

Assessment of existing concrete bridge girder for reuse based on Acoustic Emission data from load testing

Master thesis

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Preface

This thesis marks the final stage of my Master's degree in Civil Engineering at Delft University of Technology. I am deeply grateful to all those who supported me throughout this journey.

First, I would like to thank the members of my thesis committee: Yuguang Yang, Fengqiao Zhang, Oswaldo Morales Napoles, and Rik Cederhout. I am especially grateful to my chair, Yuguang Yang, for helping me find a thesis subject that matched my interests. I particularly appreciated his willingness to provide insightful feedback and to discuss my questions. I also extend my sincere thanks to my daily supervisor, Fengqiao Zhang, for assisting me with the planning, critically assessing my results, sharing her insights on the findings, and discussing all the questions I had. Furthermore, I would like to thank Oswaldo Morales Napoles for his role as my third supervisor, providing critical feedback, especially on probability. Lastly, I would like to thank my company supervisor, Rik Cederhout, for his expertise in structural design and for providing critical feedback on the results.

Furthermore, I would like to thank Nobleo Bouw & Infra for supporting me during this thesis, and all my colleagues who were available to discuss specific aspects of the work. Their input helped me to correctly implement the design codes and to improve my research.

I would also like to thank Nebest, Strukton, and Mistras for sharing the data from their tests and for providing information on how they were conducted and what the results were.

I would also like to note that, in preparing this report, I used AI-based tools to assist with writing and identifying sources. These tools helped me to find literature that was difficult to locate on my own, to check spelling and grammar, and to refine passages that were harder to read.

Working on this thesis has been a valuable opportunity to deepen my understanding of structural engineering in practice and to bridge the gap between academic research and real-world applications.

Lastly, I would like to thank my family and friends for their support and encouragement throughout this journey. I am grateful for the conversations we had about the research, in which their thoughtful questions inspired me to investigate further. In particular, I would like to thank Robin, who was always available to discuss my options and to support me whenever I needed it.

*Timo Heij
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Abstract

Cement production is a major contributor to global warming, responsible for around 6% of annual greenhouse gas emissions. Rijkswaterstaat aims to operate fully circularly by 2030, which makes high-value reuse of structural elements an attractive strategy to reduce CO₂ without compromising safety. This thesis investigates whether non-destructive proof-load testing with acoustic emission (AE) can provide sufficient evidence to justify circular reuse of prestressed concrete girders that fail current code checks on paper.

The work develops and applies a framework that combines code-based resistance models from Eurocode 2:2004 and the next-generation Eurocode 2:2023, a controlled proof-load protocol with AE monitoring, and a probabilistic reliability analysis. The case study concerns reclaimed inverted T-girders without effective shear reinforcement. At the shear control section, the governing ultimate shear effect is $V_{ULS} = 613.13$ kN, while code checks give $V_{Rd,c} = 417.11$ kN (EC2:2004) and $V_{Rd,c} = 357.37$ kN (EC2:2023). On this basis alone, reuse at consequence class 3 over 100 years would be rejected.

In the test, one girder was proof-loaded to 550 kN without irreversible damage under the green light criteria and then taken to failure for research purposes. AE processing, using a data-driven peak-frequency threshold and a calibrated wave speed, revealed first flexural activity around 462 kN and first shear-type activity around 589 kN, ahead of visible cracks. The girder ultimately failed at an applied load of 1046 kN, corresponding to a sectional shear of about 875.1 kN at the control section, highlighting a significant gap between sectional code predictions and the observed global capacity.

AE also enabled back-calculation of the effective prestress from the first flexural cracking, refining it from 2507 kN to 2686 kN. Reliability was quantified using Monte Carlo simulation over a 100-year horizon. Without updating, the 100-year reliability indices are $\beta_{100} = 2.81$ (EC2:2004) and $\beta_{100} = 2.76$ (EC2:2023), well below the CC3 target of 4.27. Conditioning on survival at 550 kN and incorporating the AE-informed prestress update raises the reliability index only to $\beta_{100} = 2.98$ (+0.17) (EC2:2004) and $\beta_{100} = 2.88$ (+0.12) (EC2:2023), still below the target.

The main conclusion is that AE-assisted proof-load testing substantially improves observability and strengthens the evidence for reliability assessment, but for this girder type, probabilistic reliability model, and load model it does not, by itself, achieve CC3-level reliability at full capacity. For circular reuse, a different resistance model in probabilistic framework or additional measures are required, such as demand reduction in the receiving structure, selective strengthening, or refined resistance modelling calibrated to a broader test set. Future work should focus on validating AE with digital image correlation, optimizing proof-load resistance based on traffic light criterion, optimizing instrumentation, and quantifying model discrepancy between code predictions and observed behaviour.

Contents

Preface	i
Abstract	ii
1 Introduction	1
1.1 Background and motivation	1
1.2 Objective of the research	2
1.3 Scope of the research	2
1.4 Research question	2
1.5 Outline of the report	2
2 Literature review	3
2.1 Flexural behaviour	3
2.1.1 Design according to Eurocode 2:2004	3
2.1.2 Design according to Eurocode 2:2023	4
2.2 Shear behaviour	4
2.2.1 Historical background and early approaches	4
2.2.2 Shear crack development without shear reinforcement	5
2.2.3 Shear transfer mechanisms	6
2.2.4 Mechanics-based models	8
2.2.5 Comparison between STM, CSCT and CSDT for prestressed concrete	9
2.2.6 Design code evolution	9
2.2.7 Shear-specific stop criteria	10
2.2.8 Key parameters	10
2.3 Monitoring and measuring techniques	11
2.4 Acoustic emission (AE)	14
2.4.1 Noise filtering	14
2.4.2 AE data analysis	15
2.4.3 AE in inverted T-beams	18
2.4.4 Shear failure analysis	18
2.5 Reliability analysis	20
2.5.1 Statistical evaluation of beam capacity based on sample testing	20
2.5.2 Updating the reliability index using acoustic emission	21
2.6 Research Gap	21
3 Methodology	23
3.1 General framework	24
3.2 Explanation framework	24
3.2.1 Material properties	25
3.2.2 Shear load and resistance	25
3.2.3 Update reliability index	25
3.2.4 Update prestress level	26
3.3 Application in the case study	26
4 Case Study	27
4.1 Experimental Overview	27
4.2 Tested girders A9	27
4.2.1 Geometry girders	27
4.2.2 Prestressing layout	29
4.2.3 Reinforcement layout	29
4.2.4 Material properties	29

4.3	Structural behaviour of prestressed girders	30
4.3.1	Shear behaviour	30
4.4	Measuring Techniques	32
4.4.1	Acoustic emission	32
4.5	AE data processing	33
4.5.1	AE hits	33
4.5.2	AE source localisation	33
4.6	Update reliability index	34
4.6.1	Probabilistic model	34
4.7	Loads in the use phase	36
5	Results	37
5.1	Acting loads on the structure	37
5.1.1	Bending moment	38
5.1.2	Shear effects	38
5.2	Resistance girder Eurocode	40
5.2.1	Shear resistance Eurocode 2 2004	40
5.2.2	Shear resistance Eurocode 2 2023 8.2.2	40
5.2.3	Shear resistance Eurocode 2 2023 I.8.3.1	41
5.3	Girder resistance test	42
5.3.1	Bending resistance	43
5.3.2	Prestress level	45
5.3.3	Shear resistance	46
5.3.4	Localisation of cracks	49
5.4	Resistance girder failure test	53
5.5	Probabilistic reliability girders	53
6	Discussion	58
6.1	Framework	58
6.2	Acting loads and control section selection	59
6.3	Code based resistance: 2004 versus 2023 models	59
6.4	AE-assisted proof test	59
6.5	AE hit rate vs Historical Index	60
6.6	Proof load outcome and failure test findings	60
6.7	Probabilistic reliability	61
6.8	Use RBK 1.2 for next generation Eurocode	62
6.9	Reduce load effect	64
6.10	AE contributions	65
6.11	AE-based reliability updating	66
6.12	Effect of testing additional girders on assessed reliability	67
6.13	Reflection on correlation between variables	68
6.14	Limitations	68
7	Conclusion & recommendations	70
7.1	Research question	70
7.1.1	Acoustic emission	70
7.1.2	Reliability girder	71
7.1.3	Assess reliability by load testing using acoustic emission	71
7.2	Recommendations	72
	Bibliography	73
A	Appendix Probabilistic reliability script	78

1

Introduction

1.1. Background and motivation

Climate change is one of the most pressing issues of our time. Human activities, such as the burning of fossil fuels and the production of materials like steel, cement, and plastics, have significantly increased the concentration of CO_2 in the atmosphere. This excess carbon dioxide contributes to global warming and has already led to a noticeable rise in global temperatures. If no substantial changes are made, these effects are expected to intensify further in the coming years (Ritchie et al., 2023).

Emissions from the cement industry are among the major contributors to global warming. Concrete is the most widely used construction material in the world, and cement is a key component of concrete. The cement industry is a significant source of emissions, including CO_2 , NO_x , and SO_x (Devi and V, 2018). It is estimated that the cement industry accounts for nearly 6% of global greenhouse gas emissions (Ali et al., 2015).

To reduce greenhouse gas emissions, extensive research is being conducted into alternatives to traditional cement. One promising development is alkali-activated concrete, which consists of a cementitious component activated by alkaline solutions. Various industrial by-products and waste materials are being explored as substitutes for the cementitious part of this alternative concrete. However, much remains unknown about the long-term behaviour and performance of alkali-activated concrete, which is why it is not yet widely used in practice (Shi et al., 2011).

The Dutch government aims to become a fully circular economy by 2050. However, for Rijkswaterstaat, this timeline is not fast enough, they have set the more ambitious goal of becoming circular by 2030. This means operating without waste and focusing on the reuse of raw materials. They strive for high-quality reuse and the use of sustainably produced resources. In doing so, they aim to reduce CO_2 emissions and minimize their impact on the climate (Rijkswaterstaat, 2025).

With reusing elements the needs for new concrete elements will be lowered. Cement production is a major global source of CO_2 emissions. In practice, a bridge or viaduct typically has a functional lifespan of 40 to 50 years, even though most were originally designed to last 100 years. Research has even shown that structural beams can remain functional for at least 150 years (Royal HaskoningDHV, 2025). This means that precast beams, for example, can be reused in new bridges, provided they meet current requirements for load-bearing capacity and reinforcement. Reusing structural elements significantly reduces the demand for primary raw materials, thereby lowering CO_2 emissions (Royal HaskoningDHV, 2024).

According to Vergoossen et al., 2022, several projects are underway in the Netherlands where precast girders are reused in new bridge constructions. The first such project took place at Hoog Burel, where girders were reused from a bridge in Groningen. These girders initially did not meet current regulations, as they were originally built according to older standards. Moreover, the design of the new bridge had already been finalized, which meant the reused girders had to be shortened to fit the new structure.

As a result, the additional reinforcement at the ends of the girders was cut off, leading to insufficient shear reinforcement according to the current Eurocode standards. To verify whether the girders could still be used safely, a finite element model (FEM) was developed. This model demonstrated that the girders had sufficient shear capacity even without the original shear reinforcement.

Another large project is the A9, where a total of 11 viaducts are being demolished and the girders are reused in other bridges or viaducts. Some of these girders will be used in the A76 project. One of the major differences between older design codes and the current Eurocode lies in the approach to shear capacity. As a result, many of the reused girders do not meet current shear requirements. Since shear failure, especially in the absence of sufficient reinforcement, can occur suddenly and without warning, further research is essential (Nuijens and Quartel, 2022).

To ensure reliable results, load tests to failure have been conducted on girders similar to those being reused in the new A76 viaduct, specifically to study shear failure. The results showed that the actual shear capacity was almost twice as high as predicted by calculations. These findings are highly valuable for further research and for the broader application of reused girders in infrastructure projects (J. P. Pineda, 2025).

1.2. Objective of the research

This study aims to investigate the potential for reusing precast girders by applying load testing to girders that do not fully comply with current design codes. This approach makes it possible to assess the structural condition of girders without causing damage, allowing them to be reused in new bridges if the results are satisfactory. Therefore, research is needed to determine how load testing can be applied effectively and how reliable conclusions can be drawn about the structural performance of reclaimed girders.

1.3. Scope of the research

This study focuses specifically on inverted T-beams, as these are currently the most commonly reused type of girder in the Netherlands. The primary focus is on precast concrete girders with insufficient or missing shear reinforcement, which presents a major challenge for their reuse. The only non-destructive testing method employed in this study is acoustic emission. The research will concentrate on load testing such girders, those that do not meet the shear capacity requirements of the Eurocode, to assess whether girders from demolished bridges can still be reused based on their demonstrated actual capacity.

1.4. Research question

The main research question is:

How can load testing with acoustic emission be applied to assess the reliability of reused concrete girders subject to bending and shear?

This main research question will be addressed through the following subquestions:

1. How can acoustic emission techniques support load testing to provide useful information for the new Eurocode about the concrete girder?
2. How to assess the reliability of the concrete girders based on the information obtained by load testing and the new Eurocode?

These questions will be answered by performing a literature review and analytical research.

1.5. Outline of the report

The structure of this report is as follows. Chapter 2 presents the findings from the literature review. Chapter 3 outlines the research methodology and framework used in this study. Chapter 4 introduces the case study. Chapter 5 presents the results obtained from the analyses. Chapter 6 provides a discussion of these results in relation to the research objectives. Finally, Chapter 7 concludes the report with final remarks and recommendations for future research.

2

Literature review

This chapter reviews the state of the art relevant to the present work. It synthesizes prior research on acoustic emission monitoring in concrete, flexural and shear behaviour of girders, and probabilistic reliability assessment to establish the context, identify gaps, and motivate the research questions pursued in this thesis.

2.1. Flexural behaviour

Prestressed concrete girders are designed to resist flexure through a combination of concrete compressive strength, prestressing force and conventional reinforcement. Their flexural behaviour is influenced by tendon layout, concrete strength, cross-sectional shape, and time-dependent effects such as creep and shrinkage.

The design of flexural resistance in Eurocode 2 (EN 1992-1-1) is based on the ultimate limit state (ULS) and serviceability limit state (SLS) principles. The ULS ensures sufficient moment capacity under extreme loading, while the SLS limits cracking and deflection under normal service conditions (The Concrete Centre, 2024).

2.1.1. Design according to Eurocode 2:2004

The 2004 version of Eurocode 2 adopts a sectional approach, using strain compatibility and equilibrium to determine the moment resistance. The flexural capacity is obtained by assuming a linear strain distribution and using a rectangular or parabolic-rectangular stress block (The Concrete Centre, 2024). The strain block is the assumed linear strain profile across the section at ULS. The neutral axis position x is obtained from strain compatibility, with concrete compression at the extreme fibre limited to ε_{cu} and tensile reinforcement strain ε_s . For prestressed members, the tendon strain comprises the initial prestrain after losses, $\varepsilon_p(0)$, plus an increment from imposed curvature, $\Delta\varepsilon_p$. From this strain block, the corresponding concrete stress block (rectangular or parabolic-rectangular in EC2) and steel stresses are derived. With the sum of horizontal forces and the moments of the internal forces, the ultimate bending resistance can be calculated (The Concrete Centre, 2024).

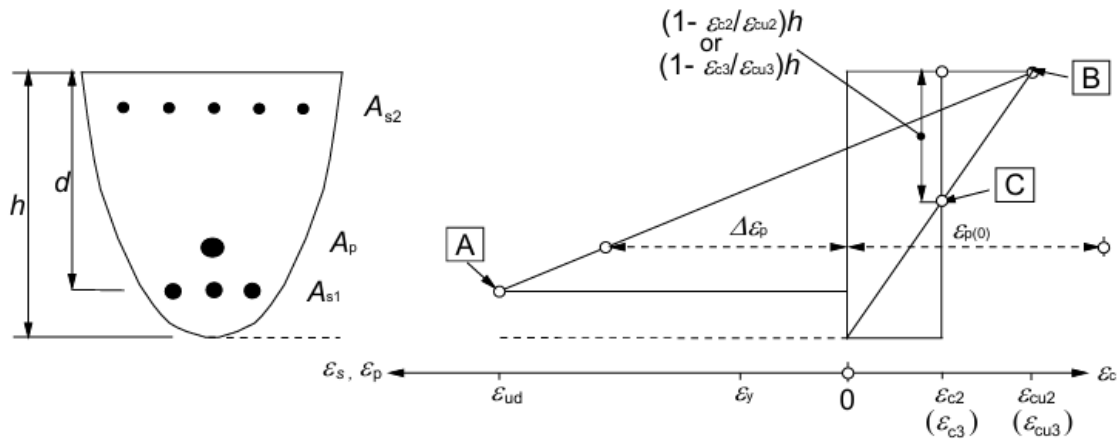


Figure 2.1: Strain distribution in ultimate limit state (NEN-EN 1992-1-1, 2015)

2.1.2. Design according to Eurocode 2:2023

The next-generation Eurocode 2 keeps the general framework. For flexural design, the 2023 version uses a similar equilibrium-based approach, so the difference is very small (FprEN 1992-1-1, 2023).

