scale experimental and analytical investigations of a PCI BT-54

girder with 30.5 m span length subject to solar radiation on the top and sides of the girder. Wind speed, internal strain, air temperature, internal temperature, and surface temperature were monitored to evaluate additional sweep and rotation resulting from non-uniform thermal effects. It was found that despite a thermally induced increase in sweep of approximately 40%, non-uniform heating had a negligible effect on rotation of the girder cross section.

Comparative Synthesis

Table 4 provides a comparative synthesis of the studies reviewed in this section, highlighting differences in modeling approaches, parameters considered, treatment of cracking and lift-off, validation or verification methods, key findings, limitations, and recommendations. This comparison

helps identify common modeling assumptions and remaining research needs discussed in the following section.

Gaps and Future Research

The research gaps identified in the literature can be organized into four priority areas: (1) analytical modeling and experimental validation, (2) bearing behavior and girder–bearing interaction, (3) construction tolerances and lateral restraint, and (4) girder-specific and time-dependent effects.

Analytical modeling and experimental validation

Existing analytical models of rollover stability rely on several simplified assumptions that require further evaluation Mast.^{4,7} idealized prestressed concrete girders as torsionally rigid based on their relatively high torsional stiffness, and

Table 4. Comparative synthesis of previous research on rollover stability

Study	Analysis type	Girder type	Bearing model	Parameters considered	Validation	Main limitations
Mast ⁷ (Journal)	Analytical & Numerical. (No FEA model)	PCI BT72	Linear elastic bearing rotational stiffness	Initial sweep; Bearing inclination; Bearing stiffness; Cracking; Girder self-weight; Wind	Test of girders with rigid supports	Idealized torsionally rigid girder; Twist and major-axis bending deformation neglected; Simplified bearing rotational stiffness; Thermal effects, bearing shear deformation and lift-off not considered
Mast ⁴² (Journal)	Experimental	PCI BT72	Steel cradles	Initial sweep; Cracking; Girder self-weight	Full-scale test	Steel-cradle supports rather than elastomeric bearings; Bearing inclination, bearing shear deformation, thermal effects, lift-off and wind not considered
Burgoyne and Stratford ⁴³ (Journal)	Analytical (No FEA model)	Prestressed concrete beams (M10 and SY6)	Equivalent rotational springs	Initial sweep; Cracking; Girder self-weight	None	Bearing represented by equivalent rotational stiffness; Torsional deformation ignored; Stiffness degradation due to cracking neglected; Bearing inclination, bearing shear deformation, thermal effects and lift-off not considered
Consolazio & Hamilton ¹¹ (Report)	Analytical & Numerical (FEA model developed with ADINA)	FBT54,63,72,78	Simplified linear springs	Initial sweep; Girder type; Girder cross-sectional properties; Span length; Bracing stiffness; Skew angle; Bearing inclination; Bearing creep; Lift-off; Girder self-weight; Wind	None	Simplified bearing representation; Bearing shear deformation, cracking and thermal effects not considered
Lee ¹³ (Thesis)	Analytical (3D nonlinear FEA model developed with ABAQUS)	PCI BT63	A series of nonlinear springs	Initial sweep; Bearing inclination; Thermal effects; Girder self-weight	None	Bearing behavior simplified using vertical springs; Bearing shear deformation, lift-off, cracking and wind not considered; Limited girder type and length
Hurff ⁵⁴ (Thesis)	Analytical & Experimental (No FEA model)	PCI BT54	Discrete strips with bilinear vertical stiffness	Initial sweep; Bearing inclination; Bearing stiffness; Lift-off; Girder self-weight & midspan vertical load; Thermal effects	Full-scale test of girder on elastomeric bearings	Bearing model did not fully reproduce nonuniform contact; Bearing shear deformation and wind not considered; Overly stiff analytical response at large rotations

Table 4. (Continued)

Study	Analysis type	Girder type	Bearing model	Parameters considered	Validation	Main limitations
Hurff and Kahn ⁴⁴ (Journal)	Numerical (No FEA model)	PCI BT54	Discrete strips with bilinear vertical stiffness	Initial sweep; Bearing inclination; Bearing stiffness; Lift-off; Girder self-weight & midspan vertical load; Thermal effects	Hurff ⁵⁴ data	Bearing shear deformation neglected; Idealized support contact behavior; Bearing model did not fully reproduce nonuniform contact; Overprediction of stiffness at larger rotations; Cracking and wind not considered
Plaut and Moen ⁵⁰ (Journal)	Analytical & Numerical (No FEA model)	Horizontally curved prestressed concrete girders	Elastic rotational springs	Initial sweep; Bearing stiffness; Bearing friction; Girder self-weight; Wind	None	Linear-elastic girder response; Linear bearing rotational stiffness; Cracking, lift-off; bearing shear deformation and thermal effects not considered
Chamorro-Varilla and Aristizábal-Ochoa ⁴⁵ (Journal)	Analytical & Numerical (FEA model developed with ABAQUS)	PCI BT54	Nonlinear elastic rotational supports (Richard–Abbott model)	Initial sweep; Bearing inclination; Overhang effects; Girder self-weight & midspan vertical load	Hurff ⁵⁴ data	Prestressing effects, concrete cracking, bearing shear deformation, girder vertical curvature, lift-off, thermal effects and wind not considered; Residual stresses neglected
Lee et al. ⁵¹ (Journal)	Analytical & Numerical (FEA model developed with ABAQUS)	AASHTO girders type IV, V, VI	Nonlinear compression only springs (Grillage method)	Girder length and cross-sectional properties; Girder self-weight; Wind	None	Initial sweep, bearing inclination, bearing shear deformation, thermal effects, lift-off and cracking not considered
Lee ⁴⁶ (Journal)	Analytical & Numerical (FEA model developed with ABAQUS)	PCI BT54,63,72	Distributed nonlinear compression-only springs, with bilinear compressive stiffness	Initial sweep; Lift-off; Girder self-weight	None	Limited to girder sections and sweep range; Simplified bearing behavior; Bearing inclination, bearing shear deformation, thermal effects, cracking and wind not considered
Krahl et al. ⁴⁷ (Journal)	Analytical & Numerical (FEA model developed with ABAQUS)	PCI BT54	Nonlinear compression only springs (Grillage method)	Initial sweep; Skew angle; Concrete elastic modulus; Top flange width; Span length; Lift-off; Girder self-weight	Hurff ⁵⁴ data	Linear-elastic concrete behavior; Bearing shear deformation, thermal effects, cracking and wind not considered
Honing et al. ⁵³ (Journal)	Analytical & Numerical (FEA model developed with ADINA)	PCI BT63, AASHTO Type V, FIB36,45,54,63, 72,78,84,96	6-DOF elastic springs	Initial sweep and camber; Girder type; Length; Environmental conditions; Lift-off; Girder self-weight; Wind	None	Bearings represented by equivalent 6-DOF springs; Nonlinear contact degradation not directly captured; Limited parametric study; Lift-off, cracking, bearing shear deformation and bearing inclination not considered
Krahl et al. ⁴⁸ (Journal)	Analytical (No FEA model)	PCI BT54 & rectangular section	Nonlinear compression only springs (Grillage method)	Initial sweep; Concrete modulus of elasticity; Top flange width; Span length; Skew angle; Lift-off; Girder self-weight and concentrated load at midspan, and symmetrical two-point loads	Hurff, Consolazio & Hamilton ^{12,54} data	Simplified equivalent bearing rotational stiffness; Linear analytical solution; Pre-buckling and post-buckling behavior not captured; Bearing shear deformation, thermal effects and cracking not considered
Cardoso and Lima ³⁷ (Journal)	Numerical (No FEA model)	AASHTO Type IV	Nonlinear compression only springs (Grillage method)	Initial geometric imperfection; Concrete strength and material-nonlinearity; Bearing stiffness; Cracking; Lift-off; Girder self-weight	None	Limited to one girder; Limited bearing configurations; Bearing shear deformation, girder vertical curvature, thermal effects and wind not considered

Table 4. (Continued)

Study	Analysis type	Girder type	Bearing model	Parameters considered	Validation	Main limitations
Cardoso and Lima ⁵² (Journal)	Numerical (No FEA model)	AASHTO Type IV	Nonlinear compression only springs (Grillage method)	Initial sweep; Concrete strength; bearing stiffness; Cracking; Lift-off; Girder self-weight; Wind	None	Limited to one girder; Limited bearing configurations; Bearing shear deformation and thermal effects not considered; Convergence limitations in highly nonlinear cases
Krahl et al. ³⁵ (Journal)	Analytical (No FEA model)	PCI BT54, BT63, BT72	Nonuniform elastic springs	Bearing stiffness; Lift-off; Girder self-weight	Consolazio & Hamilton ¹² data	Initial sweep, bearing inclination, thermal effects, wind and cracking not considered; Torsional deformation neglected
Krahl et al. ⁴⁹ (Journal)	Analytical (No FEA model)	PCI BT54	Rotational springs using secant rotational stiffness with nonlinear moment–rotation behavior	Initial sweep and camber; Bearing inclination; Lift-off; Girder self-weight	None	Bearing vertical and shear deformation, thermal effects, wind and cracking not considered; Torsional and warping effects neglected
Oliveira et al. ³⁸ (Journal)	Analytical (No FEA model)	PCI BT54, BT63, BT72	Equivalent rotational springs	Initial sweep; Initial bearing inclination; Concrete compressive strength; Girder cross-section; Bearing dimension & stiffness; Cracking; Lift-off; Girder self-weight	None	Simplified bearing model; Bearing shear deformation, wind and thermal effects not considered; Torsional and warping effects neglected; Limited girder and bearing configurations, sweep ranges, bearing inclination and concrete strength
Costa et al. ³⁹ (Journal)	Analytical (No FEA model)	UHPC I-girder & PCI BT54	Nonlinear compression-only springs with bilinear moment–rotation relationship	Initial sweep and camber; Bearing inclination; Lift-off; Girder self-weight & construction loads	None	Idealized bearing contact; Torsional and warping deformation neglected; Bearing shear deformation, cracking, wind and thermal effects not considered; Limited UHPC girder

therefore neglected both twist deformation and major-axis bending in his analysis. This assumption has been retained by nearly all analytical models used to assess rollover factors of safety. However, under rollover conditions, excluding deformation modes that may influence the initiation of cracking and the loss of equilibrium can lead to non-conservative stability estimates⁵⁵). Mast⁴² also investigated the rollover behavior of prestressed concrete girders supported on rigid supports rather than flexible bearings. Unlike flexible supports, which provide rotational flexibility that allows the girder to roll laterally and develop lateral bending, rigid supports restrain such movements and alter the rollover response. Currently, experimental testing of rollover instability of girders seated on elastomeric bearings is limited to the single test reported by Hurff.⁵⁴ Additional full-scale experimental testing, and high-fidelity finite element modeling validated against the experimental observations, are needed to validate rollover capacity equations and refine analytical models for reliable use in design.^{11,54}

Bearing behavior and girder-bearing interaction

The nonlinear response of elastomeric bearings under girder self-weight remains a major source of uncertainty in rollover stability assessment. Current bearing stiffness estimates are generally based on service-level loading, whereas rollover failure typically occurs under self-weight alone. As a result,

experimental studies are needed to quantify the axial and rotational stiffness of elastomeric bearing pads under self-weight conditions.⁵⁴ Additional research is also required to evaluate how bearing pad roll stiffness influences rollover behavior across different girder types, span lengths, and loading conditions^{13,56,12}. All existing approaches for estimating the nonlinear rotational stiffness of elastomeric bearings neglect potential softening due to interaction with shear deformations in the bearing, which may be significant if there is initial support inclination. Further experimental and analytical studies are needed to better characterize this behavior and incorporate it into rollover stability assessments. The influence of bottom-flange flatness imperfections represents another important issue that requires further investigation, as it may help identify situations in which the use of an embedded steel plate is necessary.^{48,54}

Construction tolerances and lateral restraint

The review of current codes and guidelines indicates that a defined limit for initial rotation of prestressed concrete bridge girders is still needed.⁵⁴ While the PCI Bridge Design Manual⁸ specifies allowable tolerances for initial sweep, it does not clearly define corresponding limits for initial rotation of girders associated with construction. In addition, the required strength and stiffness of lateral bracing needed to prevent rollover instability are not explicitly specified.^{13,54}

The PCI Bridge Design Manual⁸ recommends the use of lateral bracing but provides limited guidance on requirements for the design of temporary bracing. Most existing studies have focused on the effectiveness of bracing at girder end supports. Further analytical and experimental research is needed to evaluate the role of intermediate bracing systems that connect multiple girders and allow them to act together as a system, particularly during construction stages.¹¹

Girder specific and time-dependent effects

Costa et al.³⁹ showed that UHPC girders are more sensitive to initial geometric imperfections than conventional concrete beams. As a result, allowable limits for imperfections in UHPC girders, including sweep and initial inclination of the girder, should be carefully evaluated and clearly defined. Additional factors that remain insufficiently addressed include prestress losses, nonuniform stress distribution in strands, time-dependent effects such as creep and shrinkage, and impact loads during construction. Further research is needed to determine the influence of these factors on rollover stability.

Conclusion

This state-of-the-art review discussed existing research regarding the rollover behavior of prestressed concrete bridge girders supported on flexible supports. First, the fundamental theories of rollover stability and key controlling parameters were introduced. Then, the specifications in current design codes and guidelines related to rollover stability and their limitations were discussed. In addition, the influence of initial geometric imperfections, skew and slope angle, initial bearing inclination, and thermal sweep on rollover stability under self-weight and lateral wind loading, acting both individually and in combination, was discussed, and the related analytical, numerical, and experimental studies were comprehensively reviewed. It is shown that initial sweep and support inclination are the primary imperfections that reduce lateral stability by increasing tensile stresses and promoting cracking, thereby lowering effective weak-axis stiffness. Bearing behavior has a major influence on rollover stability; reductions in rotational stiffness, increases in support inclination, and the combined effects of skew angle and end slope decrease rollover capacity. Wider bearing pads provide greater stability, while bearings with low stiffness or low friction increase rotational and sliding demands. In addition, bottom-flange flatness imperfections may introduce unintended initial rotation, contributing to earlier onset of instability. Lateral wind loading increases the overturning moment and reduces the vertical load capacity, especially for long-span girders, while thermal effects become critical primarily when combined with sweep or support inclination. The effects of girder length and cross-sectional properties on lateral stability vary depending on the loading and failure conditions considered. Under wind loading, girder length was found to have a greater influence on critical wind load than cross-sectional properties. However, other studies showed that cross-sectional properties significantly affect

rollover and buckling capacity. Thus, the effects of these parameters should be evaluated based on the specific stability condition considered. Cracking reduces both flexural and torsional stiffness, confirming the need to consider stiffness degradation during construction.

Overall, the reviewed studies demonstrate that rollover instability is governed largely by support flexibility and geometric imperfections rather than material strength. However, differences remain among the existing analytical models, particularly in the representation of bearing lift-off and nonlinear rotational resistance. Current design provisions also provide limited guidance on initial support inclination, nonlinear bearing behavior, and temporary bracing requirements. These limitations can affect the assessment of rollover stability during transportation and erection. Further experimental studies are needed to validate nonlinear bearing models, establish acceptable limits for support inclination, and evaluate the stiffness and strength requirements of temporary and intermediate bracing systems.

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Data Availability Statement

Some or all data, models, or code generated or used during the study are available from the corresponding author by request.

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Corrosion-Induced Bond-Transfer Deterioration in Bridge Deck Concrete: Tension Stiffening, Crack Spacing, and Member-Level Response in Fly Ash and Normal Portland Cement Systems

Lamya Amleh*

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Abstract: Chloride-induced reinforcement corrosion degrades steel–concrete interaction, affecting tension stiffening and cracking in reinforced concrete. This study re-analyzes a legacy direct-tension dataset comprising 30 specimens from three concrete systems: normal Portland cement (NPC) concrete with ($w/c = 0.52$), higher-quality NPC concrete with ($w/c = 0.32$), and high-volume fly ash (HVFA) concrete ($w/cm = 0.32$; 58% fly ash). Twenty-four specimens were subjected to accelerated corrosion in 5% NaCl under 5 V DC, while six served as controls. Archived specimen-level data were re-audited for measured mass loss, cracking, crack spacing, loads, chloride content, and failure response. Increasing corrosion reduced tension stiffening and increased the transfer length required for formation of new transverse cracks. The crack-spacing ratio (S_0/S) is interpreted as a crack-spacing-derived relative bond stress (u/u_0), rather than a direct measure of local bond strength. For Series C7, u/u_0 decreased from 0.83 at 8% mass loss to 0.17 at 24% mass loss; for Series C3, $u/u_0 \approx 0.23$ at 14.5% mass loss. At 27% mass loss, specimen C7-9C developed no new transverse crack before steel yielding, precluding determination of u/u_0 . HVFA specimens exhibited no corrosion-induced longitudinal splitting, although localized corrosion and spalling occurred. Comparison with a published bond-reduction relationship showed reasonable agreement for Series C7 but generally lower relative bond stress for the lower- w/cm C3 and C4 series. The results demonstrate that corrosion level, localized deterioration, concrete system, and cracking response should be considered collectively and highlight limitations in applying a single mass-loss-based bond-reduction relationship across different concrete systems and corrosion ranges.

Author keywords: Bond deterioration; corrosion; tension stiffening; crack spacing; fly ash; high-volume fly ash (HVFA); supplementary cementitious materials (SCMs)

Introduction

The bond between reinforcing steel and concrete governs not only local anchorage capacity but also the distribution of deformations and cracks across a member under load. In the presence of corrosion, this bond deteriorates progressively, and the structural consequences extend well beyond the conventional concern with rebar cross-section loss. As bond weakens, the tension-stiffening contribution of concrete between cracks diminishes, crack spacings widen, crack widths increase at a given load level, and ultimately the load-carrying capacity of the member is compromised.^{1,2}

The majority of experimental studies on corrosion-induced bond deterioration have employed pullout-type specimens, which provide valuable local bond-slip information but do not reproduce the distributed cracking and tension-stiffening response of a reinforced-concrete tension zone. Systematic reviews report substantial dispersion among corrosion-bond models because of differences in specimen configuration, corrosion protocol, confinement, and damage metric.³ Recent direct-tension testing of 28 reinforced-concrete specimens further showed that advanced corrosion markedly reduces tension stiffening and tensile stress transfer between cracks and that current crack-spacing and crack-width models remain inadequate for corroded members.⁴ The present dataset is therefore revisited not as a new experimental campaign, but because it contains member-level direct-tension observations extending from the uncorroded state to severe measured corrosion, together with crack mapping, chloride profiles, localized corrosion observations, and a field-referenced bridge concrete.

The pullout program that forms Part I of the broader research program was originally reported in Amleh⁵: 192

*Corresponding Author: Lamya Amleh.
Email: lamleh@torontomu.ca
Department of Civil Engineering, Toronto Metropolitan University,
Toronto, Ontario, Canada

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specimens across six cementitious systems and four cover thicknesses, including empirical relationships for bond deterioration with corrosion. A separate Part I journal manuscript is being developed from those thesis data; because it is not yet a published source, the present paper cites Amleh⁵ for the underlying pullout data. The present article addresses the companion direct-tension program: 30 specimens across three concrete types representing member-level response to the same corrosion process. Together, the archived datasets span local interface behavior (pullout) and distributed member response (direct tension), with chloride profiling and field context from the Dickson Bridge in Montreal.

The three concrete types selected reflect distinct practical scenarios. The Dickson Bridge concrete (NPC, $w/c = 0.52$) represents a field-referenced conventional bridge-deck concrete with documented chloride-induced deterioration. The bridge was constructed in 1959 in the east end of Montreal and had been exposed to a corrosive service environment for approximately 35 years when the detailed field investigation was undertaken. Its superstructure was approximately 366 m long and 27 m wide and comprised a 150 mm heavily reinforced concrete deck supported primarily on continuous reinforced-concrete beams, with the central spans over railway tracks supported on steel plate girders. Series C7 reproduced the original deck concrete using information from contract drawings and specifications, corroborated by petrographic examination. The NPC 0.32 concrete provides a higher-quality conventional reference, while the Sundance HVFA concrete represents a low-permeability SCM system used to examine how delayed corrosion initiation may coexist with different post-initiation member-level deterioration.

Legacy dataset and contribution of the present study. The experimental program was originally conducted as part of Amleh,⁵ and the underlying tests are not claimed here as newly generated data. The new contribution of this manuscript is the consolidated and critically re-audited analysis of the direct-tension dataset in the context of current corrosion research. Specifically, the paper (i) reconstructs and verifies specimen-level geometry, cracking, mass-loss, chloride, and load-response data from the archived records; (ii) examines the transition from local corrosion damage to member-level loss of tension stiffening and composite force transfer over corrosion levels extending to approximately 27% measured mass loss; (iii) retains the original relative-bond formulation, $u/u_0 = S_0/S$, while explicitly defining u/u_0 as a crack-spacing-derived relative bond stress rather than a direct local bond measurement; (iv) distinguishes corrosion-initiation resistance from post-initiation structural interaction through comparison of Portland-cement and high-volume fly-ash concretes; and (v) places the laboratory observations alongside detailed Dickson Bridge field measurements; and (vi) tests the transferability of a contemporary direct-tension bond-reduction relationship against the re-audited specimen-level predeterminations. The direct-tension data complement the predominantly local

bond-test literature and the recent renewed interest in tension stiffening of corroded members.^{3,4}

The objectives are (i) to characterize load-elongation response across the three concrete types and corrosion stages; (ii) to quantify the progressive loss of tension stiffening; (iii) to document transverse-crack spacing and crack-width evolution; (iv) to use crack spacing as a normalized member-level indicator of bond-mediated force transfer; (v) to compare the laboratory Series C7 results with the Dickson Bridge field observations while explicitly recognizing differences between accelerated and natural corrosion; and (vi) to assess the transferability of a contemporary direct-tension bond-reduction relationship across the three concrete systems.

Background

Tension stiffening is the contribution of the concrete between cracks to the overall stiffness of a reinforced concrete tension member, which arises from the bond between steel and concrete that allows partial force transfer even at cracked sections. The fundamental bond-corrosion relationship in pullout specimens has been characterised experimentally across a range of cementitious systems and corrosion levels, establishing that bond strength initially increases at low mass loss and then declines sharply as cover cracking develops.^{6–8} In an uncorroded member, this contribution is well characterized by models based on the fundamental bond-slip relationship.^{9,10} As corrosion develops, several interacting mechanisms degrade this contribution.

First, the expansive corrosion products generate bursting pressure that propagates longitudinal splitting cracks along the bar axis.^{11,12} These cracks reduce the concrete area surrounding the bar that can develop hoop tensile stresses, directly reducing bond resistance. Second, the corrosion products form a weak, friable layer at the steel–concrete interface that disrupts adhesion and reduces the friction bond.⁷ Third, the loss of bar rib profile, through corrosion pit development and rib degradation, reduces the primary mechanical interlocking mechanism for deformed bars. The net effect is that as corrosion progresses, the stabilized transverse-crack spacing S increases because the bond over a given length of bar is insufficient to introduce enough force into the concrete to form a new crack. This increase in crack spacing is both a symptom and a measure of bond deterioration.^{13,14}

Corrosion-induced deterioration of tension stiffening has received growing attention in recent literature. The steel–concrete bond mechanism under corrosion has been modeled analytically and numerically, with finite-element approaches demonstrating that the expansive pressure distribution is the primary driver of bond loss.^{15–17} Kunawisarut et al.¹ tested directly loaded corroded RC members and proposed an exponential bond-reduction relationship derived from crack-spacing-based residual bond capacity and corrosion mass loss. Syll and Kanakubo³ reviewed 84 published

bond-corrosion studies and noted that the tension specimen approach, while providing the most structurally relevant data, has been used in only a small fraction of studies. A further overview of the corrosion-bond loss relationship confirms that bond failure is a primary structural consequence of corrosion, with the transition from local adhesion loss to global member degradation governed by cover cracking, bar lug profile loss, and confining reinforcement,¹⁸ with most relying on pullout geometry. Recent work has demonstrated that current tension-stiffening models are insufficient for corroded members, underestimating stiffness loss and overpredicting crack spacing at intermediate corrosion levels.^{1,4,19} Even for uncorroded RC ties, tension-stiffening relationships exhibit strong dependence on concrete strength class, reinforcement ratio, and shrinkage history²⁰; these sensitivities are substantially compounded by corrosion. Experimental programs across multiple concrete grades have confirmed that the bond stress-slip response is nonlinear and that higher-grade concretes show better corrosion tolerance, with the initial bond-strength increase at low corrosion levels followed by a non-linear decline.^{21,22}

For SCM concretes specifically, the tension-stiffening literature is sparse.²³ In the broader bond-corrosion literature, fly ash concretes and alkali-activated slag concretes have been shown to delay corrosion cracking and maintain higher residual bond strength than OPC concretes at equivalent low-to-moderate corrosion levels, while showing comparable or lower bond strength once cracking occurs.²⁴ Bilodeau and Malhotra²⁵ noted that high-volume fly ash concretes exhibit slightly lower direct tensile strength than NPC concretes of equivalent compressive strength, a fact with direct implications for cracking load and initial crack spacing in tension members. The present study contributes a rare member-level direct-tension dataset on tension-stiffening deterioration in high-volume fly ash concrete under accelerated corrosion conditions.

Experimental Program

Materials and mixture proportions

Three concrete mixtures were used in this study, selected to span the range of cementitious systems relevant to Canadian infrastructure:

(1) NPC ($w/c = 0.52$) “Dickson Bridge Concrete” (Series C7): Normal Portland cement (CSA Type 10), limestone coarse aggregate ($1,020 \text{ kg/m}^3$), St. Gabriel de Brandon sand (720 kg/m^3), water (210 kg/m^3). This mixture replicates the concrete used in the Dickson Bridge deck as built in 1959, verified through contract drawings and petrographic studies. Compressive strength: 42.0 MPa (365-day); effective tensile strength inferred from the tension-member cracking response: 1.92 MPa; splitting tensile strength: 2.48 MPa.

(2) NPC ($w/c = 0.32$) “Normal Cement Concrete” (Series C3): As used for the companion pullout specimens.

Compressive strength: 60.2 MPa (365-day); effective tensile strength inferred from the tension-member cracking response: 2.84 MPa; splitting tensile strength: 3.23 MPa.

(3) Sundance fly ash concrete ($w/cm = 0.32$) “Sundance Concrete” (Series C4): Sundance high-calcium Class F fly ash ($\text{CaO} = 12.5\%$, Blaine = $408 \text{ m}^2/\text{kg}$) at 58% replacement of total cementitious material mass. Compressive strength: 66.6 MPa (365-day); effective tensile strength inferred from the tension-member cracking response: 2.59 MPa; splitting tensile strength: 3.32 MPa.

Reinforcing steel: 20M deformed bars ($d_b = 19.5 \text{ mm}$, yield strength 432 MPa) conforming to CSA G30.14-M83/ASTM A615-72. Bare bar load-elongation curve was recorded for each series.

Here, effective tensile strength denotes the value inferred in the archived analysis from the tension-member first-cracking response; it was not measured in a separate concrete tensile-strength test. These values are used only as comparative descriptors of the cracking response among the three series.

Specimen geometry and preparation

The direct-tension specimens were cylindrical, 1000 mm long and 120 mm in diameter, with a single centrally placed 20M deformed bar ($d_b = 19.5 \text{ mm}$). The bar extended approximately 150 mm beyond each end of the concrete cylinder for gripping. The nominal radial concrete cover was $(120 - 19.5)/2 = 50.25 \text{ mm}$, reported herein as approximately 50 mm ($c/d_b \approx 2.58$). The bar extensions and approximately 20 mm of bar within each specimen end were protected from accelerated corrosion using epoxy coating and tape. A $100 \text{ mm} \times 100 \text{ mm}$ surface grid was marked for crack observations. Six specimens (two per concrete mixture) were uncorroded controls, and 24 specimens were subjected to accelerated corrosion. The specimen concrete length was 1000 mm, the cylindrical diameter was 120 mm, the centrally placed 20M bar ($d_b = 19.5 \text{ mm}$) extended approximately 150 mm beyond each end, the nominal radial cover was approximately 50 mm, and elongation was measured over a 1010 mm extensometer gauge length as shown in Fig. 1. Specimens were moist-cured at $23 \pm 2^\circ\text{C}$ for 91 days, matching the pullout specimen curing protocol, to ensure completion of secondary pozzolanic reactions in the Sundance fly ash mixture.²⁵

Accelerated corrosion

The accelerated-corrosion program used an electrolytic solution of 5% NaCl by weight of water. The tension specimens were immersed in the solution maintained approximately 50 mm above the top of the concrete specimens, and the solution was replaced weekly to limit changes in NaCl concentration and pH. The embedded reinforcing bar served as the anode, and a steel bar/mesh assembly served as the cathode under a constant applied potential of 5 V DC. Current was recorded for each specimen approximately every 48 h using a digital multimeter after the reading

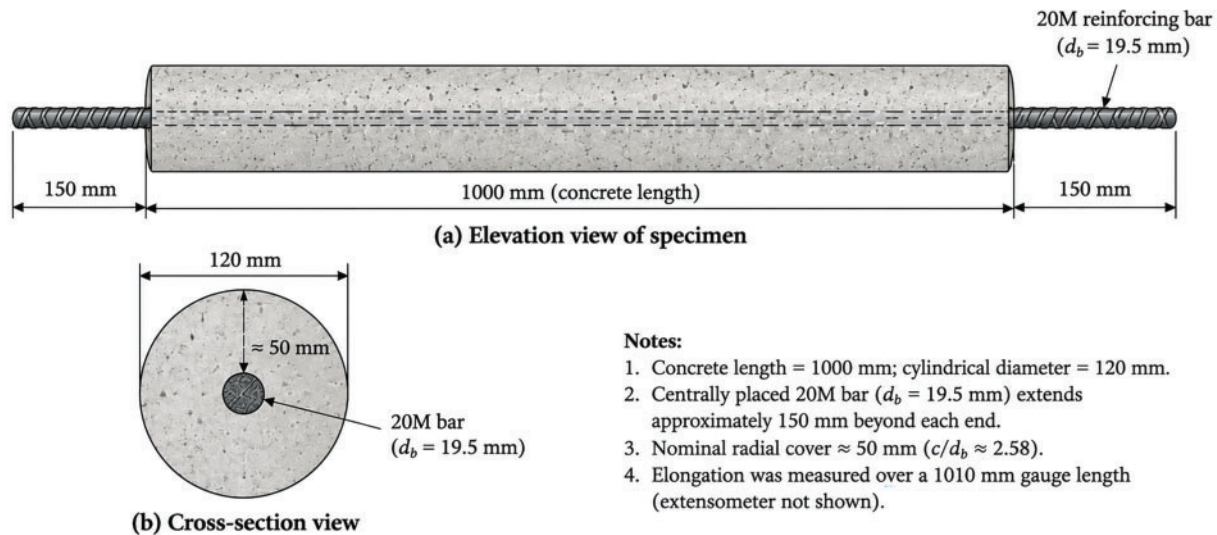


Figure 1. Schematic of the tension-specimen geometry

stabilized for approximately 5–10 min. Half-cell potentials were also monitored periodically. For the tension specimens, corrosion-rate measurements were made at three locations after the target corrosion stage was reached. The original accelerated program was designed to reach severe corrosion over approximately 15–20 weeks. Cumulative charge histories and specimen-specific Faraday-law estimates were not retained in a form that can be reconstructed reliably and are therefore not reported retrospectively. Corrosion stages were defined using multiple observations, including current response, longitudinal crack development, measured mass loss, cross-sectional loss, and chloride content.

A further limitation concerns corrosion-product chemistry. Accelerated impressed-current corrosion can produce corrosion-product assemblages and spatial distributions that differ from those formed during long-term natural chloride exposure. Because different iron oxides, oxyhydroxides, and hydroxides have substantially different volumetric expansion relative to the consumed steel, the relationship among gravimetric mass loss, interfacial pressure, cracking, and bond deterioration is not unique. Molina et al.²⁶ demonstrated the strong sensitivity of corrosion-induced expansion to assumed corrosion-product composition. Corrosion products were not chemically characterized in the present program; consequently, differences in product chemistry constitute an additional uncertainty when extrapolating the accelerated-corrosion observations to naturally corroded bridge members.

Direct tension testing

Direct-tension tests were conducted in the Structures Laboratory at McGill University using a servo-hydraulic MTS machine with a 1000 kN tension capacity. The specimens were centered to achieve axial tension and loaded incrementally under displacement control through grips attached to the reinforcing bar at both ends. Load, displacement, and strain data were acquired every 5 s during loading. An

external MTS extensometer was clamped to the reinforcing bar immediately outside the two concrete end faces; thus, the 1010 mm gauge length measured the elongation across the 1000 mm concrete member plus the short exposed bar segments between the concrete faces and extensometer clamps, not the full approximately 1300 mm bar length including the gripping extensions. Loading was paused at key stages for crack mapping and photography. Cracks were traced as soon as visible, the corresponding load was marked, and crack widths were measured with a crack-width comparator; a 10x hand microscope was used where necessary to follow crack propagation.

A concrete shrinkage pre-strain correction of approximately (0.3×10^{-3}) , based on Bischoff,²⁷ was applied to the control-specimen elongation measurements. No corresponding correction was applied to the corroded specimens because they had remained continuously immersed in NaCl solution and showed no shrinkage pre-strain.

Chloride profiles

After tension testing, chloride ion content used for the specimen profiles was determined at nominal depths of approximately 7, 25, and 50 mm from the concrete surface using the specific ion probe method (ASTM C1152-90).²⁸ The deepest measurement level corresponds approximately to the reinforcing-steel level in the 120 mm diameter specimens.

Results

Corrosion stages and bar profile

The experimental program is summarized in Table 1. Table 2 summarizes selected representative corrosion and cracking observations. The complete 30-specimen dataset is in Supplementary Table S1. Table 2 displays selected stages;

Table 1. Experimental program overview, three tension specimen series, 30 specimens total

Series	Concrete type	w/cm	f'_c (MPa)	f_t (MPa)	No. of specimens	Corrosion stages
C7	NPC–Dickson Bridge deck ($w/c = 0.52$)	0.52	42.0	1.92	10	1–8 + 2 control
C3	NPC ($w/c = 0.32$)	0.32	60.2	2.84	10	1–7 + 2 control
C4	Sundance fly ash, 58% FA ($w/cm = 0.32$)	0.32	66.6	2.59	10	1–8 + 2 control

Note: f'_c = 365-day compressive strength (MPa); f_t = effective tensile strength inferred from the tension-member cracking response (MPa). All specimens: 20M deformed bar ($d_b = 19.5$ mm), 120 mm specimen diameter, approximately 50 mm radial concrete cover ($c/d_b \approx 2.58$), accelerated corrosion in 5% NaCl at 5 V DC, 91-day moist cure. FA = fly ash.

Table 2. Selected representative corrosion and cracking observations from the tension program

Specimen	Stage	Long. cracks	Avg. long. width (mm)	ML (%)	No. trans. cracks	Avg. spacing (mm)	1st crack load (kN)	Yield load (kN)
C7-1C (ctrl)	0	0	0	0	11	83	18	127
C7-4C	1	1	1.2	8	9	100	23	116
C7-5C	2	1	1.5	11	7	125	24	102.5
C7-9C	8	2	4.0	27	0	N/A	N/A	45
C3-1C (ctrl)	0	0	0	0	12	77	19	127
C3-5C	1	0	0	1.5	12	77	34	125
C3-6C	5	3	2.0	7.7	4	200	31	115
C3-4C	7	3	3.0	14.5	2	333	37	83
C4-2C (ctrl)	0	0	0	0	10	91	21	128
C4-10C	1	0	0	0.9	11	83	31	127
C4-4C	6	0	0	2.5	7	125	36	124.5
C4-8C	8	0	0	4.5	6	143	30	90

Note: Control-specimen first-cracking values were adjusted for concrete shrinkage pre-strain. For C7-9C, no transverse cracks formed before steel yielding. ML = rebar mass loss (%). Control specimens: 2 per series (one shown as representative). Long. = longitudinal.

the complete specimen-by-specimen archival dataset is provided in Supplementary Table S1. Stage numbers denote the archived corrosion progression within each mixture and are not assumed to represent identical measured mass loss across mixtures. Cross-series comparisons are therefore made using measured mass loss and observed response rather than nominal stage number. A representative bar profile examination (Fig. 2) revealed localized corrosion concentrated at discrete regions along the bar length, with deep corrosion pits developing preferentially at rib roots. This localized character meant that some bar ribs remained in good condition adjacent to heavily corroded zones, accounting for the residual concrete contribution observed even in severely corroded specimens. During the early corrosion stages, visual examination indicated localized attack rather than a regularly spaced sequence of pits at every rib root. Some rib roots showed preferential attack while adjacent ribs remained comparatively intact, and rust accumulation was correspondingly non-uniform along the bar. The original program documented pit depth, bar-diameter change, rib degradation, and general corrosion morphology after testing but did not record a statistically complete pit-to-pit spacing distribution or rust thickness at each rib root. The

revised manuscript therefore reports the observed localization without assigning an unsupported characteristic pitting frequency or spacing.

The Sundance fly ash specimens (Series C4) were distinctive in that none developed visible longitudinal splitting cracks due to corrosion (Table 2). Nevertheless, localized corrosion and local spalling were observed. Restricted migration of corrosion products within the denser fly-ash matrix is one possible explanation for this localization, but corrosion-product transport and interfacial pressure were not measured directly. The permeability-related localization mechanism is therefore treated only as a working hypothesis.

Load-elongation responses

Figs. 3 through 5 present the load-elongation responses for representative specimens from each series alongside the corresponding control specimen and the bare bar response. The following features are common to all three concrete types and consistent with the standard load-elongation model for reinforced tension members:

(i) An initial uncracked elastic phase, in which the load-elongation response is stiffer than the bare bar due to the concrete contribution.

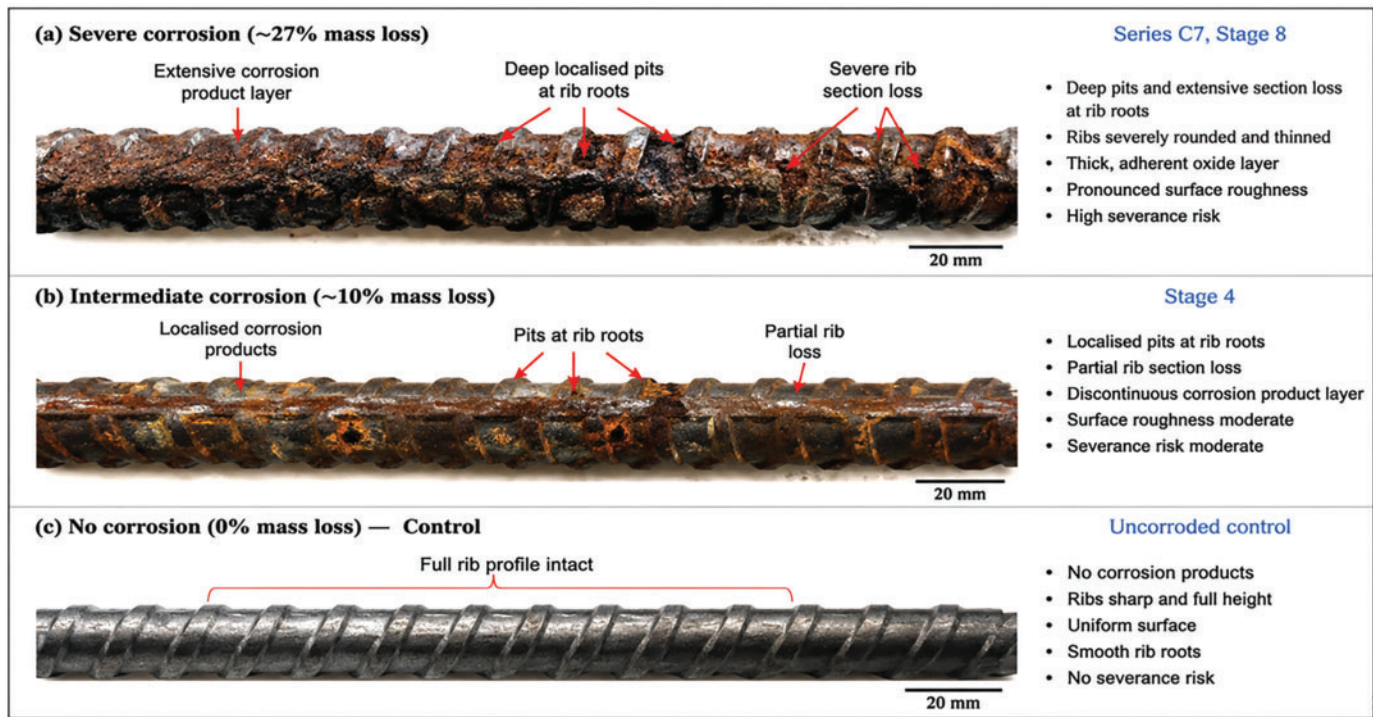


Figure 2. Representative post-test bar profiles: (a) heavily corroded Series C7 bar at Stage 8 showing localized pits and severe rib degradation; (b) intermediate-corrosion bar showing selective rib loss; and (c) uncorroded control bar

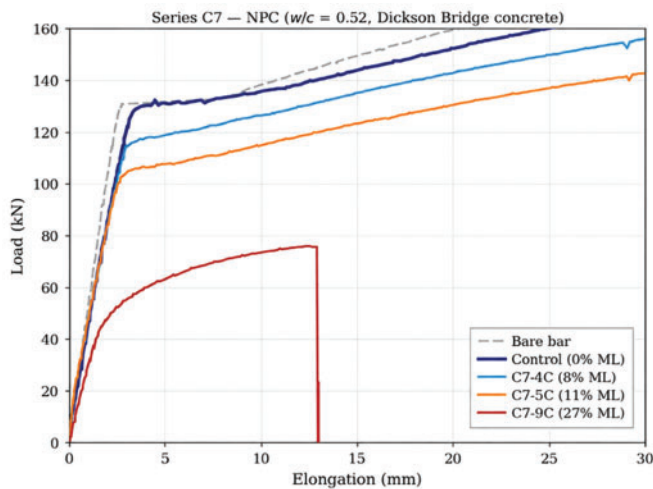


Figure 3. Load-elongation responses: Series C7 (NPC, $w/c = 0.52$, Dickson Bridge concrete). Control specimen C7-2C, corroded specimens C7-4C (8% ML), C7-5C (11% ML), and C7-9C (27% ML), compared with the bare-bar response

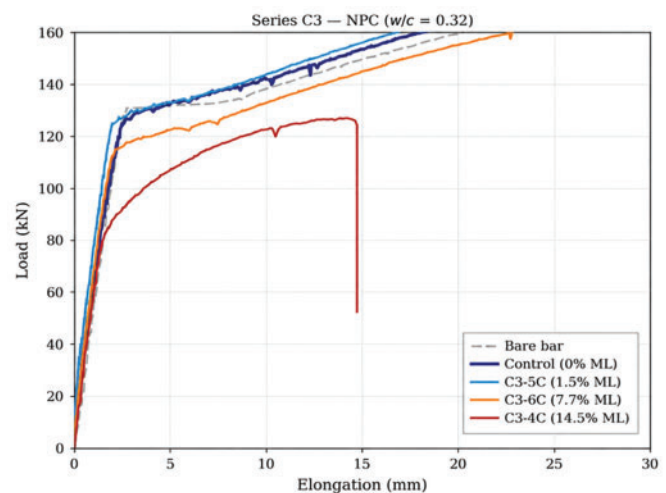


Figure 4. Load-elongation responses: Series C3 (NPC, $w/c = 0.32$). Control specimen C3-1C, C3-5C (1.5% ML), C3-6C (7.7% ML), C3-4C (14.5% ML), and bare-bar response

(ii) A cracking transition, in which the tensile strength of the concrete is exceeded and transverse cracks form. At each cracking event, the load drops slightly as the concrete contribution at that section is lost.

(iii) A stabilized cracking phase, in which the load-elongation curve becomes parallel to the bare bar response. Following cracking, the specimen response remains above

and to the left of the bare-bar response because of the concrete contribution between cracks. This tension-stiffening contribution may be expressed either as an additional load at a given elongation or, as adopted in Section 4.3, as a reduction in elongation at a given load level.

(iv) Onset of yielding of the steel bar. The yield loads reported in Tables 2 and S1 denote the identified onset of steel yielding; the recorded load-elongation response may continue beyond this point because of post-yield steel strain

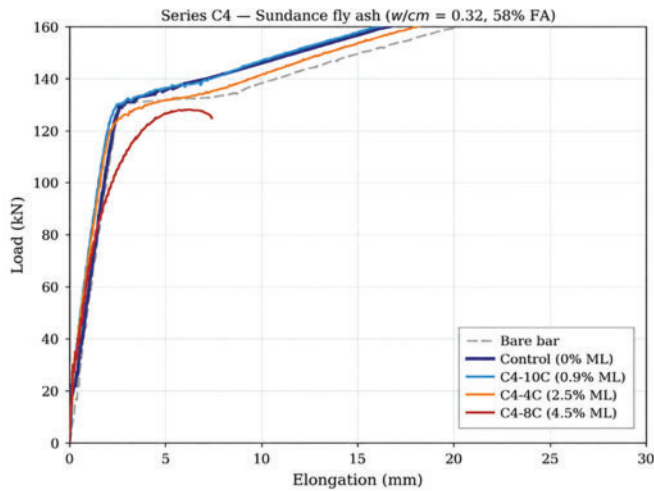


Figure 5. Load-elongation responses: Series C4 (Sundance fly ash, $w/cm = 0.32$). Control specimen C4-2C, C4-10C (0.9% ML), C4-4C (2.5% ML), C4-8C (4.5% ML), and bare-bar response

hardening. The terminal vertical drop in a plotted curve represents unloading or rupture at the end of the recorded test and should not be interpreted as the yield load.

As corrosion level increases, the load-elongation curve shifts progressively towards the bare bar response, indicating progressive loss of tension stiffening. This pattern is consistent with recent direct tension tests on corroded RC members using Digital Image Correlation, which similarly found that corrosion significantly reduces tension stiffening, with larger bar sizes exacerbating the effect.⁴ Syll et al.²⁷ similarly demonstrated that bond degradation due to corrosion-induced longitudinal cracking reduces the number of transverse cracks and diminishes tension stiffening, with the effect most pronounced at deformations below 1.5 mm. The most severely corroded specimens (C7-9C at 27% ML, C3-4C at 14.5% ML, C4-8C at 4.5% ML) show virtually no concrete contribution in the load-elongation plot; their response is essentially that of the bare bar with reduced yield load due to cross-section loss.

For severely pitted specimens, a yield load calculated from the average residual area and the uncorroded-bar yield stress is not expected to reproduce the measured onset of yielding. Average mass loss does not represent the minimum load-carrying cross-section, and localized pitting may govern yielding before the average residual section reaches the nominal uncorroded yield stress.

At very low corrosion levels (Stage 1), a slight increase in the cracking load relative to control specimens was observed in all three series. This is attributable to the absence of concrete shrinkage pre-strain in the continuously immersed corroded specimens,²⁷ rather than a genuine improvement in tensile strength or bond.

Tension stiffening quantification

The tension-stiffening response was first evaluated from the horizontal separation between the specimen and bare-bar load-elongation curves at equivalent load levels. This separation represents the reduction in average elongation associated with the contribution of the surrounding concrete.

To account for corrosion-induced loss of reinforcement cross-sectional area, the measured responses were also expressed as derived nominal stress-strain relationships. For each specimen, the equivalent residual steel area was calculated as $A_{ms} = A_{so} (1 - ML/100)$, where $A_{so} = 300 \text{ mm}^2$ is the nominal cross-sectional area of the 20M reinforcing bar, and ML is the verified gravimetric mass loss in percent.

The derived nominal stress was calculated as $\sigma = P/A_{ms}$, where P is the measured tensile load. Average strain was calculated as $\varepsilon = \Delta L/1010$, using the 1010 mm extensometer gauge length.

The resulting relationships for representative specimens from Series C7, C3, and C4 are presented in Figs. 6–8, respectively. Normalizing the measured load by the reduced residual steel area increases the derived nominal stress for a given load, particularly at higher mass-loss levels. Consequently, increased separation from the bare-bar response in this representation should not be interpreted as a physical increase in the concrete contribution with corrosion. It reflects the combined influence of tension stiffening and normalization by the reduced steel area.

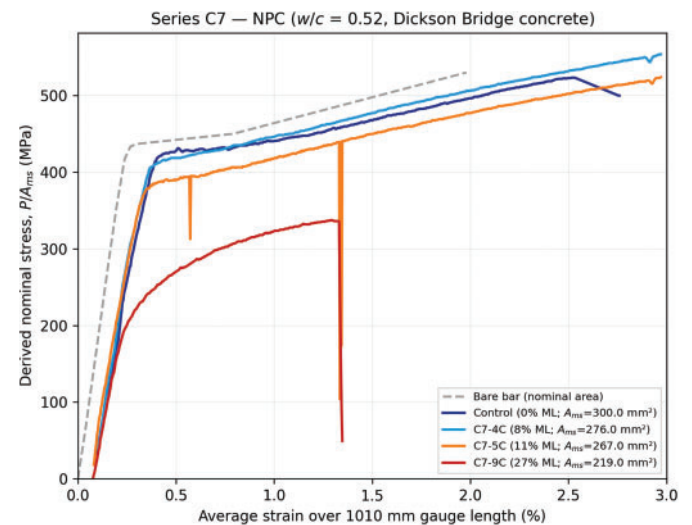


Figure 6. Derived nominal stress-strain relationships for representative Series C7 specimens. Derived nominal stress was calculated using the specimen-specific equivalent residual steel area, A_{ms} , determined from the verified mass loss. Average strain was calculated over the 1010 mm extensometer gauge length

The two representations therefore provide complementary information. The load-elongation curves in Figs. 3–5 describe the measured member response, including changes in load capacity and deformation capacity. The derived nominal stress-strain relationships in Figs. 6–8 facilitate

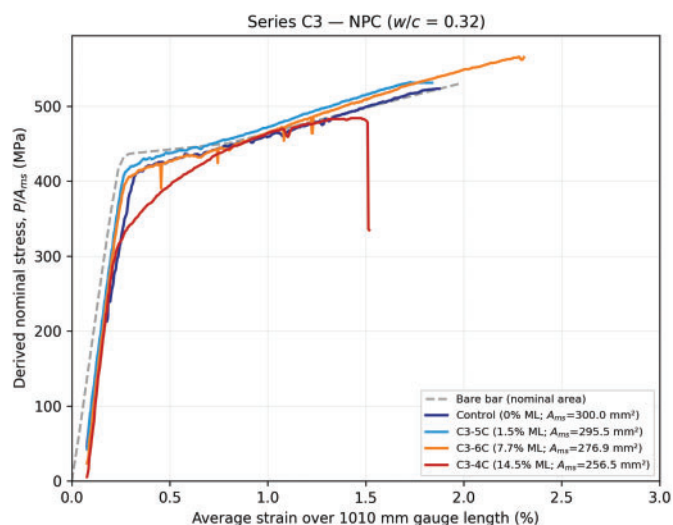


Figure 7. Derived nominal stress–strain relationships for representative Series C3 specimens. Derived nominal stress was calculated using the specimen-specific equivalent residual steel area, A_{ms} , determined from the verified mass loss. Average strain was calculated over the 1010 mm extensometer gauge length

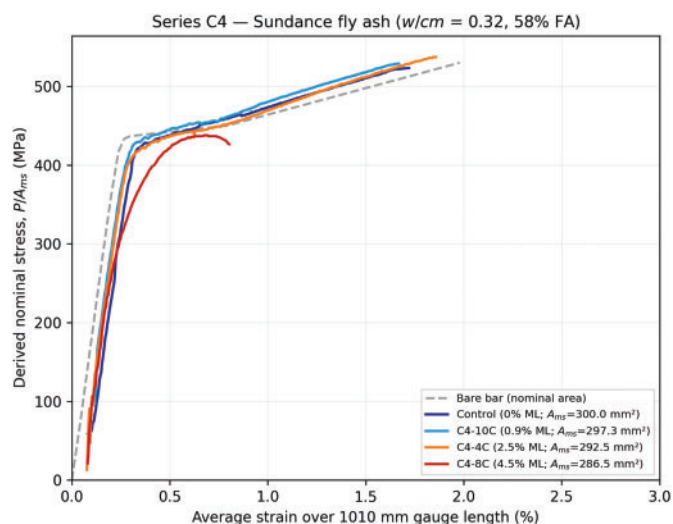


Figure 8. Derived nominal stress–strain relationships for representative Series C4 specimens. Derived nominal stress was calculated using the specimen-specific equivalent residual steel area, A_{ms} , determined from the verified mass loss. Average strain was calculated over the 1010 mm extensometer gauge length

comparison after accounting for the estimated corrosion-induced reduction in steel area; they do not independently separate the force carried by the concrete.

Cracking behavior

NPC Concrete ($w/c = 0.52$) Dickson Bridge Series: Control specimens C7-1C and C7-2C developed 11 transverse cracks

with an average stabilized spacing of 83 mm. With an approximately 50 mm radial cover, twice the cover is approximately 100 mm. The observed 83 mm average spacing is therefore of the same order as, and somewhat below, this simple cover-based characteristic length; it should not be interpreted as a strict theoretical minimum because stabilized crack spacing also depends on bond-transfer length, concrete tensile strength, bar geometry, and the distribution of cracks along the member. The first cracking load (adjusted for shrinkage pre-strain) was approximately 18–19 kN.

With increasing corrosion, the number of transverse cracks decreased, and their spacing increased. At the intermediate C7 condition represented by C7-8C at 16% measured mass loss, six transverse cracks formed, with an average stabilized spacing of 143 mm. At C7-9C (27% measured mass loss), no new transverse crack appeared before yielding of the steel bar. Accordingly, a finite stabilized crack spacing and the crack-spacing-derived relative bond stress u/u_0 cannot be calculated for this terminal specimen. Its response is discussed qualitatively as severe loss of bond-mediated force transfer and tension stiffening. The maximum crack width increased substantially with corrosion level at a given steel stress (Table 3).

NPC Concrete ($w/c = 0.32$): Control specimens C3-1C and C3-2C developed 12 transverse cracks at an average spacing of 77 mm, reflecting the higher concrete quality ($f'_c = 60.2$ MPa vs. 42.0 MPa for Series C7) and more effective initial bond-mediated force transfer. The first cracking load was approximately 34 kN, substantially higher than for Series C7, consistent with the higher splitting tensile strength.

The Series C3 specimens showed the same systematic increase in crack spacing with corrosion level. At Stage 7 (C3-4C, 14.5% mass loss), only 2 transverse cracks formed. At 1.5% measured mass loss, C3-5C retained a yield load essentially equal to the control, an average transverse-crack spacing of 77 mm, and $u/u_0 = 1.00$, with no major longitudinal splitting. This is consistent with the substantial approximately 50 mm radial cover and relatively dense NPC $w/c = 0.32$ concrete, for which this early measured corrosion state had apparently not produced visible cover splitting or a measurable increase in stabilized transverse-crack spacing. Because corrosion-product pressure was not measured directly, this observation does not establish a quantitative cover-cracking threshold. The appearance of two longitudinal cracks in the separate C3-8C specimen at 2.2% measured mass loss is qualitatively consistent with the threshold-like nature of corrosion-induced splitting once expansive stresses become sufficient relative to the tensile resistance and confinement of the surrounding concrete. Because C3-5C and C3-8C are different specimens, the interval between 1.5% and 2.2% ML should not be interpreted as a measured transition threshold for an individual member.

Sundance Fly Ash Concrete ($w/cm = 0.32$): Control specimens C4-1C and C4-2C developed 10 transverse cracks at an average spacing of 91 mm, yielding at approximately 127 kN (steel stress 423 MPa). The first cracking load was 22 kN, lower than for C3 (34 kN) despite the higher compressive strength, consistent with the lower effective tensile strength

Table 3. Maximum transverse crack width (mm) at selected steel-stress levels for representative specimens

Specimen	100 MPa	150 MPa	200 MPa	250 MPa	300 MPa	350 MPa	At yield
C7-1C (control)	0.10	0.12	0.18	0.22	0.28	0.35	0.40
C7-4C (8% ML)	0.20	0.35	0.55	0.70	1.00	—	—
C7-5C (11% ML)	0.20	0.40	0.70	0.80	—	—	—
C3-1C (control)	0.08	0.10	0.15	0.17	0.20	0.28	0.35
C3-5C (1.5% ML)	0.08	0.10	0.15	0.18	0.22	0.30	0.40
C3-6C (7.7% ML)	0.20	0.25	0.50	1.00	—	—	—
C4-2C (control)	0.08	0.15	0.20	0.50	0.55	—	—
C4-8C (4.5% ML)	0.15	0.20	0.23	0.33	0.43	0.50	0.55

Table 4. Crack-spacing-derived relative bond stress as a function of measured reinforcement mass loss for selected representative specimens

Series/specimen	Measured mass loss (%)	S_0 (mm)	S (mm)	$u/u_0 = S_0/S$
C7-4C	8	83	100	0.83
C7-5C	11	83	125	0.66
C7-6C	24	83	500	0.17
C7-9C	27	83	Undefined	N/A: no new transverse crack before yield
C3-5C	1.5	77	77	1.00
C3-4C	14.5	77	333	0.23
C4-10C	0.9	91	83	1.10
C4-8C	4.5	91	143	0.64

inferred from the tension-member response of the fly ash concrete (2.59 MPa vs. 2.84 MPa for C3), as reported by Bilodeau and Malhotra²⁵ for HVFA concretes.

Critically, the Series C4 specimens showed no visible longitudinal splitting cracks due to corrosion. Despite this apparently favorable surface-cracking behavior, increasing corrosion was accompanied by wider transverse-crack spacing and reduced bond-mediated force transfer. At approximately matched low corrosion levels, the crack-spacing-derived relative bond stress for C4 was slightly lower than for C3, but the available observations do not isolate fly ash as the controlling cause. The results instead indicate that visible longitudinal splitting alone is not sufficient to characterize post-initiation structural deterioration in HVFA concrete. Localized rib damage, local spalling, concrete tensile properties, and corrosion-product localization are plausible contributors; the present program did not measure these mechanisms independently.

Crack-spacing-derived relative bond stress

Following the original mechanics-based formulation, a normalized relative bond stress was inferred from the measured stabilized crack spacing:

$$u/u_0 = S_0/S \quad (1)$$

where S_0 is the average stabilized crack spacing of the uncorroded control specimens, and S is the corresponding spacing of a corroded specimen. The normalized quantity

u/u_0 is interpreted as a crack-spacing-derived indicator of relative bond-mediated force transfer and not as a direct measurement of local interfacial bond strength.

The maximum transverse crack widths recorded at selected steel-stress levels are summarized in Table 3. These measurements complement the crack-spacing analysis by illustrating the progression of localized cracking with corrosion.

The dash indicates that a crack-width value was unavailable in the archived record at the stated steel-stress level. For several corroded specimens, this reflects yielding or termination of the test before that stress level was reached. For the uncorroded C4-2C control at 350 MPa, the dash is treated as an archival data gap because the specimen yielded at approximately 423 MPa. Where an exact stress level was not recorded, the crack width was obtained by interpolation between adjacent archived observations.

The crack-spacing-derived relative bond-stress values calculated using Eq. (1) are summarized for selected representative specimens in Table 4. The specimen selections differ from those in Table 2 because the tables serve different purposes. The complete specimen-level dataset is provided in Supplementary Table S1 in the online supplementary material.

ML denotes measured reinforcement mass loss. For C7-9C, no new transverse crack formed before steel yielding; therefore, a finite stabilized crack spacing could not be established, and S and u/u_0 are undefined.

The progression of u/u_0 with measured reinforcement mass loss is presented separately for Series C7, C3, and C4 in Figs. 9–11, respectively. The symbols represent individual specimen observations. Lines connect the observations within each series to illustrate the measured response progression and do not represent fitted regressions or predictive relationships.

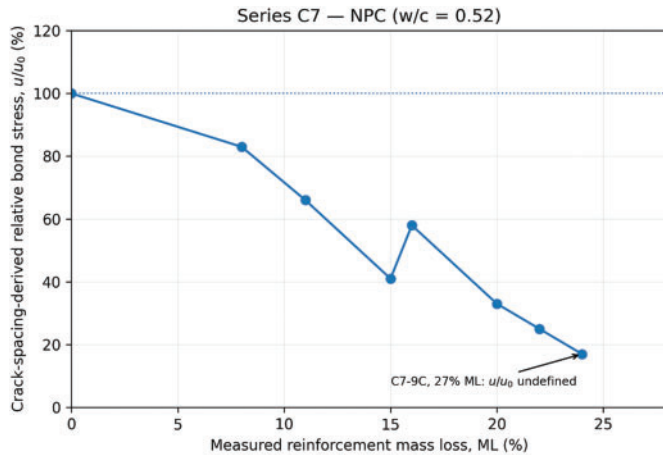


Figure 9. Crack-spacing-derived relative bond stress, $u/u_0 = S_0/S$, versus measured reinforcement mass loss for Series C7 (NPC, $w/c = 0.52$). Symbols represent verified specimen observations, and lines connect observations only to illustrate response progression. C7-9C at 27% ML is not assigned a numerical u/u_0 because no new transverse crack formed before steel yielding

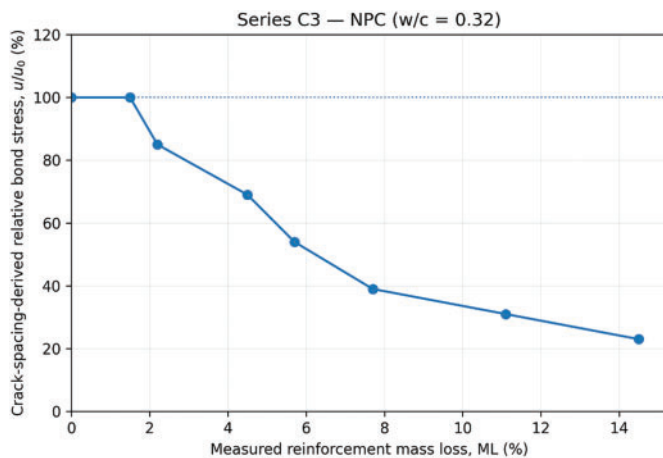


Figure 10. Crack-spacing-derived relative bond stress, $u/u_0 = S_0/S$, versus measured reinforcement mass loss for Series C3 (NPC, $w/c = 0.32$). Symbols represent verified specimen observations, and lines connect observations only to illustrate response progression

NPC ($w/c = 0.52$). As shown in Fig. 9, the verified spacing was 100 mm at 8% measured mass loss, compared with 83 mm for the control, giving $u/u_0 = 83/100 = 0.83$. The crack-spacing-derived relative bond stress decreased

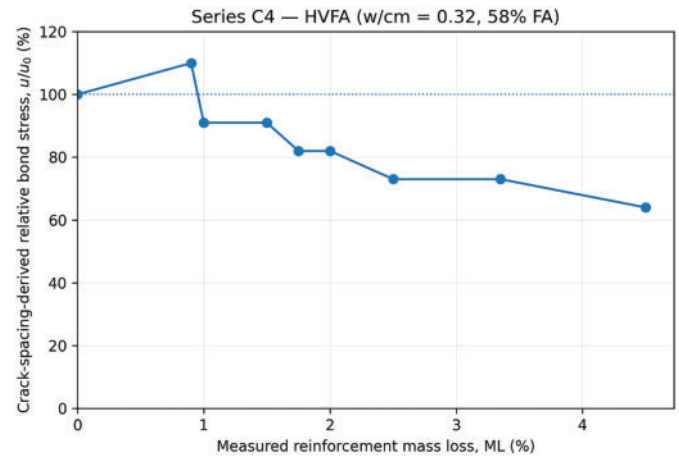


Figure 11. Crack-spacing-derived relative bond stress, $u/u_0 = S_0/S$, versus measured reinforcement mass loss for Series C4 (Sundance HVFA, $w/cm = 0.32$). The lowest-corrosion specimen exhibits an apparent value above unity; this response is interpreted cautiously because crack spacing is influenced by concrete tensile response and crack stabilization as well as bond-mediated force transfer

further with advanced corrosion, reaching approximately 0.17 at 24% mass loss. For C7-9C at 27% mass loss, no new transverse crack formed before steel yielding. A finite stabilized crack spacing could not therefore be established, and Eq. (1) could not be applied. This specimen is reported qualitatively as a terminal condition with severely impaired bond-mediated force transfer rather than being assigned a numerical residual value.