2.2. Shear behaviour

Shear failure in prestressed concrete girders can be a brittle and sudden mode of collapse, characterized by the formation of inclined cracks and rapid loss of load-carrying capacity. Unlike flexural failures which are usually preceded by yielding of reinforcement and large deformations, shear failures often occur with little warning, making them especially dangerous. Ensuring a safe design against such brittle failures has long been a fundamental principle in structural engineering (Muttoni and Simões, 2023). In beams without stirrups (shear reinforcement), the shear is resisted only by prestress and the concrete through mechanisms such as aggregate interlock along crack surfaces, dowel action of longitudinal bars, uncracked compression zone action, and the concrete's tensile strength. Once a critical diagonal crack forms and these mechanisms are exhausted, the member can fail catastrophically without significant prior deflection or visible distress (Yang, 2014).

2.2.1. Historical background and early approaches

Many mid-20th-century bridges were designed with little or no stirrup reinforcement for shear, especially when analytical checks suggested concrete alone was sufficient. The design philosophy at that time sought a simple, reliable lower-bound solution for shear capacity, often treating shear as a secondary check after flexure. Experimental studies in the 1960s revealed that larger structural members without stirrups can be significantly weaker in shear than smaller specimens, even when geometrically similar. Kani demonstrated the "size effect" in shear: as the beam depth or shear span increases, the nominal shear strength (stress at failure) drops (Kani, 1967).

Early models evolved as researchers sought to predict these observations. The truss analogy proposed by Ritter and Mörsch idealised the cracked beam as a 45° truss (Grandić et al., 2015). One foundational theory, the shear-friction theory by Birkeland and Birkeland, proposed that shear can be sustained by friction across crack faces if clamping pressure is provided by reinforcement crossing the crack (Birkeland and Birkeland, 1966). Mattock and Hawkins extended this with experimental studies examining shear transfer across cracks under various reinforcement and concrete configurations (Mattock and Hawkins, 1972).

Another major development was the aggregate interlock mechanism. Walraven studied shear transfer across cracked concrete interfaces, demonstrating how crack width, surface roughness, and aggregate size influence shear transfer (Walraven, 1981). Randl expanded this by integrating cohesion, friction, and dowel action into a unified shear transfer model (Randl, 1997).

2.2.2. Shear crack development without shear reinforcement

Prestressed concrete beams without shear reinforcement can fail in shear through several distinct mechanisms, each characterised by different crack patterns and structural responses. These include shear-tension failure, shear-compression failure, and shear-flexural failure. In inverted T-beams, such as those often used in bridge construction, the failure mechanisms are influenced by the unique geometry and load transfer paths. This subsection discusses each failure mode in detail and contextualises them within current and forthcoming Eurocode provisions.

Shear-tension failure

Shear-tension failure is a brittle failure mode that occurs when tensile stresses within the web, induced primarily by shear forces, exceed the concrete's tensile strength. The top and bottom flanges typically remain uncracked. This failure mode is common in members with thin webs, such as prestressed hollow-core slabs or T-beams (Sarkhosh, 2014; Yang, 2014). Figure 2.2 gives a crack pattern of a beam with shear-tension failure.

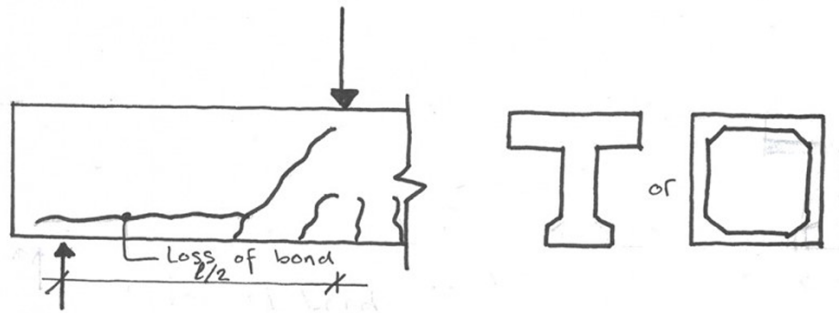


Figure 2.2: Cracking pattern of beam with shear-tension failure (Straathof, 2025)

Shear-compression failure

Shear-compression failure arises when a diagonal crack leads to crushing of concrete in the compression zone. It often occurs in beams with low span-to-depth ratios ($l/d < 2.5$), where internal force flow resembles a strut, redirecting shear through a diagonal compression zone. The concrete strut formed between the load and the support may become excessively steep, concentrating compressive stresses that can exceed the crushing strength of concrete (Sarkhosh, 2014). Figure 2.3 gives a crack pattern of a beam with shear-compression failure.

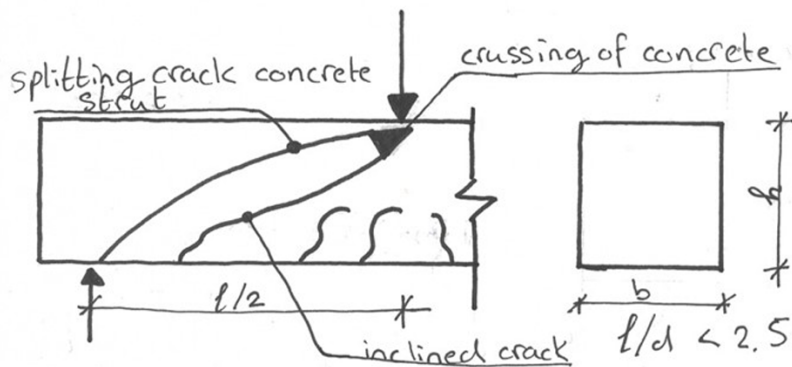


Figure 2.3: Cracking pattern of beam with shear-compression failure (Straathof, 2025)

Shear-flexural failure

This mode occurs when a flexural crack, originating at the tension face of the beam, transitions into an inclined crack under the influence of shear force. Unlike pure shear-tension failure, shear-flexural failure is a combined effect of flexural and shear actions, and is often observed in slender beams with long spans ($2.5 < l/d < 6.0$). The diagonal crack typically starts from a flexural crack and propagates

diagonally due to the increasing shear force and principal tensile stresses dominate the flexural effects causing the cracks to change direction. Diagonal cracks form, typically at an angle of around 45 degrees (Sarkhosh, 2014). Figure 2.4 gives a crack pattern of a beam with shear-flexural failure.

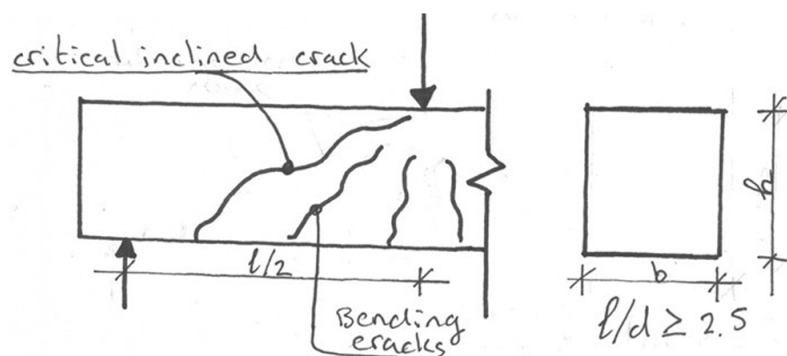


Figure 2.4: Cracking pattern of beam with shear-flexural failure (Straathof, 2025)

2.2.3. Shear transfer mechanisms

After a crack forms in a prestressed concrete beam, residual shear capacity depends on a combination of mechanisms that enable force transfer across the cracked section. These include aggregate interlock, dowel action, and tension softening. These mechanisms act simultaneously and interactively, forming the basis for post-cracking shear resistance in concrete members. Their significance has been confirmed through experimental studies and is now embedded in modern shear design models, including the Critical Shear Crack Theory (CSCT), which underpins the upcoming Eurocode 2 (2023) (FprEN 1992-1-1, 2023).

Aggregate interlock

Aggregate interlock (or crack interface shear transfer) refers to the mechanism by which aggregate particles along a crack interface transmit shear stresses. In normal concrete, the aggregates are harder and stiffer than the surrounding cement paste. Consequently, when a crack forms, it tends to pass around the aggregate particles, leaving a rough, tortuous crack surface (Walraven, 1981). Exposed aggregates from one face engage with the opposite face, providing mechanical resistance to relative slip, much like interlocking teeth, while friction across the rough surface further contributes to shear transfer (Taylor, 1970). Walraven's model, verified by extensive tests, predicted that aggregate interlock capacity diminishes markedly as crack width increases, since wider cracks reduce the contact area and engagement of aggregate asperities (Völgyi and Windisch, 2017).

Aggregate interlock can contribute a significant fraction of the shear resistance in beams without stirrups, often on the order of 33–50% at ultimate load. However, its contribution is highly sensitive to crack kinematics (Kaszubska and Kotynia, 2022). Bažant and Kim (1984) analyzed the lower shear strength using the principles and concepts of fracture mechanics: larger depth leads to larger crack openings at a given load, thus less aggregate interlock efficiency. Modern formulations like the Critical Shear Crack Theory explicitly incorporate this by relating shear strength to the width/opening of a critical crack (Muttoni, 2008). Figure 2.5 is an illustration of the development aggregate interlock in a concrete cracked beam.

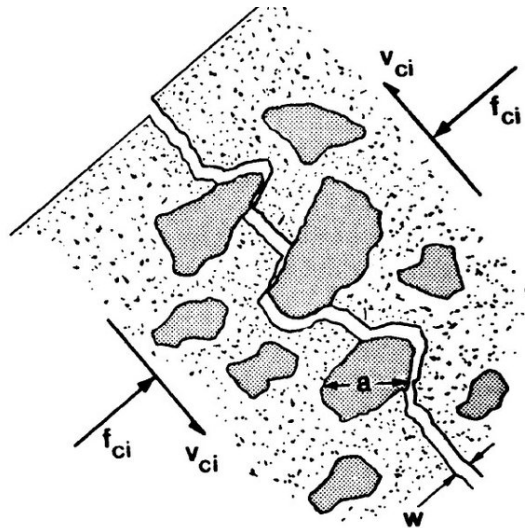


Figure 2.5: Transmitting shear stresses across crack with aggregate interlock (Vecchio and Collins, 1986)

Dowel action

Dowel action refers to the shear resistance provided by longitudinal reinforcing bars as they cross a crack. At a diagonal crack, the longitudinal reinforcement is forced to carry transverse shear, acting like a dowel embedded in the surrounding concrete. The bar resists this shear through bending and bearing against the concrete, similar to a dowel pin in a mechanical joint. As the bar deflects across the crack, it develops shear and bending stresses, while the concrete around the bar provides a reactive support. Eventually, either the bar yields, or the concrete under and around the bar crushes and splits, limiting the dowel capacity (Vintzeleou and Tassios, 1986).

Experiments generally find the dowel action contribution to the total shear is smaller than that of aggregate interlock or compression zone shear, often on the order of 15–25% of the ultimate shear in typical beams (Kaszubska and Kotynia, 2022). Nonetheless, in cases with thick reinforcement (large diameter and high ratio) or very wide cracks (where interlock has degraded), the dowel action becomes more prominent. Modern analyses confirm that as aggregate interlock stresses diminish at large crack openings, the steel dowel forces increase to compensate, and vice versa (Collins et al., 2008). Figure 2.6 is an illustration of the development of dowel forces in a concrete cracked beam.

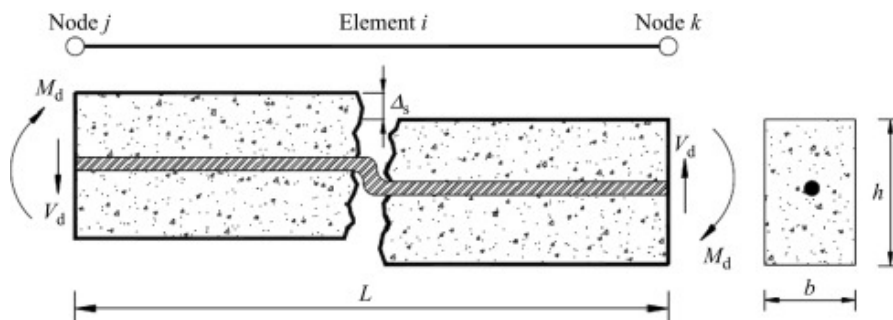


Figure 2.6: Dowel action mechanism along a reinforced concrete cracked beam (Liberati et al., 2016)

Tension softening

Tension softening refers to the phenomenon that concrete, after cracking, can still transmit tensile stress across the crack as it opens. Vecchio and Collins (1986) incorporated tension softening rigorously into a shear model via the Modified Compression Field Theory (MCFT). They calibrated a tension softening response such that the average tensile stress in cracked concrete could be related to the crack opening strain. This effectively captures the influence of concrete bridging and aggregate interlock in tension.

Fracture mechanics characterises this by a softening curve. For example, an initial tensile strength at crack initiation, dropping gradually to zero stress at a certain crack opening (typically 0.1–0.3 mm for normal concrete). The area under the softening curve equals the fracture energy, which is higher for concretes with tougher aggregate and is proportional to the ability to sustain stress at larger crack openings. This residual tensile stress contributes a vertical component of force across an inclined crack and enhances shear transfer by restraining crack growth (Völgyi and Windisch, 2017).

Modern shear theories, such as the Critical Shear Crack Theory (CSCT) and the Modified Compression Field Theory (MCFT), incorporate tension softening using parameters like fracture energy or residual tensile strength. These models account for the declining tensile stress across a crack as a function of its width or rotation, integrating these stresses to calculate the residual tensile force bridging the crack, which decreases with increasing crack opening and vanishes when the crack is fully developed (Muttoni, 2008; Vecchio and Collins, 1986). Figure 2.7 is an overview of the development of shear transfer mechanisms in concrete beams with an example of tension softening as residual tension.

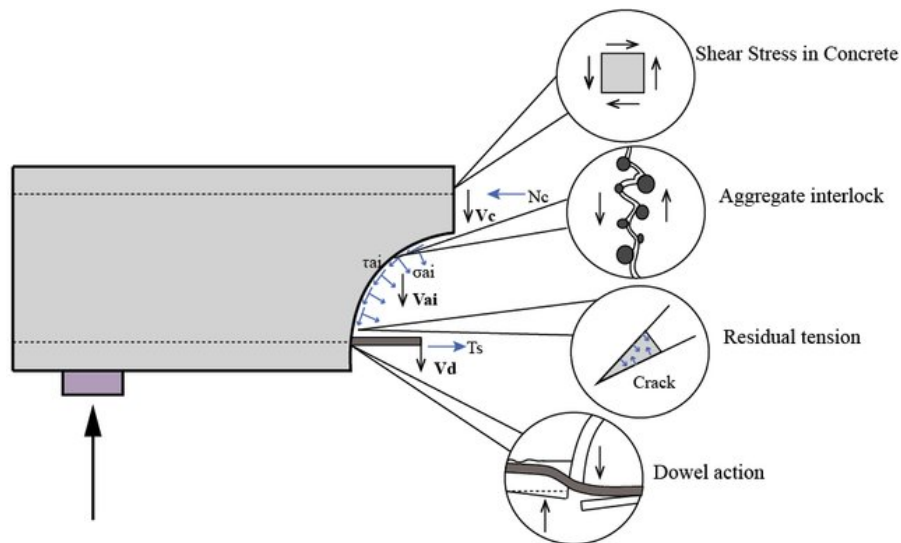


Figure 2.7: Overview of shear transfer mechanisms (Filian et al., 2019)

2.2.4. Mechanics-based models

By the 1980s, empirical models gave way to mechanics-based models. Vecchio and Collins (1986) introduced the Modified Compression Field Theory (MCFT) treating cracked reinforced concrete girders as orthotropic and considering rotating principal stress directions. Their model included tension softening and tension stiffening, leading to more accurate shear predictions.

Muttoni (2008) developed the Critical Shear Crack Theory (CSCT), first for punching shear, then extended to one-way shear. CSCT relates shear strength to the width of the critical crack that forms before failure. As crack width increases, shear capacity drops due to reduced aggregate interlock and stress transfer. CSCT captures size effects and incorporates reinforcement strain and concrete properties. CSCT is primarily formulated for flexural-shear failure (Muttoni and Simões, 2023).