NPC ($w/c = 0.32$). Fig. 10 shows that, at 1.5% mass loss, the average crack spacing remained 77 mm, equal to that of the control, giving $u/u_0 = 1.00$. With increasing corrosion, spacing increased progressively. For C3-4C at 14.5% mass loss, $S = 333$ mm and $u/u_0 = 77/333 \approx 0.23$.

Sundance fly ash ($w/cm = 0.32$). As shown in Fig. 11, the lowest-corrosion specimen exhibited a small apparent increase in u/u_0 because its measured crack spacing was slightly smaller than that of the control. At higher corrosion levels, transverse-crack spacing increased and u/u_0 decreased, reaching approximately 0.64 at 4.5% mass loss. This response is interpreted cautiously because concrete tensile strength and crack stabilization influence crack spacing in addition to bond-mediated force transfer, and the achieved corrosion states were not exact replicates.

Chloride profiles

Fig. 12 shows the chloride ion content profiles across the cover depth for all corroded specimens with archived profile data. Nominal profile depths are approximately 7, 25, and 50 mm from the surface, with the deepest level at approximately the reinforcing-steel level (120 mm diameter specimen, 50 mm radial cover). The three panels use different horizontal scales to accommodate the markedly different

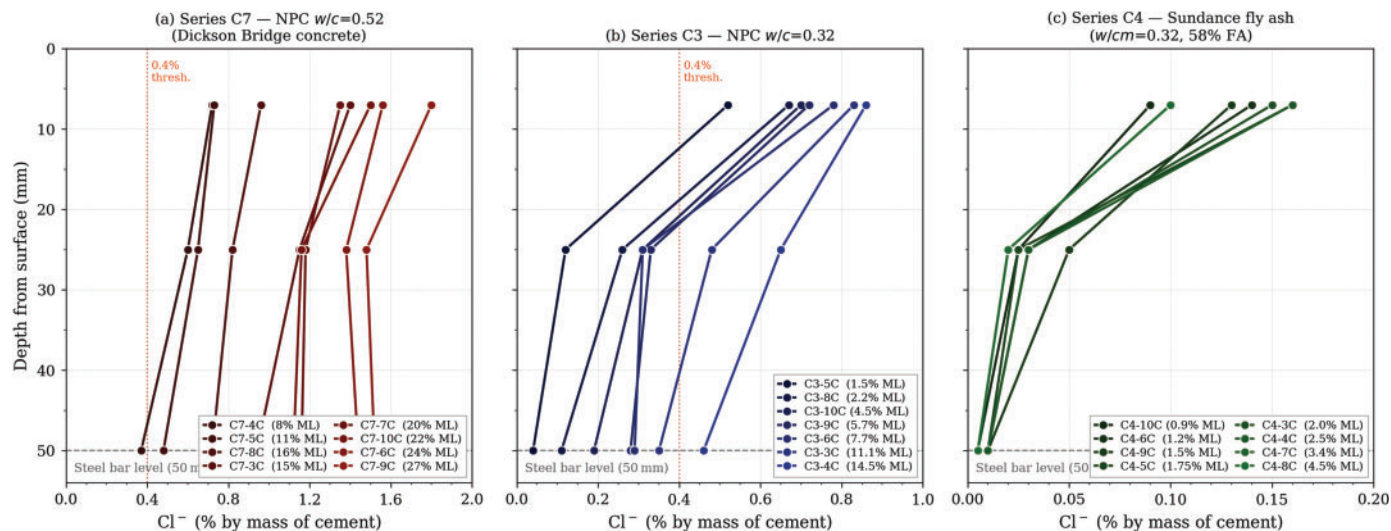


Figure 12. Chloride ion content profiles across cover depth for all corroded specimens with archived profile data: (a) Series C7; (b) Series C3; and (c) Series C4

chloride-concentration ranges; comparisons should therefore be made using the axis values rather than visual slope alone. Key observations:

(i) The Dickson Bridge concrete ($w/c = 0.52$) shows substantially higher chloride ingress at all depths, consistent with its higher permeability, with chloride contents at the rebar level reaching 0.7–1.4% by mass of cement at the higher corrosion stages. These values are broadly consistent with the range observed in the Dickson Bridge field survey (Section 4.7).

(ii) The NPC 0.32 and Sundance fly ash concretes show markedly lower chloride penetration than Series C7, but their rebar-level ranges differ materially. Series C3 shows approximately 0.04–0.46% chloride by mass of cement at the steel level, whereas Series C4 remains substantially lower, generally approximately 0–0.02%. The markedly lower C4 values are consistent with reduced chloride ingress,²⁹ although the present data do not isolate chloride ingress, corrosion morphology, and structural response as independent effects. The difference between the two low- w/cm systems was less pronounced in the companion pullout specimens, which included a wider 25–100 mm cover range than the fixed approximately 50 mm radial cover of the tension specimens.

(iii) Preferential chloride transport through cracks relative to intact concrete has been documented in the literature.¹⁹ However, the present chloride profiles (Fig. 12) were measured at single representative depths per specimen rather than as paired crack-location and inter-crack-location profiles; they therefore describe the overall chloride concentration gradient through the cover and do not directly demonstrate crack-enhanced transport in these specimens.

Correlation with Dickson bridge field data

Fig. 13 compares Series C7 laboratory observations with Dickson Bridge field measurements. The field investigation included detailed testing of four randomly selected approximately 5 m × 6 m deck areas, with many measurements collected on a 0.25 m × 0.25 m grid.^{30,31} Field points showed considerable scatter, particularly around chloride contents of approximately 0.3–0.5% and mass losses of roughly 5–20%. This is expected because measured chloride content is a local state variable at the time of sampling, whereas steel mass

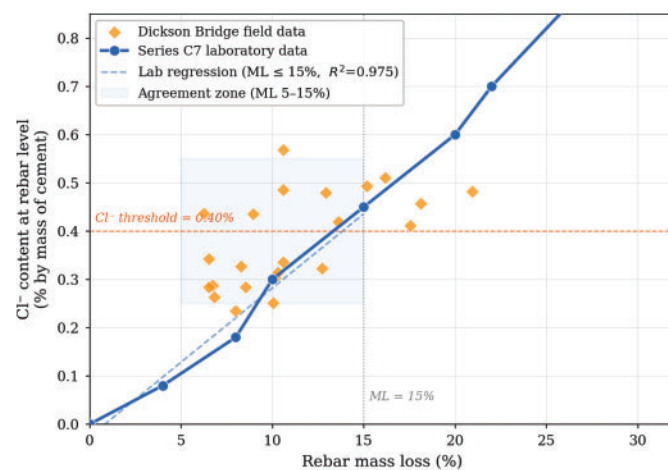


Figure 13. Correlation between Series C7 laboratory data and Dickson Bridge field survey: rebar mass loss vs. chloride content at steel level. Field data cluster between 0.3–0.5% Cl^- and 5–20% mass loss, showing qualitative consistency with laboratory C7 data for $ML < 15\%$

loss integrates the preceding exposure history. Deicing-salt use ceased after the bridge was abandoned in 1993, and subsequent snowmelt could redistribute and leach chlorides within the deck. Spatial variability in cover, cracking, permeability, moisture availability, and localized pitting also contributes to the scatter. For mass losses below approximately 15%, the average trends in the field and Series C7 datasets are qualitatively consistent; this comparison is treated as field consistency rather than formal validation of the accelerated-corrosion method.

The relatively large spread in field mass loss around approximately 0.4–0.5% chloride content is physically plausible and should not be interpreted as a unique chloride-to-mass-loss relationship. The Dickson Bridge field investigation documented substantial spatial variability in concrete quality, compaction, cover, electrical resistivity, moisture/exposure history, and localized surface condition. In addition, de-icing salt application ceased after bridge decommissioning, and subsequent seasonal melt could redistribute or leach chlorides, so the chloride concentration measured at the time of sampling need not equal the concentration that initiated or sustained the accumulated corrosion. Local pitting further decouples gravimetric average mass loss from a single chloride measurement. Future programs should therefore use replicate specimens, spatially resolved chloride and moisture/resistivity measurements, repeated electrochemical monitoring, and quantitative pit-depth/section-loss mapping, and should analyze uncertainty explicitly rather than fitting a deterministic relationship to single specimens.

Discussion

Mechanistic interpretation: six-stage cracking model

The experimental observations are consistent with a six-stage deterioration model for bond and cracking in tension members under progressive corrosion:

Stage 1: Initial corrosion products: A thin layer of iron oxide corrosion products forms at the bar surface, occupying a substantially larger volume than the parent steel, typically two to seven times,²⁵ and increasing hoop stress and frictional resistance. The crack-spacing-derived relative bond stress may show a small apparent increase at very low corrosion; C4-10C gave $u/u_0 = 1.10$ at 0.9% measured mass loss. This is treated as an apparent low-corrosion enhancement rather than a direct measurement of increased local bond strength because crack spacing is also influenced by concrete tensile response and crack stabilization.

Stage 2: Expansive pressure buildup: Further corrosion product accumulation generates sufficient bursting pressure to create tensile stresses exceeding the concrete tensile strength along the bar axis. Finite-element modelling has confirmed that this pressure distribution is highly non-uniform, with stress concentrations at rib roots.¹⁵ Longitudinal splitting cracks initiate.³² Experimental pullout

data confirm that bond strength peaks in the range of 0.5–1.5% mass loss before declining.^{6,7}

Stage 3: Cover cracking and bond-loss onset: Longitudinal crack propagation disrupts the surrounding concrete ring and reduces effective confinement of the bar. Adhesion at the steel-concrete interface is progressively lost, and bond-mediated force transfer begins to decline measurably, consistent with analytical interface modelling.¹⁶

Stage 4: Force transfer to ribs: Load transfer shifts from adhesion to mechanical interlocking at the ribs. Internal cracks form adjacent to main transverse cracks,¹³ redistributing force transfer.

Stage 5: Internal cracking proliferation: Progressive internal cracking around the bar reduces the rate of force transfer, increasing the stabilized transverse-crack spacing S . Fewer transverse cracks form during loading, and those that form are more widely spaced. This bond-spacing relationship has been confirmed experimentally in corroded RC members under direct loading.^{1,33}

Stage 6: Near-complete bond loss: Rib profile degradation, heavy corrosion product layer, and extensive longitudinal cracking combine to reduce bond transfer capacity to negligible levels. At the limiting condition, no new transverse cracks may form before steel yielding, as observed for C7-9C at 27% measured mass loss. Other severely corroded specimens, such as C3-4C at Stage 7, still formed a small number of widely spaced transverse cracks (two at 333 mm spacing) but with strongly reduced tension stiffening, approaching the bare-bar response. The state-of-the-art review by Zhao and Lin²² confirms this as the terminal stage of the corrosion-bond deterioration process, and Apostolopoulos and Koulouris¹⁸ document that rib profile loss is a primary driver of near-complete bond loss at high corrosion levels.

The mass-loss level at which these response stages become apparent is not universal. It depends on cover, concrete tensile properties, confinement, corrosion localization, corrosion-product transport, and exposure history. Thinner cover and lower tensile resistance generally reduce resistance to splitting, while differences in permeability and corrosion-product migration may alter the localization and surface expression of damage. The present dataset does not isolate these influences sufficiently to assign universal stage-transition thresholds.

Comparison of three concrete types

Practical implications for bridge engineering. The results demonstrate that corrosion-related changes in crack spacing and tension stiffening can become pronounced well before complete section loss. Because the crack-spacing index is indirect, and because the original paired target-condition design produced generally unique achieved mass-loss states rather than replicated observations at identical measured corrosion levels, the data should be used to inform mechanistic understanding and model assessment rather than as stand-alone rehabilitation thresholds.

NPC ($w/c = 0.52$): Dickson Bridge: This series had the highest permeability and most extensive longitudinal cracking and reached advanced corrosion states over the

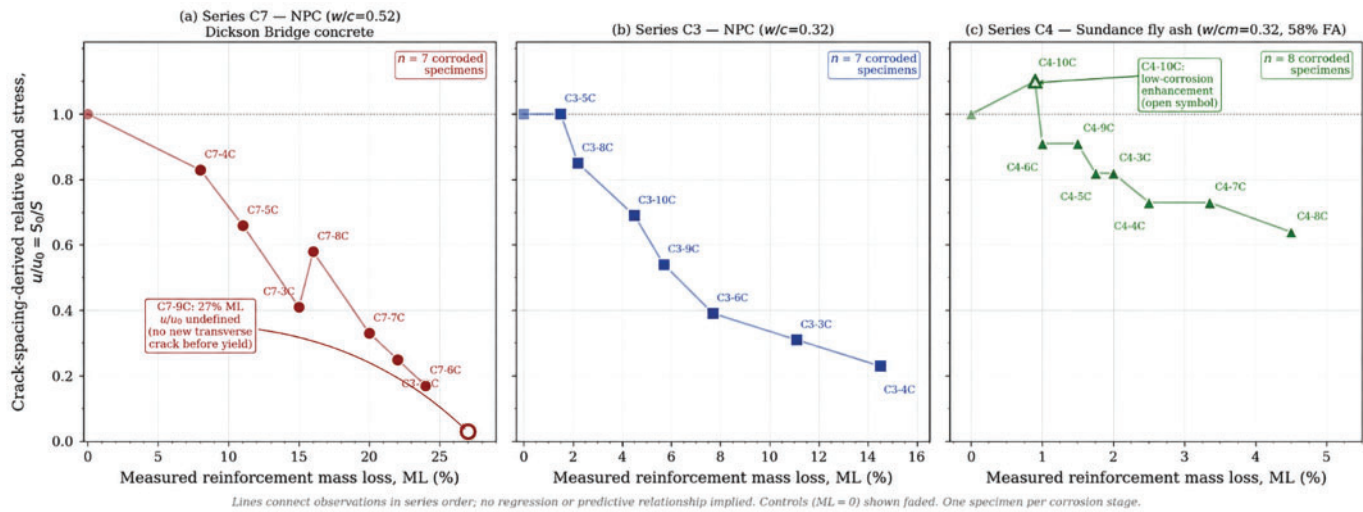


Figure 14. Crack-spacing-derived relative bond stress, $u/u_0 = S_0/S$, versus measured reinforcement mass loss for the three concrete series

accelerated exposure program. Bond-mediated force transfer decreased progressively over the measured 0–24% mass-loss range, while C7-9C at 27% ML reached a terminal condition in which no new transverse crack formed before steel yielding. The laboratory observations show qualitative consistency with the Dickson Bridge field data over the moderate mass-loss range, but differences in corrosion history, spatial variability, and accelerated versus natural exposure preclude treating the comparison as formal validation.

NPC ($w/c = 0.32$): Highest effective tensile strength inferred from the tension-member cracking response and the smallest uncorroded stabilized crack spacing among the three series (77 mm vs. 83 mm for C7 and 91 mm for C4), consistent with effective initial bond-mediated force transfer. Most resistant to longitudinal splitting. The crack-spacing-derived relative bond stress follows the same qualitative deterioration pattern but is delayed in mass loss terms relative to Series C7.

Implications for structural assessment and model development

The verified crack-spacing data provide quantitative member-level indicators that can support evaluation and future calibration of tension-stiffening and corrosion-damage models. Their particular value is that they arise from direct-tension members rather than pullout specimens, thereby preserving distributed transverse cracking and the progressive loss of concrete contribution between cracks. The quantity $u/u_0 = S_0/S$ is therefore reported as a descriptive, crack-spacing-derived relative bond stress and not as a direct local bond measurement, design resistance factor, or universally transferable predictive relationship.

For Series C7, the re-audited data give $u/u_0 = 0.83$ at 8% mass loss, decreasing to 0.17 at 24% mass loss. At 27% mass loss, no new transverse crack formed before yielding and u/u_0 is therefore not calculable. For Series C3, the index decreases to approximately 0.23 at 14.5% mass loss.

These trends demonstrate substantial degradation of bond-mediated force transfer but should not be converted directly into rehabilitation trigger criteria without independent calibration. The measured progression for all three concrete series is summarized in Fig. 14.

Symbols represent individual specimens, and lines connect observations within each series to illustrate the measured response progression; no regression or predictive relationship is implied. C7-9C is identified as a terminal response but is not assigned a numerical u/u_0 because no new transverse crack formed before steel yielding.

The data in Fig. 14 do not define universal rehabilitation thresholds. Although the original program was designed with two specimens per nominal target corrosion condition, the achieved corrosion states were generally unique rather than replicated at identical measured mass-loss levels. The response also depends on concrete tensile strength, corrosion localization, crack stabilization, confinement, and exposure history. The results are therefore best used as mechanistic evidence and for external model assessment rather than as statistically calibrated rehabilitation criteria.

For the Sundance HVFA concrete, the absence of longitudinal splitting should not be interpreted as evidence that member-level steel–concrete interaction was preserved. Localized corrosion and spalling were observed, while the crack-spacing and load–elongation responses still changed with corrosion. These observations support distinguishing resistance to corrosion initiation from post-initiation structural response. However, the proposed mechanism requires confirmation through replicated tests and spatially resolved corrosion measurements.

No design resistance factor or regression equation is recommended from the present dataset. Future calibration should include replicate specimens at matched measured corrosion levels, independent bond measurements, uncertainty quantification, and validation against naturally corroded members. These limitations are also relevant when applying

recent bond-strength and development-length models for corroded reinforcement.³⁴

Contemporary interpretation: from corrosion metrics to loss of composite action

The re-audited results suggest that the most useful contemporary interpretation is not a single mass-loss-to-bond equation but a progression in member response. At low corrosion, transverse cracking remains distributed, and tension stiffening is largely retained. The slight Stage-1 increase in first-cracking load is attributed primarily to the absence of shrinkage pre-strain in the continuously immersed corroded specimens rather than to a demonstrated increase in bond. Separately, the C4 crack-spacing index exceeds unity at 0.9% ML; this may reflect low-corrosion confinement or interfacial effects, but because crack spacing is also sensitive to concrete tensile response and crack stabilization, it is treated as an apparent rather than independently measured bond enhancement.

With increasing damage, fewer new transverse cracks form, average crack spacing increases, and the load-elongation response approaches that of the bare or equivalently corroded bar. At advanced deterioration, severe rib degradation, localized pitting, longitudinal splitting or local spalling, and reduced steel area act together; in the limiting C7-9C specimen, no new transverse crack formed before steel yielding. This sequence is consistent with recent direct-tension observations of corrosion-induced loss of tension stiffening.⁴

Average gravimetric mass loss should not be treated as a unique structural-damage variable. Mass loss is a useful global index, but specimens with similar average loss can differ in pit severity, rib degradation, corrosion cracking, confinement, and spatial distribution of corrosion products. The archived observations of deep localized pits and partially intact adjacent ribs indicate that global mass loss and local force-transfer capacity are complementary rather than equivalent descriptors. This heterogeneity also explains why specimens exposed under nominally similar accelerated-corrosion conditions need not achieve identical measured corrosion states.

For bridge assessment, the practical implication is therefore multi-indicator interpretation. Chloride content and half-cell or corrosion-rate measurements describe exposure and electrochemical state, while crack morphology, crack spacing, tension-stiffening response, and localized section loss provide information on structural consequence. The Dickson Bridge comparison illustrates both the value and the limitation of linking these domains: field chloride levels and mass loss show qualitative consistency with the laboratory trend, but spatial variability and post-decommissioning chloride redistribution preclude a one-to-one conversion. This interpretation aligns with current recommendations to integrate laboratory and in-situ observations when assessing chloride-induced corrosion.³⁵

Fig. 15 synthesizes the verified specimen-level data as a descriptive deterioration-response map. It does not define calibrated thresholds. Instead, it places the measured

$u/u_0 = S_0/S$ observations in the context of the progression from effective force transfer through increasing transfer length and reduced tension stiffening to the terminal C7-9C response, for which u/u_0 is not calculable because no new transverse crack formed before yielding. The regions are descriptive and do not represent calibrated deterioration or rehabilitation thresholds.

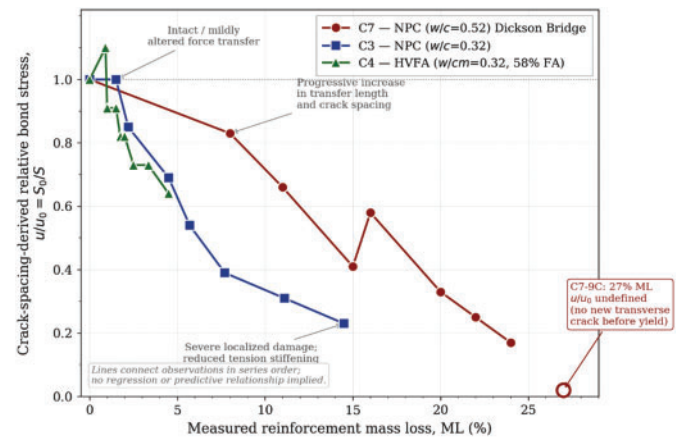


Figure 15. Descriptive deterioration-response map based on the verified direct-tension observations

The ordinate for Fig. 15 is the crack-spacing-derived relative bond stress $u/u_0 = S_0/S$. Region labels summarize the observed progression in force-transfer response; they are interpretive descriptors rather than statistically calibrated thresholds. C7-9C is shown as a terminal response but is not assigned a numerical u/u_0 value because no new transverse crack formed before steel yielding. The verified u/u_0 dataset can therefore serve as an external assessment benchmark rather than a calibrated design relationship. Section 5.5 applies this use by evaluating whether a contemporary direct-tension bond-reduction model reproduces the observed crack-spacing-derived relative bond-stress trends across the three concrete systems.

Comparison with a contemporary direct-tension bond-reduction model

Kunawisarut et al.¹ provide a particularly relevant external comparison because their experimental program also used single-bar reinforced-concrete members subjected to direct tension. Their bond formulation was derived through force equilibrium using average tensile-crack spacing, and the residual bond capacity was normalized to the corresponding uncorroded condition:

$$R_K = \tau/\tau_{\text{exp}} \exp(-0.054 ML) \quad (2)$$

where R_K is the residual bond ratio predicted by Kunawisarut et al. and ML is the average reinforcement mass loss expressed as a percentage. The published relationship was fitted after excluding the low-corrosion bond-enhancement regime and was proposed for corroded reinforced-concrete members subjected to direct tension. Kunawisarut et al. also observed that direct-tension members generally exhibit less bond degradation than pullout

specimens, emphasizing the importance of test configuration when transferring corrosion–bond models.

The normalized Kunawisarut relationship is mechanically comparable to the present within-series quantity $u/u_0 = S_0/S$ although neither quantity represents a direct local bond-stress measurement.

Geometric comparability. One geometric difference between the two experimental programs warrants consideration. The Kunawisarut et al. specimens had a 150×150 mm square cross-section with a reinforcing bar approximately 19 mm in diameter, whereas the present specimens were 120 mm diameter cylinders with a centrally positioned 20M bar ($d_b = 19.5$ mm). The corresponding geometric factor $A_c/(\pi d_b)$, which enters the force-equilibrium expression for average bond stress, is approximately 377 mm for the Kunawisarut specimens and 185 mm for the present specimens.

This difference can influence absolute crack spacing and inferred bond stress. However, $u/u_0 = S_0/S$ is normalized within each concrete series, so the fixed geometric terms cancel to first order. Geometry therefore does not preclude comparison of normalized deterioration trends, although differences in section geometry, concrete tensile response, reinforcement characteristics, and crack development remain relevant. The comparison is consequently treated as an external trend assessment rather than a demonstration of quantitative equivalence.

Model comparison. Eq. (2) was evaluated against the 22 corroded specimens for which a finite stabilized transverse-crack spacing, and hence u/u_0 , could be calculated. The six uncorroded controls define the within-series reference condition $u/u_0 = 1.0$ but were not included in the error calculations. C7-9C was excluded because no new transverse crack formed before steel yielding, and C3-7C was excluded because its test was not completed. For each concrete series, the mean absolute error was calculated as

$$MAE = \frac{1}{n} \sum_{i=1}^n \left| \left(\frac{u}{u_0} \right)_{obs,i} - \left(\frac{u}{u_0} \right)_{pred,i} \right|,$$

and the mean signed error was calculated using the observed-minus-predicted residuals. Each calculable corroded specimen received equal weight; the errors were not weighted by mass loss.

For Series C7 over the 8–24% ML range, $MAE = 0.093$ and the mean signed error was $+0.035$. For Series C3 over the 1.5–14.5% ML range, $MAE = 0.163$ and the mean signed error was -0.141 . For Series C4 over the 0.9–4.5% ML range, $MAE = 0.095$ and the mean signed error was -0.058 . All eight calculable corroded C4 observations were included in the external-model error assessment. Except for the 0.9% ML observation, which exhibits apparent low-corrosion enhancement, all C4 observations lie below the published relationship. The specimen observations and model comparisons are presented in Fig. 16.

Symbols represent individual corroded specimens, the dashed curve represents the published Kunawisarut relationship, and the solid curve represents the descriptive exponential fit to the present observations. C7-9C is excluded because u/u_0 is undefined.

Descriptive exponential comparison. In addition to the published model, Fig. 16 presents a one-parameter descriptive exponential fit for each series:

$$\frac{u}{u_0} = \exp(-a ML). \quad (3)$$

The coefficient a was obtained by unweighted nonlinear least-squares minimization of residuals in the original u/u_0 domain, with the uncorroded intercept fixed at $u/u_0 = 1.0$. The C4-10C observation at 0.9% ML and $u/u_0 = 1.10$ is displayed in Fig. 16 but was excluded from the C4 descriptive fit because it represents the low-corrosion enhancement regime. It was retained in the external-model error calculation because that calculation evaluates model performance over the complete observed range.

The fitted coefficients are $a = 0.050, 0.100$, and 0.100% ML^{-1} for Series C7, C3, and C4, respectively, compared with $a_K = 0.054\%$ ML^{-1} for the Kunawisarut et al. relationship. The coefficients and their ratios to a_K are summarized in Fig. 17.

The dashed horizontal line represents the Kunawisarut et al.¹ reference coefficient, $a_K = 0.054\%$ ML^{-1} . The C4 low-corrosion enhancement observation at 0.9% ML was excluded from the descriptive coefficient fit.

The ratios a/a_K are approximately 0.93, 1.85, and 1.86 for Series C7, C3, and C4, respectively. These ratios describe differences in fitted curve shape and should not be interpreted as indicating that one concrete deteriorates a corresponding number of times faster. The three series cover substantially different measured mass-loss ranges; therefore, the fitted coefficients are descriptive comparison metrics rather than calibrated material constants or statistically validated deterioration rates.

The nearly identical descriptive coefficients for Series C3 and C4 indicate that the deviation from the published relationship cannot be attributed to fly ash alone. Both series have a lower w/cm than Series C7, and their shared concrete-quality characteristics may contribute to the observed similarity. However, the present dataset does not isolate the governing material parameter.

The Series C4 observations remain scientifically notable. At approximately matched low corrosion levels, the crack-spacing-derived relative bond stress for C4 tends to be slightly lower than that of C3. For example, $u/u_0 = 0.91$ for C4 compared with 1.00 for C3 at 1.5% ML, and 0.64 compared with 0.69 at 4.5% ML. This difference occurs even though the C4 specimens developed no visible corrosion-induced longitudinal splitting cracks.

The C4 response indicates that the absence of longitudinal splitting is not sufficient evidence that member-level bond-mediated force transfer has been preserved. Localized rib damage, local spalling, concrete tensile properties, corrosion-product localization, and the substantially lower chloride contents measured at the reinforcement level in Series C4 may all have influenced the observed response. The present program cannot separate their individual effects.

Uncertainty and scope. No formal confidence bands or coefficient confidence intervals are reported. Although the

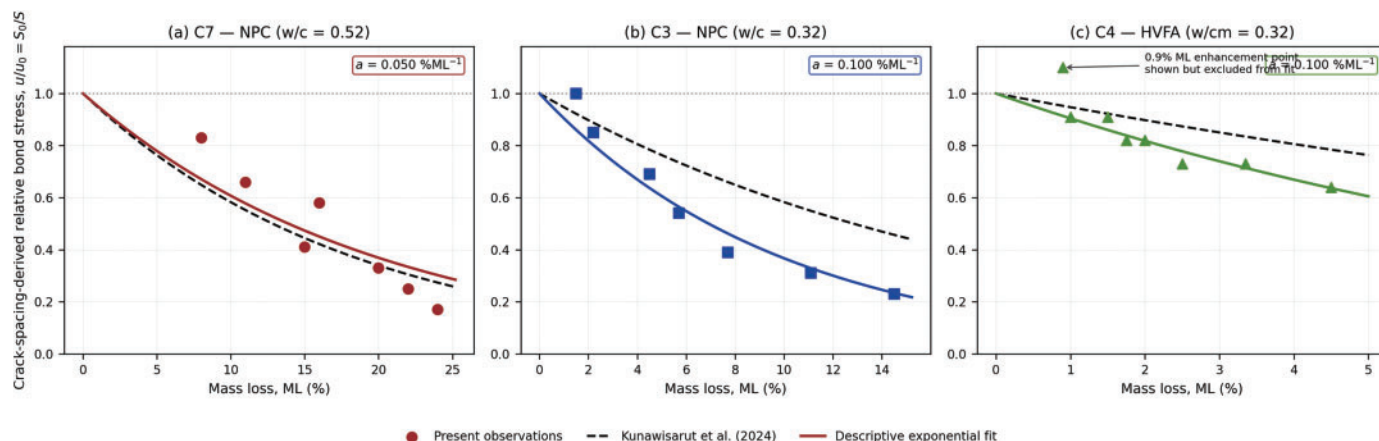


Figure 16. Comparison of the Kunawisarut et al.¹ direct-tension bond-reduction relationship, $R_K = \exp(-0.054ML)$, with the present crack-spacing-derived relative bond stress, $u/u_0 = S_0/S$: (a) Series C7, NPC ($w/c = 0.52$); (b) Series C3, NPC ($w/c = 0.32$); and (c) Series C4, HVFA ($w/cm = 0.32$)

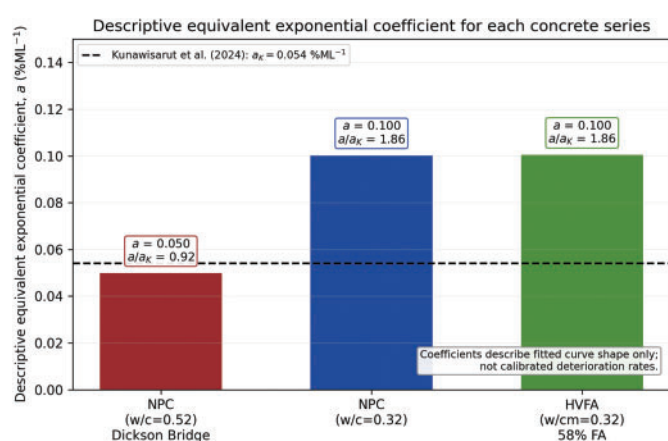


Figure 17. Descriptive equivalent exponential coefficient a obtained from $u/u_0 = \exp(-a ML)$ for the three present concrete series

program was designed around nominal target corrosion conditions, the specimens generally reached different measured mass-loss levels. Treating observations from adjacent corrosion stages as statistical replicates would combine actual corrosion progression with specimen-to-specimen variability and could produce a misleading uncertainty estimate.

The control pairs exhibited essentially identical stabilized crack spacings within each series, but they do not quantify uncertainty under corroded conditions. The present analysis is therefore an external model assessment using individual observations and descriptive error metrics rather than a statistical validation. Replicated tests at matched measured corrosion levels are required to quantify predictive uncertainty.

Conclusions

1. Thirty direct-tension specimens from three concrete systems were examined using the archived experimental program. The re-audited specimen geometry is

1000 mm concrete length and 120 mm cylindrical diameter, with a centrally placed 20M reinforcing bar ($d_b = 19.5$ mm), approximately 50 mm radial concrete cover ($c/d_b \approx 2.58$), and a 1010 mm extensometer gauge length.

2. Tension stiffening decreased progressively with corrosion in all three concrete systems. At advanced corrosion levels, the load-elongation response approached that of the corroded steel bar, indicating limited concrete contribution.
3. Transverse-crack spacing increased with corrosion. The revised analysis retains the original relation $u/u_0 = S_0/S$ and explicitly defines u/u_0 as a crack-spacing-derived relative bond stress rather than a direct measurement of local bond capacity. For C7-9C, u/u_0 is not calculable because no new transverse crack formed before steel yielding.
4. The Sundance HVFA specimens did not develop corrosion-induced longitudinal splitting cracks, although two specimens experienced highly localized corrosion and local spalling. A permeability-related localization mechanism is presented only as a working hypothesis.
5. The verified archived data show $u/u_0 = 0.83$ for Series C7 at 8% mass loss and 0.17 at 24% mass loss; approximately 0.23 for Series C3 at 14.5% mass loss; and approximately 0.64 for Series C4 at 4.5% mass loss. These values are descriptive trends from a dataset whose original paired target-condition design yielded generally unique achieved mass-loss states rather than replicated observations at identical measured corrosion levels.
6. Series C7 laboratory results show qualitative consistency with Dickson Bridge field observations for moderate mass loss, but the comparison does not constitute full validation of accelerated corrosion. Field chloride and mass-loss data display substantial spatial and exposure-history variability.

7. The study is a contemporary re-analysis of a legacy experimental dataset rather than a new experimental campaign. Its contribution is the verified specimen-level consolidation of the direct-tension records, reinterpretation of crack spacing as a member-level bond-transfer indicator, external assessment against a contemporary direct-tension bond-reduction relation, explicit comparison with current high-corrosion tension-stiffening evidence, and integration of laboratory response with Dickson Bridge field observations.
8. Comparison with Kunawisarut et al.¹ shows that the published exponential direct-tension bond-reduction relation captures the average C7 trend reasonably well but generally overpredicts retained crack-spacing-derived bond response for the lower-w/cm C3 and C4 series. Using descriptive one-parameter exponential fits and excluding the C4 low-corrosion enhancement point from the C4 coefficient fit, the equivalent coefficients are approximately 0.050, 0.100, and 0.100 %ML⁻¹ for C7, C3, and C4, respectively, compared with 0.054 %ML⁻¹ in the published relation (the slight difference between the C3 (1.85) and C4 (1.86) ratios reflects fitting precision before rounding to 0.100.). Because C3 and C4 show nearly identical descriptive coefficients and the three series span different corrosion ranges, the deviation cannot be attributed to fly ash alone, and the coefficients should not be treated as calibrated material deterioration rates. The C4 observations are nevertheless notable because reduced bond-mediated force transfer occurred despite the absence of visible corrosion-induced longitudinal splitting cracks.
9. The absence of reconstructable specimen-specific cumulative-charge histories prevents retrospective comparison with Faraday-law estimates. Future studies should retain time-resolved electrochemical records and include true replicates at matched measured corrosion levels. For bridge assessment, the findings support a multi-indicator approach combining chloride exposure, electrochemical condition, crack morphology and spacing, tension-stiffening response, and localized section loss rather than relying on average mass loss alone. The six-stage framework is therefore retained as a qualitative interpretation of the progression from distributed cracking and tension stiffening to severely impaired composite action, not as a calibrated deterioration law.

Overall, the six-stage framework is retained as a qualitative mechanistic interpretation of the observed progression from distributed cracking and tension stiffening toward severely impaired composite action. It is not proposed as a calibrated deterioration law. The legacy dataset is most valuable when used transparently alongside contemporary replicated experiments and field observations.

Declaration of Competing Interest

The author declares no competing financial interests or personal relationships that could have influenced the work reported in this paper.

Data Availability

The complete experimental dataset (load-elongation curves, crack width measurements, crack spacing data, chloride profiles, and mass loss values for all 30 specimens) is available from the corresponding author upon reasonable request.

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Supplementary Table S1. Complete specimen-by-specimen archival dataset used in the revised analysis, provided in the online supplementary material. C3-7C was not completed because of an MTS malfunction; unavailable quantities are reported as N/A

Series	Specimen	Stage	Mass loss (%)	Major long. cracks	Avg long. width (mm)	1st crack load (kN)	Yield load (kN)	No. trans. cracks	Avg spacing (mm)	Cross-sec. loss (%)	Cl at steel (%)	u/u0
C7	C7-1C	0	0	0	0	18	127	11	83	0		1.00
C7	C7-2C	0	0	0	0	19	127	11	83	0		1.00
C7	C7-4C	1	8	1	1.2	23	116	9	100	8.7	0.37	0.83
C7	C7-5C	2	11	1	1.5	24	102.5	7	125	19.3	0.48	0.66
C7	C7-8C	3	16	1	1.2	23	96	6	143	24.4	0.72	0.58
C7	C7-3C	4	15	2	1.3	17	87.5	4	200	31	0.94	0.41
C7	C7-7C	5	20	1	3	28	83	3	250	34.6	1.16	0.33
C7	C7-10C	6	22	2	4	59	81.5	2	333	35.8	1.12	0.25
C7	C7-6C	7	24	3	2	63	70	1	500	44.9	1.44	0.17

Supplementary Table S1. (Continued)

Series	Specimen	Stage	Mass loss (%)	Major long. cracks	Avg long. width (mm)	1st crack load (kN)	Yield load (kN)	No. trans. cracks	Avg spacing (mm)	Cross-sec. loss (%)	Cl at steel (%)	u/u0
C7	C7-9C	8	27	2	4	N/A	45	0	N/A	64.6	1.52	N/A
C3	C3-1C	0	0	0	0	19	127	12	77	0		1.00
C3	C3-2C	0	0	0	0	19	127	12	77	0		1.00
C3	C3-5C	1	1.5	0	0	34	125	12	77	1.6	0.04	1.00
C3	C3-8C	2	2.2	2	0.08	48	124	10	91	2.4	0.11	0.85
C3	C3-10C	3	4.5	2	0.35	35	121	8	111	4.7	0.19	0.69
C3	C3-9C	4	5.7	3	1.5	30	122	6	143	3.9	0.28	0.54
C3	C3-6C	5	7.7	3	2	31	115	4	200	9.4	0.29	0.39
C3	C3-3C	6	11.1	2	3	48	93	3	250	26.8	0.35	0.31
C3	C3-4C	7	14.5	3	3	37	83	2	333	34.6	0.46	0.23
C3	C3-7C	N/A*	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
C4	C4-1C	0	0	0	0	22	126	10	91	0		1.00
C4	C4-2C	0	0	0	0	21	128	10	91	0		1.00
C4	C4-10C	1	0.9	0	0	31	127	11	83	0	0.002	1.10
C4	C4-6C	2	1.0	0	0	29	124	9	100	2.4	0.003	0.91
C4	C4-9C	3	1.5	0	0	27	126	9	100	0.8	0.013	0.91
C4	C4-5C	4	1.75	0	0	27	125	8	111	1.6	0.004	0.82
C4	C4-3C	5	2.0	0	0	32	124	8	111	2.4	0.02	0.82
C4	C4-4C	6	2.5	0	0	36	124.5	7	125	2.0	0.01	0.73
C4	C4-7C	7	3.35	0	0	22	110	7	125	13.4	0	0.73
C4	C4-8C	8	4.5	0	0	30	90	6	143	29.1	0.006	0.64

Modal Analysis, Health Monitoring, and Train-Induced Resonance of Multi-Span Bridges

Lutz Auersch*; Samir Said and Rolf Rohrmann

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Abstract: Resonances of railway bridges due to the passage of trains have been mainly investigated for single-span bridges. When multi-span bridges are considered, it is of interest whether stronger resonance amplifications compared to a single-span bridge must be taken into account. Measurements of eight multi-span structures are presented with their natural frequencies and mode shapes: wooden floor beams, a concrete beam, a steel truss footbridge, three road bridges, and two railway bridges. Typical effects are shown, such as multiple fundamental frequencies due to parallel beams and multiple spans, close eigenfrequencies of different mode shapes, clusters for simply supported or weakly coupled spans, and regularly separated frequencies for continuous beams and bridges. The mode shapes are generally global, but sometimes more local mode shapes exist; for example, a longer midspan is dominant in the first eigenmode. The consequences of multi-span bridges for train passages are discussed based on a frequency-domain theory in which three spectra determine the resonance excitation: the axle-sequence spectrum of the train, the transfer function of the bridge, and the modal force spectrum. The method is applied to the two railway bridge examples, which are treated as single-span bridges at first, and then these examples are extended to two-, three-, and multi-span bridges. The resonance of a railway bridge occurs when a maximum of the axle-sequence spectrum (the second or third car-length frequency in the examples) coincides with an eigenfrequency of the bridge. The resonance amplitude is determined by the mode shape, which results directly in the modal force spectrum and the modal mass, for which some rules are presented, as well as for the resonance amplitude of multi-span bridges.

Author keywords: Railway bridge; train passage; resonance; multiple spans; mode-shape spectrum

Introduction

When trains pass over bridges, the response to a moving load is higher than that for a static load. Higher amplifications occur due to the repeated axle loads. For special combinations of train speed, train configuration, bridge length, and eigenfrequency, a strong resonance can occur. For high-speed trains on short bridges, the amplitudes were sometimes so high that the ballasted track was damaged.

The topic of resonances of railway bridges has attracted many researchers in the past,¹⁻⁴ and many articles about methods, phenomena, and standards have been published.³⁻⁷ The resonance of bridges under train passage is a complex phenomenon in which, besides amplification, cancellation (of resonances) is also observed.⁸⁻¹⁰ Most theoretical analyses are performed by time-domain methods, and only a few articles use frequency-domain methods.^{4,11,12} A modal analysis of the bridge is usually a fundamental step in the

investigation. In the attempt to get simpler methods and rules, the problem is divided into the phase of bridge passage and the free vibrations after the bridge passage, e.g.,¹² and into the passage of the static train loads and the full vehicle-bridge interaction, e.g.^{13,14} Finally, many experiments on bridge resonances under train passages have been reported, for example, in.¹⁵⁻¹⁹ All these articles deal with single-span bridges, in many cases with simply supported bridges. Only a few articles present examples or methods for multi-span railway bridges.²⁰⁻²⁵ More general rules for multi-span bridges are rarely found.

Modal analysis is also a basic concept in structural health monitoring.²⁶⁻³⁰ The recent research focuses on better tools for operational modal analysis,³¹⁻³³ on removing environmental conditions, especially the influence of the temperature,^{34,35} and on automated monitoring,^{36,37} statistical methods,³⁸⁻⁴⁰ and the use of artificial intelligence.⁴¹

The research on bridges at the Federal Institute of Material Research and Testing (BAM) was initiated by Rohrmann,²⁷ and many single-span and multi-span bridges have been measured and monitored mainly with respect to damage detection.⁴²

The experience with bridges from health monitoring is used here for the topic of resonances of railway bridges. This contribution focuses on multi-span bridges and their special effects compared to single-span bridges. What are the typical

*Corresponding Author: Lutz Auersch.
Email: lutz.auersch-saworski@bam.de
Federal Institute of Material Research and Testing, 12200, Berlin, Germany

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resonance frequencies (resonance frequency patterns), and what are the typical mode shapes of multi-span bridges? Are multi-span railway bridges more dangerous or less dangerous than single-span bridges? Can a multi-span bridge be analyzed by looking at only one span?

The structure of the article is as follows.

The first section presents some theory of multi-span structures with different coupling conditions. The next section, with eight subsections, presents eight measurement examples of multi-span structures: beams, floors, four road bridges, and two railway bridges. The examples have different numbers of spans and different structures, such as weakly coupled simply supported spans or parallel beams, continuous spans, and an arch with nearby eigenfrequencies. After the measurement examples, the next section presents the frequency-domain theory for the response to train passages, where characteristic spectra of the train, the transfer functions of the bridge, and the mode shape (modal force) spectra are discussed. This theory is applied to the two railway bridge examples in the next section, where these bridges are analyzed as single-span bridges at first. Then, in the next section, these examples are extended to two-, three-, and multi-span continuous and simply supported bridges, and the differences between single and multiple spans in case of resonant train passages are evaluated.

Theory of Multi-Span Structures

The principal behavior of multi-span floors and bridges can be analyzed by one-dimensional beam models. The differential equation for the displacements u of a beam

$$EI u'''' + \mu u'' = 0 \quad \text{or in frequency domain } \omega$$

$$EI u'''' + \mu \omega^2 u = 0 \quad (1)$$

with bending stiffness EI and mass per unit length μ has four solutions

$$u = A_1 \cos ax + A_2 \sin ax + A_3 \exp(+ax) + A_4 \exp(-ax), \quad (2)$$

if the parameter a is fitted to the differential equation

$$a = \sqrt[4]{EI/\mu\omega^2}. \quad (3)$$

The constants A_i are fitted to the boundary conditions at the supports.

At first, the full coupling of identical beams is considered. The assembly of all beams yields a continuous beam with continuous rotations at the supports. This is demonstrated for 5 m span concrete beams in Fig. 1. For coupled beams, the support conditions depend on the vibrations of the adjacent beams. If two beams vibrate in anti-phase, the mid support is “hinged” (Fig. 1a), whereas the mid support is “clamped” if the beams are in phase (Fig. 1c). Therefore,

the two coupled beams have two eigenfrequencies (Fig. 1e): the hinged–hinged and the higher hinged–clamped eigenfrequency of a single beam. In the case of many coupled beams, more than one eigenfrequency exists. The eigenfrequencies lie in a frequency band between the hinged–hinged and the clamped–clamped eigenfrequency of a single beam (Fig. 1f). Generally, all beams participate in all eigenmodes (Fig. 1b,d).

In contrast to this strong coupling of beams, a weak coupling of simply supported beams is analyzed by inserting rotational springs at the supports. Thus, two adjacent beams can have discontinuous rotations. The resulting spectra and a mode shape are demonstrated in Fig. 1g,h for the same beam model as before. The fourth mode of the 6-span beam clearly shows the additional rotations due to the weaker coupling (Fig. 1g). The different eigenfrequencies of the weakly coupled beams (Fig. 1h) are in a narrower frequency range compared to the strong coupling. Similar multi-span models and their special behavior have also been discussed for the health monitoring of bridges.³⁷

Observations at Eight Example Structures and Bridges

The following eight subsections present modes and eigenfrequencies of eight multi-span structures that have been measured during health monitoring tasks. The purposes of these monitoring tasks and some related results are given as supplementary information. The observations about the mode shapes, eigenfrequencies, and eigenfrequency clusters are used for the train-induced resonances in the second part of the article.

Example 1: Continuous two-span prestressed concrete beam

A two-span prestressed concrete beam with a total length of 24 m (2×12 m) and a width of 1 m has been built on the BAM test area (Fig. 2a). A line of 25 vertical geophones was installed at 1 m intervals along the bridge, and the modal analysis was done by hammer tests. The natural frequencies appear in pairs, where the square roots of the frequencies always keep the same distance (Fig. 2b). The first six modes are presented in Fig. 3 as theoretical modes (lines) and experimental results (square symbols). Each pair of natural frequencies corresponds to the antisymmetric mode with a lower frequency (Fig. 3, left) and the corresponding symmetric mode with a higher frequency (Fig. 3, right). The agreement between theory and experiment is very good.

The prestress of the beam could be varied (max 1200 kN) by the external tendons below the u-shaped cross section (Fig. 4a). The beam was damaged by releasing the prestress, and many cracks developed. A second modal analysis was done in this released state, and then the beam was prestressed again, and a third measurement was done. A clear reduction

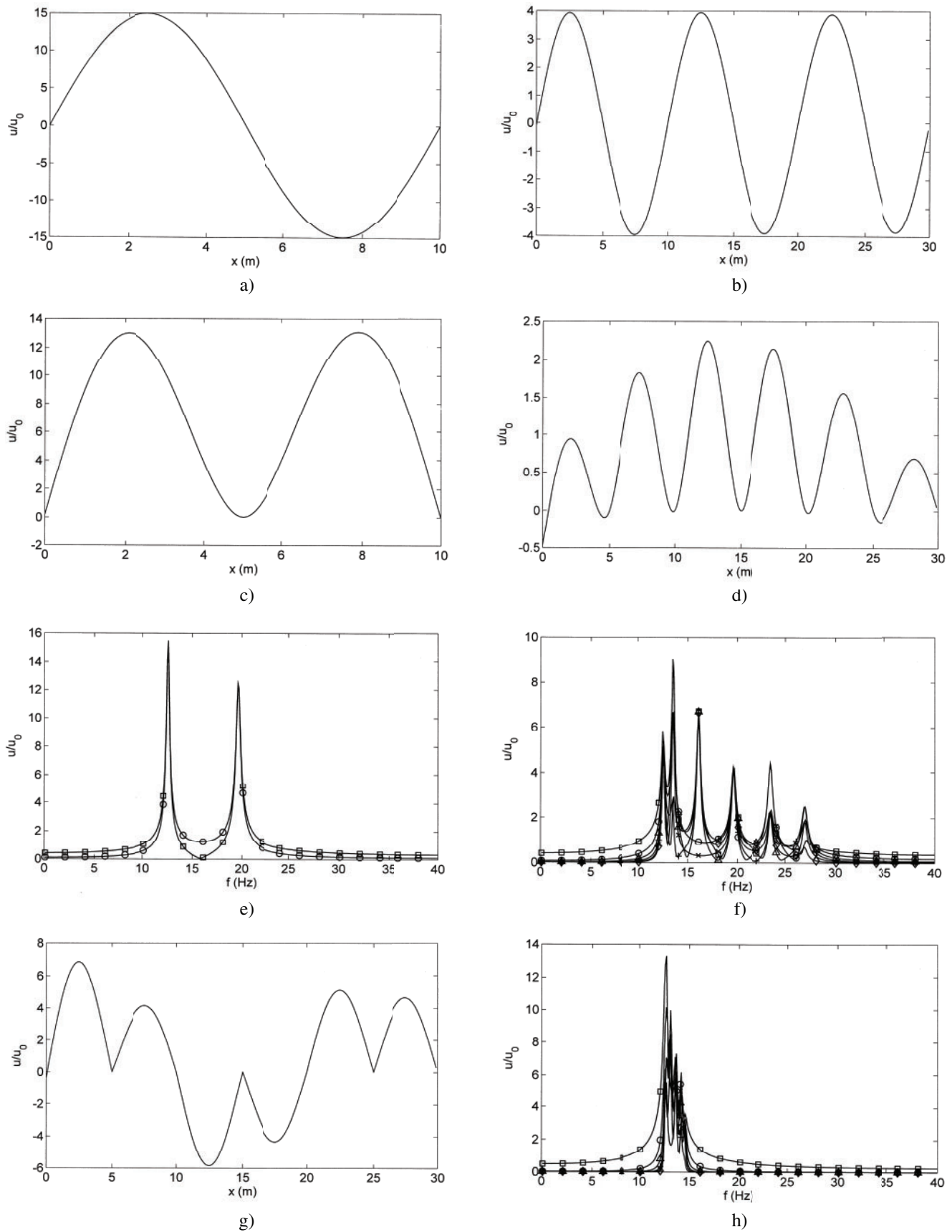
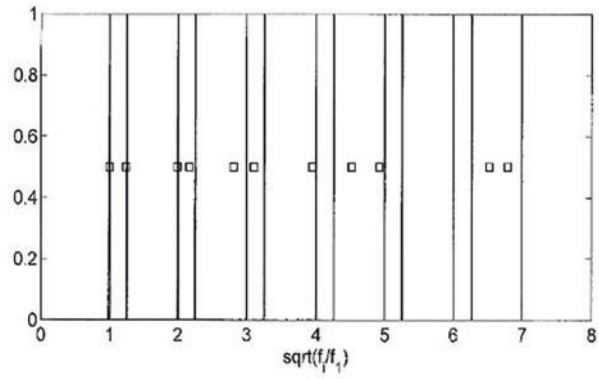


Figure 1. Coupled concrete floor beams of 5-m length, 0.2-m thickness: (a, c, e) 2-span continuous, (b, d, f) 6-span continuous, (g, h) 6-span weakly coupled by rotational springs, (a–d, g) vibration modes, and (e, f, h) spectra



a)



b)

Figure 2. Two-span beam (a) and regular sequence of the natural frequencies (theory) and $\square$ measurements (b)

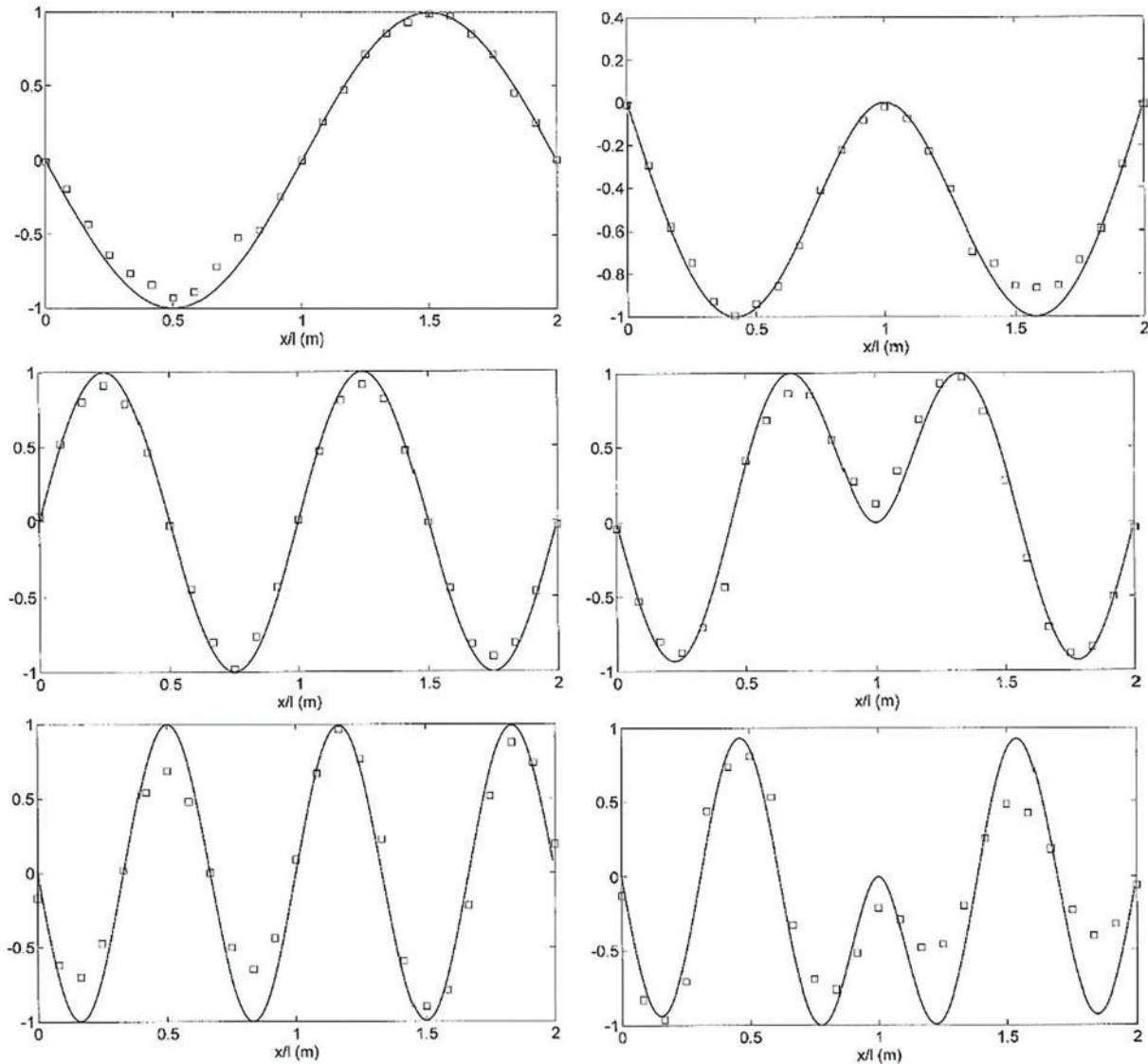


Figure 3. Calculated mode shapes of the two-span beam of length $2l = 24$ m, first, second, and third antisymmetric (left) and symmetric (right) modes and $\square$ measurement results at 3.9, 6.0, 15.4, 18.3, 37.4, 45.9 Hz, with 2.1, 1.3, 1.0, 6.3, 2.2, 4.2% damping

of the first eigenfrequencies was found for the intermediate state without prestress and with subsequent damage, but

the eigenfrequencies are almost identical for the undamaged and damaged states with prestress (Fig. 4b). As the total

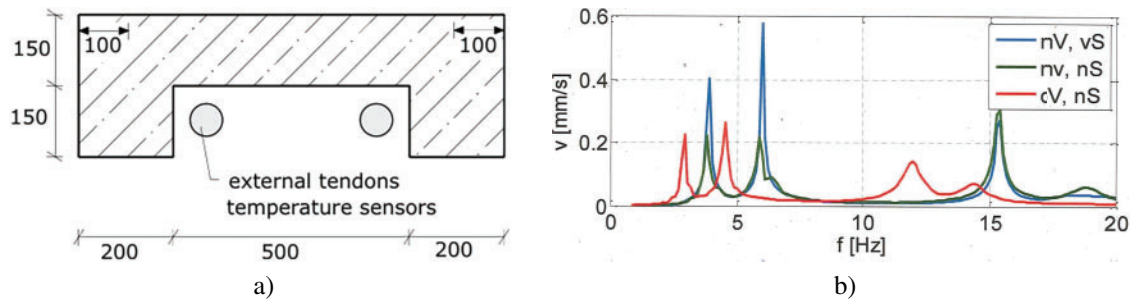


Figure 4. U-shaped cross section of the two-span beam with two external tendons for variable prestressing (a), transfer functions of the beam from hammer impact: with prestress without damage (blue), without prestress after damage (red), and after damage with prestress (green)

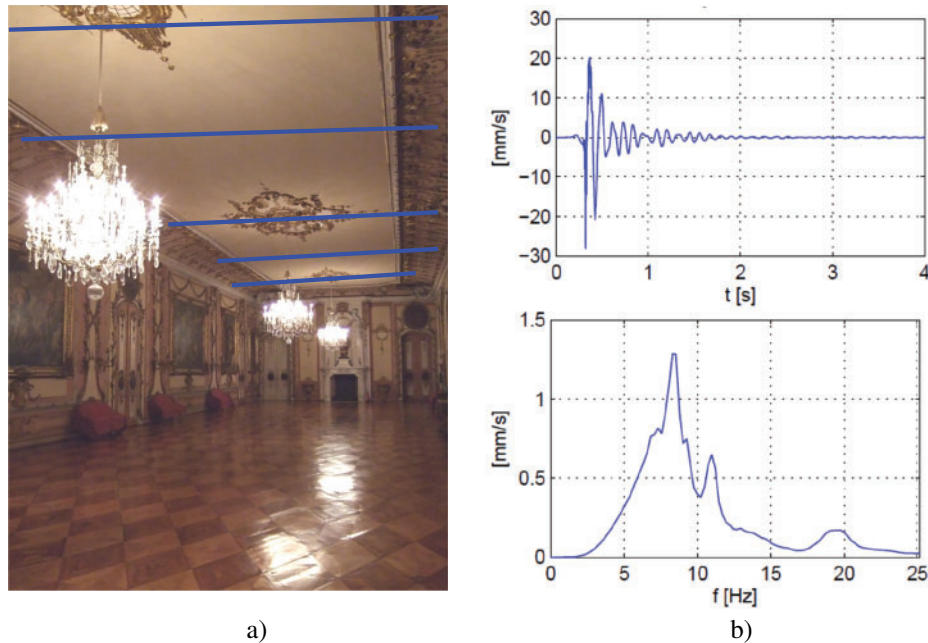


Figure 5. Floor above a large ballroom with 6 rooms on the second story (a) and time history and spectrum of a heel-drop test (b)

loss of prestress gives the maximum crack damage, it can be concluded that cracks under prestress are difficult to detect by modal analysis. The tests were repeated for different lower prestress levels, and the stress levels could be identified.^{43,44}

Example 2: Eleven coupled wooden floor beams (“Neues Palais” in Potsdam)

The large (30 m × 10 m) wooden floor above a ballroom in the “Neues Palais” in Potsdam was dynamically tested (Fig. 5a). Eleven wooden floor beams (0.3 m × 0.4 m) span a length of ten meters and are connected by three layers of floorboards. The floor is divided into six rooms by non-load-bearing walls. The measurement line of 18 sensors along the middle axis of the floor was shifted to three more positions at the quarter lines and along the outer bearing wall. Hammer, heel-drop, and ambient measurements were performed on this floor. In addition to the measurements, a 2-dimensional model of eleven 10-m-long beams was built, which are clamped at both ends and coupled by springs along

the middle axis, and where every second beam is stiffened and loaded by a non-load-bearing wall.

All measured spectra show a maximum region between 5 and 10 Hz with some relative maxima, but without sharp peaks (see Fig. 5b for an example). The wide resonance region indicates the coincidence of several natural frequencies, and a high damping can be assumed for this system. A special method to extract nearby modes was developed and applied to some of the floor modes.⁴⁵

The measured and calculated vibration modes are shown in Fig. 6 as amplitudes along the long axis of the floor. The first mode at about 7 Hz shows a global behavior of the floor, where the floor beams at 2.5, 7.5, ..., 27.5 m have higher amplitudes than the wall beams at 5, 10, ..., 25 m. The second mode at 7.7 Hz consists of vibrations of the floor beams, where every two neighboring rooms are in phase. The next mode at 8.5 Hz consists of vibrations of all floor beams out of phase with their neighbors, while the wall beams are at rest. At a higher frequency of 11.7 Hz, the wall beams show strong amplitudes that are in anti-phase to the floor beams.