Yang proposed the Critical Shear Displacement Theory (CSDT), which takes a displacement-based perspective on shear failure in beams without stirrups. In this model, failure occurs when the shear displacement across a flexural shear crack reaches a critical value, Δ_{cr} . The total shear capacity is expressed as the sum of contributions from the concrete compression zone, dowel action of the longitudinal reinforcement, and aggregate interlock. CSDT explicitly incorporates deformation mechanisms and has demonstrated strong agreement with experimental results. Unlike CSCT, which uses crack width as a failure indicator, CSDT focuses on the critical shear displacement as the governing parameter. CSDT is built for flexural-shear failure. (Yang, 2014).

2.2.5. Comparison between STM, CSCT and CSDT for prestressed concrete

Prestressing is a fundamental strategy in concrete design that not only enhances the flexural performance of concrete girders but also significantly affects their shear behaviour. In prestressed girders, axial compression from the prestress delays the initiation of flexural and shear cracks, improves the aggregate interlock along cracks, and increases the overall stiffness of the member. These effects influence both the magnitude and the nature of shear resistance. The effect of prestress has been studied in the context of different analytical models such as the Strut-and-Tie Model (STM), Critical Shear Crack Theory (CSCT), and Critical Shear Displacement Theory (CSDT), each of which incorporates the influence of prestress differently.

Strut-and-Tie Model (STM)

In the Strut-and-Tie Model, a prestressed girder is visualised as a truss composed of compressive struts and tensile ties. Prestressing introduces axial compressive forces and often vertical components of tendon force (from draped profiles), which reduce the net shear force that needs to be resisted by the concrete. The prestressing tendons themselves can act as ties in the truss model, supplementing the capacity otherwise provided by stirrups. Furthermore, prestressing increases the cracking load, delays the onset of diagonal cracks, and enhances compressive strut action by increasing the effective compressive force in the web. This improves the beam's resistance to shear failure and increases its overall ductility. STM accounts for prestress through equivalent loads and through modified stress fields, which is especially important in regions near tendon anchorages or deviators where shear forces are highest (Schlaich et al., 1987).

Critical Shear Crack Theory (CSCT)

CSCT models shear strength based on the width and development of a critical shear crack. In prestressed members, the axial compression increases the cracking moment and reduces the opening of diagonal cracks under loading. Since CSCT links shear capacity to crack width, prestressing indirectly raises the shear capacity by limiting crack development. The theory incorporates the rotational demand of the section and uses longitudinal strain to predict crack widths, where the compressive prestress reduces longitudinal tensile strains and thereby the width of the critical crack. Consequently, prestressed girders reach the limiting crack width associated with shear failure at higher load levels compared to non-prestressed members. This makes CSCT particularly suited to quantify the beneficial effects of prestress in shear-dominated members (Mari, 2023).

Critical Shear Displacement Theory (CSDT)

CSDT focuses on the displacement along a shear crack, integrating contributions from concrete compression, aggregate interlock, and reinforcement dowel action. Prestressing improves the effectiveness of all three mechanisms by reducing initial cracking and keeping cracks narrow, thus maintaining higher interlock and compressive stress transfer. In prestressed girders, the longitudinal compressive force delays crack propagation and reduces shear displacement, which according to CSDT, raises the shear failure threshold. The theory can be extended to account for prestress by including the effect of initial compressive stress in reducing the critical crack sliding displacement. Experimental applications of CSDT have shown that prestressed members exhibit higher ultimate shear loads and more ductile crack patterns, confirming the theory's applicability to prestressed concrete design (S. Han et al., 2021; Rueda-García et al., 2023).

2.2.6. Design code evolution

NEN-EN 1992-1-1 (2015) provided an empirical shear design equation based on a large test database for members without shear reinforcement. It introduced a size factor k and accounted for reinforcement ratio and concrete strength. However, the model lacked direct physical basis.

fib Model Code for Concrete Structures 2010 (2013) marked a turning point by codifying MCFT-based shear design within a levels-of-approximation framework, providing mechanics-based provisions that explicitly account for crack-width-dependent shear transfer (aggregate interlock), rotation of principal strains, and tension stiffening—aspects not explicitly modelled in the current Eurocode 2 shear method (Sigrist et al., 2013).

fib Model Code for Concrete Structures 2020 (2023) continues and strengthens this mechanics-based approach, adopting a strain-/size-effect, CSCT-consistent formulation for shear in members without

transverse reinforcement, where shear resistance is related to the opening of a critical shear crack, itself linked to longitudinal reinforcement strain and effective depth. This enables more physically consistent predictions and calibration against large databases, especially for large-D members.

FprEN 1992-1-1 (2023) 8.2.2 provisions for members without shear reinforcement are based on the Critical Shear Crack Theory (CSCT) and the original formulation proposed by Muttoni and Fernández-Ruiz (2008). That formulation has been modified for the Eurocode (Ibrahim et al., 2025). The new Eurocode adopts the following expressions for members without shear reinforcement (FprEN 1992-1-1, 2023).

An additional update is the parameter d_{dg} , representing the effective maximum aggregate size, to account for aggregate interlock along shear cracks. This is introduced through the size-dependent term d_{dg}/d , which captures the influence of crack-surface roughness on shear transfer: larger aggregates enhance interlock and increase shear resistance. Conversely, at higher concrete compressive strengths the relative benefit of aggregate interlock diminishes, because cracking may propagate more through the cement and across aggregate particles, reducing the mechanical interlock contribution. a_{cs} denotes the effective shear span at the control section, a_v the mechanical shear span used for short-span effects, and k_{vp} a factor that modifies shear resistance based on the axial force level and action ratio at the section.

There is an additional Annex in the next-generation Eurocode for cases where a more refined verification is needed to assess existing structures, as an alternative to the above equations. The design value of the shear resistance of members without shear reinforcement can be calculated taking into account the same extra parameters used in section 8.2.2.

In this formula two additional parameters are introduced. One is γ_{def} , which is a partial safety factor and covers the uncertainties related to the calculation of the deformation. It is 1.33 unless the National Annex specifies a different value. The other is ε_v , the strain in the longitudinal reinforcement, which should be calculated in accordance with the assumptions in section 8.1.1 and using sections 5.2.4 and 5.3.3, about the strain stress relation of longitudinal reinforcement.

2.2.7. Shear-specific stop criteria

With load testing, all behaviour must remain elastic with full recovery after unloading. Use incremental loading with brief unload–reload loops and verify negligible residuals in accordance with proof-load practice (recovery of deflection/rotation and closure of cracks). In addition to these global checks, continuously observe the shear spans for the first appearance and evolution of inclined cracks and monitor their opening and sliding, together with local compressive strains in the web near the load–support path. Halt the test if crack kinematics accelerate, if residual crack opening or residual deformation is detected after unloading, if reinforcement strains approach yield, or if web compressive strains indicate onset of crushing. Check the flexural region in parallel (midspan curvature or chord rotation, and steel/prestressing strains) and stop if any flexural residuals or yielding are observed (Deutscher Ausschuss für Stahlbeton, 2000).

2.2.8. Key parameters

Accurate calculation of the shear capacity of prestressed concrete girders depends on several geometric, material, and prestressing-related parameters. Other parameters are also important to measure during load testing. The key parameters are listed down below:

- **Effective depth** (d): Distance from compression face to the centroid of longitudinal reinforcement or prestressing tendons.
- **Web width** (b_w): Cross-sectional width of the web where shear is transferred.
- **Concrete compressive strength** (f_{ck}): Characteristic 28-day cylinder compressive strength.
- **Longitudinal reinforcement ratio** (ρ_l): Ratio of steel area to concrete area, affecting crack development and ductility.
- **Yield strength of reinforcement** (f_{yk}): Characteristic yield stress of the steel reinforcement.
- **Axial stress in concrete** (σ_{cp}): Often derived from prestressing force and cross-sectional area; contributes to shear resistance.

- **Maximum aggregate size** (d_g): Influences crack roughness and aggregate interlock, now explicitly included in the 2023 Eurocode.
- **Prestressing force and eccentricity** (N_p, e_p): Needed to determine pre-compression and stress distribution, often inferred from load testing.
- **Strain** (ε): Needed to measure during load testing to find if there is plastic deformation or to find crack opening.

These parameters must be known or accurately estimated during assessment of existing structures, particularly in cases where documentation is incomplete. In such cases, proof load testing and model calibration provide valuable means to infer the required inputs for safe shear capacity evaluation.

2.3. Monitoring and measuring techniques

As mentioned above, monitoring techniques are required to track the stop criteria for the shear behaviour of prestressed concrete structures without shear reinforcement during load testing. Furthermore, it is necessary to determine structural parameters that govern shear capacity, many of which cannot be observed directly. Non-destructive testing (NDT) methods are therefore vital tools to assess the internal condition and integrity of concrete bridge girders without causing irreversible damage. Internal damage is not always visible from the outside, so methods capable of providing information about the internal structure are essential. This section introduces different types of measurement techniques.

Linear Variable Differential Transformer (LVDT)

An LVDT is an electromechanical displacement transducer that measures linear motion by detecting the position of a movable ferromagnetic core within a coil assembly. When the core moves, the voltages induced in the secondary windings change, providing a precise measurement of displacement. LVDTs are highly accurate, on the order of 0.01 mm, and are often used for precise deflection monitoring on bridges (Tang et al., 2024).

Advantages: High accuracy and resolution. Suitable for continuous monitoring. Unaffected by environmental conditions.

Disadvantages: Requires physical attachment. Limited to point measurements. Setup can be labor-intensive.

Electrical Resistance Strain Gauges

A bonded electrical resistance strain gauge consists of a thin metal foil pattern (on a flexible backing) that is affixed to the concrete surface with adhesive. As the concrete deforms under load or thermal change, the foil stretches or compresses, changing its electrical resistance. This change is measured with a Wheatstone bridge to infer the surface strain (FPrimeC Solutions, 2020).

Advantages: High sensitivity. Well-established technology. Compact and inexpensive.

Disadvantages: Limited to surface measurements. Vulnerable to environmental effects such as temperature and moisture.

Optical Fibre Sensors

Optical fibre sensors (especially fibre Bragg Gratings (FBGs) or distributed sensing based on Brillouin/Rayleigh scattering) use light in a fibre to measure strain and temperature. In an FBG sensor, a small periodic variation of refractive index in the fibre reflects a narrow wavelength (the Bragg wavelength). When the fibre is strained or heated, this reflected wavelength shifts proportionally (Innovations, 2014).

Advantages: Immune to electromagnetic interference. Enables distributed sensing over long distances. Durable and lightweight.

Disadvantages: High initial cost. Requires specialised installation and interpretation equipment.

Digital Image Correlation (DIC)

Digital Image Correlation is a non-contact optical method that captures full-field displacement and strain by tracking a speckle pattern on the concrete surface. High-resolution cameras acquire images of a random dot pattern on the girder, and correlation algorithms compute sub-pixel movements between images under load to produce a map of surface deformation. DIC can measure 2D or 3D displacement fields over the entire visible area of the girder (Blikharsky et al., 2022).

Advantages: Provides full-field strain mapping. Non-contact. Highly accurate.

Disadvantages: Sensitive to lighting and environmental conditions. Requires careful surface preparation and calibration.

Ground Penetrating Radar (GPR)

GPR uses electromagnetic pulses to probe inside concrete. A ground-coupled antenna emits short electromagnetic pulses that travel into the concrete. Interfaces (changes in dielectric constant, such as rebar or voids) reflect part of the signal back to a receiver. By scanning along the girder, one collects a series of A-scans or B-scans that reveal the location and depth of embedded features. GPR is commonly used to locate reinforcement, measure concrete cover, and detect large voids or delaminations (as reflections or signal attenuation) (Azari, 2016).

Advantages: Fast scanning over large areas. Effective at locating embedded objects. Modern GPR software can generate 3D images of reinforcement.

Disadvantages: Interpretation can be complex. Results affected by material conductivity and moisture.

Infrared Thermography (IR)

Infrared thermography detects thermal contrasts on the concrete surface. Active thermography involves heating the concrete (e.g. with lamps or heat pulse) and watching heat flow. Subsurface delaminations or voids alter heat conduction, creating hot/cold spots visible as temperature differentials in an IR camera. Passive thermography uses ambient or solar heating differences. IR provides a visual heat map of the surface that can indicate hidden defects (delaminations) and material variations (Tomita and Chew, 2022).

Advantages: Non-contact and fast. Suitable for large-scale assessments. Can detect hidden anomalies.

Disadvantages: Requires sufficient thermal gradients. Surface conditions and environmental factors may affect accuracy.

Radiography

Radiography uses ionizing radiation (X-rays or gamma rays) to image the internal structure of an object. A powerful source emits rays through the concrete girder, and a detector or film on the opposite side captures the transmitted image. Variations in density and thickness (such as voids, cracks, or reinforcement) appear as contrasts on the radiograph. Radiography can provide a volumetric “picture” of the interior, similar to medical X-rays for bones (Shumway, 2024).

Advantages: High-resolution imaging. Can detect voids, cracks, and inclusions.

Disadvantages: Safety concerns due to radiation. Expensive and time-consuming. Requires access to both sides of the component.

Ultrasonic Pulse Velocity (UPV)

In UPV testing, an ultrasonic transducer, usually 50 kHz, sends a pulse through the concrete, and another transducer on the same or opposite face receives the pulse. The travel time gives the pulse velocity ($v = \text{path length}/\text{time}$). Higher velocities indicate sound, dense concrete. Slower velocities indicate cracks, voids, or poor quality. UPV is standardized (e.g. ASTM C597) and can estimate concrete uniformity, detect large flaws, and correlate roughly to compressive strength or elastic modulus (Bire, 2024).

Advantages: Simple and portable equipment. Effective for quality control and flaw detection.

Disadvantages: Requires good surface contact. Influenced by moisture content and temperature. Limited depth penetration.

Impact Echo (IE)

Impact Echo is an acoustic pulse-echo method. A small impact, usually a handheld hammer, generates stress waves that propagate through the concrete. A nearby sensor, displacement or velocity transducer, records the surface vibration. The frequency spectrum of the received signal is analyzed: reflections from internal defects or the opposite surface produce characteristic resonant peaks. From the peak frequencies, using known sound speed, one can compute thickness or detect delaminations and voids (Amphora Consulting, 2023b).

Advantages: Effective for locating delaminations and thickness changes. No need for couplant.

Disadvantages: Requires skilled interpretation. Less effective in highly heterogeneous materials.

Acoustic Emission (AE)

Acoustic Emission is a passive monitoring technique that listens for ultrasonic stress waves emitted by active damage processes. Piezoelectric sensors attached to the concrete surface continuously record transient elastic waves generated by phenomena like crack initiation, crack growth, fibre breakage, or delamination under load. In essence, AE detects the sound of structural events as they happen. By analysing arrival times and amplitudes at multiple sensors, one can sometimes locate the source of emission (Amphora Consulting, 2023a).

Advantages: Real-time monitoring. Sensitive to active damage processes.

Disadvantages: Complex data analysis. Requires quiet conditions. Sensitive equipment needed.

Concrete compressive strength

Concrete compressive strength for existing girders is best established from drilled cores tested in the lab. Cores are extracted and conditioned per EN 13791, then tested in compression. Strength is interpreted with location/diameter/aspect-ratio and moisture corrections (Federal Highway Administration, 2024).

Advantages: Cores provide direct strength with low uncertainty.

Disadvantages: Coring is semi-destructive, needs repair and access.

Reinforcement yield strength

Yield strength is most reliably obtained from tensile tests on extracted bar samples following specimen preparation and testing requirements in ASTM A370 or EN ISO 15630-1. Sampling should account for bar size and straightening limits to avoid pre-damage. Where extraction is impractical, in RBK 1.2 there is also given what kind of reinforcement is used at certain times.

Advantages: Coupon tests provide definitive f_y and stress–strain response. Establishes ductility and actual overstrength.

Disadvantages: Requires local demolition and bar replacement/anchoring. Hardness-based estimates are indirect and sensitive to surface condition and calibration.

Prestress in existing girders

Effective prestress and tendon eccentricity can be established by (i) a records-based drawings and design calculations, use effective prestress after losses for the calculation, and verify geometry in situ with inspection or (ii) a proof-loading back-calculation—incrementally load to first flexural cracking, set the extreme-fibre tensile stress equal to the mean flexural tensile strength $f_{ctm,fl}$ per NEN-EN 1992-1-1 (2015), solve for the effective prestress from combined permanent and test loads, and confirm negligible residuals on unload (Federal Highway Administration, 2024).

Advantage: Combines documentary evidence with a physically anchored, non-invasive verification method, enabling a reliable estimate even when records are incomplete. **Disadvantage:** Accuracy

depends on the quality of records, detection of first cracking, and assumptions on stiffness and boundary conditions. The proof-loading route must remain strictly elastic and is sensitive to test execution (NEN-EN 1992-1-1, 2015).