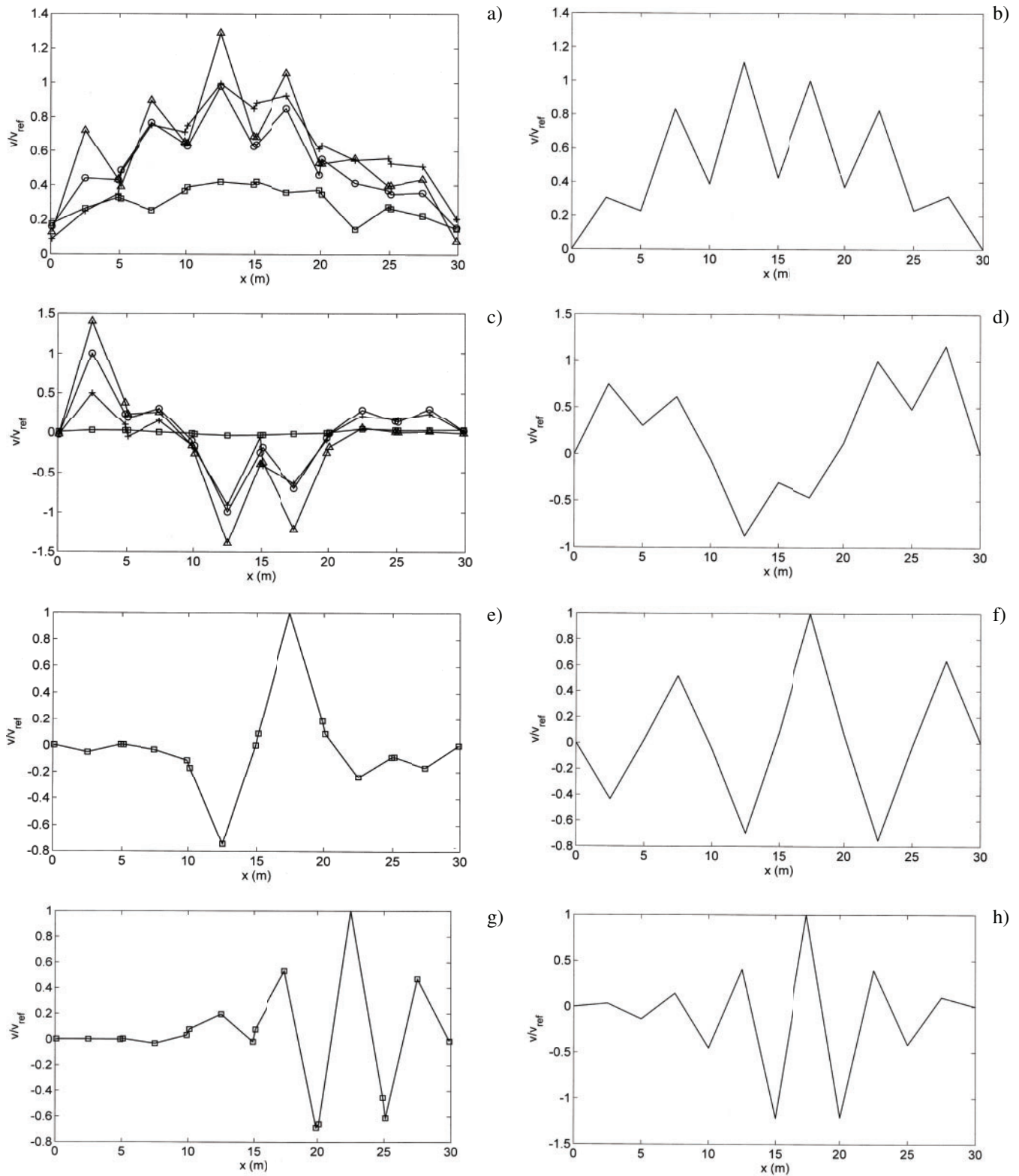


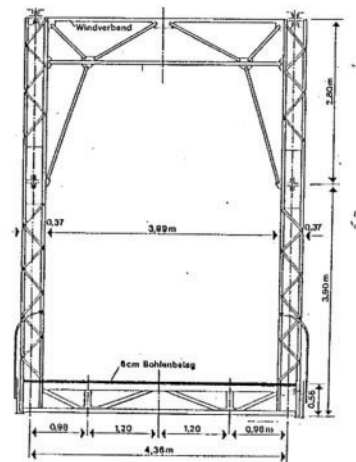
Figure 6. Measured (left) and calculated (right) vibration modes of the 11 coupled floor and wall beams: (a) 6.5 Hz (ambient), (b) 7.2 Hz, (c) 7.7 Hz (ambient), (d) 7.7 Hz, (e) 8.5 Hz (hammer), (f) 8.3 Hz, (g) 11.7 Hz (hammer), (h) 11.7 Hz, Δ middle, $\circ$ outer and $+$ inner quarter axis, and $\square$ outer wall axis in ambient measurements (a, c)

It was found that all floor beams contribute to each natural mode even for a weak coupling of the beams, so that

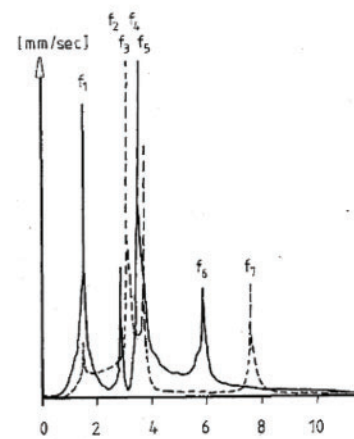
it was impossible to assess the state of each individual beam, neither by global ambient tests nor by local hammer tests.



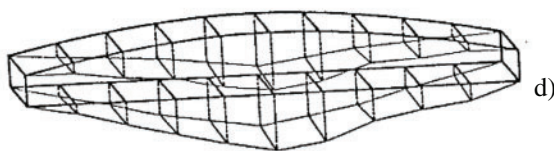
a)



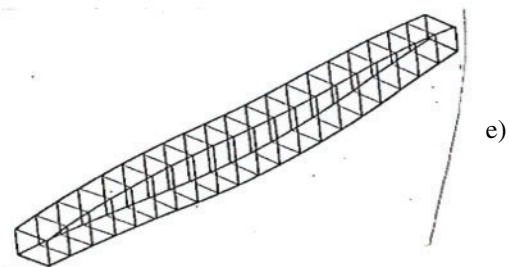
b)



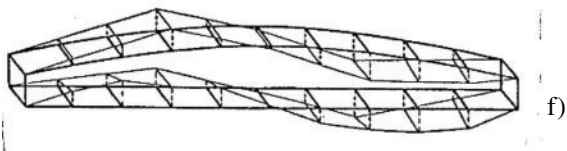
c)



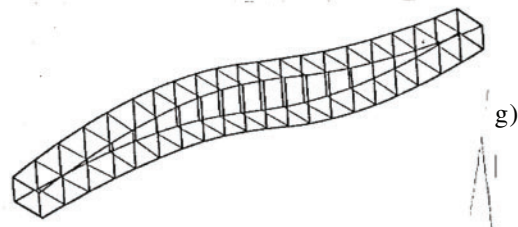
d)



e)



f)



g)

Figure 7. Steel truss arch bridge: view (a), cross section (b), spectra with first eigenfrequencies (c), first vertical eigenmode measured (d) and calculated (e), and second vertical eigenmode measured (f) and calculated (g)

Example 3: Steel truss arch bridge with close eigenfrequencies

The steel truss arch bridge (Fig. 7a,b) is a footbridge with a length of 63.7 m, a width of 4.5 m, and a maximum height of 6.6 m which was sensitive to crowd movement. The positions of dampers should be optimized with the help of the modal analysis. The horizontal and vertical vibrations

due to walking, jumping, wind, and other excitation were measured on the arch and the deck, on both sides of the bridge. The first two vertical modes are shown in Fig. 7d-g. They are the symmetric first bending mode and the anti-symmetric second bending mode. Completely different from a straight beam, the eigenfrequencies of these modes are very close, at 3.16 and 3.63 Hz. This is a ratio of 1.15, whereas

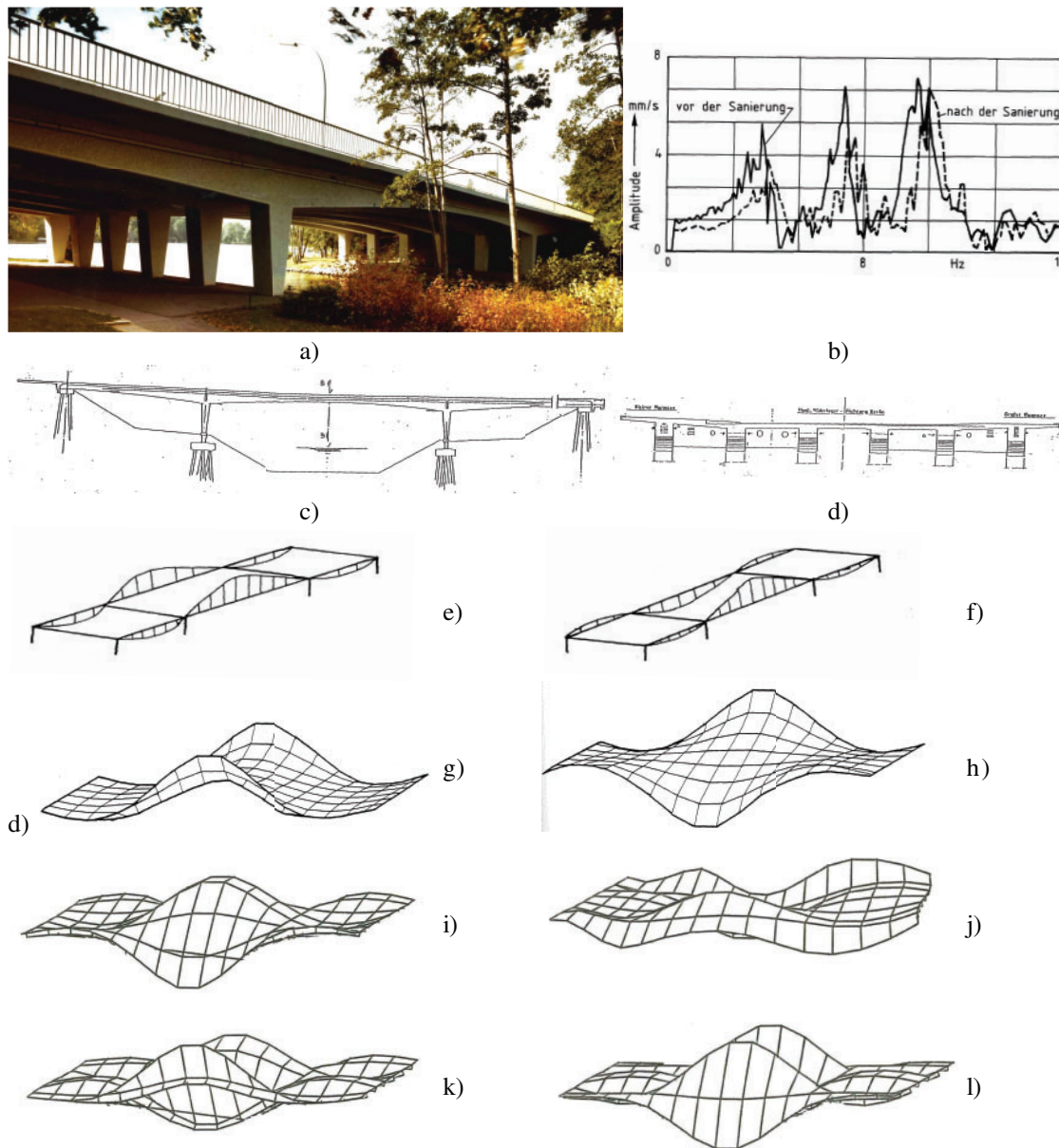


Figure 8. Three-span bridge with six parallel beams: view (a, c), cross section (d), mid-point spectrum before and after renovation (b), the first two mode shapes from measurement (e, 3.9 Hz; f, 4.4 Hz), and the first six mode shapes from finite-element calculation (g–l, 4.1, 4.2, 4.4, 4.5, 4.7, and 4.8 Hz)

the ratio between the second and first eigenfrequency would be 2.75 for a clamped–clamped beam and even 4 for a simply supported beam. The specific behavior of the arch bridge was confirmed by finite-element calculations, which resulted in eigenfrequencies of 2.9 and 3.5 Hz and a ratio of 1.2. Nearby eigenfrequencies of the first and second modes have been found in other measurement examples of arch bridges.^{44–49} Moreover, the symmetric fundamental mode is sometimes missing. This can be explained by the stiffening of the mid-beam due to the curvature, which shifts the first bending mode to a higher frequency. For a strong curvature,

the arch tends to behave like a two-span structure that has a first antimetric mode.

Example 4: Three-span bridge with six parallel beams

The beam-type bridge has a midspan of 35 m and two side spans of 20 m (Fig. 8a). Six prestressed concrete beams are equally distributed across the width of 24 m. The bridge had been repaired by adding steel plates under the beams. The bridge vibrations were measured with 40 geophones on the left and right lanes before and after the repair.

The spectra show a maximum region between 3 and 4 Hz (Fig. 8b) where several modes are assumed. The two first fundamental bending modes could be measured by the two measurement lines. For the first (symmetric or bending) mode (Fig. 8e), the left and right sides of the bridge are in phase, and the shorter side spans are in anti-phase with the midspan. For the second (antimetric or torsional) mode (Fig. 8f), the left and right sides of the bridge are in anti-phase. More modes can be calculated with a 3-dimensional finite-element model (Fig. 8g–i), which consist of different combinations of upward and downward (in-phase and out-of-phase) amplitudes of the six beams. Thus, the full set of fundamental modes could be obtained from the calculation. The amplitudes of the midspan are dominant for all these fundamental modes. The higher resonance frequencies in Fig. 8b belong to higher bending modes of the bridge. An increase in the frequencies was observed after the repair, which indicates a 15% increase in stiffness as a result of the repair.

Example 5: Seven-span continuous box-type highway bridge—health monitoring and modal analysis of the Westend bridge

The Westend highway bridge was built in the early 1960s as a prestressed concrete box-girder bridge. It has a length of 240 m and consists of seven continuous spans of approximately 35 m length and 14 m width (Fig. 9a), which are supported on hinged circular columns, the abutments at the ends, and a clamped column in the middle of the bridge.

The preliminary studies concentrated on the critical third span, where cracks in the box girder and in the shear wall above the column occurred. The modal analysis of this span clearly showed the first and second bending and torsion modes between 2 and 11 Hz (Fig. 9e–h). The first bending mode appears several times with different frequencies. The effects of different excitations, for example, microseismic impacts, sudden release, nearby rail traffic, and light and heavy road traffic, were analyzed, and a first beam model of the whole bridge was updated (Fig. 10).

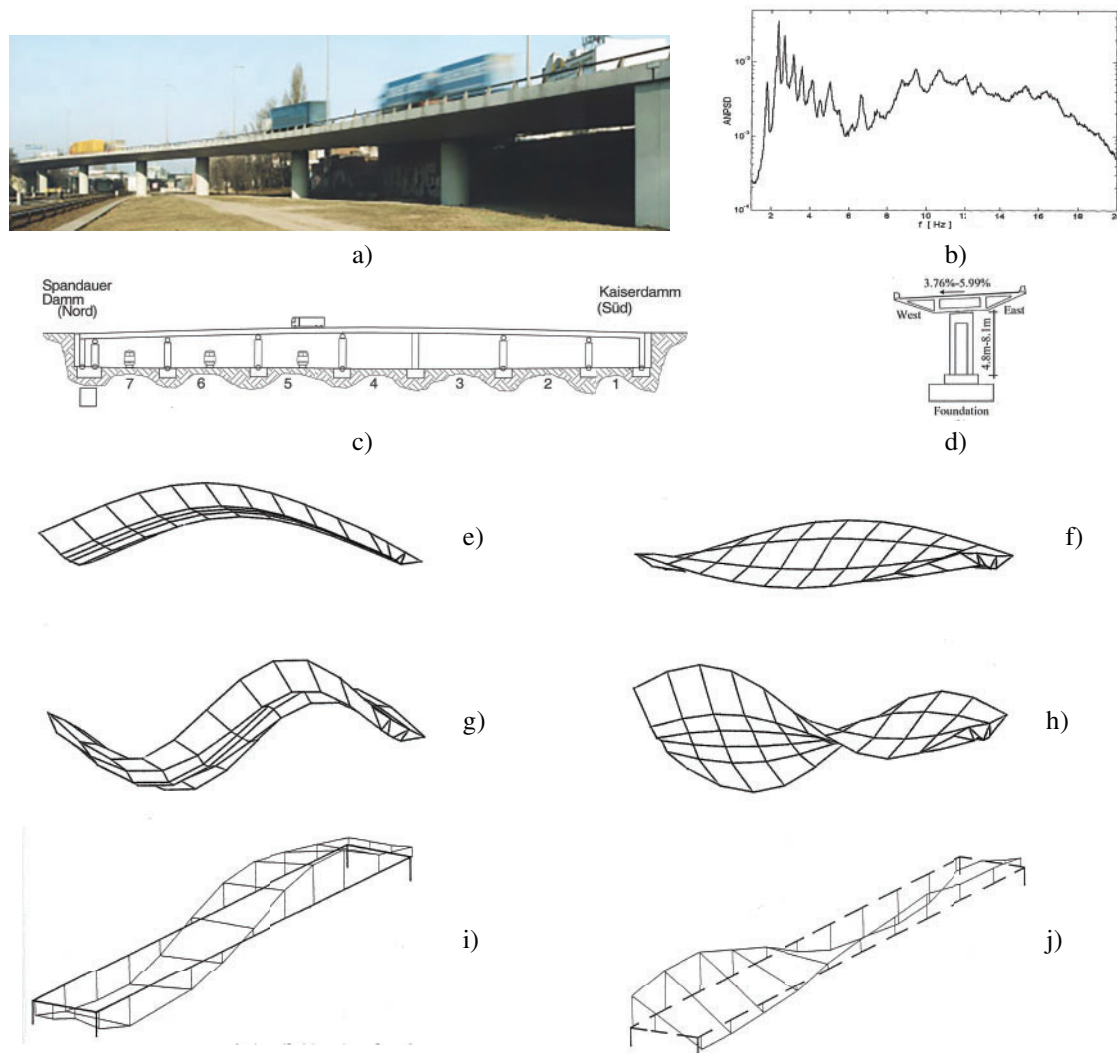


Figure 9. Seven-span highway bridge: view (a,c), cross section (d), averaged spectrum (b), the first four mode shapes of the third span from finite-element calculation (e–h, 2.1, 3.9, 7.1, 10.5 Hz), and the second bending and torsional mode shapes from measurement (i–j, [2.4, 4.6,] 7.6, 11.4 Hz)

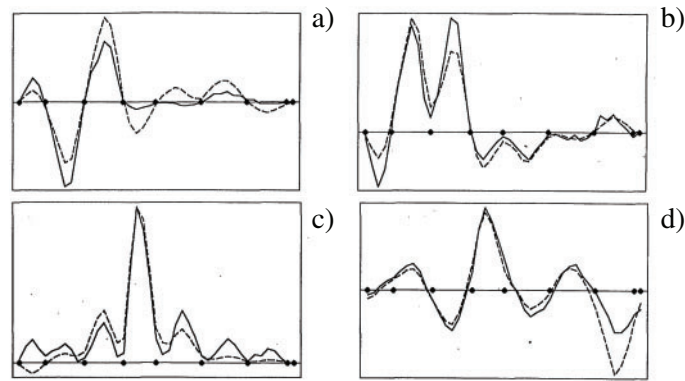


Figure 10. 1D mode shapes of the whole seven-span highway bridge from measurement (continuous line, 2.7, 3.5, 4.6, 6.6 Hz) and calculation

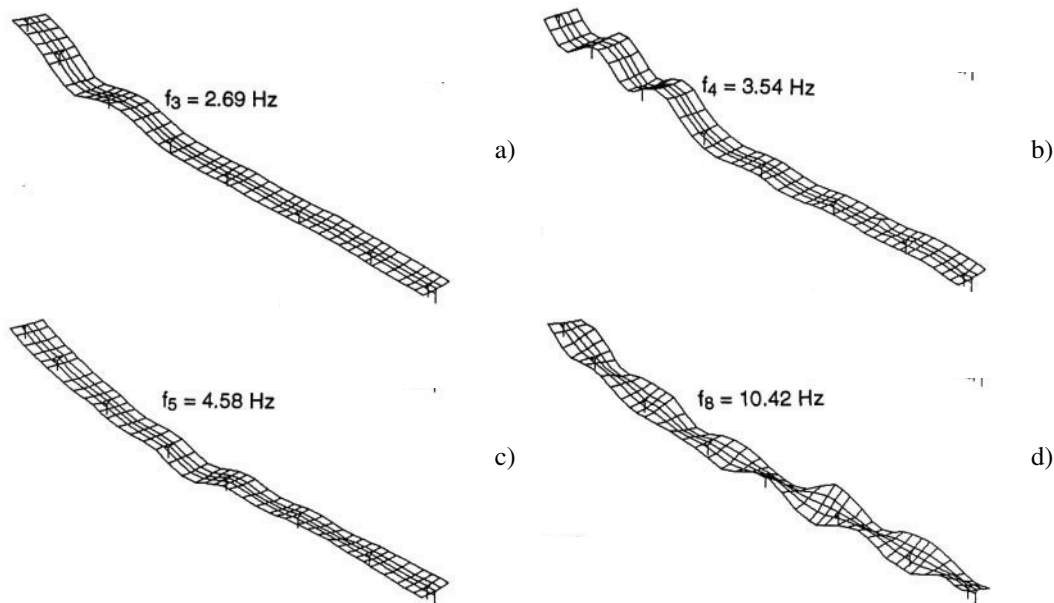


Figure 11. 3D mode shapes of the whole seven-span highway bridge from measurement

A monitoring system was installed and started in 1994. It has been modified and extended several times. It was extended from 16 to 32 and 48 channels, from the critical span 3 to the neighboring spans 1, 2, and 3. Geophones (velocity transducers) are used for the vibrations, eigenfrequencies, and mode shapes. Acceleration sensors and a seismometer were used to check the accuracy at the low-frequency eigenfrequencies. The monitoring software includes the measurement and the immediate evaluation of the raw data.⁵⁰ The characteristics of the vibration, such as max/min values, root-mean-square values, and frequency peaks, are extracted and stored together with selected time segments, single spectra, and averaged spectra. Additional sensors were installed, such as temperature, strain gauges, inclination, and crack-width sensors. A calibration measurement with a 60-t truck has been done, and additional evaluation procedures were implemented for the monitoring of the steadily increasing loads from the road traffic.

Three modal analysis measurement campaigns were performed on the whole seven-span bridge, twice together with

EMPA of Switzerland.⁵¹ The spectrum shows several clearly separated peaks (Fig. 9b). The mode shapes in Fig. 11 show the coupled bending of the different spans, where all spans have alternating minima and maxima at 2.8 Hz, pairs of neighboring spans are in phase at 3.5 Hz, and all spans are in phase at 4.5 Hz. A torsional mode with alternating minima and maxima is found at 10.4 Hz. Note that the three modes at 2.8, 3.5, and 4.5 Hz give almost the same fundamental mode shape at span 3, which often has the highest amplitude of all spans. The different amplitudes of the different spans can be due to the differences in the span lengths of 31 to 38 m, the varying curvature and inclination of the bridge, or the clamped support (rigid pier) between spans 3 and 4, which partly separates the four north spans from the three south spans. Very similar mode shapes were achieved from an updated finite-element model.⁵¹

The stored monitoring data were analyzed for the influence of the traffic loads⁵² and the temperature.⁵³ The monitoring data from 2000 to 2014 were further evaluated by Hu in the context of statistical pattern recognition⁵⁴

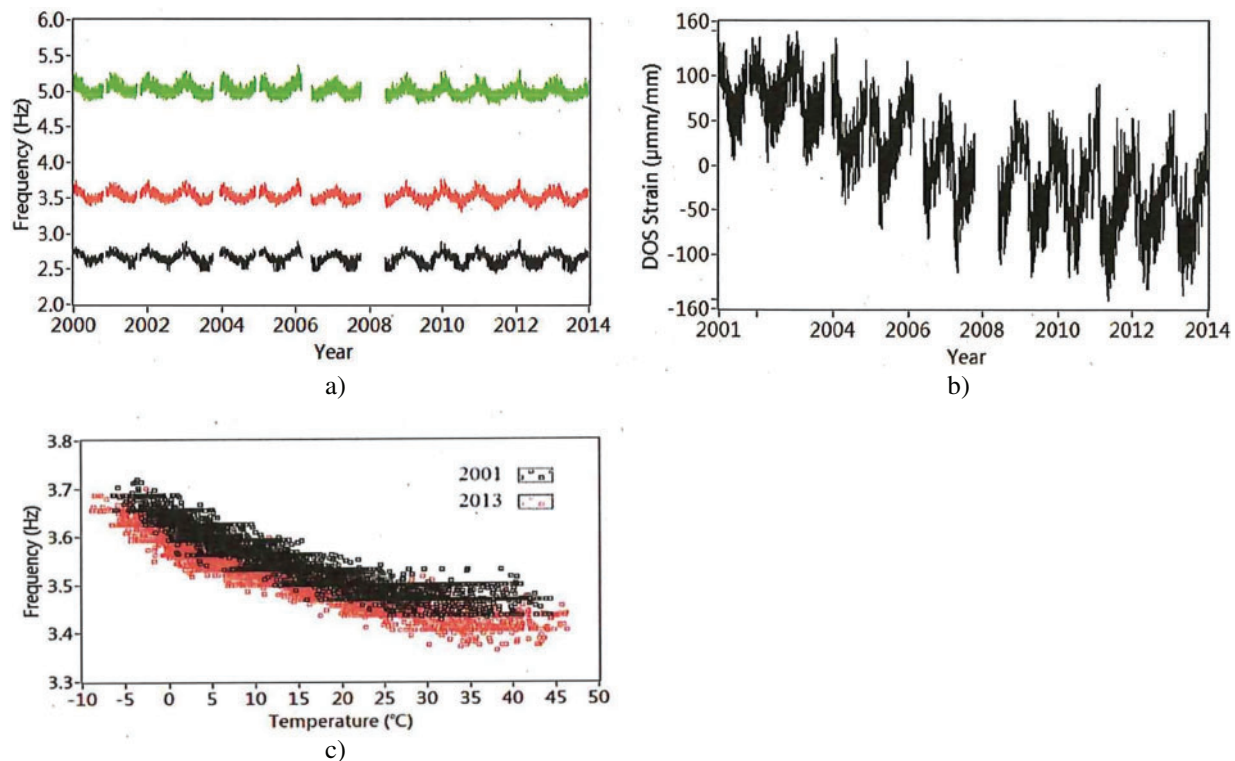


Figure 12. Monitoring data of the Westend bridge: (a) eigenfrequencies, (b) strain of a tendon over 14 years, and (c) influence of the temperature on the eigenfrequency in 2001 (black) and 2013 (red)

(Fig. 12). The influence of temperature was removed, and a trend in some health indices was found, which was related to a loss of prestress. In 2025, the bridge was demolished.

Example 6: Seven-span continuous steel road bridge

The bridge has a length of 264 m and a width of 18 m (Fig. 13a). The main mid-span is 66 m long, and on both sides are three shorter spans of half the length (33 m). There are two steel boxes along the bridge, regularly spaced cross beams, and a concrete deck. A modal analysis was performed after the construction to provide a reference for a later comparison with a possibly damaged state. Five measurement axes with 2×33 geophones were measured to represent the mode shapes. Five examples are shown in Fig. 13c. The first two modes are fundamental bending and torsional modes of the longer mid-span, with small contributions from the nearest side spans. The third mode consists of coupled bending vibrations of both outer spans. The mode at 4.58 Hz is the second torsional mode of the long mid-span and at the same time the first torsional mode of the shorter side spans. The bending mode at 8 Hz has regular minima and maxima at equal distances. The wavelength is half the side span and a quarter of the mid-span, so this mode is the second bending mode for the shorter spans and the fourth bending mode for the long mid-span. At this frequency, the bridge behaves like a bridge with eight equal spans and an additional support at the mid-span.

Example 7: Long three-span concrete rail bridge

The continuous rail bridge (Fig. 14a) has an overall length of 300 m, where the support distances are 135 m for the mid-span and 82 m for the side spans.⁵⁵ The concrete box has a height of 4.5–6 m and supports two railway lines. The monitoring of the bridge during some years includes strains, inclinations, temperatures, acceleration, and 13 geophones along the bridge, which were used to find the vibration modes during ambient excitation and the passages of freight and high-speed trains. The fundamental mode at 1.95 Hz corresponds only to the vibration of the longest mid-span (Fig. 14c). The modes between 3 and 3.5 Hz (Fig. 14d–g) have maxima at the shorter side spans in different combinations, but also some contributions from the mid-span. At 4.85 Hz, the second bending mode of the mid-span was measured. This bridge will be further considered under train passage in the second part of the article.

Example 8: Short two-span concrete rail bridge

The short two-line railway bridge “Arroyo Bracea I” (Fig. 15a) was analyzed by the Spanish colleagues⁵⁶ for the typical resonance effect from high-speed trains. It consists of two simply supported spans of 15.25 m length and 11.6 m width. Each span consists of five concrete beams of 1 m height and a concrete deck of 0.25 m thickness. The modal analysis was done on one of the two spans with 3 axes of 3–4 accelerometers. After stabilization, three modes were found in close neighborhood (Fig. 15b), all of which are first bending modes of the beams. All beams are in phase

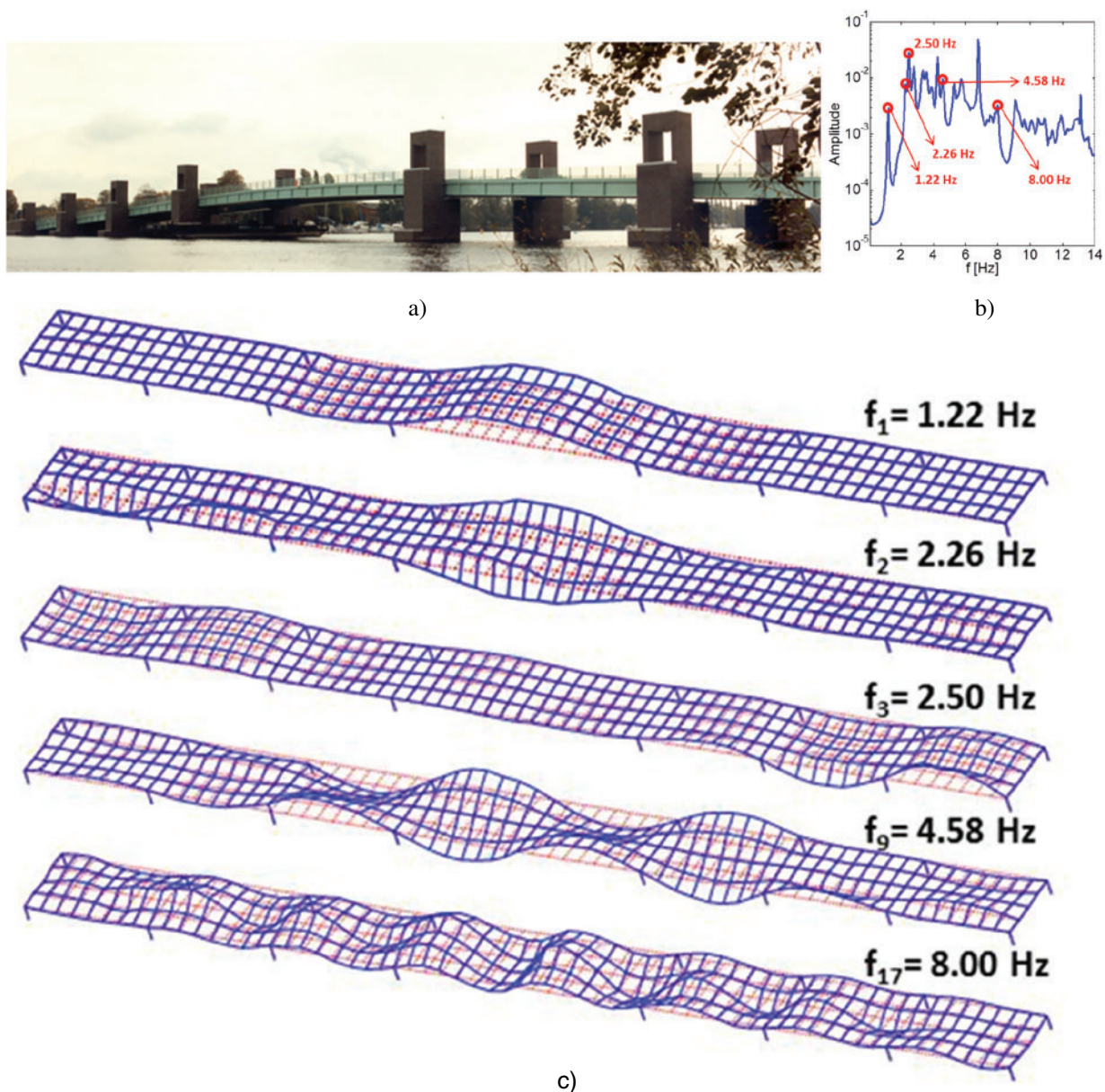


Figure 13. Seven-span regular continuous bridge (a), averaged spectrum (b), and 3D mode shapes from measurement (c)

at 9.25 Hz, the outer beams are out of phase at 10.63 Hz (“torsion”), and the outer beams are in phase while the mid beam is out of phase at 12.75 Hz. The same mode shapes were calculated by two different models of this skew bridge, producing identical or slightly different natural frequencies (see Fig. 10 and Table 4 in Ref⁵⁶).

Summary of observations

The multi-span bridges with n spans generally have n fundamental modes, where the spans contribute with different positive and negative amplitudes.

- In the case of equal, simply supported, weakly coupled spans, a cluster of eigenfrequencies occurs (Example 2).
- The arch bridge (example 3) has nearby frequencies of the first and second bending modes.

- The beam-type bridges (examples 4 and 8) also have a cluster of eigenfrequencies.
- The continuous beams and bridges (examples 1, 5, and 6) show modes with mixed simple and clamped support conditions and more separated fundamental eigenfrequencies.

Theory for the Bridge Resonance from Train Passages

The problem of resonances of single-span or multi-span railway bridges is now addressed. The passage of a train excites the different modes of the railway bridge with (angular) eigenfrequencies ω_j and mode shapes $\omega_j(x)$. The solution for the bridge vibration can be expressed in the frequency

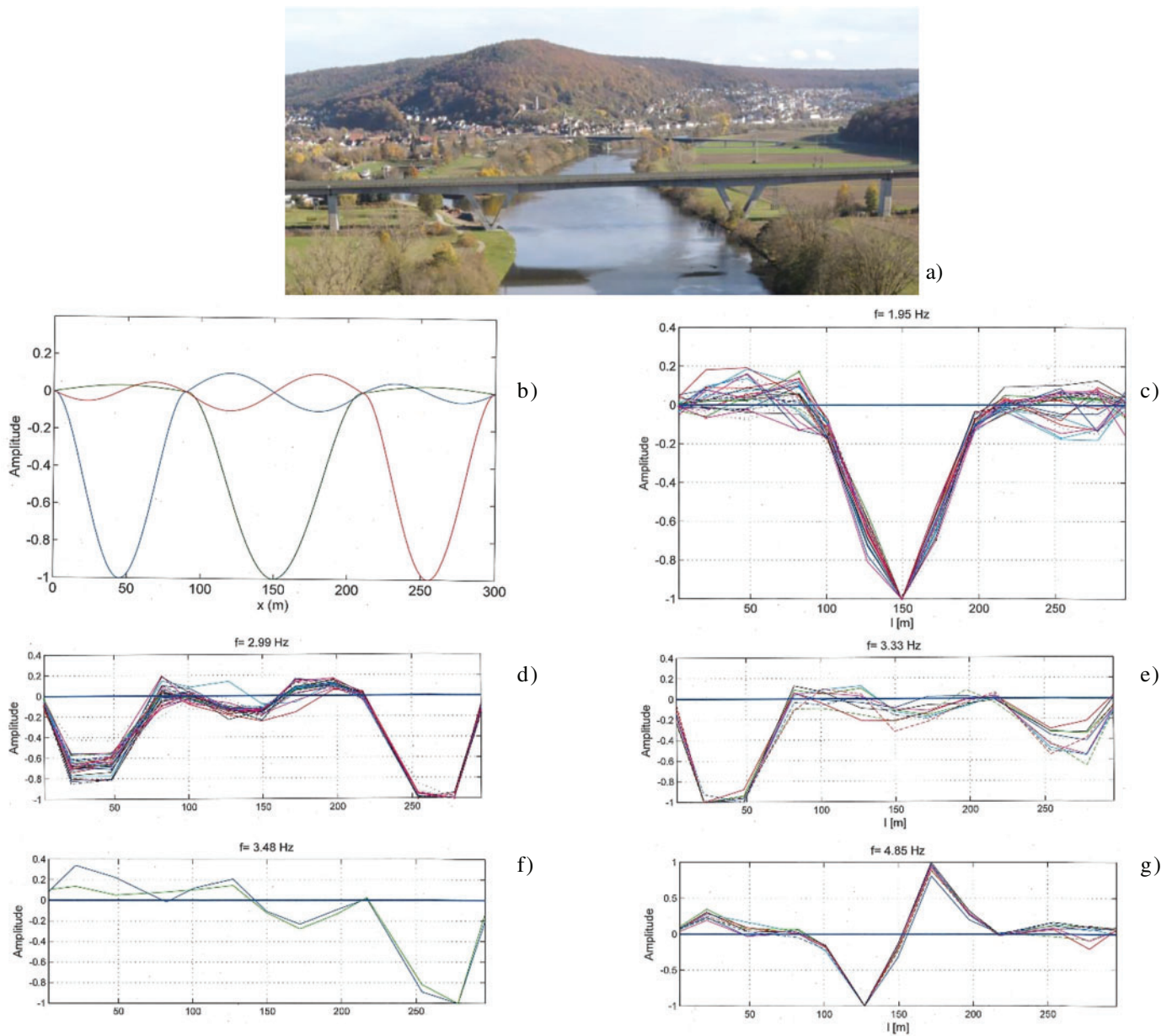


Figure 14. Three-span railway bridge (a), theoretical mode shapes of the first (green) and the second and third (blue and red) eigenfrequencies (b), measured mode shapes of the first eigenfrequency (c, 1.95 Hz), and the next four eigenfrequencies from measurement (d–g, 2.99, 3.33, 3.43, 4.85 Hz)

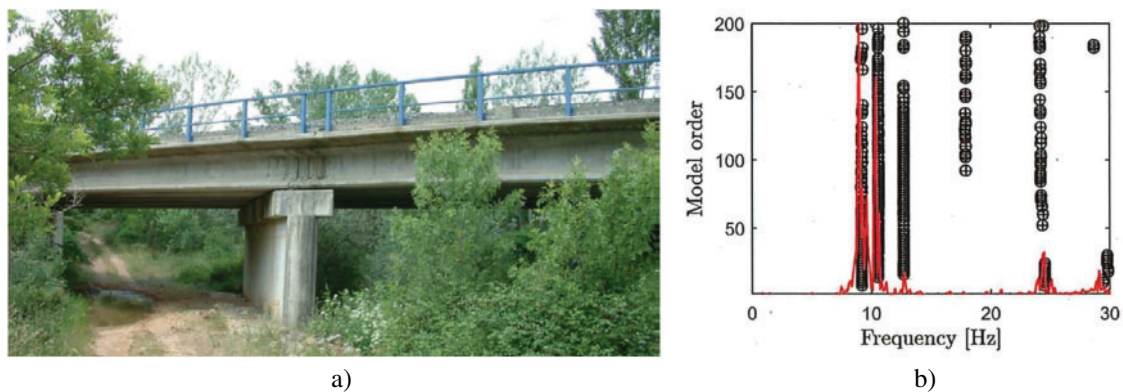


Figure 15. Two-span simply supported bridge (a) and stabilization diagram from stochastic subspace identification (b)

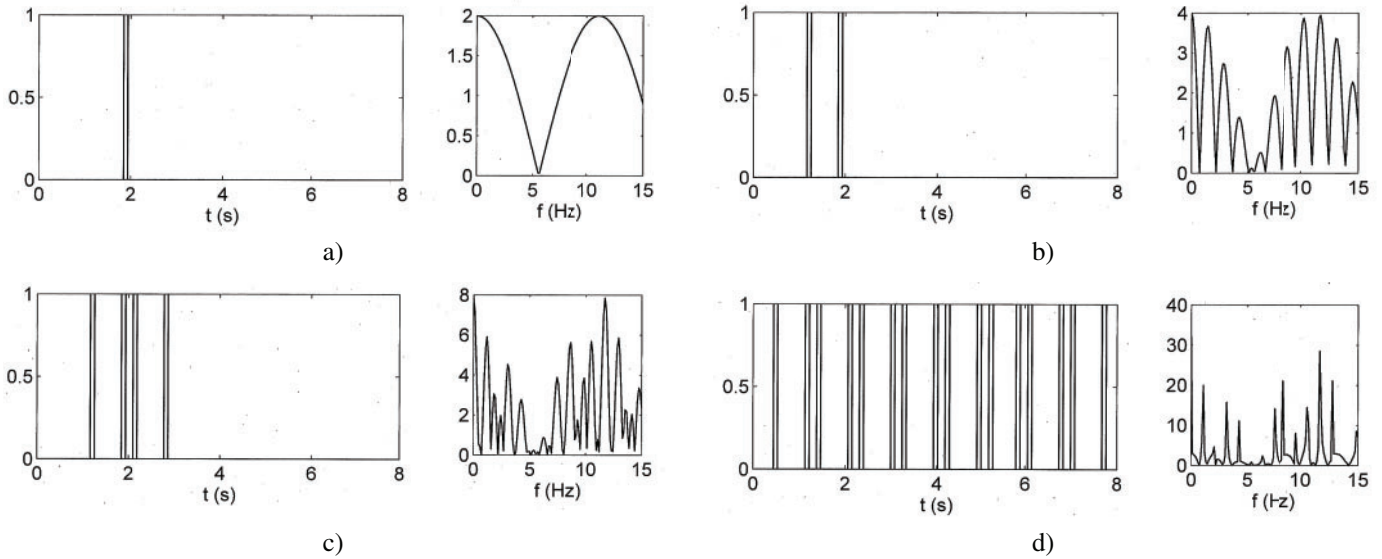


Figure 16. Axle sequence in time and frequency domain for (a) a bogie (axle distance 2.5 m), (b) a car (bogie distance 17 m), (c) two cars (car length 25 m), and (d) eight cars traveling at 100 km/h

domain as the superposition of the eigenmodes, where each contribution is a product of three spectra:

$$v(\omega) = \sum_j v_j(\omega) = F_S X(\omega) \sum_j W_j(\omega) H_j(\omega) \quad (4)$$

where F_S is one static axle load, $X(\omega)$ is the axle-sequence spectrum of the train, $W_j(\omega)$ is the spectrum of the mode shape $\omega_j(x)$, and $H_j(\omega)$ is the corresponding transfer function of the bridge. This result is derived in detail in⁵⁷. It holds for any type of railway bridges: simply supported, elastically supported, integral, single-span, multi-span, and continuous bridges. The time-domain solution is obtained via the inverse Fourier transform.

The axle-sequence spectrum of the train is

$$X(\omega) = \sum_{k=1}^n A_k \exp(-i\omega T_k) \quad (5)$$

where T_k is the delay time of the k th axle and A_k is an amplitude factor compared to the single axle. Fig. 16 shows the axle sequence and the axle-sequence spectrum for a bogie, a car with two bogies, two cars, and for a train with eight cars. The single bogie shows characteristic zeros, the whole car shows many characteristic maxima (and zeros), and these maxima turn into sharp peaks for the whole train.

The mode-shape spectrum is the frequency-dependent part of the modal force spectrum. The modal force in general is

$$F_j(\omega) = \int F'(x, \omega) w_j(x) dx \quad (6)$$

and the modal component F_j of the moving constant load F_S is

$$F_j(t) = \int F_S \delta(x - v_T t) \omega_j(x) dx = F_S \omega_j(v_T t) \quad (7)$$

$$F_j(\omega) = F_S W_j(\omega)$$

in time and frequency domain, where v_T is the train speed. The mode-shape spectrum $W_j(\omega)$ is the Fourier transform

of $\omega_j(v_T t)$, the time history of a single unit modal force. The mode-shape spectrum $W_1(\omega)$ of a single bridge span has characteristic zeros due to the bridge length L or the passage time T , respectively. The mode-shape spectrum of a multi-span bridge consists of the sum of the mode-shape spectra of all bridge spans:

$$W(\omega) = \sum_{k=1}^n W_k(\omega) \exp(-i\omega T_k) \quad (8)$$

In the case of identical bridge spans, this is simply

$$W(\omega) = W_1(\omega) \sum_{k=1}^n A_k \exp(-i\omega T_k) = W_1(\omega) Y(\omega) \quad (9)$$

a bridge-sequence spectrum $Y(\omega)$. This is illustrated for two out-of-phase spans and for two in-phase spans in Fig. 17. As for the axle-sequence spectrum, there are characteristic zeros, and the maxima can become sharper for more in-phase or out-of-phase bridge spans.

The frequency response function $H_j(\omega)$ of the bridge is

$$\frac{v_j(\omega)}{F_j(\omega)} = H_j(\omega) = \frac{i\omega}{m_j(\omega_j^2 + 2D_j i\omega_j^2 - \omega^2)} \quad (10)$$

for the velocity, and its maximum at the resonance frequency ω_j is

$$\frac{v_j(\omega_j)}{F_j(\omega_j)} = H_j(\omega_j) = \frac{1}{m_j 2D_j \omega_j} \quad (11)$$

where D_j is the modal damping, and m_j is the modal mass

$$m_j = \int m' w_j(x)^2 dx \quad (12)$$

The resonance amplification is inversely proportional to the modal mass, the modal damping, and the (angular) eigenfrequency.

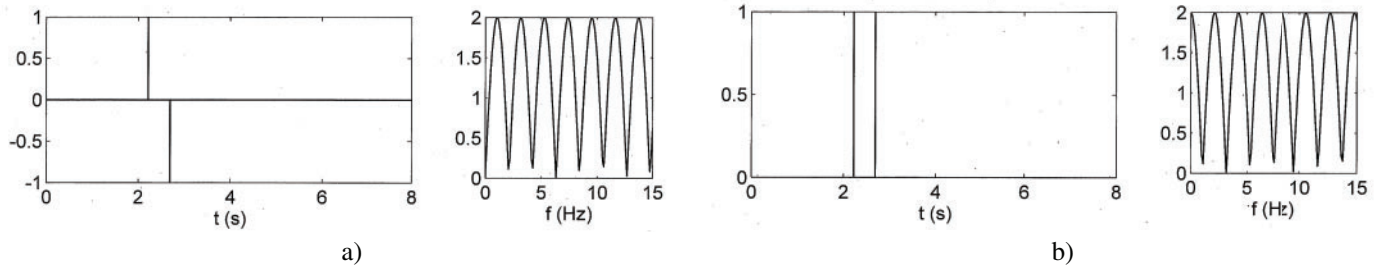


Figure 17. Bridge sequence in time and frequency domain for (a) the out-of-phase mode and (b) the in-phase mode

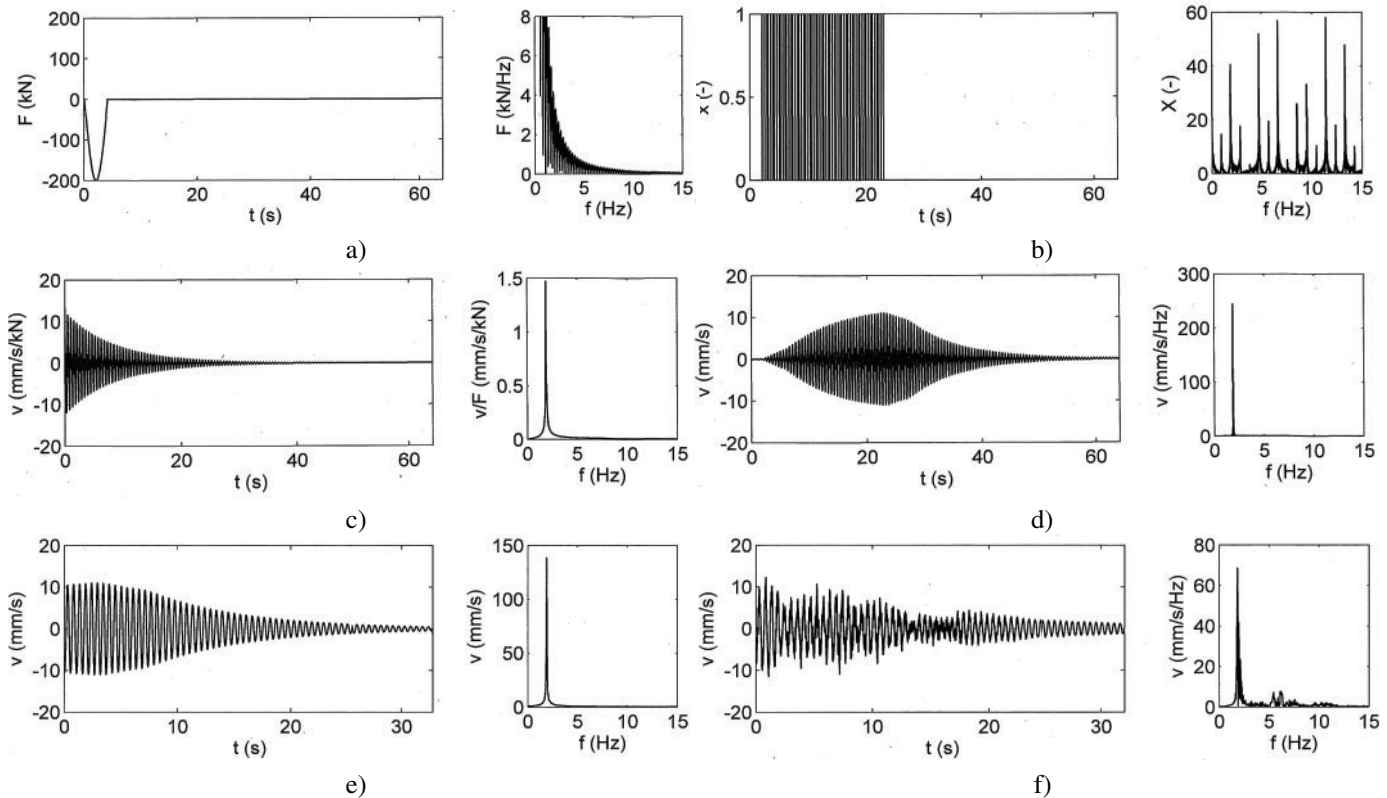


Figure 18. The passage over the long-span railway bridge with $v_T = 100$ km/h, modal force (a), the axle sequence of the twenty 3-axle freight cars (b), impulse and frequency response function of the bridge for $f_l = 1.9$ Hz, $D = 1$ % (c), response of the bridge (d), response of the bridge (partly, e), and measured response (f)

Application to the Rail Bridge Examples

Long bridge with slow freight train

The long rail bridge (example 7; Fig. 14a) has three spans with different lengths and frequencies, and the resonance of the longest mid-span is analyzed as a single-span bridge. The passage of a long freight train at a low speed of 100 km/h over the long bridge lasts about 20 seconds, and a time period of 65.5 s has been investigated. The corresponding modal force (Fig. 18a) shows a passage time of about 2 seconds for a single axle and a spectrum with very narrow minima or zeros. The freight train consists of 20 three-axle cars and has an axle-sequence spectrum (Fig. 18b) with very sharp peaks. If such a peak meets a zero of the modal force spectrum, the cancellation of this resonance would occur. In the present case, the second car-length frequency meets nearly

a maximum of the modal force spectrum, which means an amplification of the resonance. The frequency response function is given for the first bending mode at 1.9 Hz, with a damping of 1 % (Fig. 18c). The mass is at least proportional to the length of the bridge so that the modal mass is high. The resonance of the long-span bridge is reduced by the modal mass, increased by the low resonance frequency, and increased due to the low damping.

The response of the bridge to the passage of the freight train (Fig. 18d) follows from the three spectra in Fig. 6a–c. The spectral density is very high at 240 mm/s/Hz, which is partly due to the very long train. The time history shows an increase during the train passage up to a maximum of 10 mm/s. The following vibration lasts another 20 seconds (Fig. 18e). The resonance with 10 mm/s due to a slow, long

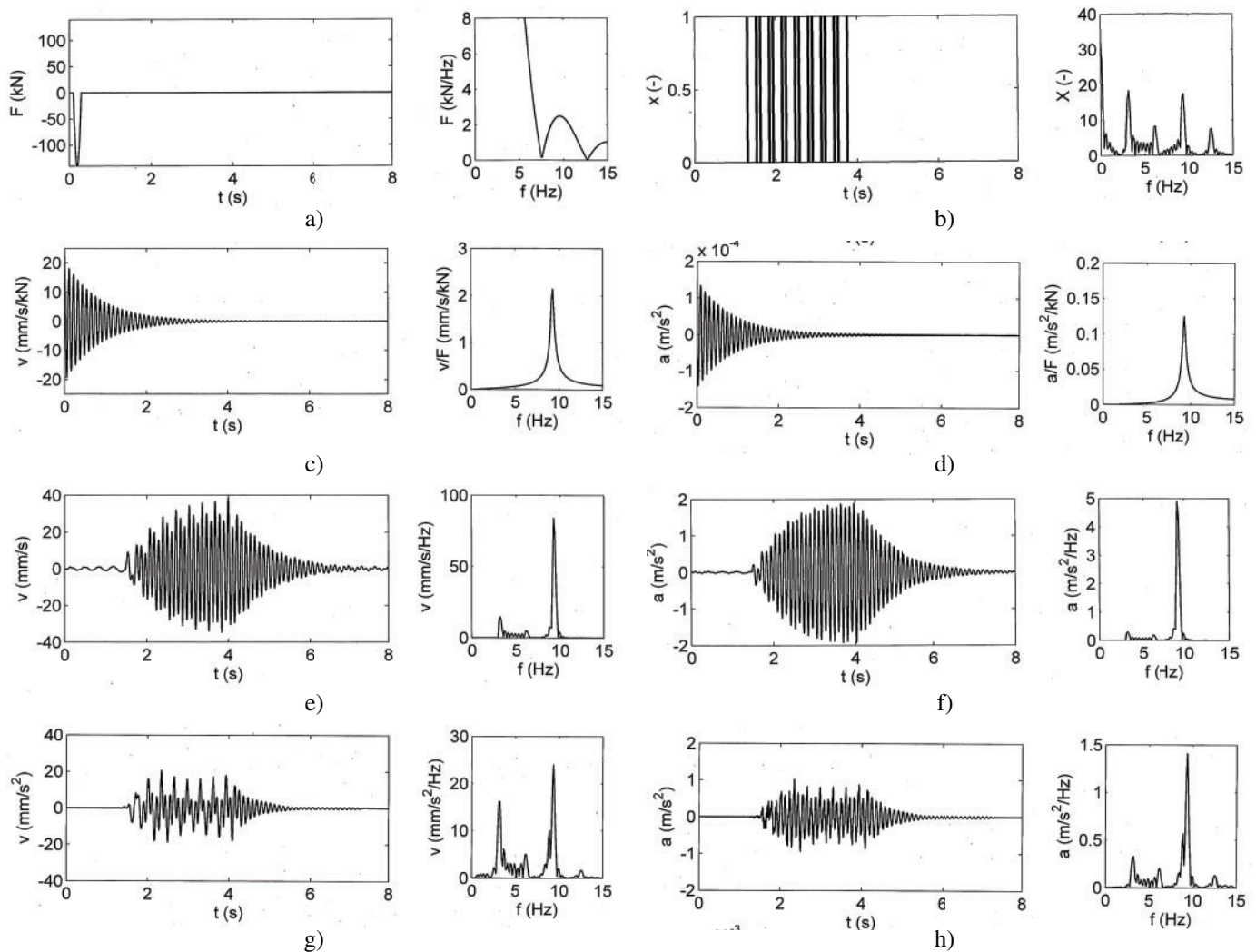


Figure 19. The passage over the short-span simply supported railway bridge with $v_T = 280$ km/h: (a) modal force, (b) the axle sequence of the ICE3 train, (c) velocity response function and (d) acceleration response function of the bridge for $f_l = 9.3$ Hz, $D = 2\%$, (e) velocity response and (f) acceleration response of the bridge, (g) measured velocities, and (h) measured accelerations

freight train on a long-span bridge was also found in the measurements (Fig. 18f).

Short bridge with high-speed train

The Arroyo Bracea bridge (example 8; Fig. 15a) is analyzed for the passage by the Spanish ICE3 train, the S103, at a train speed of 280 km/h. At first, it is analyzed as a single simply supported bridge using the three steps in Fig. 19a–c. The very short passage time yields a high-frequency spectrum of the modal force, where the first side maximum is at about 10 Hz. The axle-sequence spectrum of the ICE3 (Fig. 19b), which is a conventional train with two bogies per car, has a strong third car-length frequency that lies at 9.3 Hz for the train speed of 280 km/h. This is close to the first bending eigenfrequency of the bridge, which is shown in the frequency response function in Fig. 19c for the velocities and in Fig. 19d for the accelerations. The resonance amplification is quite high due to the lower modal mass of the shorter bridge. If all spectra of Figs. 19a–c are put together in

Fig. 19e, the response of the bridge shows high amplitudes of 40 mm/s in the time domain and 80 mm/s/Hz in the frequency domain. This strong resonance builds up during the two seconds of the train passage. The calculated velocity and acceleration responses are compared with the experimental results of the Spanish team⁵⁶ in Fig. 19g and h. The build-up of the resonance is not as strong as in the calculation. The calculated maximum accelerations give higher values of 2 m/s² compared to 1 m/s² for the measurements. The prediction, however, is very similar to the prediction from time-domain calculation in.⁵⁶

The Consequences of Multi-Span Bridges for Rail Traffic

Multi-span bridges of equal or different spans have coupled modes, where other spans contribute more or less to the mode shape of the one dominant span. Therefore, the modal mass of this mode is always higher than the modal mass of

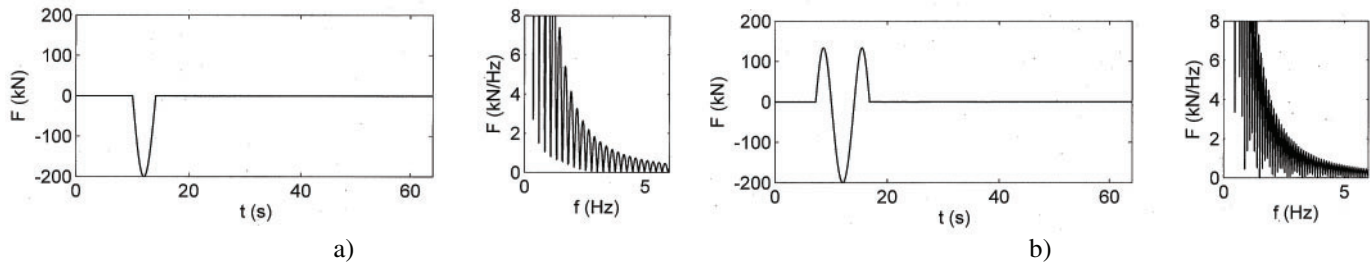


Figure 20. Modal force in time and frequency domain for (a) the long single-span bridge and (b) the long 3-span bridge; 100 km/h train speed

a single-span bridge. In the case of n equal spans, which are all equally present in the mode shape, we have

$$m_{jn} = \int m' w_{jn}(x)^2 dx = n \int m' w_{j1}(x)^2 dx = n m_{j1} \quad (13)$$

where the modal mass is n times the modal mass of the single span. Correspondingly, the maximum amplitude in the frequency response function would be only $1/n$ of the maximum of the single bridge. The modal force, on the other hand, could be higher for the multi-span bridge. The effect on the modal force is analyzed for some examples that are based on the two rail bridges.

Long continuous bridge with three different spans

A 3-span bridge with the dimensions of the long bridge in example 7 (80 m, 120 m, 80 m) is considered. The mode shape of the lowest eigenfrequency (Fig. 20b) is assumed with maxima of 0.67, 1.0, and 0.67 and is compared with the single mid-span mode in Fig. 20a. The corresponding modal force spectra show many maxima due to the length of the bridge and the low speed of 100 km/h. The frequency bands around each maximum are very narrow, especially for the three-span bridge. The envelope of all maxima is similar for the single-span and the three-span bridge. The only difference between the two bridges is the higher modal mass for the three-span bridge, where

$$m_{13} = \int m' w_{13}(x)^2 dx = (1 + 2 \times 0.67 \times 0.67^2) \int m' w_{11}(x)^2 dx = 1.6 m_{11} \quad (14)$$

so that a lower resonance amplitude might be expected for the 3-span bridge. If the contributions of the shorter side spans are smaller, as in the bridge example 6 at 1.2 Hz (Fig. 13), the resonance amplitude is closer to the resonance amplitude of the single-span bridge.

Simply supported bridges with two or more spans

The bridge example from Spain is now idealized as two identical consecutive simply supported bridges. Two fundamental modes are considered here, which have almost the same eigenfrequencies. The corresponding modal force

time histories and spectra are shown in Fig. 21a,b for alternating (out-of-phase) amplitudes and for equal (in-phase) amplitudes. The in-phase mode has a maximum at 10 Hz, which is twice the maximum of a single span (Fig. 19a). The out-of-phase mode has a zero at 10 Hz. The modal mass is twice the modal mass of a single span, so that the effects of the modal mass and the modal force compensate for the in-phase mode, and the resonance is the same as that of a single-span bridge.

If n bridge spans are considered (Fig. 21), n fundamental modes exist. The mode-shape spectra are evaluated at the resonance frequency of the single-span bridge (10 Hz, Fig. 19a). The in-phase modes of the 2-, 3-, and 4-span bridges in Fig. 21b,d,f also have a strong maximum at 10 Hz. The out-of-phase mode of the 3-span bridge (Fig. 21c) has a smaller maximum near 10 Hz, and the out-of-phase modes of the 2- and 4-span bridges (Fig. 21a,e) have a zero at 10 Hz. Besides the zero at 10 Hz, maxima exist above and below that are clearly lower than the maximum for the in-phase mode. Resonances could exist at a lower or higher train speed, but only with a lower resonance amplitude. The same conclusions hold for the mixed mode shapes in Fig. 21g,h. A zero can be found at 10 Hz, and the neighboring maxima are smaller than the maximum of the in-phase mode, which is the most relevant mode with the strongest resonance. The maximum of the in-phase mode-shape spectra (Fig. 21b,d,f) increase proportionally to the number n of spans, so that the effect of the modal mass, which is also proportional to n , is compensated. This means that the resonance of the in-phase mode of the multi-span bridge is the same as the resonance of the single-span bridge. It is expected that these observations can be generalized to all bridges with equal spans.