2.4. Acoustic emission (AE)

Acoustic Emission (AE) stands out among other measurement techniques for detecting internal damage. AE refers to transient stress waves generated by the sudden release of energy within a material, typically caused by events such as cracking in concrete. These stress waves propagate from the source to AE sensors, where they are converted into electrical signals (F. Zhang, 2022). Each wave detected by a sensor is referred to as an AE hit, or simply a 'hit'. The waveform associated with each hit can be used to calculate various parameters, including amplitude, rise time, duration, signal strength, energy, and counts (see Figure 2.8) (ElBatanouny et al., 2014). This information can then be used to estimate the location of the source, classify the type of source event, and assess the structural integrity of the material (Ohtsu, 2010).

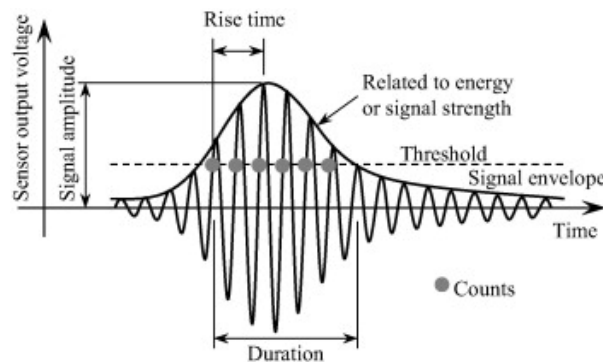


Figure 2.8: A schematic waveform of acoustic emission (ElBatanouny et al., 2014)

There is a need for an effective non-destructive testing method to assess the condition of existing structures. Condition assessment is often performed through load testing. The American Concrete Institute (ACI) Committee 437 has proposed a method based on the application of a series of load cycles, referred to as the Cyclic Load Test (CLT). The CLT assesses the structural condition primarily by analysing the relationship between the applied load and the resulting deflection. However, obtaining accurate deflection measurements in the field is often challenging and, in some cases, may lead to misleading interpretations.

Acoustic emission (AE) enables the detection of stress waves caused by sudden energy releases, such as crack initiation and propagation. Several studies have been conducted to develop analytical methods for structural assessment using AE-based parameters. Most of these methods have been developed from experiments on fibre-reinforced polymer (FRP) or reinforced concrete structures. Examples of such methods include intensity analysis, calm and load ratios, peak cumulative signal strength ratio, relaxation ratio, and b-value analysis (ElBatanouny et al., 2014).

2.4.1. Noise filtering

Acoustic Emission monitoring produces large volumes of data, much of which may be attributed to environmental noise or irrelevant activity. To enhance the reliability and interpretability of AE signals, various data reduction and filtering methods are employed. These include threshold-based filtering, time and amplitude discrimination, and noise reduction algorithms. Filtering techniques help isolate AE events associated with structural damage, while data reduction ensures manageable storage and processing. Common practices involve rejecting hits below a set amplitude threshold and using clustering or classification methods to group signals with similar characteristics (Aggelis and Shiotani, 2007; Grosse et al., 2022).

Not all signals captured during acoustic emission (AE) monitoring originate from cracking events. In addition to burst-type signals associated with damage, AE systems often record non-relevant signals

arising from environmental or mechanical sources. These noise signals typically appear as continuous waveforms and can significantly interfere with data interpretation. As demonstrated by Van Steen et al. (2019), examples include mechanical vibrations from surrounding equipment or structural elements, which may exhibit steady or oscillatory patterns, see figure 2.9.

To discriminate such mechanical noise from true AE activity, a time-window-based filtering technique can be employed. The raw signal is divided into multiple temporal segments, and for each segment, characteristic amplitude metrics, such as the peak value or root mean square (RMS), are computed. If the signal's amplitude characteristics (peak or RMS) are similar in both windows, it is likely to be noise, because true burst signals from events like cracking typically change over time (Van Steen et al., 2019).

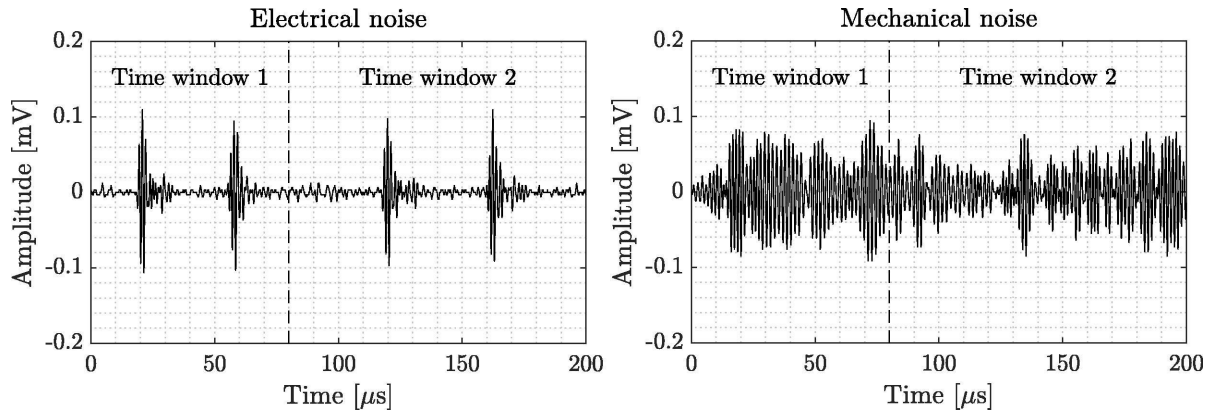


Figure 2.9: Examples of electrical and mechanical AE noise (Van Steen et al., 2019)

2.4.2. AE data analysis

Hit analysis

Acoustic Emission (AE) monitoring is a non-destructive technique used to detect active damage processes in prestressed concrete girders. An AE event refers to a physical occurrence such as crack growth, while an AE hit is the detection of that event by a sensor (EN 14584, 2011). Each hit is characterized by signal parameters: arrival time (first threshold crossing, used for timing and source location), peak frequency (dominant frequency, helps identify damage mechanisms), amplitude (maximum signal strength in dB, reflects event intensity), duration (time the signal remains above threshold), energy (signal envelope area, linked to damage severity), and counts (number of oscillations above threshold) (Grosse et al., 2022).

AE hit rate, derived from hit arrival times, indicates activity trends. When combined with peak frequency, it helps differentiate damage types: low-frequency activity suggests micro-cracking, while high-frequency clusters can signal steel wire breaks (Dubuc et al., 2019). In the absence of formal protocols, AE data are often classified by frequency bands and cumulative parameters, enabling real-time damage tracking in structural health monitoring applications (Amphora Consulting, 2023a).

Source localisation

Traditional AE localisation algorithms assume homogeneous media and constant wave speeds, which are not applicable in reinforced concrete. The basic method uses arrival time, sensor locations, and wave speed to estimate the source location. F. Zhang (2022) identifies that localisation errors arise from uncertainties in wave velocity, sensor placement inaccuracies, and arrival time picking. These errors become particularly problematic when interpreting the precise location of cracks. A recent study on wave speed in a concrete girder estimated it to be 4100 m/s, and the material attenuation factor is 20 dB/m for signals with a central frequency of 60 kHz. The amplitude drops further from the source due to material attenuation and geometric spreading loss. The further the sensor is from the source, the more attenuated the signal becomes, which can lead to larger arrival time picking errors and, consequently, greater localisation errors.

According to measurements, the sensor-to-source distance must be within 1 meter to ensure that direct waves can be detected. Missing an AE event can occur when not enough sensors receive the direct

waves. The required number of sensors is three for 2D and four for 3D localisation (F. Zhang, 2022).

To address these challenges, F. Zhang (2022), introduced a real-time probability density field update approach that can rapidly incorporate new AE events. This method significantly improves computational efficiency, reducing the total processing time (including calculation and visualization) to approximately 0.12 seconds per AE event, compared to around 50 seconds per event in earlier probabilistic approaches (Schumacher et al., 2012). By incrementally updating the probability field rather than recalculating it from scratch, the method enables near-instantaneous feedback on likely crack locations, making it particularly suitable for real-time structural health monitoring in complex, heterogeneous materials like reinforced concrete.

Reused girders most likely contain cracks that developed during their service life. Identifying these existing cracks is important for assessing structural integrity. Applying a small load and monitoring AE events during the subsequent unloading phase can help locate such pre-existing cracks (F. Zhang, 2022).

pdAE in AE testing

Probability density of AE events (pdAE) refers to a probabilistic description of where acoustic emission sources are likely located within a structure. Rather than pinpointing each AE event at a single estimated coordinate, pdAE treats the source location as a random variable and assigns a probability density over the material volume or area. In essence, it produces a spatial probability density field for AE events, accounting for uncertainties in classical source localisation (e.g., sensor timing errors, wave speed variability, etc.). Integrating this density over any region gives the probability (or expected fraction) of AE events originating in that region. The pdAE approach thus provides a "heat map" of likely AE source locations, offering a clearer visualization of damage zones than discrete point estimates (F. Zhang, 2022).

Mathematical formulation of pdAE

Formally, pdAE is based on a probability density function (PDF) that describes localisation error. This error is often modeled as an isotropic Gaussian distribution centered at the estimated location of the AE source. For a single AE event a located in a k -dimensional space, the PDF is given by:

$$f_a(x, x_{g,a}) = \frac{1}{(2\pi)^{k/2}} \cdot \frac{1}{\sigma^k} \exp\left(-\frac{|x - x_{g,a}|^2}{2\sigma^2}\right) \quad (2.1)$$

where x_g is the estimated coordinates and σ is the standard deviation of the localisation error.

For multiple events A , the total pdAE field is the sum:

$$p_A(x) = \sum_{a \in A} f_a(x, x_{g,a}) \quad (2.2)$$

This gives the event density field, with $\int p_A(x) dx = N$ for N AE events (F. Zhang et al., 2022).

Calculation from raw AE data

1. *Data collection*: Instrument the structure with AE sensors and record AE hits, noting each hit's timestamp, sensor ID, amplitude, energy, etc.
2. *Event formation*: Group hits across sensors into AE events using temporal coincidence and arrival time picking methods.
3. *Source localisation*: Estimate the event's location $x_{g,a}$ using triangulation or a grid search method based on time-of-arrival data and known wave velocity.
4. *Error estimation*: Derive or assume a standard deviation σ based on calibration experiments, prior simulations, or known localisation uncertainty.
5. *Compute pdAE*: For each AE event, construct its probability distribution $f_a(x)$ using the Gaussian model. Sum these over all events to obtain $p_A(x)$.

6. *Normalization*: Normalize $p_A(x)$ by the total number of events N to interpret the result as a probability density.

Error distribution calibration

The accuracy of pdAE depends on the appropriate estimation of σ . This can be done through:

- Monte Carlo simulations incorporating arrival time errors and sensor placement variations
- Calibration experiments with known AE sources (e.g., pencil lead breaks)
- Analytical estimates based on sensor geometry and wave speed uncertainties

Incorporating AE parameters

While standard pdAE assigns equal weight to each AE event, signal properties like amplitude or energy can be used for weighted pdAE fields:

$$p_A^{\text{weighted}}(x) = \sum_{a \in A} w_a \cdot f_a(x) \quad (2.3)$$

where w_a represents a weighting factor such as energy or amplitude of the event.

Applications of pdAE The probability density acoustic emission (pdAE) technique has been applied in various areas of structural health monitoring, with significant benefits for both damage detection and interpretation.

One primary application is crack and damage localisation. Compared to traditional scatter plots of point-based AE localisations, pdAE offers a much clearer and more reliable depiction of crack positions. Because it incorporates the uncertainty of source localisation, pdAE consolidates spatially scattered event estimates into continuous high-probability regions or “hotspots.” For example, in laboratory tests on concrete beams, pdAE maps accurately traced crack paths and aligned well with visually confirmed damage. In similar studies on concrete slabs, internal crack networks that were not easily visible were successfully revealed using pdAE maps. This approach effectively filters measurement noise and smooths AE data, highlighting real damage zones while suppressing misleading outliers caused by localisation errors.

Beyond identifying the location of damage, pdAE can also contribute to assessing the severity and progression of cracking. The magnitude of the pdAE field near a crack has been correlated with the crack’s width and the amount of shear displacement. Studies have shown that a greater accumulation of AE probability density along a crack typically reflects more intense activity, either due to wider openings or higher levels of frictional slip. Interestingly, two distinct patterns have been observed depending on crack type: for tensile (mode-I) cracks, pdAE may decrease as the crack opens and surfaces lose contact. Conversely, for shear-dominated cracks, a high number of AE events may occur even at small displacements due to surface interaction. These insights suggest that monitoring the temporal evolution of pdAE peaks can be used to infer damage growth. In practice, thresholds on pdAE values can be used to establish warning levels. For example, a moderate pdAE concentration may represent a “yellow light” condition, while a rapid increase in a critical region may trigger a “red light” alarm indicating imminent failure. This concept has been successfully demonstrated in tests on prestressed concrete beams, where pdAE activity in the compressive strut region was used to predict and classify stages of shear failure in real time.

In the context of long-term structural health monitoring, the pdAE method is particularly well suited for real-time applications. Because the computation of the density field is relatively efficient, it can be updated incrementally as each new AE event is recorded. This enables the creation of live probability maps that show the evolving distribution of damage within a structure. An increasing concentration of pdAE in a new area can suggest that a crack is initiating or extending, prompting timely inspection or preventive action. In experimental setups, pdAE has proven capable of warning of failure moments before collapse, based on a noticeable surge in event probability density in the critical region. For permanent monitoring of infrastructure such as bridges, pdAE can be integrated into AE software systems to continuously track AE activity clusters. Observing the development or migration of pdAE hotspots over time helps identify structural degradation with reduced risk of false alarms, thanks to its inherent

noise tolerance. In this way, pdAE adds a valuable probabilistic dimension to AE-based monitoring and improves reliability in real-world damage detection (F. Zhang, 2022).

Visualization and tools

Two-dimensional contour plots or heat maps are suitable when analysing a planar cross-section, such as the web of a beam or the surface of a slab. The pdAE field is visualized as a color density map using actual spatial coordinates, allowing direct correlation with structural features like supports or load points. Contour lines can highlight regions of high acoustic activity. Common tools include Matlab's `contourf` or Python's `pcolormesh`, often overlaid on a schematic of the structural element for context.

Three-dimensional visualization applies to volumetric AE data, where slices or isosurfaces can be used to display regions with high probability density. Software such as ParaView or MATLAB's slice viewer is typically used. In many practical cases, especially when one structural dimension is significantly smaller than the others, it is effective to project the pdAE field onto a two-dimensional plane. This simplification enables faster computation and clearer interpretation, while still capturing the relevant spatial characteristics of AE activity (Fengqiao Zhang et al., 2025).

Source classification

Acoustic emission (AE) signal parameters can be utilized to classify the source mechanisms occurring within a concrete structure. These parameters are influenced by the characteristics of wave propagation from the source to the receiver. Previous research has investigated methods for distinguishing between sources generated by tensile cracking and those resulting from frictional mechanisms, based on full-scale shear tests. The findings indicated that, when considering the peak frequency, signals with frequencies exceeding 70 kHz were predominantly associated with tensile cracking, whereas those below 70 kHz were attributed to frictional cracking. An alternative approach involved the use of partial power, where values greater than 0.5 were indicative of tensile cracking and values below 0.5 corresponded to frictional cracking (Fengqiao Zhang et al., 2022).

2.4.3. AE in inverted T-beams

In inverted T-beams, both overall size (depth) and cross-sectional geometry (flange and web) systematically alter AE measurements, even at comparable damage states. First, deeper members impose longer propagation paths, causing stronger attenuation and frequency downshift: amplitudes and energies decay with distance, average frequency decreases (Li et al., 2020; Mandal et al., 2022; Mao et al., 2022). This produces fewer and weaker hits per load increment compared to shallow beams, which often show numerous intense, localised emissions (Behnia et al., 2016). Consequently, lab-calibrated thresholds may not transfer directly to bridge-scale elements, and scale-aware interpretation is required (Deresse et al., 2021).

Second, inverted T-beam geometry acts as a spatial filter on AE waves. Thick, stiff top flanges behave as low-pass filters, preferentially attenuating high-frequency content and biasing detected peak frequencies (C. Han et al., 2021). Flanges introduce acoustic shadowing: signals from web-initiated cracks can reflect or scatter at flange–web junctions, reducing amplitudes at flange-mounted sensors (Kosnik, 2009). Thicker webs further increase path length and internal damping, yielding lower amplitudes and longer durations (Yang et al., 2016). Mode conversion at geometric discontinuities (longitudinal $\leftrightarrow$ shear/surface waves) distorts waveforms and complicates arrival-time picks (C. Han et al., 2021).

Practically, these effects mean (i) sensor layouts should account for flange–web filtering and shadow zones (e.g., web-side or embedded sensors, shorter spacing, or waveguides), (ii) detection thresholds and frequency-based indices need depth- and path-sensitive calibration, and (iii) localisation and pdAE mapping should allow for increased uncertainty and potential undercounting in flange-dominated regions (Larosche et al., 2015; Savino et al., 2023; Schumacher et al., 2012; Shousha et al., 2023).