Multi-span continuous bridges

Similar rules hold for continuous multi-span bridges. Mode shapes that are similar to the mode shapes of the four-span simply supported bridge in Fig. 21e–h have been measured at a four-span continuous highway bridge.⁵⁸ Continuous bridges with more spans can have mode shapes with different amplitudes for different spans (see for example Fig. 1d and Ref^{37,59}). The corresponding effect on the resonance amplitude is analyzed for the most relevant mode, the in-phase mode. The maximum amplitude is 1 and the other

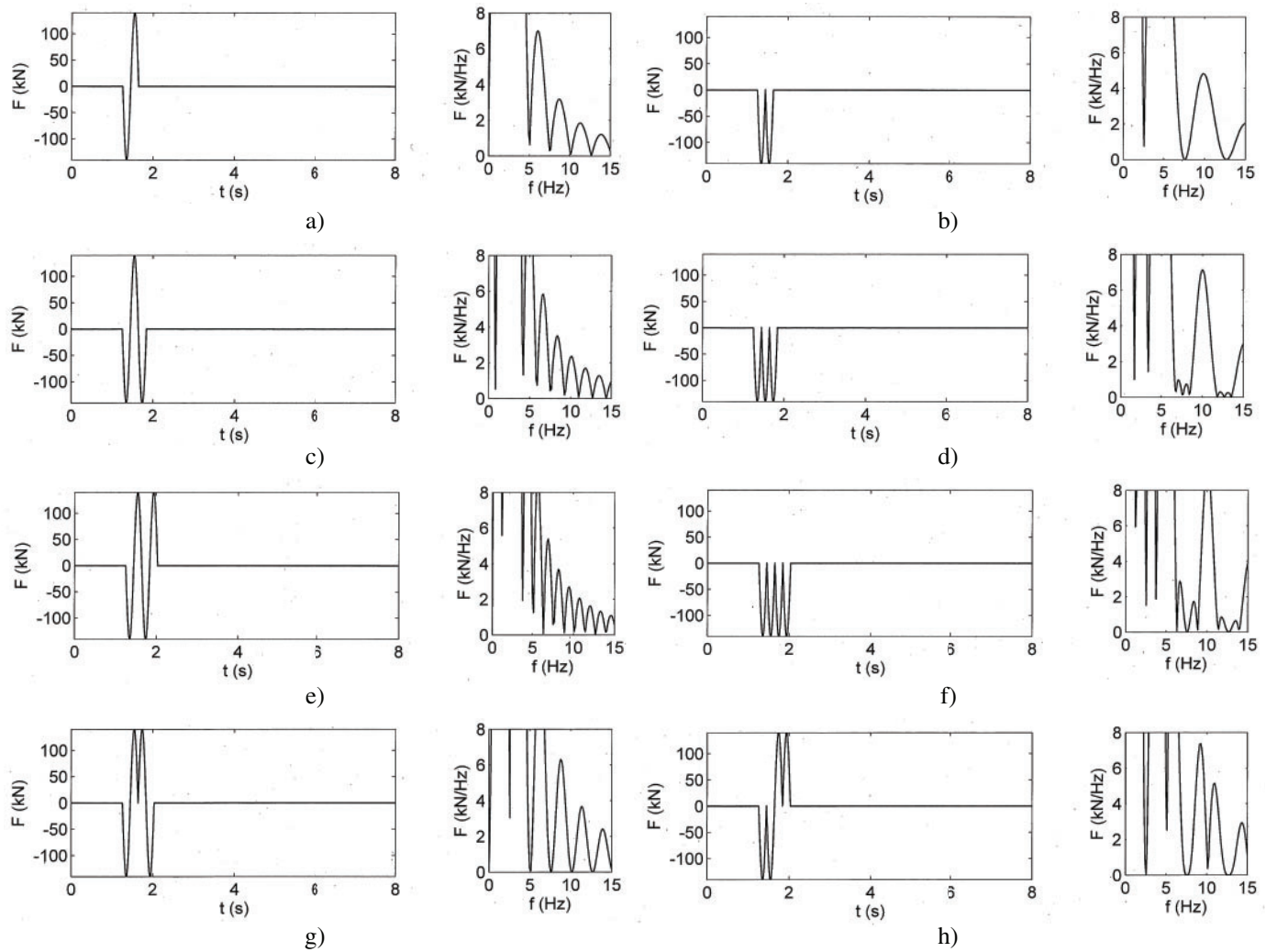


Figure 21. Modal force in time and frequency domain for (a, c, e) the out-of-phase mode of a two-, three-, and four-span bridge, (b, d, f) the in-phase mode of a two-, three-, and four-span bridge, and (g, h) the second and third fundamental modes of the four-span bridge; 280 km/h train speed

amplitudes are $A_k < 1$. The modal mass is

$$\frac{m_n}{m_1} = 1 + \sum A_k^2 < n. \quad (15)$$

and the modal force for the in-phase fundamental mode is

$$\frac{W_n}{W_1} = 1 + \sum A_k < n. \quad (16)$$

The effect on the modal mass is stronger, so that the resonance amplitude is somewhat higher than for a single span.

$$\frac{V_n}{V_1} = \frac{W_n m_1}{W_1 m_n} = \frac{1 + \sum A_k}{1 + \sum A_k^2} > 1. \quad (17)$$

This effect can be studied for the 6-span continuous beam in Fig. 1d. The mid-spans have the highest amplitudes, and the outer spans have the lowest amplitudes. The modal mass is 3.6 instead of 6, and the modal force is 4.5 instead of 6. Finally, the resonance amplification for the 6-span bridge

is 23% higher than for the single-span clamped-clamped bridge. In this case of different amplitudes, the highest fundamental frequency can have a higher resonance amplitude. The ratio between the resonance amplitudes of the multi-span and the single-span bridge depends on the number of spans and the amplitudes A_k . Some calculations have shown that the ratio increases for more spans and smaller A_k . For a specific multi-span bridge, the A_k must be determined from calculated or measured mode shapes. The present rules may help to improve the national and international standard assessment procedures.^{60–63}

For equal amplitudes of all spans, as for the south three-span bridge of example 5 at 3.5 Hz (Fig. 11b) and for the four-span bridge in Ref.⁵⁸ the in-phase mode has similar resonance amplitudes, and all lower fundamental eigenfrequencies have smaller resonance amplitudes compared to the single-span bridge.

Conclusions

Eigenfrequencies and mode shapes of multi-span structures have been analyzed in theory and using eight measurement examples. Whereas a single bridge has only one fundamental mode, multi-span bridges generally have n fundamental modes, where the spans contribute with different positive and negative amplitudes. In the case of equal, simply supported, weakly coupled spans, a cluster of eigenfrequencies occurs. Continuous beams show modes with mixed simply supported and clamped support conditions and separated fundamental eigenfrequencies.

Resonances of railway bridges to train passages have been analyzed in the frequency domain using the axle-sequence spectrum, the transfer function of the bridge, and the modal force (the mode-shape) spectrum. The modal mass increases with the number of spans; the modal force usually does not. Only the in-phase mode has a corresponding increase in the modal force and therefore the same resonance amplitude as a single-span bridge. If the in-phase mode has different amplitudes at the different spans, a higher resonance amplitude is possible. Although higher resonance amplitudes are possible for the in-phase mode, all lower fundamental eigenfrequencies have lower resonance amplitudes than the single-span bridge.

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Behavior of Reinforced Concrete-Filled Steel Tube Columns with External Socket Connections Under Seismic Loading

Julio A. Samayoa^{1,*}; Giorgio T. Proestos² and Mervyn J. Kowalsky³

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Abstract: This study evaluates a new external socket connection for reinforced concrete-filled steel tube (RCFST) columns intended for seismic applications. The connection incorporates a steel-tube discontinuity at the column-to-socket interface, where the main tube is intentionally terminated, and a thinner embedded sleeve is used to reduce moment transfer into adjoining elements and facilitate plastic-hinge formation away from the connection. Two full-scale specimens were tested under reversed cyclic loading to assess global behavior, failure modes, strain demands, displacement capacity, and energy dissipation. The specimens had identical column geometry, while socket height, column concrete strength, and grout strength were selected using an analytical framework to produce two connection configurations with distinct predicted failure modes. One specimen exhibited flexural failure with hinge formation in the column outside the external socket, whereas the other developed shear failure in the portion of the column embedded within the socket, consistent with the respective analytical predictions. Strain measurements indicated limited localized yielding of the external socket wall reinforcement and steel sleeve in the shear-governed specimen, while these components remained elastic in the flexural specimen. No visible damage was observed in the external socket wall or footing in either test, supporting the intended capacity-protected behavior of these components. The results provide experimental support for the analytically predicted failure mechanisms for the two configurations investigated and demonstrate that the tested connections developed the calculated nominal moment capacity of the corresponding cast-in-place columns.

Author keywords: Reinforced concrete-filled steel tube; external socket connection; large-scale tests; seismic design; bridge design

Introduction

Reinforced concrete-filled steel tube (RCFST) columns are widely used in bridge construction, particularly within regions of high seismicity.^{1–4} These systems consist of a steel tube filled with concrete and internal longitudinal and transverse reinforcement, forming a composite section that provides enhanced strength and ductility under lateral loading when compared with conventional reinforced concrete or steel piers.^{5–8} RCFST columns offer several advantages, including their ability to function simultaneously as a deep foundation element below ground and as a column above

ground, the use of the steel tube as permanent formwork that expedites construction, and the high level of confinement provided by the tube, which increases flexural strength and deformation capacity while exhibiting minimal levels of damage.^{1,3,4,6–10} An example of their implementation is shown in Fig. 1 for the O'Malley Bridge in Alaska,¹¹ where the RCFST serves continuously as both the foundation and the above-grade columns.



Figure 1. Example of an RCFST multicolumn bent used in the O'Malley Bridge, AK

*Corresponding author: Julio A. Samayoa.

Email: jsamayo@ncsu.edu

¹Postdoctoral Researcher, Department of Civil, Construction and Environmental Engineering, North Carolina State University, Raleigh, NC, 27695

²Associate Professor, Department of Civil, Construction and Environmental Engineering, North Carolina State University, Raleigh, NC, 27695

³Christopher W. Clark Distinguished Professor, Department of Civil, Construction and Environmental Engineering, North Carolina State University, Raleigh, NC, 27695

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For RCFST pile-column systems, the geometric configuration can vary, with some applications using an oversized below-grade pile relative to the above-grade column, while others adopt a constant diameter for the pile and column.^{6,12,13} Regardless of the geometric configuration, these systems can develop plastic hinges under lateral loading, either below grade or above grade. Under transverse loading in a multi-column bent, a plastic hinge commonly forms at the column–cap beam interface due to the fixed-head boundary, whereas longitudinal loading may induce hinging below grade.^{1,3,4,14,15}

Focusing on the interface between the column and the adjoining structural element, RCFST columns are often terminated just below this element, creating a gap or discontinuity in the steel tube between the column and the adjacent member.^{1,3,4,16} This discontinuity is intended to limit moment transfer, such that the connection to the superstructure is achieved through a reinforced concrete region, thereby facilitating the design of the cap beam or foundation as a capacity-protected element.

For seismic design, it is essential to detail the connection between the RCFST column and the adjoining structural elements so that plastic demands are shifted away from the cap beam or footing, ensuring these components remain capacity protected and that the plastic hinge develops at the intended discontinuity in the RCFST. However, strain penetration will still occur at the interface, increasing the likelihood of cracking in the adjoining elements. These considerations have motivated the study and development of an alternative connection system capable of relocating plastic demands away from protected components while maintaining reliable seismic performance. External socket connections provide a feasible solution by ensuring that inelastic demands concentrate outside the connection region and preserving the capacity-protected behavior of the adjoining structural elements.¹⁷

An external socket connection consists of an external wall integrated with the cap beam or footing, forming a voided opening into which the column is inserted. The column and the external socket wall are bonded using cast-in-place concrete or grout, eliminating the need for additional reinforcement within the connection.¹⁷ These connections simplify the construction of adjoining members compared to internal socket connections by reducing steel congestion in the cap beam or foundation and allowing for smaller cap beam cross-sections, since the void is located outside the structural element. These characteristics make external socket connections convenient for use in cap beams while also complementing their established application in footings for internal socket connections.^{18–22} Despite the advantages of external socket connections, no research to date has examined their seismic behavior when used with RCFST columns, whereas only limited studies have investigated the seismic performance of RCFST columns with internal socket connections.¹⁶

Understanding the system behavior is essential for developing connection details that ensure the plastic hinge will develop outside the connection while preserving the structural integrity of the adjoining elements, achieving seismic

performance comparable to that of a cast-in-place RCFST column. Although the development of an analytical model is beyond the scope of this paper, analytical evaluations informed the selection of key design parameters of the external socket connection, such as socket height (H_s), external socket wall thickness (t_w), and grout strength (f'_g), for the experimental program.²³

This paper introduces a new connection detail for RCFST columns that use external socket connections for seismic applications. The detail mimics the RCFST-to-cap beam connection employed by AKDOT&PF^{4,14} and is adapted for a socket connection. This study evaluates the seismic performance of RCFST columns with external socket connections through two large-scale experimental tests with identical RCFST column geometry but different socket heights, column concrete strengths, and grout strengths. The two configurations were designed to develop distinct failure modes and assess their overall seismic response.

The specimen parameters were selected based on analytical evaluations,²³ which were developed using results from six previous experimental tests of RC columns with socket connections.¹⁷ The previous study identified socket height as a primary design variable, with the required socket height also dependent on the material properties of the column and grout. In the present study, the analytical framework was used to select two RCFST socket connection configurations intended to develop distinct failure modes. Therefore, the specimens were not designed to independently evaluate the effects of socket height, column concrete strength, or grout strength, but rather to experimentally assess the response of RCFST columns for two connection configurations associated with different predicted failure modes. The system performance is quantified in terms of lateral deformation capacity, hysteretic energy dissipation, and observed failure modes. The results provide experimental evidence of the response and failure mechanisms of the two selected connection configurations, including the conditions associated with plastic-hinge formation in the RCFST column outside the external socket connection, and provide insight into the implementation of RCFST columns with external socket connections in seismic regions.

Experimental Test Methodology

Two large-scale tests on RCFST columns with external socket connections were conducted at the Constructed Facilities Laboratory at North Carolina State University. The columns were connected to a footing or inverted cap beam using external socket connections and incorporated a steel discontinuity detail intended to reduce moment transfer into the connection. To implement this discontinuity, the steel tube of the RCFST column was terminated 51 mm before the interface with the external socket, and a separate steel tube with reduced thickness was installed for the portion embedded within the socket, as shown in Fig. 2. This thinner steel tube functioned primarily as formwork and provided confinement but it did not contribute significantly to flexural

strength due to the limited bond between the concrete and the steel tube.⁸

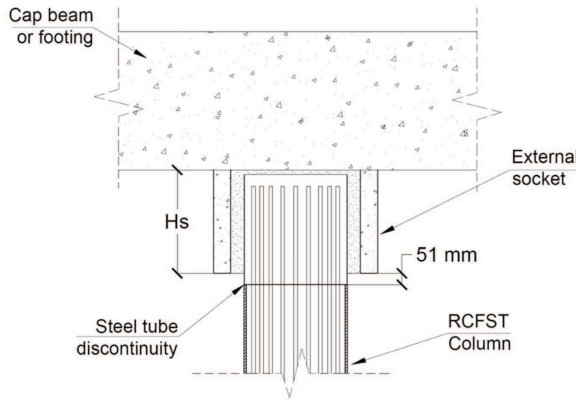


Figure 2. Discontinuity detailing of the RCFST column

For both large-scale specimens, the thickness of the thinner tube was one-third that of the main RCFST column tube, ensuring that the embedded sleeve was weaker than the primary steel tube. This connection detail ensured that the moment transferred to the external socket and footing or cap beam was governed by the confined reinforced concrete section alone, rather than by the full composite capacity of the RCFST column.

The differences between the two experiments are summarized in Table 1 and focus on variations in socket height and the material strengths of the grout and column concrete, while the RCFST column detailing was kept constant for both specimens. The specimen configurations were selected using an analytical framework,²⁴ which relates the socket geometry and material strengths to the nominal flexural capacity of the column through three potential governing mechanisms. The analytical framework is summarized herein through Eqs. (1) to (3) to clarify the basis used for the specimen design. Eq. (1) represents the condition associated with flexural failure of the column, Eq. (2) represents the shear failure mechanism and incorporates the revised UCSD shear model,²⁵ and Eq. (3) represents the failure mechanism governed by the grout. For a given set of geometric and material properties, each equation provides the required socket height associated with its corresponding failure mechanism, and the largest required socket height governs the connection design.

$$\chi \geq \frac{0.6 * P * D_c + 1.2 * M_n}{0.1 * f'_c * D_c^3} \quad (1)$$

$$\chi \geq \frac{M_n}{1.3 * \left(\frac{2 * A_{sp} * f_{yd} * D_c^2}{S} + 0.21 * \sqrt{f'_{cd}} * D_c^3 \right)} [MPa] \quad (2)$$

$$\chi \geq 3.7 * \sqrt{\frac{M_n}{f'_{cg} * D_c^3}} \quad (3)$$

Where P = Axial force acting on the column; Mn = Nominal moment capacity; Dc = Diameter of the column; f'_{cd} = Design concrete compressive strength; f'_{cg} = Design grout compressive strength (Cylinder); f_{yd} = Design yield stress; A_{sp} = Area of spiral for transverse reinforcement;

S = Spacing of transversal reinforcement; and χ = Ratio between socket height and diameter of the column.

Using this framework, two connection configurations were selected to experimentally evaluate distinct expected failure modes: one specimen was designed to develop a flexural failure mode, with the plastic hinge forming in the RCFST column outside the socket connection, while the second specimen was designed to develop a shear-governed failure mode. The nominal flexural capacity of the RCFST column was maintained the same for both specimens, while the socket height and the grout and column concrete strengths were selected using the analytical framework to obtain the targeted response. The analytical design parameters, required socket heights, and expected failure modes for both specimens are summarized in Table 1. Therefore, the variations in socket height and material strengths represent components of each analytically designed connection configuration rather than independent experimental variables.

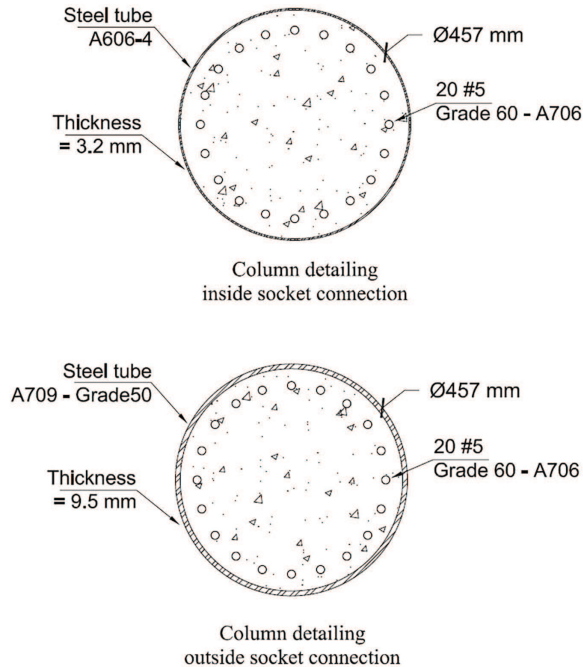
The RCFST column specimens, SJS7 and SJS8, had a clear height of 2286 mm and a diameter of 457 mm, resulting in a height-to-diameter ratio of 5.0. The longitudinal reinforcement consisted of 20 Grade 60-A706 #5 steel bars (16 mm nominal diameter), corresponding to a reinforcement ratio (ρ_l) of 2.4%. The external steel tube was made of A709 Grade 50, which provided confinement and shear capacity to the concrete and had a thickness of 9.5 mm. The concrete core did not contain transverse reinforcement. The clear distance between the outer surface of the longitudinal reinforcing steel and the inner surface of the steel tube was 25 mm.

The portion of the column embedded in the external socket maintained the same longitudinal and transverse reinforcement details as the section outside the socket. However, the steel tube within the socket region was made of A606-4 and had a thickness of 3.2 mm, equivalent to one-third of the thickness of the steel tube outside the socket. Within the socket region, this thin steel tube primarily served as permanent formwork and was not considered to provide additional shear capacity to the embedded portion of the column. Fig. 3 shows the plan view of the RCFST column inside and outside the external socket connection, while Fig. 4 presents the elevation view of the RCFST column detailing.

The longitudinal reinforcement of the external socket wall for both specimens consisted of 20 Grade 80-A706 #6 steel bars (19 mm nominal diameter), anchored to the footing with standard hooks and extended to the level of the bottom reinforcement of the footing. The external socket was encased by steel sleeves made of A36 steel with a thickness of 6.3 mm. The thickness of the external socket wall (tw) was 83 mm, and the gap between the column and the external socket was 25 mm. The concrete compressive strength of the external socket wall was 77.8 MPa for SJS7 and 96.5 MPa for SJS8. The footing and cap beam were designed as capacity-protected components and featured identical detailing for all specimens. Figs. 5 and 6 illustrate the reinforcement details of the external socket connection.

Table 1. Specimen parameters and predicted failure mechanisms

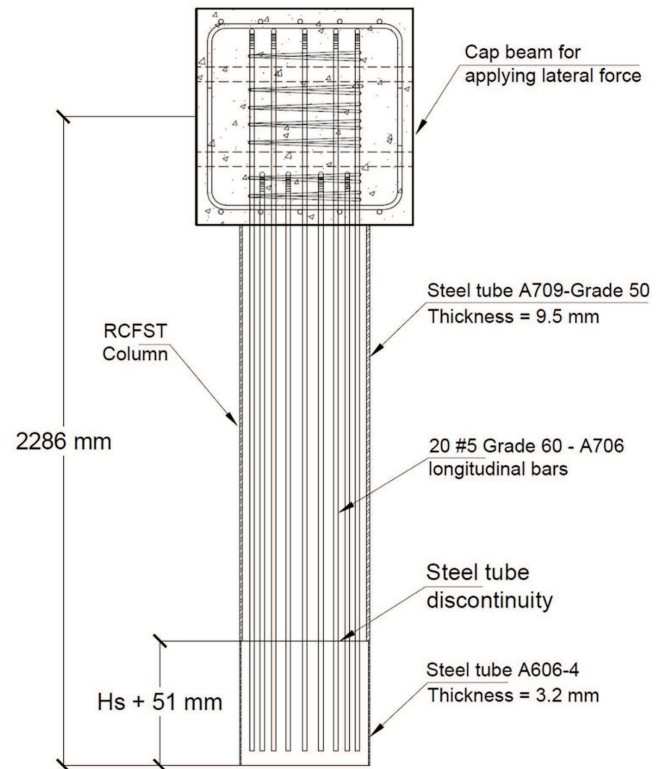
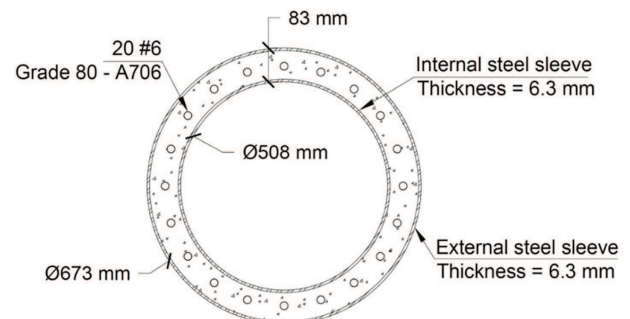
Test specimen	Column diameter [D_c] (mm)	Hs (mm)	Hs(% D_c)	Column f'_c (MPa)	Grout f'_g (MPa)	Predicted failure mechanism
SJS7	457	389	0.85 D_c	38.6	62.5	Shear
SJS8	457	559	1.22 D_c	48.3	40.1	Flexural

**Figure 3.** Plan view of RCFST column

Concrete and grout compression tests were conducted on 100 mm x 200 mm cylinders following ASTM C39.²⁶ This standard was selected because most grout mixes in the experimental program included a coarse aggregate, making ASTM C39 more appropriate for assessing their compressive strength. A constant axial load equal to 5% of the design axial column capacity (343 kN) was applied to all specimens. This percentage value was chosen to maintain consistency with previous tests conducted at NCSU and is considered a lower-bound axial load level representative of minimum gravity load demands.

Test setup and instrumentation

SJS7 and SJS8 were tested under lateral loading using a 978-kN hydraulic actuator with a 1016 mm stroke, secured to a strong wall through a steel plate. The specimens were positioned at the actuator's mid-stroke. An axial load ratio of 5% of the design axial column capacity was applied to both specimens. This axial load ratio was selected to maintain consistency with previous column tests conducted at NCSU, including the six RC columns with external socket connections used in developing the analytical framework, and to facilitate comparison with the existing experimental database. The 5% axial load ratio also represents a

**Figure 4.** Elevation view of RCFST column**Figure 5.** Plan view of external socket detailing

lower-bound gravity demand consistent with the bridge applications considered in the experimental program. A spreader beam supporting two 490-kN hydraulic jacks, placed above the cap beam, was used to apply the constant axial load during testing. A self-regulated axial load system was used to maintain the constant axial force, incorporating a third jack within the Universal Testing Machine operating under force-controlled mode.²⁷

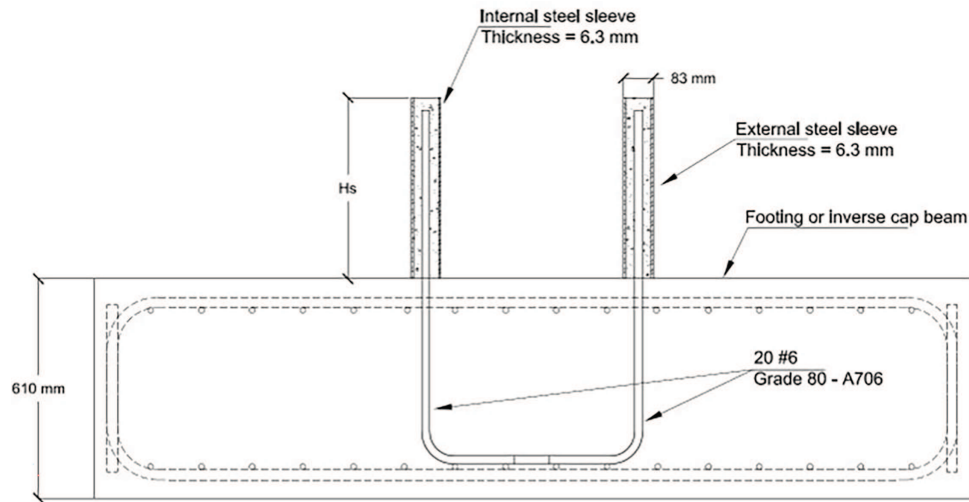


Figure 6. Elevation view of external socket detailing

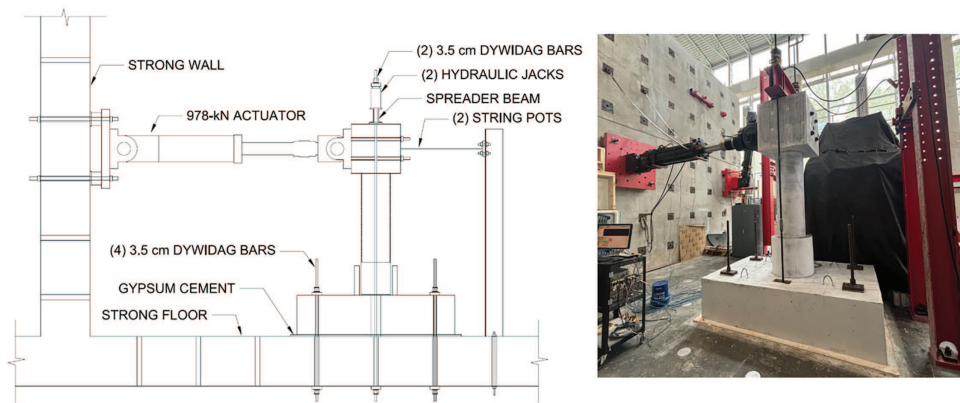


Figure 7. Experimental test setup of RCFST columns

Gypsum cement was applied between the footing and the strong floor to enhance surface contact and friction. Additionally, the foundation was anchored to the strong floor using four 3.5 cm diameter bars, each post-tensioned to 445 kN. Specimen displacement was monitored with two string potentiometers, positioned at the center and edge of the cap beam to detect any potential rotation. Fig. 7 presents the test setup.

Digital Image Correlation (DIC) was employed to monitor the displacement field and strains in the steel tube of the RCFST column and the external socket sleeve. The DIC data was also used to determine the rotation of the external socket and the RCFST column. Additionally, 12 strain gauges were installed on the longitudinal reinforcement within the external socket wall to measure axial strains and verify the transmission of strains to the footing or cap beam.

Loading protocol

The specimens were subjected to a reverse cyclic unidirectional quasi-static load applied at the top of the column. Positive force and displacement indicated a push toward the north direction of the specimen, while negative values represented a pull toward the south direction.

The loading protocol for both experimental tests consisted of a single cycle at 25%, 50%, 75%, and 100% of the theoretical first yield force of the longitudinal column reinforcement. Beyond the yield point, three cycles were applied at progressively increasing displacement ductility levels. Displacement ductility of one was determined by extrapolating the experimentally measured first yield displacement (Δ'_y) to the nominal moment capacity of the cross-section, as defined in Eq. (4), where M'_y represents the theoretically calculated first yield moment of the column section. It is important to note that the measured first yield displacement accounts for various deformation components, including shear and flexural deformation of the column, strain penetration of the longitudinal column reinforcement into the embedded portion within the socket, rocking of the column, as well as shear deformation and rotation of the external socket.

$$\mu_1 = \frac{M_N}{M'_y} \Delta'_y \quad (4)$$

The displacement for any given ductility level “n” was calculated by multiplying the displacement at ductility one by “n,” as shown in Eq. (5).

$$\mu_n = n * \mu_1 \quad (5)$$

The tests continued until the recorded laterally applied force dropped by approximately 40% from the maximum observed value. Extending the lateral loading beyond this point was considered unnecessary, as it would likely not provide additional valuable information and could potentially compromise the safety of the test setup. Fig. 8 presents an example of the loading protocol used in the experimental tests.

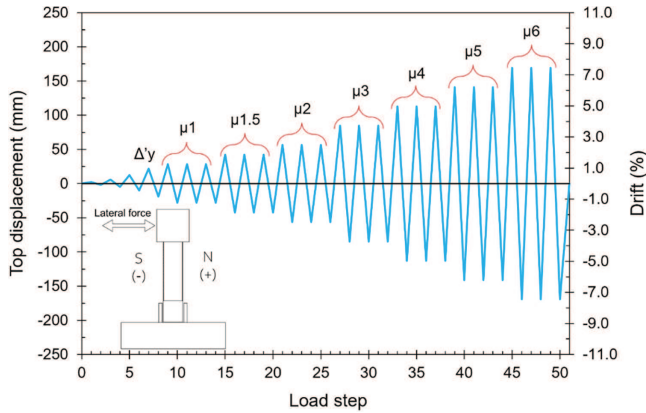


Figure 8. Example of loading protocol

Experimental Results and Discussion

Global response and associated failure modes

The global response of the two RCFST large-scale tests was evaluated using moment-displacement curves. These curves highlight key characteristics such as stiffness, ductility, and strength degradation. Fig. 9 presents the moment-displacement response for both specimens and includes a comparison with the nominal moment capacity of the corresponding CIP RC section at the steel tube discontinuity, calculated using the actual specimen geometry and measured material properties. While no cast-in-place (CIP) column was physically tested in this study, previous large-scale tests on CIP RCFST columns^{1–3,6,14} provide a basis for comparison. These CIP columns typically incorporate a gap at the interface with the footing, similar to the steel tube discontinuity applied in this study. At the discontinuity, the RCFST column transitions to behave as a reinforced concrete (RC) column, with strength governed by the fully confined RC core, independent of any contribution from the steel tube.⁸

While both SJS7 and SJS8 developed flexural plastic hinges in the column region above the external socket connection, their failure mechanisms differed. SJS7 exhibited a shear failure within the portion of the column embedded in the socket after reaching a peak moment of 428 kNm at a displacement ductility of five, which was approximately 4.6% higher than the nominal flexural capacity of a CIP column. This failure mode was identified based on consistency with the observed response of other external socket connection specimens that exhibited shear failure,¹⁷ as well as post-test inspection of the RCFST column within the socket,²³ where distinct shear cracking was observed.

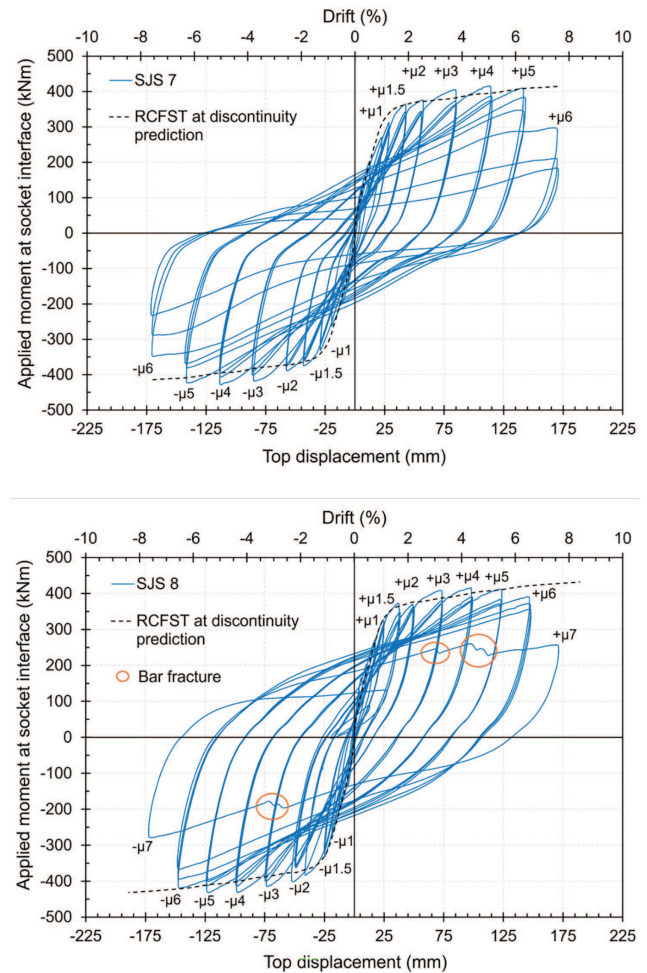


Figure 9. Measured moment–displacement response at external socket interface

As shown in Fig. 9, a drop of the maximum applied moment of approximately 55% occurred during the second cycle of ductility six, coinciding with extensive concrete core crushing and column rocking, leading to the conclusion of the test after the third cycle of ductility six. Although a plastic hinge developed on the column at the socket interface, longitudinal bar fracture did not occur, as the shear failure of the column inside the external socket connection prevented the development of sufficient force to cause bar rupture of the longitudinal steel of the column.

In contrast, SJS8 experienced a flexural failure characterized by longitudinal bar fracture above the socket connection. The specimen reached a peak moment of 432 kNm, which was approximately 3.3% higher than the CIP nominal capacity at a displacement ductility of seven. The observed response is consistent with the behavior reported for other socket connection specimens exhibiting a flexural failure mechanism.^{17,19,20,22} As shown in Fig. 9, a drop of the maximum applied moment of approximately 45% was observed during the first cycle of ductility seven. At this stage, three longitudinal bars fractured on the south side during the first positive cycle of ductility seven, followed by two additional bar fractures on the north side during the first negative cycle, leading to the conclusion of the test. Due

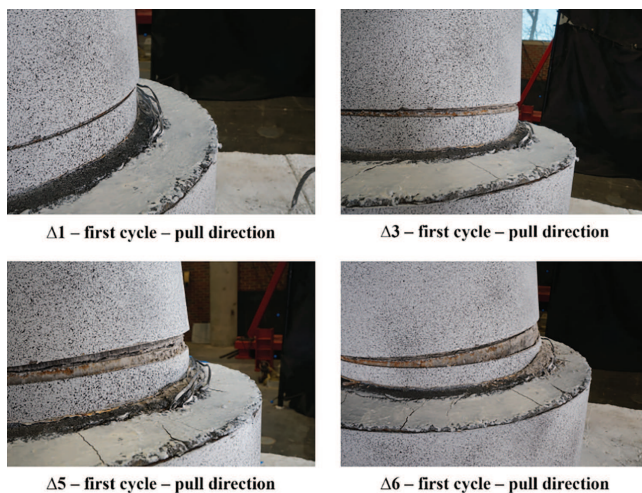
Table 2. Summary of global response of test SJS7 and SJS8

Test specimen	Column Mn at discontinuity (kNm)	Peak moment on column at socket interface (kNm)	Failure mode (reached limit state)	$\Delta \mu 1$ (mm)	Ductility at limit state	Drift at limit state
SJS7 (0.85 D_c)	409.1	428.0	Shear failure of the column inside the connection	28.2	$+\mu 5$	6.2%
SJS8 (1.22 D_c)	418.3	432.0	Flexural failure of the column. Bar fracture	24.4	$+\mu 7$	7.5%

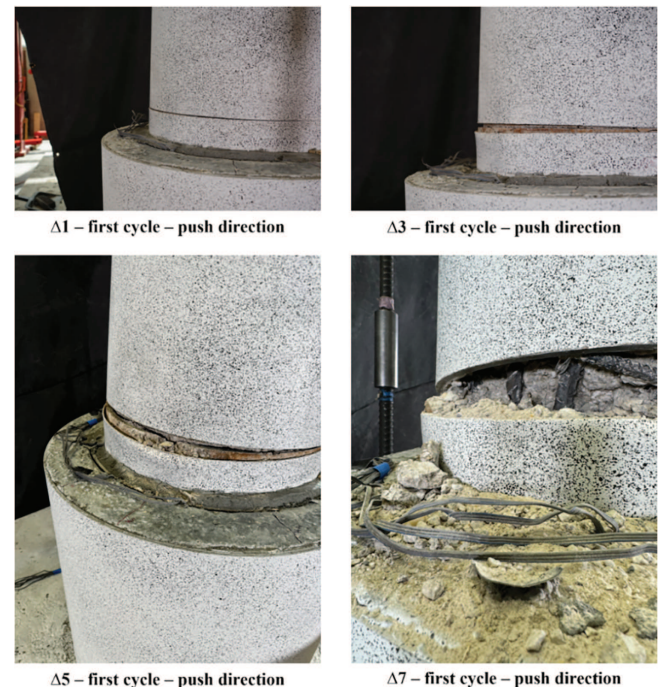
to the external steel sleeve, it was not possible to visually identify which bars fractured during testing.

Despite the differences in failure type, both specimens demonstrated the formation of plastic hinges outside the connection region, confirming the effectiveness of the external socket connection and steel discontinuity detail in enabling the desired inelastic mechanism. Table 2 summarizes the global response characteristics and complements the information presented in the moment-displacement curves.

The damage progression of SJS7 is illustrated in Fig. 10, which presents damage observations at displacement ductility of 1, 3, 5, and 6, focused on the north side of the column at the discontinuity interface during pull cycles. The images highlight the progressive increase in the gap between the steel tubes and the concrete damage of the column. At ductility one, a 4 mm gap was observed between the steel tubes. By the final cycle of ductility two, this gap increased to 9 mm, accompanied by local buckling of the thinner steel tube at the discontinuity interface as it underwent compression.

**Figure 10.** Damage progression for SJS7

As loading progressed, the gap between the steel tubes further increased, reaching 15 mm at ductility three and 22 mm at ductility four. Additionally, during the first cycle of ductility three, a 2 mm crack appeared in the concrete at the interface between the steel tubes. Strength degradation initiated at ductility five, coinciding with core crushing at the

**Figure 11.** Damage progression for SJS8

socket interface. By the final cycle of ductility five, the gap had increased to 30 mm, and visible concrete core crushing was observed at the interface between the steel tubes. During the last cycle of ductility six, the gap expanded to 42 mm, and the concrete core experienced extensive crushing at the interface between the steel tubes, marking the final stage of damage progression before the test was ended.

The damage progression of SJS8 is illustrated in Fig. 11, which presents damage observations at displacement ductility of 1, 3, 5, and 7, focused on the south side of the column during push cycles. The images highlight the progressive increase in the gap between the steel tubes and the concrete damage of the column. At ductility one, a 3 mm gap was observed at the discontinuity interface of the steel tubes. By the final cycle of ductility two, this gap increased to 11 mm, accompanied by local buckling of the thinner steel tube at the discontinuity interface as it underwent compression.

As the test progressed, the gap between the steel tubes continued increasing, reaching 15 mm at ductility three and 22 mm at ductility four. Additionally, during the first cycle

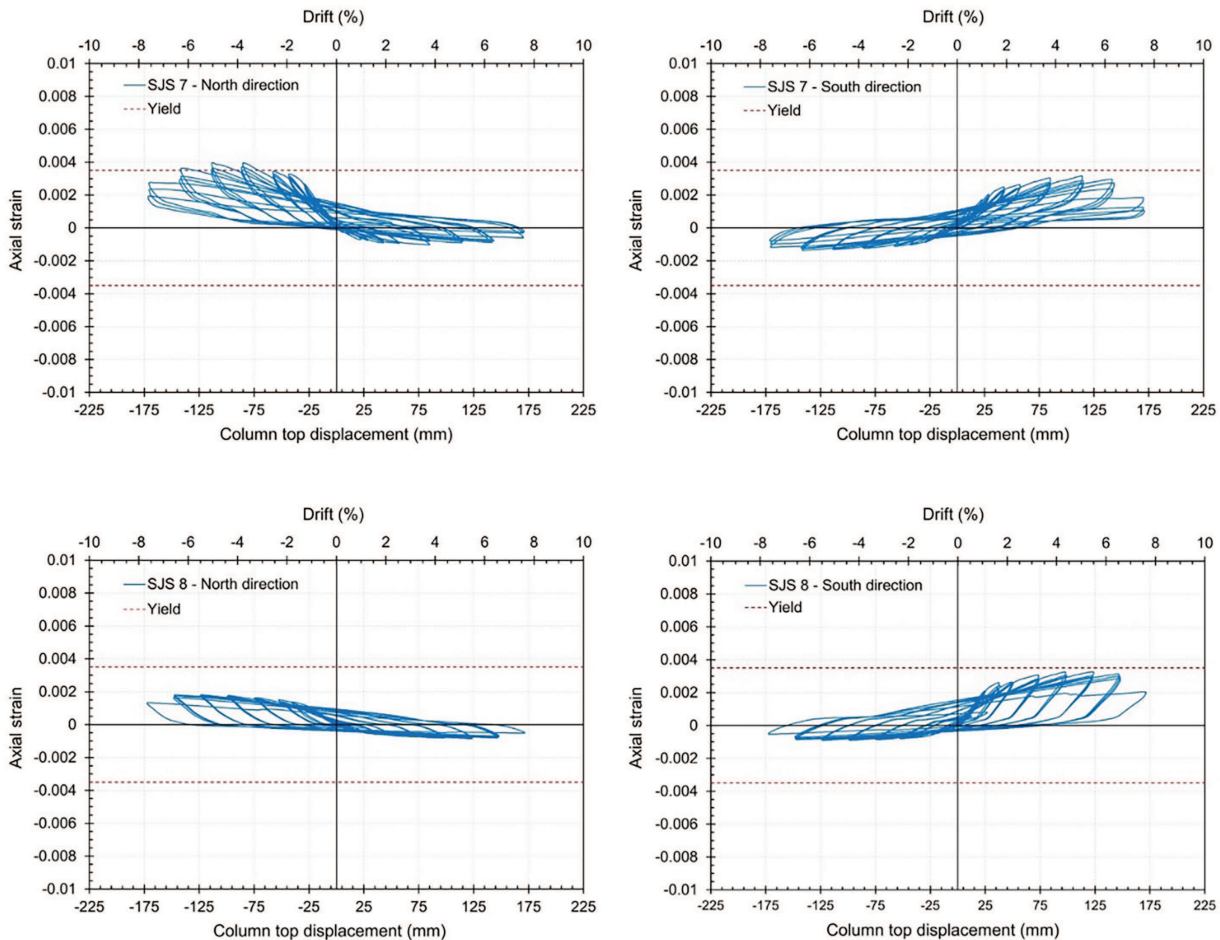


Figure 12. Axial strain of external socket reinforcement

of ductility three, a 2 mm crack appeared in the concrete at the interface between the steel tubes. By the final cycle of ductility five, the gap had increased to 30 mm, and visible concrete core crushing was observed at the interface between the steel tubes. During the last cycle of ductility six, the gap expanded to 42 mm, and the concrete core experienced extensive crushing at the interface between the steel tubes. At ductility seven, the column experienced three bar fractures on the south side and two additional fractures on the north side. Due to the external steel sleeve, it was not possible to directly observe which bars fractured during testing.

The formation of a gap at the discontinuity interface was observed in both tests and is an expected and desirable outcome. This ensures that the steel tube does not contribute to the flexural capacity of the RCFST column at the discontinuity interface and inside the external socket, allowing the forces transferred to the external socket connection to be considered only from the RC column with a fully confined cross-section.

For both specimens, the external socket wall did not sustain significant damage, with only minor radial cracks in the concrete and grout appearing before the first yield displacement of the RCFST column. The inner and outer steel sleeves of the socket did not exhibit any visible inelastic

deformations, and the footing remained undamaged, confirming its role as a capacity-protected element.

Axial strains and associated failure modes

For both experimental tests, the axial strain history of the extreme longitudinal reinforcement bars in the external socket wall was monitored using strain gauges placed 50 mm above the footing. Additionally, axial strains on the external socket sleeve and the steel tube of the RCFST column were determined from the DIC data. The measured strains in the steel sleeve and steel tube reflect different deformation modes: horizontal strains capture the circumferential deformations caused by hoop forces around the external socket, while vertical strains result from the flexural deformation of the socket wall. Tension tests were performed to quantify the stress–strain properties of the reinforcing steel, in accordance with ASTM A370.²⁸ The yield strain of the Grade 80 steel was determined as 0.0035 at the transition between the linear and nonlinear portions of the stress–strain curve, and the yield strain for the steel sleeves was determined as 0.0024.

Fig. 12 presents the strains in the external socket reinforcement along both the north and south sides of the column for both large-scale tests. The strain measurements were consistent with this observation. In SJS7, the external socket wall reinforcement at the interface with the footing

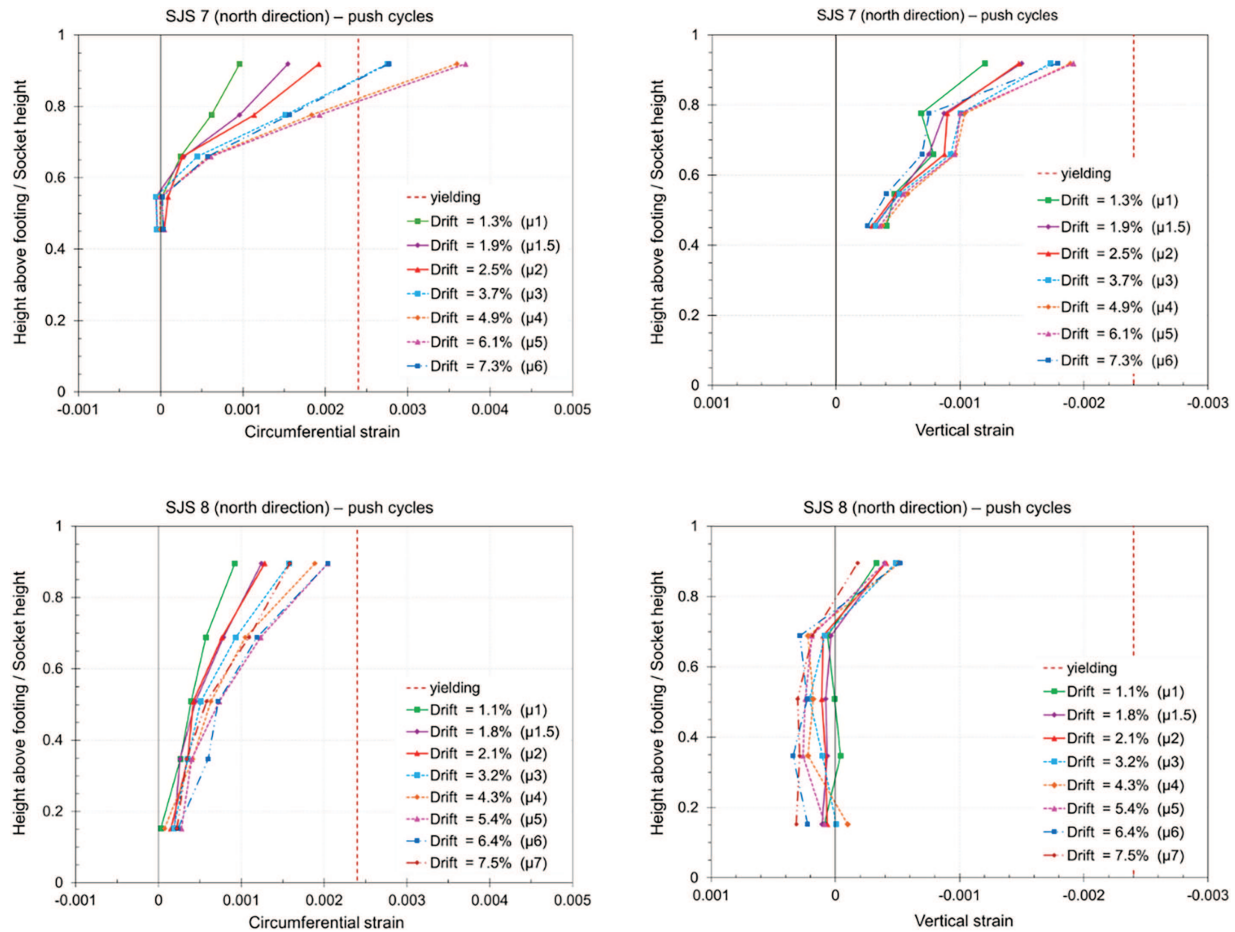


Figure 13. Circumferential and vertical strains on the external socket sleeve

reached approximately 1.15 times the yield strain, indicating localized yielding with limited plastic strain, while the measured strains in SJS8 remained below the yield strain. Therefore, although localized yielding occurred in the external socket wall reinforcement of SJS7, the limited exceedance of the yield strain was consistent with the absence of visible damage to the external socket wall. The external socket connections effectively confined plastic strains to the column, preventing them from extending into the foundation or cap beam, confirming their role as capacity-protected elements.

Following the analysis of reinforcement strains in the socket wall, strain measurements on the steel sleeves of both the external socket and the RCFST columns were evaluated. Fig. 13 presents the circumferential and vertical strains, measured along the height of the external socket sleeve. No visible damage was observed on the sleeves during either test, suggesting that they did not experience significant inelastic deformation.

In specimen SJS7, the circumferential strains reached approximately 1.5 times the yield strain within the top 20% of the socket height, indicating localized yielding of the steel sleeve in this region. Below this region, the measured circumferential strains remained below the yield strain. Vertical strains remained within elastic limits throughout the test,

indicating that the steel sleeve did not significantly contribute to the flexural resistance of the external socket wall.

For specimen SJS8, the highest circumferential strains also occurred in the top 20% of the socket height but remained below the yield strain, indicating fully elastic behavior across the entire sleeve. Vertical strains remained elastic during all loading cycles. Although compressive vertical strains were expected during push cycles, the recorded values were small and within the margin of measurement error, suggesting negligible flexural demand on the steel sleeve.

Fig. 14 presents the circumferential and vertical strains measured in the steel tube of the RCFST columns, just above the discontinuity interface. The data confirm that both circumferential and vertical strains remained elastic throughout the experiment. This behavior confirms that the steel tube above the discontinuity contributed primarily to confinement and shear capacity but did not significantly contribute to the flexural resistance. Consequently, the moment transferred to the external socket connection can be considered equivalent to that of a fully confined reinforced concrete column, because the discontinuity detail effectively isolates the steel tube from participating in the flexural resistance.

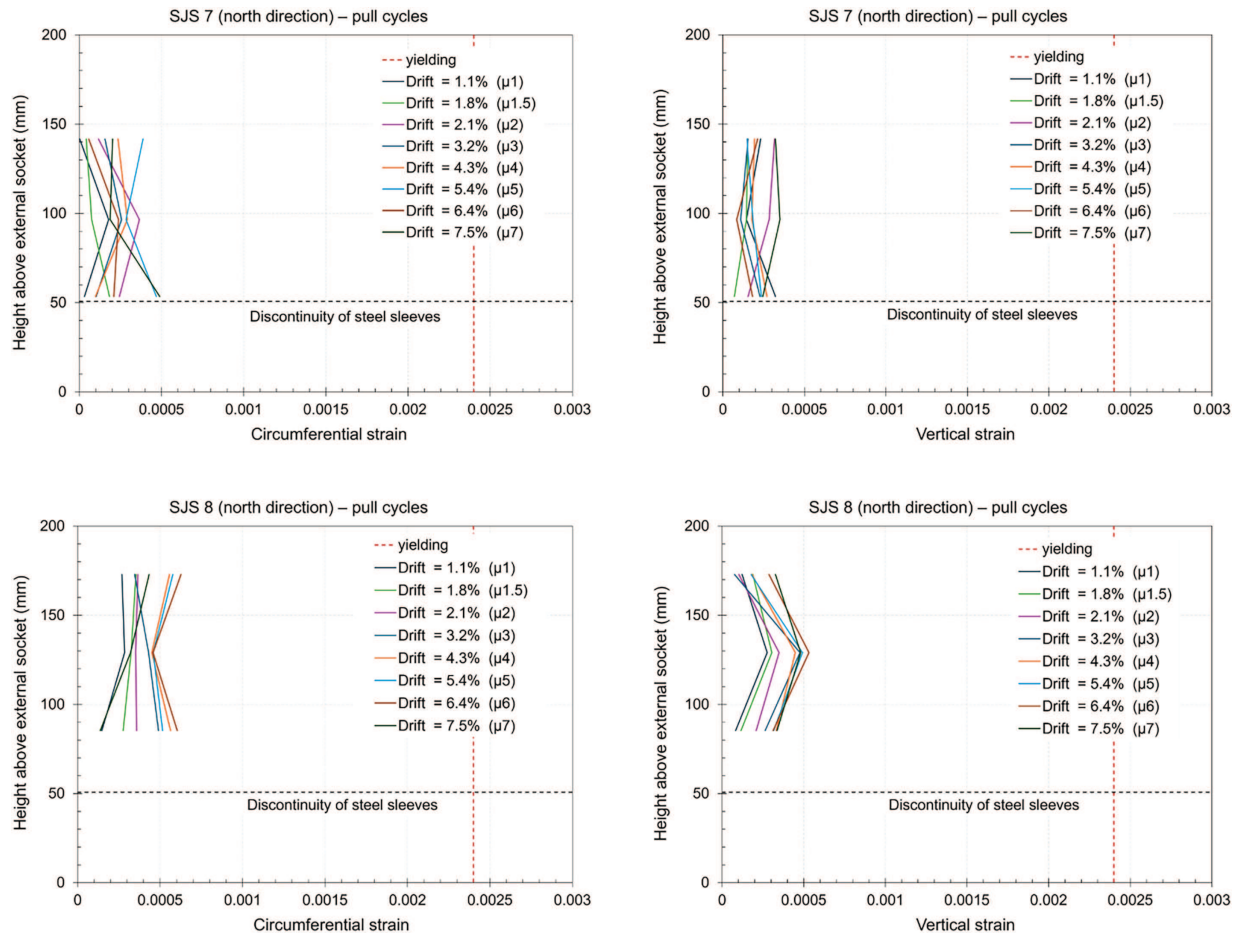


Figure 14. Circumferential and vertical strains on the Steel tube of the column

Hysteretic damping and associated failure modes

Hysteretic damping is a key parameter for evaluating how structures dissipate energy under large inelastic deformations, which is particularly relevant for assessing seismic performance. In this study, hysteretic damping was calculated using Jacobsen's area-based method,²⁹ which defines damping as the ratio of the area enclosed by the force–displacement hysteresis loop to the total elastic energy stored during the same loading cycle. The values reported herein correspond directly to the damping calculated using the Jacobsen method and do not represent inherent structural damping or total equivalent viscous damping.

Fig. 15 illustrates the hysteretic damping values for all cycles across the various ductility levels for specimens SJS7 and SJS8. As expected, damping increased with higher ductility levels, reflecting greater energy dissipation as inelastic deformation progressed. Higher hysteretic damping was observed during the cycles prior to first yield in both specimens compared with the 5% value commonly reported for conventional CIP RC columns and RCFST columns at the location of the discontinuity.^{27,30,31} This increase is attributed to initial cracking of the grout and socket wall, which allowed limited rocking at the connection before the

column reached first yield. The resulting rocking affected the shape and enclosed area of the measured hysteresis loops and is therefore inherently reflected in the damping calculated using the Jacobsen area-based method.

For specimen SJS7, the elastic damping ratio was approximately 14%, representing an increase of about 9% relative to typical CIP columns. For specimen SJS8, the elastic damping ratio was approximately 10%, also exceeding the CIP baseline, indicating enhanced energy dissipation in the elastic range for both cases.

Both specimens exhibited comparable hysteretic damping across ductility levels up to ductility six. At that stage, SJS7 began to experience shear failure and reached a peak hysteretic damping of approximately 28%, whereas SJS8, which exhibited flexural failure, reached a slightly higher peak damping of approximately 31%. The difference in damping behavior at higher ductility levels can be attributed to their distinct failure mechanisms. For SJS7, the increased damping was likely due to enhanced rocking and localized damage following shear failure. In contrast, for SJS8, the energy dissipation was primarily due to progressive flexural damage in the concrete and the eventual fracture of the column longitudinal reinforcement.

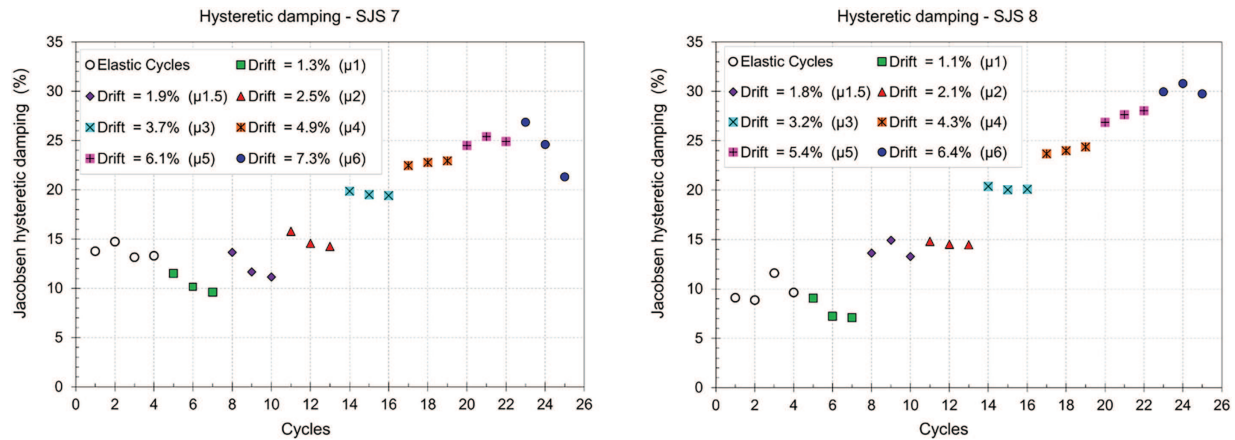


Figure 15. Hysteretic damping for all cycles of each ductility displacement

Summary of results and discussion

The experimental results from the two large-scale tests provide insight into the seismic behavior of RCFST columns using external socket connections. Both specimens developed a plastic hinge in the column outside the external socket connection; however, their failure modes and overall performance differed.

SJS8 demonstrated the preferred seismic behavior, with a flexural failure mechanism characterized by the fracture of the column longitudinal reinforcement above the external socket. This confirms the successful development of the plastic hinge outside the connection and represents a preferred and repairable failure mode. In contrast, SJS7 experienced a shear failure within the portion of the column embedded in the socket. Although a hinge formed above the connection, shear failure led to excessive rocking and strength degradation before bar fracture could occur, consistent with the observed damage and shear failure mechanisms reported in other external socket connection tests.¹⁷ This failure mode is difficult to identify during loading because the damage is confined to the embedded portion of the column within the socket and is most clearly confirmed through post-test inspection, thereby compromising post-earthquake inspection and reparability.

The experimental program was intentionally designed to produce distinct failure modes to evaluate the seismic behavior under different parameter combinations. The predicted failure mechanisms were established based on prior research on external socket connections¹⁷ and an analytical framework,²³ and the corresponding design variables for each specimen are summarized in Table 1. The analytical framework, summarized by Eqs. (1) to (3), was used to select combinations of socket height and material strengths associated with the targeted failure mechanisms. SJS8 was designed with a combination of socket height, column concrete strength, and grout strength intended to prevent the shear failure mechanism from governing and allow flexural failure to develop outside the socket connection. In contrast, the parameter combination selected for SJS7 was intended to produce a shear-governed response within the embedded

portion of the column. The experimentally observed failure modes were consistent with these analytical predictions.

In both specimens, a physical gap formed at the steel discontinuity interface and widened with increasing ductility, consistent with the intended interruption of the steel-tube contribution to the RCFST flexural capacity at the column-to-socket interface. Within the socket, the embedded steel tube primarily served as permanent formwork and was not considered to provide additional shear capacity, consistent with the analytical design assumption that the main steel tube did not provide continuous flexural resistance into the socket connection. Strain measurements showed localized yielding of the external socket wall reinforcement and steel sleeve in SJS7, with measured strains slightly exceeding the corresponding yield strains and indicating limited localized plastic demands, while these components remained elastic in SJS8. Despite the localized yielding in SJS7, no visible damage was observed in the external socket wall or footing, supporting the intended capacity-protected behavior of the socket connection.

Both specimens also demonstrated effective energy dissipation, with hysteretic damping during cycles prior to first yield reaching approximately 14% for SJS7 and 10% for SJS8, compared with the approximately 5% value typically reported for CIP columns. The higher values are attributed to early cracking of the grout and socket wall and the resulting limited rocking at the connection. As inelastic deformations progressed, the hysteretic damping calculated using the Jacobsen method increased further, with SJS8 reaching a peak of approximately 31% and SJS7 reaching 28%, indicating considerable energy dissipation in both specimens despite their different failure modes. The measured moment capacity of both specimens exceeded the calculated nominal moment capacity of the corresponding CIP column, with SJS7 and SJS8 reaching approximately 5% and 3.3% above the calculated values, respectively. These results indicate that, when properly detailed with appropriate socket height and material strengths, the external socket connection can develop the nominal flexural capacity of the corresponding CIP column.

These findings provide experimental support for the use of the analytical framework to select combinations of socket height and material strengths associated with different failure mechanisms. For the two configurations investigated, the selected combinations of socket height, column concrete strength, and grout strength resulted in observed failure modes consistent with the analytical predictions. The results therefore emphasize the importance of considering these parameters together when designing the external socket connection to promote plastic hinge formation outside the connection and prevent a shear-governed response within the embedded portion of the column.

Conclusions

The two large-scale tests demonstrated that the tested RCFST columns with external socket connections developed the calculated nominal moment capacity of the corresponding CIP column, although the specimens developed different failure mechanisms. A key outcome of the study was the successful implementation of a new discontinuity detail at the column-to-socket interface intended to reduce moment transfer into the connection. This was achieved by terminating the RCFST steel tube before the interface and introducing a separate, thinner steel tube segment inside the socket. The embedded steel tube primarily served as permanent formwork and was not considered to provide additional shear capacity to the embedded portion of the column. The thinner embedded tube was selected to remain weaker than the main RCFST tube such that, upon contact between the two steel tube segments at the discontinuity, premature buckling of the main tube would be avoided. A gap formed at the discontinuity during testing, consistent with the intended interruption of the RCFST flexural capacity at the column-to-socket interface, such that the flexural contribution of the main steel tube was not transferred into the socket connection. The external socket wall reinforcement and steel sleeve exhibited limited localized yielding in SJS7 and remained elastic in SJS8. Despite the localized yielding, no visible damage was observed in the external socket wall or footing. Both specimens also exhibited considerable hysteretic energy dissipation. The hysteretic damping calculated using the Jacobsen method reached approximately 14% for SJS7 and 10% for SJS8 during cycles prior to first yield and increased with inelastic deformation to peak values of approximately 28% and 31%, respectively.

The observed failure modes were consistent with the predictions of the analytical framework used to select the socket height and material strengths. SJS8 developed the intended flexural failure mode above the socket, including fracture of the column longitudinal reinforcement and formation of a plastic hinge, whereas SJS7 developed the predicted shear failure in the embedded portion of the column. These results provide experimental support for the analytical framework for the two configurations investigated and demonstrate the importance of considering socket height and material strengths together when establishing the governing connection failure mechanism.

The experimental findings are limited to the two connection configurations investigated and an axial load ratio of 5% of the design axial column capacity. Although the analytical framework accounts for axial load in the connection design, the response of the proposed connection at higher axial load ratios was not experimentally evaluated. In addition, both specimens used an embedded sleeve thickness equal to one-third of the main RCFST steel tube thickness; therefore, this ratio should not be interpreted as a minimum or optimized design value. Further experimental investigation is required to evaluate the connection under higher axial demands and to establish the influence and appropriate thickness of the embedded sleeve.

Overall, the two tests provide experimental evidence that the proposed external socket connection and steel-tube discontinuity detail can develop the intended flexural or shear response predicted by the analytical framework while protecting the external socket wall and footing from visible damage. The results provide experimental support for the proposed connection concept within the range of parameters investigated and establish a basis for further validation of RCFST external socket connections for seismic bridge applications.

Acknowledgments

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Data Availability Statement

The data, models, or code that support the findings of this study are available from the corresponding author upon reasonable request.

Supplemental Materials

The authors have declared that there are no supplemental materials associated with this manuscript.

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