2.4.4. Shear failure analysis

Acoustic Emission (AE) can detect the onset of both flexural and shear cracking. This enables early identification of the first flexural crack, which can be used to verify the prestress level. With an appropriate AE sensor layout, the region where cracking occurs can be located and, together with signal characteristics, used to infer the likely shear-failure mechanism.

To evaluate the condition of prestressed concrete members, a three-tier Traffic Light System is proposed based on the integrity of the compressive strut. The method compares AE activity between the strut region and a reference zone that is known to experience early-stage flexural cracking. In practice, the decision metric is the maximum probability density of AE (pdAE) in the strut relative to that reference zone. This provides a practical, data-driven indicator of damage level during loading cycles and remains informative during unloading phases.

- **Green-light criterion:** This marks the onset of AE activity in the reference region, associated with the formation of initial flexural cracks. It is characterized by a notable increase in AE energy or hit rate at a specific load step, exceeding the minor fluctuations typically caused by microcracking. This threshold serves as a baseline for identifying subsequent damage in the strut. Structurally, the green-light criterion corresponds to a safe condition, typically occurring at a load ratio of less than 60% of the ultimate shear capacity, where only flexural cracking is observed.
- **Yellow-light criterion:** This level is triggered when the maximum pdAE in the strut region surpasses the crack initiation threshold defined by the green-light criterion. It indicates the formation of the first meaningful cracks within the compressive strut itself, suggesting localised degradation but not yet severe damage. Experimental data shows that this criterion generally occurs at load ratios between 49% and 87%. Notably, the load ratio at which the yellow-light threshold is reached has been observed to decrease with increasing shear span-to-depth ratio, suggesting that slenderer beams are more susceptible to early shear-related degradation.
- **Red-light criterion:** This most severe level occurs when the pdAE in the strut exceeds that in the reference region at the same load level. This behaviour is typically associated with inclined shear cracking in the strut, where significant internal friction and shear displacement generate a high density of AE events, even if the visible crack width is small. The red-light condition is associated with imminent or ongoing flexural-shear failure and usually appears at load levels approaching 99% of the ultimate shear capacity, warranting immediate safety intervention.

It is important to note that AE comparisons between the strut and reference region cannot directly infer crack width, due to differing crack development histories. As outlined in earlier sections, the relationship between pdAE and crack width differs by region: a Type I relationship is observed in the reference region (flexural cracking), while a Type II relationship applies in the compressive strut (shear cracking). Consequently, the same pdAE level in both regions implies a smaller actual crack width in the strut. Despite this, pdAE remains a reliable indicator of internal damage progression and forms the basis for the traffic light classification framework (F. Zhang, 2022).

In Figure 2.10, an illustration is provided showing the expected crack types and associated pdAE signatures corresponding to each traffic light level.

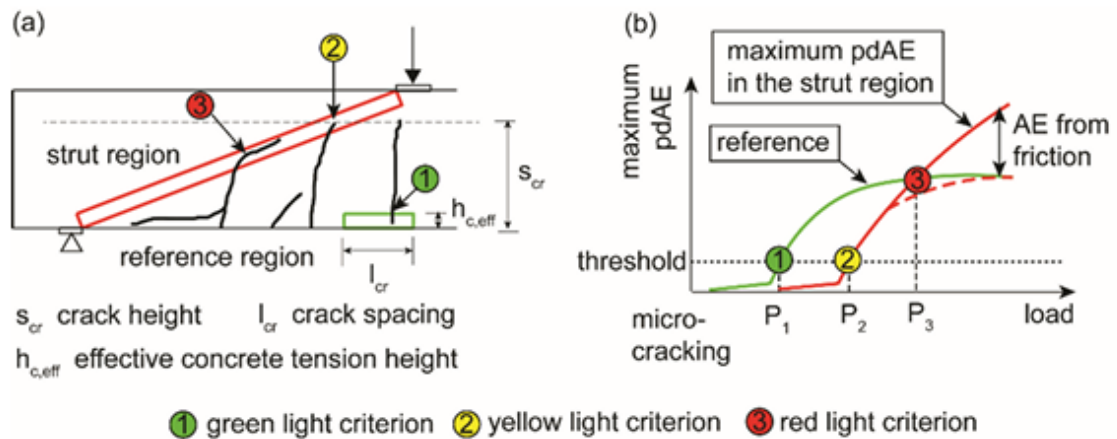


Figure 2.10: Traffic light system as shear failure indication: (a) cracking behaviour at the three traffic light criterion and (b) determination of the maximum pdAE in the strut and the reference region for the different traffic light criterion. (F. Zhang, 2022)

Decision making AE criteria

As discussed in the section above, there are three damage levels of AE-based indicators. Table 2.1 summarizes the physical background of the AE-based indicators and the possible actions.

Table 2.1: Decision making based on AE-based failure indicators for RC beams, considering user need. (F. Zhang, 2022)

User objective	AE-based indicator	Physical behaviour	Corresponding actions
Estimate the cracking load	Green-light criterion	Cracking in the reference region	Stop loading; Can repair the cracks.
Load the structure to the most, but keep safety against shear failure	Yellow-light criterion	Cracking in the strut	Stop loading; Can keep the structure, but under lower load; Can maintain the structure to prolong the service life.
Approach the actual shear capacity, but avoid collapsing of structure	Red-light criterion	Cracking in the strut that significantly reduces the bearing capacity	Take safety measures against collapse; Stop loading immediately; Replace or repair the structure.

2.5. Reliability analysis

2.5.1. Statistical evaluation of beam capacity based on sample testing

To assess the structural capacity of a population of prestressed concrete inverted T-beams, a statistical approach was applied based on the non-destructive testing of a representative subset. Key parameters to measure include initial stiffness, cracking load, and strain development. Acoustic Emission (AE) monitoring is used to detect crack initiation and localisation, following the methodology (Mandal et al., 2022).

Given the variability inherent in concrete structures and the small sample size, classical statistical methods were employed to estimate both the mean capacity and the expected variation within the population (Montgomery and Runger, 2020). The following statistical tools are used:

- **Confidence Interval (CI) for the mean capacity:** A two-sided $100(1 - \alpha)\%$ CI for the true mean capacity μ is calculated as:

$$\bar{X} \pm t_{n-1, 1-\alpha/2} \cdot \frac{s}{\sqrt{n}}$$

where $\bar{X}$ is the sample mean, s is the sample standard deviation, n is the number of tested beams, and $t_{n-1, 1-\alpha/2}$ is the critical value from the Student's t -distribution with $n - 1$ degrees of freedom.

- **Prediction Interval (PI) for individual beams:** A $100(1 - \alpha)\%$ PI, representing the range where a new, untested beam's capacity is expected to fall, is given by:

$$\bar{X} \pm t_{n-1, 1-\alpha/2} \cdot s \cdot \sqrt{1 + \frac{1}{n}}$$

- **Tolerance bounds for the population:** One-sided tolerance intervals provide conservative estimates ensuring that a specified proportion of the population exceeds a given lower bound with a certain confidence level (Hahn and Meeker, 1997). For example, a 95%/95% lower tolerance bound ensures that at least 95% of the beams have capacities greater than the calculated bound, with 95% confidence.

Acoustic Emission monitoring proved effective in identifying the first crack load without requiring visible inspection, in line with experimental findings (Mandal et al., 2022). They observed that AE signals exhibit clear onset at first microcracking and can localise damage in real time under bending.

By applying these statistical tools, it was possible to infer the expected capacity range for the entire set of beams, even with a limited number of physical tests. This approach aligns with recommended practices for structural reliability and quality assessment when full-scale testing of all elements is impractical (Montgomery and Runger, 2020).

2.5.2. Updating the reliability index using acoustic emission

Acoustic Emission (AE) monitoring provides real-time information on damage development within concrete structures during proof load testing. Recent research explores its integration into reliability-based assessment frameworks by treating AE observations as evidence that can update the probabilistic model of structural resistance.

The reliability index β is calculated as:

$$\beta = \frac{\mu_R - \mu_E}{\sqrt{\sigma_R^2 + \sigma_E^2}}$$

where μ_R and μ_E represent the mean values of resistance and load effect, respectively, and σ denotes their standard deviations. However, these parameters are often uncertain for existing structures, particularly in the absence of as-built documentation.

Alternatively, when the time-dependent failure probability $p_f(t)$ is obtained, for example from a Monte Carlo simulation on annual failure events, the reliability index can be expressed as:

$$\beta(t) = -\Phi^{-1}(p_f(t)),$$

where $\Phi^{-1}(\cdot)$ is the inverse standard normal distribution function.

AE monitoring offers a way to update the estimate of the structural resistance R based on observed crack activity (or lack thereof) during loading. If no critical AE activity is recorded up to a certain proof load level, it can be inferred that the actual resistance is higher than previously assumed. This approach allows updating the probability distribution of R , thereby shifting the reliability index β to a higher, more favorable value.

The study by de Vries et al. (2023) proposes a reliability analysis framework where proof load measurements are used and used in a Bayesian updating framework. A prior distribution for resistance is formulated based on assumptions or historical data. The likelihood of failure is updated, resulting in a new posterior distribution for R . This leads to an updated reliability index β_{updated} , which better reflects the actual structural condition as demonstrated during testing.

This methodology is especially relevant for prestressed concrete girders with uncertain properties, where AE can serve as an indicator of internal cracking or failure mechanisms that might not be visible externally. It provides a quantitative, probabilistic enhancement to structural safety assessment, allowing more refined decisions regarding maintenance, strengthening, or load restrictions.

2.6. Research Gap

From the literature review, the following key knowledge gaps have been identified:

Despite significant advancements in understanding the shear behaviour of reinforced and prestressed concrete members, several gaps persist in the current state of knowledge, especially regarding existing infrastructure and the evaluation of shear-critical regions in precast prestressed girders such as inverted T-beams.

Although the 2023 revision of Eurocode 2 and the fib Model Code 2020 provide more physically based shear models, including enhanced parameterisation such as aggregate size, their validation under full-scale loading remains limited. While this thesis investigates reliability-based assessment methods, it does not provide full-scale experimental validation.

The application of the new and current Eurocode for assessing shear and flexural capacity relies on several input parameters that are often unknown or uncertain in existing prestressed concrete girders. These include the effective prestress force, its eccentricity, reinforcement ratios, concrete strength,

and aggregate size. While some of these parameters can be estimated through indirect methods, their accuracy is limited without destructive testing. There is currently no standardized procedure for extracting some input data from load testing. Emerging methods, such as back-calculation from measured deflections or AE-based localisation of cracking, offer promise, but require further validation and integration into code-based assessment workflows. Particularly for reused prestressed girders with missing documentation, a systematic approach for deriving reliable input parameters from load and AE measurements is still lacking.

Several methods have been developed for updating the reliability index of reinforced concrete structures using load test data, often through Bayesian updating or Monte Carlo simulation. These approaches have shown promise in incorporating measured deflections, strains, and even AE signals into a probabilistic framework. However, such methods have not yet been systematically extended or validated for prestressed concrete girders, particularly those that are shear-critical and lack transverse reinforcement. There is currently a gap in adapting and applying these reliability updating frameworks to prestressed girders assessed through proof load testing.

While proof load testing and AE monitoring can provide detailed insight into the structural performance of individual girders, there is currently no standardized method to extrapolate these results to untested members from the same bridge.

While models such as the Critical Shear Crack Theory (CSCT), the Critical Shear Displacement Theory (CSDT), and the Strut-and-Tie Model (STM) offer robust theoretical frameworks, their application to prestressed members, particularly those without transverse shear reinforcement, remains limited in terms of experimental validation. Much of the literature either generalises shear behaviour across reinforced and prestressed systems or neglects the unique influence of prestressing on crack formation, shear transfer, and load redistribution. Although this represents an important research area, it lies outside the scope of this thesis.

3

Methodology

This chapter describes the framework developed for the assessment of reclaimed girders and its application in the case study.

3.1. General framework

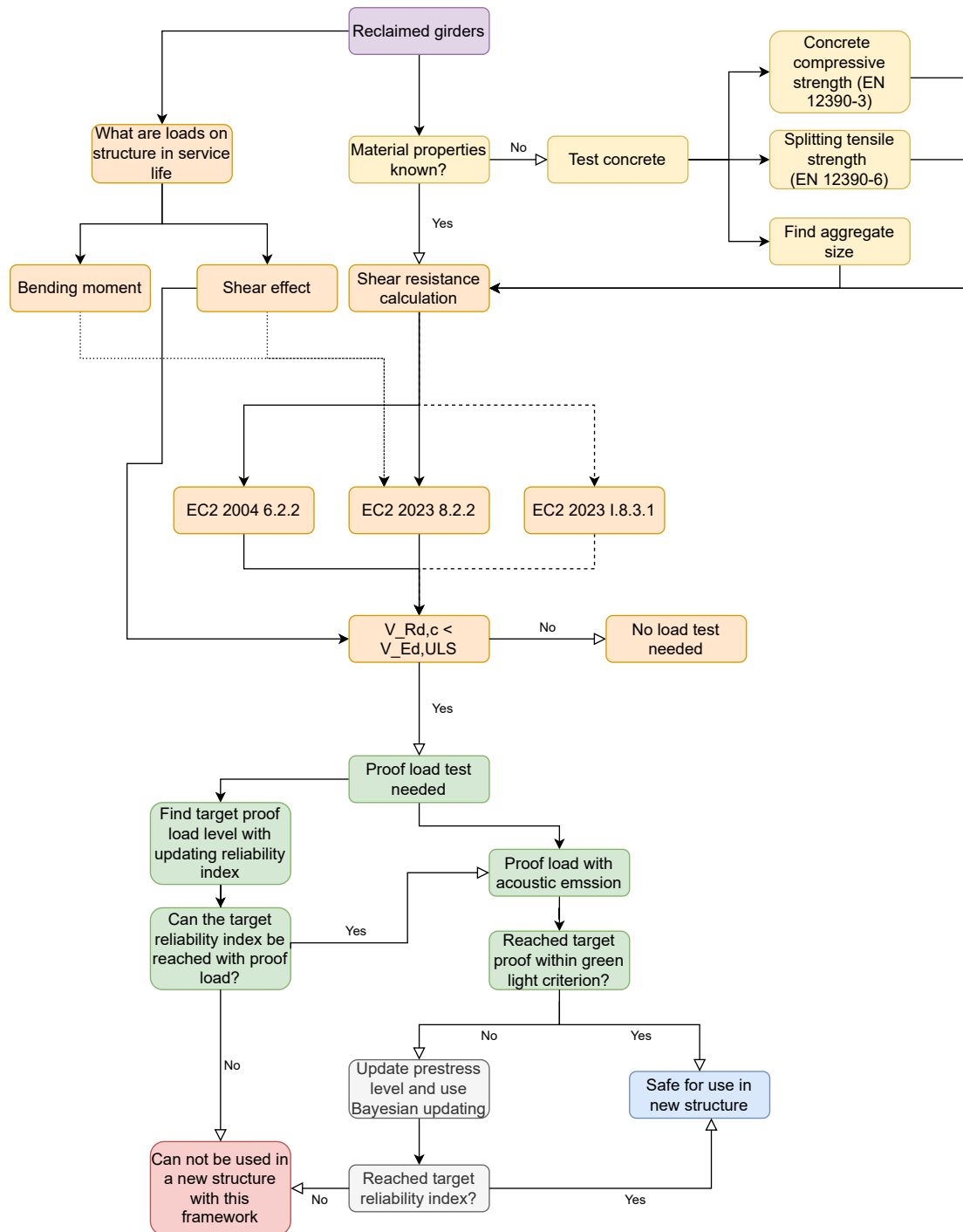


Figure 3.1: Flowchart of assessing reclaimed girders

3.2. Explanation framework

The proposed framework establishes a rigorous yet low-damage pathway to qualify reclaimed girders for reuse in a new structure. It begins with an information intake in which available drawings, fabrication

records, and design documents are used to identify key properties and potential critical regions. A systematic visual survey is then carried out to map the geometry and apparent condition of the reclaimed girders. In principle, girders with significant visible damage are not suitable for reuse. However, such damage cannot always be verified beforehand, because the sides of the girders are often not accessible when they are still in service in the original structure.

3.2.1. Material properties

If some material properties cannot be obtained from documentation (yellow part in figure 3.1), for example the concrete strength class, they can be determined by testing. Concrete compressive strength is established from cores drilled at locations with low stress so that the residual capacity of the girder is not significantly affected. The aggregate size can also be determined from laboratory tests on these cores. These properties are important inputs for calculating the shear resistance.

3.2.2. Shear load and resistance

With material and geometric information established, the anticipated actions in the intended new application are defined explicitly for both serviceability and ultimate limit states, including the relevant load combinations and partial factors (see orange part in figure 3.1). Structural verification then proceeds using the current and the next-generation Eurocode 2 formulations as the default reference. In the next-generation Eurocode there is also an extra verification for shear that may be applied to existing structures. For shear, the upcoming provisions recognize additional contributing mechanisms. Bending and other pertinent checks need to be performed in parallel, with clause references attached to the corresponding calculation records rather than dispersed throughout the narrative. Bending is mentioned explicitly in figure 3.1, only because it is needed for the calculation of certain variables in the next-generation Eurocode. This gives certain shear resistance and the shear effect from the loads. If the capacity is higher than the load effect, no load test is needed, otherwise a proof load test is needed. If so the green part from the framework needs to be done.

3.2.3. Update reliability index

If the deterministic checks indicate that the shear capacity is below the actual shear loads, a controlled proof-load test needs to be performed to assess whether the target reliability index can be reached using a Monte Carlo simulation, see green area in figure 3.1. The time-dependent reliability is then evaluated on a yearly grid from the proof year (year 0) to year 100 (the design life for new structures). At each year, the limit state is expressed in terms of an effect–resistance margin

$$Z(t) = \theta_R R(t) - \theta_E E(t),$$

where $R(t)$ denotes the available shear resistance at the control section, $E(t)$ the corresponding shear effect, and θ_R and θ_E are model uncertainty factors applied multiplicatively to resistance and effects, respectively. Random variables are sampled according to distributions given in NEN-EN 1990 (2019), and the annual probability of failure is obtained from first passage of the limit state, i.e., by counting realisations that become negative in a given year among those that had not failed earlier.

The shear effect $E(t)$ comprises a constant part due to dead load (the new situation also must be taken into account) and a variable part due to, for example, traffic, depending on the type of bridge. The resistance $R(t)$ is taken from the stochastic section and material inputs, and a deterioration trend in resistance can be implemented.

For each year, realisations that newly cross the limit state (negative Z) are identified among survivors from the previous year. The weighted fraction of these new failures over the total survivor weight yields the annual probability of failure $p_f(t)$. The corresponding annual reliability index can be calculated as

$$\beta(t) = -\Phi^{-1}(p_f(t)),$$

where Φ^{-1} is the inverse standard normal cumulative distribution function. This procedure provides a sequence of annual indices.

For updating, the proof load is implemented by truncating the resistance realisations below the observed proof-load level. This reduces the number of low-resistance samples and could increase the reliability

index. By implementing this, a target load level can be found at which the required reliability index could be reached, provided the girder survives to that load. It is also possible that there is no proof load level to reach the reliability what is needed. This way the girder cannot be reused or there needs to change something for example at the load part. An important aspect for reuse is that no significant cracks are allowed. During load testing the girder needs to stay within the green-light criterion developed by F. Zhang (2022). When the load level reaches the predefined value, the girder is safe for reuse in a new structure and moves to the blue area in figure 3.1.

Acoustic emission (AE) is the monitoring technique used to detect and evaluate internal damage in the girders during testing for this framework. AE relies on stress waves generated by the sudden release of strain energy within the material, such as cracking in concrete. These waves are captured by sensors attached to the specimen and converted into electrical signals. Each detected wave (a “hit”) yields a waveform from which key parameters, such as amplitude, rise time, duration, signal strength, peak frequency, and energy, are extracted. These parameters are used to identify damage events, classify source types, and assess the progression of structural degradation under load (F. Zhang, 2022). Peak frequency contains useful information for setting a threshold to filter background noise and for counting valid hits per sensor.

3.2.4. Update prestress level

When the reliability index cannot be increased sufficiently with load testing, the prestress level can be updated by a first-cracking (decompression) check, see gray area in figure 3.1, found using acoustic emission data. Assuming the concrete tensile strength is consistent with the concrete compressive strength (following the material properties in NEN-EN 1992-1-1, 2015), flexural cracking is deemed to initiate when the extreme-fibre tensile stress is exceeded. The bending-moment pattern used for this verification is the self-weight together with the applied proof load placed at the critical position. This provides a conservative verification aligned with the proof-loading assessment.

Updating this information in the given formulas could result in higher reliability indices, and it may be possible that a girder is safe after this update for use in a new structure (goes to blue area in figure 3.1). If still not, then this framework does not apply, or other shear-resistance calculation methods need to be used and moves to the red part in the framework, see figure 3.1.

3.3. Application in the case study

The framework is applied in the accompanying case study. The available load test was destructive and therefore not fully aligned with the “no significant damage” intent. Nevertheless, the methodology is followed as closely as possible to demonstrate the decision process and the role of updating.

4

Case Study

This chapter describes the experimental and analytical methods used in the case study. It details the loading scheme, sensor layout and AE acquisition, data processing and localisation procedures, and framework applied to quantify reliability described in chapter 3. Assumptions and parameter choices are stated to ensure reproducibility.

4.1. Experimental Overview

The experiments focused on the shear behaviour of old prestressed inverted T-beams. The tests included measurements of compressive strength and tensile splitting strength. Finite element modelling was used to determine the critical loading positions for the shear test (J. P. Pineda, 2025). The tests were conducted by Nebest, Strukton, and Mistras under the leadership of José Paredes Pineda. They subsequently shared the data with TU Delft for this research.

4.2. Tested girders A9

The girders tested in this study were originally part of the KW30 viaduct on the A9 motorway, constructed in 1968 and designed according to the RVB 1962 design guidelines. The reclaimed girders are of the HNP75/98 type F, see figure 4.2, and were recovered in 2023 for potential reuse.

The aim of the testing campaign was to evaluate the structural performance of the girders, specifically their shear capacity, in the context of reuse in a new viaduct at the A76 Daekderweg intersection near Nuth. Initial assessments indicated that the existing shear resistance might be insufficient for reuse without further measures.

The girders were previously used in a statically indeterminate structure. Additional information on the structural details and historical context is available in the study by José Paredes Pineda (J. P. Pineda, 2025).

The reclaimed girders were labelled from 1 to 33, and seven girders were damaged during reclaiming or transporting. Girders 6, 9, 15, 19, 22, 30, and 32 already showed damage at the bottom flange. Girder 19 had damage at both ends and was therefore deemed unsuitable for testing (J. E. P. Pineda, 2024). In this case study, only girder 6 is taken into account, because it was only possible to extract this test data from AEwin for Express-8.

4.2.1. Geometry girders

The original girders had a length of 22.55 meters. To reclaim the girders, they are shortened to a length of 20.3 meters, but due to bearing of 0.3 meters at each side the actual span will be 19.7 meters. (see figure 4.1).

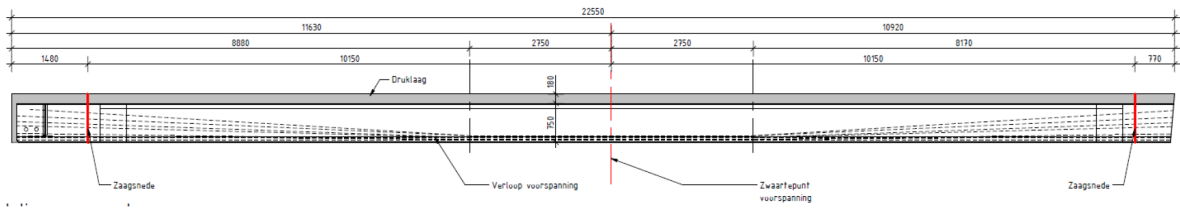


Figure 4.1: Longitudinal cross-section HNP75/98 (J. E. P. Pineda, 2024)

The total height of the HNP75/98 girder is 750 mm without the cast in situ top flange, and the width of the bottom flange is 980 mm. The web thickness decreases linearly from the thick web to the thin web. The corresponding cross-sections are shown in figures 4.2 and 4.3, while the cross-sectional properties are summarized in table 4.1. The variable web thickness primarily contributes to the additional self-weight of the girder (J. E. P. Pineda, 2024).

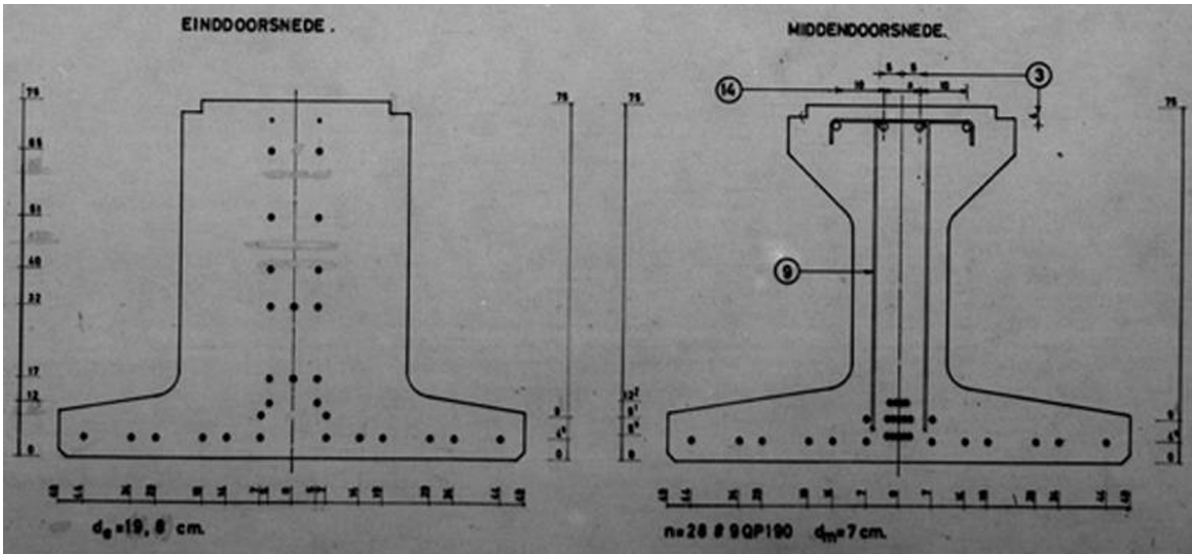


Figure 4.2: Cross-sections of prestressed girders HNP75/98 (J. E. P. Pineda, 2024)

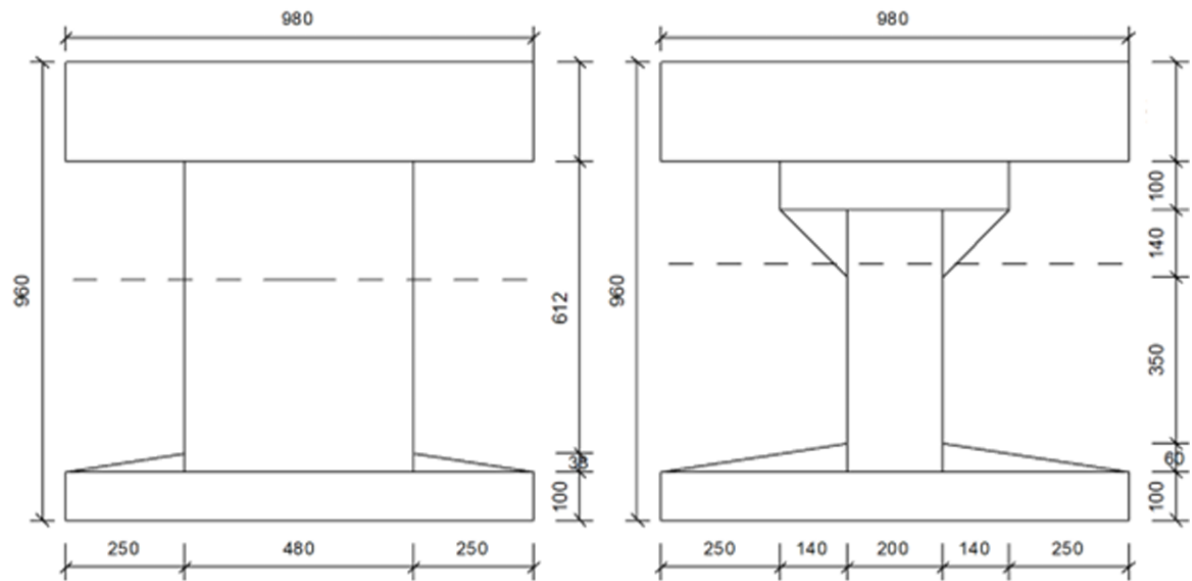


Figure 4.3: Simplified cross-section HNP75/98 (J. E. P. Pineda, 2024)

Table 4.1: Cross-sectional properties of the simplified geometry of girders HNP75/98 type F without top flange

Description	Thick web	Thin web	Units
Area	419500	299000	mm ²
Second moment of area	22015106007	19832576774	mm ⁴
Distance to c.g. from the bottom (z_b)	330.3	315.7	mm
Distance to c.g. from the top (z_t)	419.7	434.3	mm
Section modulus (bottom) (W_b)	66647290	62828263	mm ³
Section modulus (top) (W_t)	52457211	45661757	mm ³

Measurements to the thickness of the top flange was measured randomly for 5 girders. It has shown that the average thickness is 211 mm with a standard deviation of 11.5 mm. Following the construction papers the thickness should be 180 mm. The width of the bottom flange was also measured randomly for 5 girders. The average width is 983 mm with a standard deviation of 2.45 mm (J. E. P. Pineda, 2024).

4.2.2. Prestressing layout

The Type F girders are prestressed with 28 strands of 7-wire tendons (ϕ 12 mm, QP190), each with an effective cross-sectional area of 92.4 mm². The tendons are arranged across eight layers: the two bottom layers consist of 12 strand at the bottom and 2 strands above, respectively, following a straight profile, while the six upper layers follow a kinked profile that includes a straight segment over the central 5.5 m of the span.

In addition, two ϕ 5 mm prestressing wires are located near the top of the cross-section. However, these wires are neglected in the structural calculations due to their minimal contribution.

The vertical position of the centroid of the tendon group varies along the girder length, ranging from 69.9 mm from the bottom fibre at midspan to 198.2 mm near the supports. For intermediate sections, the vertical position is determined by linear interpolation. The full tendon layout is illustrated in figure 4.1 (J. E. P. Pineda, 2024).

4.2.3. Reinforcement layout

Longitudinal reinforcement

The longitudinal reinforcement in the top flange consists of QR40 ribbed bars. Both the upper and lower layers of reinforcement in the top flange are detailed as ϕ 10 mm at 150 mm spacing. In regions subject to hogging moments, the reinforcement is increased to ϕ 22 mm at 100 mm spacing. The design concrete cover was specified as 30 mm. However, measurements indicate that the actual cover is approximately 40 mm. No longitudinal reinforcement is present in the web of the girder. For the determination of ρ_l the area of the prestress is taken (J. E. P. Pineda, 2024).

Shear reinforcement

The web shear reinforcement consists of ϕ 6 mm QR40 ribbed stirrups distributed along the entire length of the girder. As illustrated in the right section of Figure 4.2, these stirrups are fully anchored at the top but not at the bottom. In accordance with RBK 1.2, such stirrups must be disregarded in shear capacity calculations due to insufficient anchorage (J. E. P. Pineda, 2024; RBK 1.2, 2022).

Interface reinforcement

The reinforcement connecting the precast girder to the top flange varies along the span. At the left end (over the first 5.6 m), interface hairpins are provided as ϕ 14 mm at 200 mm spacing. Similarly, at the right end (over 5.0 m), the same detailing is applied. In the central portion of the girder, which spans 11.4 m, the hairpins are spaced at 300 mm (J. E. P. Pineda, 2024).

4.2.4. Material properties

Concrete

The concrete compressive strength and splitting tensile strength were determined in 2021 from cores extracted from reclaimed girders. Cores were taken from five girders at three locations along each

girder. Each core had a diameter of 80 mm and an approximate length of 100 mm. Compressive strength was determined on 12 cores in accordance with EN 12390-3, and splitting tensile strength was determined on 6 cores in accordance with EN 12390-6. The found mean values are $f_{cm} = 113.1$ MPa and $f_{ctm,sp} = 7.25$ MPa. The characteristic cube compressive strength was calculated in accordance with RBK 1.1. Based on these test results, the concrete strength class of the girders is C80/95 (J. E. P. Pineda, 2024). The strength class of the cast-in-place top flange is assumed to be C35/45, based on historical design documentation.

Prestressing and reinforcement steel

According to RBK 1.2, the prestressing strand type QP190 has the following characteristic properties: tensile strength $f_{pk} = 1864$ MPa, 0.1% proof strength $f_{p0.1k} = 1619$ MPa, $f_{pd} = 1472$ MPa and ultimate strain $\varepsilon_{uk} = 3.5\%$. The reinforcement steel is of type QR40 with a characteristic yield strength of $f_{yk} = 400$ MPa (RBK 1.2, 2022).

4.3. Structural behaviour of prestressed girders

4.3.1. Shear behaviour

The shear resistance of the girders is assessed using both theoretical calculations and experimental validation. The theoretical assessment follows the provisions of the current Eurocode 2 (NEN-EN 1992-1-1, 2015) and considers upcoming updates from the second generation of Eurocodes. In this case study, the contribution of transverse shear reinforcement is neglected due to the lack of proper anchorage, as per RBK 1.2 guidelines.

Experimental validation via load testing

To validate the analytical shear assessment and to quantify the actual shear behaviour, a three-point bending test was performed. Although the effective span in the new configuration is 19.7 m, the actual span was reduced due to end damage on the tested girders. The damaged ends acted as cantilevers, effectively shortening the load-bearing span. The effective span during the test was 17 m. The self-weight resulted in a hogging moment at the support with cantilever action. The test setup is visible in figure 4.4



Figure 4.4: Test setup (van Duinen, 2025)

A point load was applied at the centre of the composite layer over an area of 400×400 mm², corresponding to the wheel contact area in Load Model 1 (NEN-EN 1991-2, 2015). Four loading positions were defined by the shear-span-to-depth ratio A/D . Specifically, girders 6, 9 and 32 were tested

at $A/D = 5.0$, girder 15 at $A/D = 4.5$, girder 22 at $A/D = 4.0$, and girder 30 at $A/D = 6.0$. This arrangement enabled comparison of the structural response at different loading positions. Finite-element analyses in DIANA indicated that $A/D = 5.0$ is critical. Accordingly, and as noted earlier, only girder 6 is considered in this case study.



Figure 4.5: Load application joint (Howse et al., 2025)

The critical loading position was determined using the FE program DIANA. The test was conducted under load controlled load test. Data acquisition included jack force measurements, digital image correlation (DIC), LVDTs and acoustic emission (AE) monitoring. For this case study the DIC and LVDT data is not used.

Loading scheme

Loading was force-controlled and applied in increments of 50 kN. After each increment, a dwell period of 3 min was imposed to allow transient effects to decay and to verify displacements from the LVDTs, as well as to monitor any continued AE activity indicative of ongoing damage. Extended load holds were scheduled at the Serviceability Limit State (SLS), the Ultimate Limit State (ULS), and at the NLFEA-predicted capacity to facilitate detailed observations at these reference points.

To assess residual (plastic) deformation and to provide additional context for the AE analysis, the protocol included controlled unloading sequences to approximately 100 kN, followed by reloading. These unload–reload cycles also served to check for plastic deformation relative to earlier stages (Howse et al., 2025).

The maximum load application rate was 3 kN/s. Due to test-stand limitations and stability considerations, the measured rate during the campaign varied between 2 and 8 kN/s. This variation was documented for each step and considered in the interpretation of both deformation and AE data.

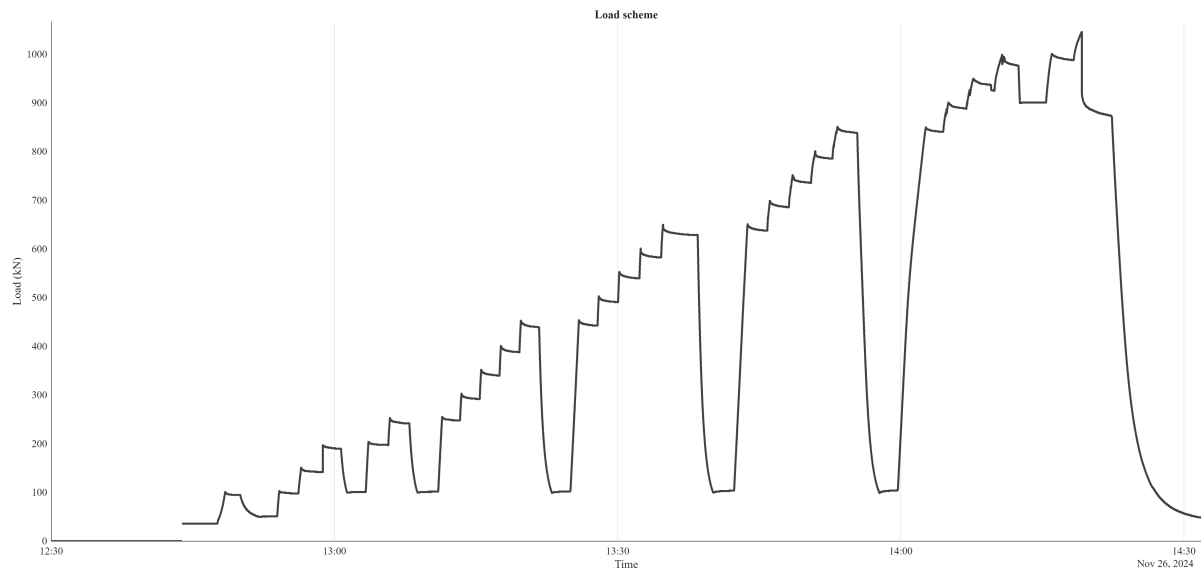


Figure 4.6: Example loading scheme

4.4. Measuring Techniques

4.4.1. Acoustic emission

Twenty-four Physical Acoustics R3I AE sensors (nominal resonance of 30 kHz) were installed on the girder surfaces. The array comprised four rows: one row on the top flange to monitor potential delamination, two rows on the webs to detect shear cracks, and one row on the bottom flange to detect flexural cracks. The test setup with sensors is shown in figure 4.7 and the sensor layout is shown in figure 4.8. Sensor numbering starts at the support (sensor 1). The two middle rows on the webs use a spacing of 1.0 m between sensors. On the top flange, sensor 23 is located at the loading point and the spacing along the row is 1.5 m. On the bottom flange, the sensor spacing is 1.0 m. A noise threshold of 40 dB was used (Howse et al., 2025).

Sensors mounted on the top flange are excluded from the quantitative AE analysis. They are placed on the flange side and therefore lie out of plane relative to the web and bottom-flange sensors, which introduces path and coupling differences that are not comparable to the other sensors. As a result, their measurements are not considered reliable for consistent interpretation with the other sensor rows.



Figure 4.7: Test setup with AE sensors (Sengers et al., 2025)

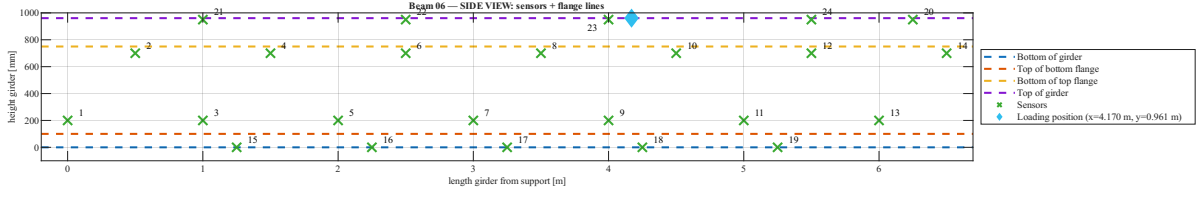


Figure 4.8: Sensor layout of beam 6

The AE sensors are placed on the girder with a magnetic hold-downs, which can be put at metallic washers that are epoxied to the girder, so there was no drilling needed, which could cause extra stresses (Howse et al., 2025).

4.5. AE data processing

Acoustic emission (AE) activity was monitored in real time during the load tests using AEwin for Express-8 (Mistras). The acquisition software wrote sequential .dta files, each limited to roughly 200 MB to prevent oversized files and reduce the risk of corruption (Howse et al., 2025). The data were exported through AEwin ASCII output by selecting the variables required for analysis and saved as plain-text .txt files. The load signal was recorded in parallel and likewise exported as a .txt file with time stamps, enabling direct synchronization of AE activity with the loading scheme. All files were imported into MATLAB, concatenated in chronological order, and aligned on a common time base using the recorded start time.

Intervals associated with the intentional unloads to approximately 100 kN, which were included in the protocol to assess plastic deformation, were excluded from the interpretation because different mechanisms dominate during unloading and are outside the scope of this study. The workflow includes routine quality control to screen non-physical records (for example, zero duration with nonzero energy) and ensure that apparent AE peaks coincide with load steps rather than with logging errors. The MATLAB scripts used for these steps are parameterized by bin size and thresholds, and produce the figures and tables referenced in the results.

4.5.1. AE hits

Hit-level features exported from AEwin included time stamp, channel ID (sensor), amplitude, energy, duration, counts, peak frequency, and, where available, partial power. The baseline MATLAB script, originally shared by Fengqiao Zhang, was adapted to the reclaimed girder dataset and the current export format. To suppress background activity not associated with load testing sources, the analysis retained only hits with peak frequency greater than 27.5 kHz. This choice was verified to remove constant low-frequency noise while preserving load-correlated activity. Further details on the choice of the peak frequency threshold are given in section 5.3. The retained hits were put into non-overlapping 1 s bins to reduce burstiness and reveal trends at the time scale of the step-and-hold loading. Three complementary visualizations were generated in a consistent layout: a combined hit-rate history for all sensors to show global activity, per-sensor hit-rate histories to localise activation, and per-sensor cumulative counts to highlight step changes and persistence. In each case, the load history was overlaid so that peaks could be related directly to load increments, constant-load holds, and unload-reload cycles. The unload intervals to approximately 100 kN were masked in the plots to avoid misinterpretation, in line with the exclusions described above.

4.5.2. AE source localisation

Source localisation was carried out in MATLAB using a time-of-arrival approach on hit-level data to estimate the 2D source position in the selected plane (x-y). A single effective wave speed of $v = 3200 \text{ m/s}$ was adopted for all events. For each burst, arrivals were taken directly from the exported AE hit times. Hits were first filtered by peak frequency ($PF > 27.5 \text{ kHz}$) and channels 15–24 were excluded. Events were formed per cluster by sorting hit times and grouping arrivals that occurred within a time window $\Delta t = \text{span}/v$, where span is the diagonal of the sensor bounding box for that cluster. Only the earliest pick per channel was retained, and a minimum of three unique sensors was required.

The forward model assumes straight-line propagation in a homogeneous medium, with predicted arrival times equal to an origin time plus the travel time over the source–sensor distance divided by v . To avoid estimating an unknown event start time, a common reference (e.g., the mean or a selected sensor's pick) is subtracted from all arrival times, and the misfit is computed on these time differences so that only relative arrivals enter the localisation. Source coordinates were obtained by minimizing an unweighted least-squares misfit over a uniform grid that spans the instrumented region for the cluster (grid step $g_d = 0.01$ m with a 0.10 m padding around the sensors). Candidate solutions were accepted only if the location lay inside the convex hull of the contributing sensors for that burst. Otherwise the burst was discarded.

4.6. Update reliability index

The traffic component of the annual shear effect is modelled as the product of an annual maximum shear (Gumbel) and a dynamic amplification factor (lognormal), scaled in time by a multiplicative traffic level. The traffic level equals one after the proof year and thereafter grows stochastically according to a lognormal annual growth factor, so that trends in loading are reflected directly in the annual hazards. In the present analysis no deterioration is applied for the resistance.

Plain Monte Carlo sampling is used for all variables (equal weights), and annual failure probabilities are evaluated by first passage over the 0–100 year horizon with the proof-year floor applied to the variable shear. All inputs, distributions, and coefficients of variation used to construct $E(t)$ and $R(t)$ are listed in Tables 4.2 and 4.3.

4.6.1. Probabilistic model

Unless stated otherwise, random variables are modelled as lognormal. Annual maxima of the traffic action are modelled as Gumbel, and permanent loads as normal. The choice of distributions follows NEN-EN 1990 (2019), de Vries et al. (2023), Joint Committee on Structural Safety (2021), and Adonizio et al. (2024). Coefficients of variation (COVs) are taken from Joint Committee on Structural Safety (2021), except for traffic. Traffic load statistics are based on TNO WIM data for Dutch highways to derive the annual maximum traffic effect and its variability (de Vries et al., 2023). Traffic growth over the design lifetime is represented by a lognormal annual growth multiplier with mean 0.0013 (i.e., $\approx 13\%$ total increase over 100 years) following Steenbergen et al. (2012). Dynamic amplification is included via a dynamic load factor $C_{0Q} \sim \text{LN}(1.23, 0.14)$ applied to the traffic component, consistent with NEN-EN 1991-2 (2015). Quantities treated as deterministic are fixed in the analysis.

Prestress is introduced via the average concrete compressive stress $\sigma_{cp} = F_p/A_c$ and the axial force $N_{Ed} = -F_p$ (compression negative). In EC2:2004, prestress enters $V_{Rd,c}$ directly through the $k_1 \sigma_{cp}$ term in both the basic and minimum expressions. In EC2:2023, prestress affects the resistance through the critical section

$$a_{cs} = \max\left(\frac{|M_{Ed} + N_{Ed} e_p|}{V_{Ed}}, d\right),$$

which defines

$$a_v = \sqrt{\frac{a_{cs} d}{4}},$$

used only in $V_{Rd,c}$. Prestress also enters via

$$k_{vp} = 1 + \frac{N_{Ed}}{|V_{Ed}|} \cdot \frac{d}{3 a_{cs}},$$

bounded below by 0.1, which appears in $V_{Rd,c}$ through the ratio $d_{dg}/(k_{vp} a_v)$. The EC2:2023 minimum expression $V_{Rd,c,\min}$ does not depend on prestress.

For the proof year ($t = 0$), the variable traffic component is floored at the proof shear V_p . The constant part uses the proof self-weight value when the proof load does not include self-weight. For plotting, if V_p excludes self-weight, the visualization cut is placed at $V_p + \mathbb{E}[G_{DL}]$. A common effects distribution

$$E = G_{DL} + G_{SDL} + C_{0Q} Q$$

is used in the “before” and “after” histograms so that resistances and effects are compared on the same axis. Importance sampling is implemented in the code but disabled in the present configuration. Plain Monte Carlo (equal weights) is used. Figures employ fixed bin edges on $[0, 800]$ kN with 160 bins. After proof, the left tails of the R histograms (EC2:2004 and EC2:2023) are shaded by truncating the precomputed bin counts at the proof cut for visualization only. The reliability analysis conditions on the proof test by flooring the variable shear at the proof level in the proof year and removing any realisations that fail at $t = 0$ from the survivor set for subsequent years. No explicit truncation of the resistance distribution is applied. Annual reliability is reported as $\beta(y) = -\Phi^{-1}(p_f(y))$ and plotted from year 2 onward (years counted from $t = 0$), since the immediate post-proof filtering dominates the first year after the test. Some values will be discussed in greater detail in Chapter 5, where the calculations are presented and the values reported in the tables below are derived.

Stochastic inputs are sampled independently. A single exception is the concrete area, which is constructed deterministically from the random geometric components to maintain sectional consistency. This ties the realized section area directly to the sampled widths and heights while leaving other basis variables (e.g. d , $b_{w,2004}$, $b_{w,2023}$, A_p , f_{cm} , f_{yd}) marginally specified and mutually independent. Note that $b_{w,2004}$ and $b_{w,2023}$ are treated as separate stochastic inputs because they enter different code formulations. No explicit cross-correlation is imposed between them. The script that is used, is shown in A.

Table 4.2: Loads and actions used in the reliability analysis

Quantity	Mean	COV	Distribution
Global bending at section M_{Ed}	492.17 kNm	0.10	Lognormal
Global shear effect at section V_{Ed}	430.94 kN	0.10	Lognormal
Axial force N_{Ed} (from prestress)	2507 kN	0.10	Lognormal
Proof shear V_p	415.09 kN	–	Deterministic
Dead load shear (proof year) G_{DL}	95.63 kN	0.04	Normal
Dead load shear (new situation) G_{DL}	120.86 kN	0.04	Normal
Dead load shear (asphalt) G_{SDL}	30.02 kN	0.25	Normal
Annual max variable shear (traffic) Q	234.33 kN	0.03	Gumbel I
Dynamic multiplier on Q , C_{0Q}	1.23	0.14	Lognormal
Traffic scale at proof year c_{t0}	1.00	0.10	Lognormal
Traffic annual growth d_{ct}	0.0013 /yr	0.14	Lognormal
Change point for resistance c_{rt0}	–	–	Deterministic
Resistance decay slope d_{cr}	0 /yr	–	Deterministic
Model uncertainty, resistance θ_R	1.00	0.05	Lognormal
Model uncertainty, effects θ_E	1.00	0.10	Lognormal

Table 4.3: Material properties and geometry

Quantity	Mean	COV	Distribution
Concrete mean compressive strength f_{cm}	88 MPa	0.06	Lognormal
Concrete characteristic strength f_{ck}	80.0 MPa	0.06	Lognormal
Steel yield strength f_{ym}	650 MPa	—	Deterministic
Axial force from prestress $N_{Ed} = -F_p$	2507 kN	0.10	Lognormal
Eccentricity of prestress e_p	338.13 mm	—	Deterministic
Aggregate lower size D_{lower}	32 mm	0.02	Lognormal
Effective depth d	795.57 mm	0.02	Lognormal
Web width b_w	200.0 mm	0.05	Lognormal
Web width b_w (EC2:2004, RBK 1.2)	250.0 mm	0.05	Lognormal
Web width b_w (EC2:2023)	200.0 mm	0.05	Lognormal
Top flange height (updated)	250 mm	0.05	Lognormal
Width top flange	980 mm	0.05	Lognormal
Hight web area	650 mm	0.05	Lognormal
Prestressing steel area A_p	2,587.2 mm ²	0.05	Lognormal
Total depth h	1,000.0 mm	0.05	Lognormal

4.7. Loads in the use phase

During the service life of a prestressed concrete girder, several permanent and variable loads act on the structure. These loads must be correctly identified and modelled to accurately assess structural behaviour under both serviceability and ultimate limit states.

Self-weight: The primary permanent load, calculated based on the girder's cross-sectional geometry and the unit weight of concrete (typically 24.5 kN/m³). For precast inverted T-beams, the self-weight includes contributions from the compressive layer, the top flange.

Superimposed permanent loads: These include the assumed weight of asphalt layer. Standard asphalt thicknesses range from 140 mm to 150 mm, depending on the road category and pavement detailing, resulting in an additional permanent load of approximately 3.22–3.45 kN/m² (Rijkswaterstaat, 2021).

Prestressing force: Treated as an imposed internal action, introduced during the transfer phase. Acting with an eccentricity, it generates a counteracting moment that reduces tensile stresses from bending due to gravity loads. Due to two kinks, there are two vertical forces acting upwards. The effective prestress, accounting for time-dependent losses (shrinkage, creep, and relaxation), is used what was accounted for in the design. The initial prestress is 1212 MPa per tendon and with accounting the losses the remaining prestress is 969 MPa per tendon. The total effective prestress is in the end 2507 kN.

Live loads: For the deterministic load-effect calculations, Eurocode 1 Load Model 1 (LM1) is adopted (NEN-EN 1991-2, 2015). LM1 consists of a tandem system (TS) of concentrated wheel loads applied within notional traffic lanes, together with a uniformly distributed load (UDL) representing the background traffic stream.

For updating the reliability index after proof loading, Weight-in-Motion (WIM) measurements reported by TNO are used (de Vries et al., 2023). The traffic loading is decomposed and modelled with a mean (baseline) component for the lane EUDL and axle/tandem effects, a dynamic component, and a future trend. These components are treated separately in the reliability analysis. Together with the proof-load outcome, the WIM-based model characterises annual maxima and supports a time-dependent reliability assessment that reflects the observed traffic at the bridge.

These loads form the basis for both numerical modelling and test result interpretation. During proof load testing, selected permanent and live loads are replicated to validate the structural response against analytical predictions.

5

Results

This chapter presents the results that address the research questions posed at the beginning of the study. It reports the acting loads and their effects, the code-based resistance checks, the acoustic-emission observations and source localisation, and the probabilistic reliability outcomes. The results are organised to answer the sub-questions step by step and to prepare the answer to the main research question.

5.1. Acting loads on the structure

The girders in the new structure are simply supported with a span of $L = 19.7$ m. The composite cross-section is idealised as web, bottom flange, and compression slab. The dimensions of the reclaimed girders are given in table 4.1. In that table the top flange of the composite section is not yet included, so the neutral axis must be recalculated with the compressive layer (top flange) taken into account. For the section-property calculations, the web is modelled using only the thin-web area. The locally thickened portion is neglected because it is small and was largely cut off. The loads are determined in accordance with ROK 2.0 and NEN-EN 1991-2. As a first step, the neutral axis including the compressive layer is established for the thin-web model.

$$\begin{aligned} z &= \frac{A_i \cdot a_i}{A_i} \\ z &= \frac{(211 \cdot 980) \cdot (0.5 \cdot 211) + 299000 \cdot (434.3 + 211)}{(211 \cdot 980) + 299000} &= 424.67 \text{ mm} \\ A_c &= 299000 + 211 \cdot 980 &= 505780 \text{ mm}^2 \end{aligned}$$

and, with unit weight $\gamma_c = 24.5 \text{ kN/m}^3$, the full-span distributed load due to the self-weight of the girder is

$$q_{self} = \gamma_c \cdot A_c \cdot 10^{-6} = 24.5 \cdot 505780 \cdot 10^{-6} = 12.39 \text{ kN/m}$$

The asphalt distributed load is determined in section 4.7 and due to a layer of 150 mm asphalt, the distributed load is

$$q_{asphalt} = 3.45 \cdot \frac{b_{f,top}}{1000} = 3.45 \cdot \frac{980}{1000} = 3.31 \text{ kN/m}$$

The traffic distributed load acts over the full span. It is assumed that the distributed load on lane one is acting on the structure and is taken as

$$q_{traffic} = \alpha_{qi} \cdot q_k \cdot \frac{b_{f,top}}{1000} = 1.15 \cdot 9 \cdot \frac{980}{1000} = 10.14 \text{ kN/m}$$

Furthermore, for the traffic load, two axle loads are to be applied to the girder. The distance between the axles is 1.2 m, and the distance between the two tyres on the same axle is 2 m. In Load Model 1, each axle load is 300 kN, corresponding to 150 kN per tyre. Since the girder's width and height are both less than 1 m, the load from the axles can be considered as distributed to the neutral axis of the girder, with an effective width equal to the tyre width plus twice the vertical distance to the neutral axis. The tyre contact area is assumed to be 0.4 m × 0.4 m. Load distribution to the neutral axis is assumed to occur at a 45 degrees spread. The axles will be positioned at different locations to produce the maximum shear force and bending moment.

$$F_{TS} = F_{tyre} \frac{b_f}{0.4 + 2d_{NA}} = 150 \frac{0.98}{0.4 + 2 \cdot 0.42461} = 117.67 \text{ kN}$$

This value is used as the longitudinal point load per axle in the following.

5.1.1. Bending moment

For a simply supported girder, the tandem system is positioned to maximize midspan bending: one axle at midspan and the second axle 1.2 m away (unfavourable side). The tandem moment is taken from Matrixframe:

$$x_1 = \frac{L}{2} \text{ m}, \quad x_2 = \frac{L}{2} + 1.2 \text{ m}$$

$$M_{TS} = 1088.48 \text{ kNm} \quad (\text{Matrixframe})$$

The other actions are distributed loads. For a simply supported span, the midspan moment is:

$$M_{\text{mid}} = \frac{q L^2}{8}$$

Hence, the bending moments from the distributed loads are:

$$\begin{aligned} M_G &= \frac{12.39 \cdot 19.7^2}{8} = 601.05 \text{ kNm} \\ M_{\text{asphalt}} &= \frac{3.31 \cdot 19.7^2}{8} = 160.57 \text{ kNm} \\ M_{\text{traffic}} &= \frac{10.14 \cdot 19.7^2}{8} = 491.90 \text{ kNm} \end{aligned}$$

The midspan SLS moment by superposition:

$$M_{\text{SLS}} = M_G + M_{\text{asphalt}} + M_{\text{traffic}} + M_{TS} = 2342 \text{ kNm}$$

Ultimate combinations per NEN-EN 1990:

$$\begin{aligned} M_{\text{ULS}}^{6.10a} &= 1.4 M_G + 1.4 M_{\text{asphalt}} + 1.2 (M_{\text{traffic}} + M_{TS}) = 2962.69 \text{ kNm} \\ M_{\text{ULS}}^{6.10b} &= 1.25 M_G + 1.25 M_{\text{asphalt}} + 1.5 (M_{\text{traffic}} + M_{TS}) = 3322.60 \text{ kNm} \end{aligned}$$

so the governing ULS moment is

$$M_{\text{ULS}} = \max(M_{\text{ULS}}^{6.10a}, M_{\text{ULS}}^{6.10b}) = \max(2962.69, 3322.60) = 3322.60 \text{ kNm}.$$

5.1.2. Shear effects

Shear is verified at the section located at $x = 1.73 d_{\text{eff}}$ from the left support, following RBK 1.2, in which an angle of 30° may be used for prestressed girders (horizontal projection $\approx 1.73 d_{\text{eff}}$). To maximise the design shear at this section (and avoid direct transfer through the bearing), the LM1 tandem is positioned so that the nearest axle is at a distance d_{eff} from the support. With LM1 tandem axle spacing $s_{\text{TS}} = 1.20 \text{ m}$, the axle locations are

$$x_1 = 1.73 d_{\text{eff}}, \quad x_2 = 1.73 d_{\text{eff}} + 1.20 \text{ m}.$$

The control section at one effective depth from the support was determined iteratively, since the effective depth $d(x)$ depends on the local section properties at a distance x from the support. The prestressing eccentricity decreases over the first 7.4 m (including a 300 mm support plate), from 198.2 mm to 69.9 mm measured from the bottom. Using a fixed-point iteration,

$$d_i = d(x_i), \quad x_{i+1} = d_i,$$

with convergence criterion $|x_{i+1} - x_i| < \varepsilon$ and $\varepsilon = 1$ mm, the procedure converged to

$$d_{\text{eff}} = 791.73 \text{ mm}, \quad x = 1.73 d_{\text{eff}} = 1.3697 \text{ m}.$$

Hence the axle positions used for design shear are

$$x_1 = 1.3697 \text{ m}, \quad x_2 = 2.7697 \text{ m}.$$

An explicit evaluation of d_{eff} at the control section is:

$$d = 961 - \left(198.2 - \frac{198.2 - 69.9}{7400} (1.73 \cdot 791 + 300) \right) \text{ mm} = 791.73 \text{ mm}.$$

For a full-span distributed load, the shear can be calculated the following way

$$V_q = q \left(\frac{L}{2} - x_i \right).$$

For point loads the shear is found using MatrixFrame:

$$V_{TS} = 211.81 \text{ kN}$$

This taken into account gives the following shear values

V_{eq}	$= 12.39 \cdot \left(\frac{19.7}{2} - 1.73 \cdot 0.79173 \right)$	$= 105.07 \text{ kN}$
$V_{asphalt}$	$= 3.31 \cdot \left(\frac{19.7}{2} - 1.73 \cdot 0.79173 \right)$	$= 28.07 \text{ kN}$
$V_{traffic}$	$= 10.14 \cdot \left(\frac{19.7}{2} - 1.73 \cdot 0.78154 \right)$	$= 85.99 \text{ kN}$

The SLS shear is obtained at $x = d_{\text{eff}}$ and is

$$V_{SLS} = V_{eq} + V_{asphalt} + V_{traffic} + V_{TS} = 430.94 \text{ kN}$$

The ultimate combinations for shear are evaluated following the NEN-EN 1990 as

$$\begin{aligned} V_{ULS}^{6.10a} &= 1.4 V_G + 1.4 V_{As} + 1.2 (V_Q + V_{TS}), \\ V_{ULS}^{6.10b} &= 1.25 V_G + 1.25 V_{As} + 1.5 (V_Q + V_{TS}), \end{aligned}$$

and the governing ULS shear is

$$V_{ULS} = \max(V_{ULS}^{6.10a}, V_{ULS}^{6.10b}) = \max(543.76 \text{ kN}; 613.13 \text{ kN})$$

5.2. Resistance girder Eurocode

5.2.1. Shear resistance Eurocode 2 2004

For the effective depth, there is taken the distance from the prestress to the top compressive fibre from 1.73 times d from the support, to neglect the the direct shear transfer. Except the Eurocode 2 2004, also the RBK 1.2 is used and gives that $b_{w,gem}$ can be used.

$$\begin{aligned} V_{Rd,c} &= (C_{Rd,c} \cdot k \cdot (100 \cdot \rho_l \cdot f_{ck})^{1/3} + k_1 \cdot \sigma_{cp}) \cdot b_{w,gem} \cdot d \\ V_{Rd,c,min} &= (v_{min} + k_1 \cdot \sigma_{cp}) \cdot b_{w,gem} \cdot d \end{aligned}$$

$$\begin{aligned} d &= 961 - (198.2 - \frac{198.2 - 69.9}{7400} \cdot (791 + 300)) = 791.73 \text{ mm} \\ b_{w,gem} &= \frac{A_{b,pro}}{d} \leq 1.25 \cdot b_{min} = \frac{791.73 \cdot 200 + 121661}{791.73} \leq 1.25 \cdot 200 = 250 \text{ mm} \\ k &= 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{791.73}} = 1.503 \leq 2.0 \\ \rho_l &= \frac{A_p}{b_{w,gem} \cdot d} = \frac{28 \cdot 92.4}{250 \cdot 791.73} = 0.0131 \leq 0.02 \\ \sigma_{cp} &= \frac{N_{Ed}}{A_{c,precast}} < 0.2 \cdot f_{cd} = \frac{2507 \cdot 10^3}{299000} < 0.2 \cdot \frac{80}{1.5} = 8.38 < 10.67 \text{ MPa} \\ C_{Rd,c} &= \frac{0.18}{\gamma_c} = \frac{0.18}{1.5} = 0.12 \\ v_{min} &= 0.035 \cdot k^{3/2} \cdot f_{ck}^{1/2} = 0.035 \cdot 1.503^{3/2} \cdot 80^{1/2} = 0.577 \\ k_1 &= 0.15 \end{aligned}$$

$$\begin{aligned} V_{Rd,c} &= (0.12 \cdot 1.503 \cdot (100 \cdot 0.0131 \cdot 80)^{1/3} + 0.15 \cdot 8.38) \cdot 250 \cdot 791.73 = 417.11 \text{ kN} \\ V_{Rd,c,min} &= (0.577 + 0.15 \cdot 8.38) \cdot 250 \cdot 791.73 = 363.40 \text{ kN} \end{aligned}$$

5.2.2. Shear resistance Eurocode 2 2023 8.2.2

The upcoming Eurocode formulation for shear resistance includes additional contributing mechanisms that must be accounted for. Consequently, for members with an effective shear span $a_{cs} < 4d$, the effective depth d in the expression for $V_{Rd,c}$ is replaced by the mechanical shear span a_v . This substitution reflects the reduced shear span and its influence on concrete shear resistance in short-span regions. Here, a_{cs} denotes the effective shear span derived from the action ratio at the control section, and a_v is the mechanical shear span used to capture the shear–tension behaviour of short members. The partial safety factor γ_V is taken as 1.4, using table 4.3, because the national annex is not online yet to the authors' knowledge at the time of research.

$$\begin{aligned} V_{Rd,c} &= \left(\frac{0.66}{\gamma_V} \cdot (100 \rho_l \cdot f_{ck} \cdot \frac{d_{dg}}{k_{vp} \cdot d})^{\frac{1}{3}} \right) \cdot b_w \cdot d \\ V_{Rd,c,min} &= \left(\frac{11}{\gamma_V} \cdot \sqrt{\frac{f_{ck}}{f_{yd}} \cdot \frac{d_{dg}}{d}} \right) \cdot b_w \cdot d \end{aligned}$$

$$\begin{aligned}
d_{dg} &= 16 + D_{lower} \cdot \left(\frac{60}{f_{ck}}\right)^2 \leq 40 = 16 + 32 \cdot \left(\frac{60}{80}\right)^2 = 34 \text{ mm} \\
\rho_l &= \frac{A_p}{b_w \cdot d} = \frac{28 \cdot 92.4}{200 \cdot 791.73} = 0.0163 \leq 0.02 \\
a_{cs} &= \left| \frac{M_{Ed}}{V_{Ed}} \right| \geq d = \left| \frac{(492.17 - 2507 \cdot 0.33813) \cdot 10^6}{430.94 \cdot 10^3} \right| \geq 791.73 = 824.99 \text{ mm} \\
a_v &= \sqrt{\frac{a_{cs}}{4}} \cdot d = \sqrt{\frac{824.99}{4}} \cdot 791.73 = 404.09 \text{ mm} \\
k_{vp} &= 1 + \frac{N_{Ed}}{|V_{Ed}|} \cdot \frac{d}{3 \cdot \alpha_{cs}} \geq 0.1 \\
k_{vp} &= 1 + \frac{-2507}{|430.94|} \cdot \frac{791.73}{3 \cdot 824.99} \geq 0.1 = 0.1
\end{aligned}$$

$$\begin{aligned}
V_{Rd,c} &= \frac{0.66}{1.4} \cdot (100 \cdot 0.0163 \cdot 80 \cdot \frac{34}{0.1 \cdot 404.09})^{\frac{1}{3}} \cdot 200 \cdot 791.73 = 357.37 \text{ kN} \\
V_{Rd,c,min} &= (\frac{11}{1.4} \cdot \sqrt{\frac{80}{1472 - 969}} \cdot \frac{34}{791.73}) \cdot 200 \cdot 791.73 = 102.82 \text{ kN}
\end{aligned}$$

5.2.3. Shear resistance Eurocode 2 2023 I.8.3.1

If a more detailed verification is required for the assessment of an existing structure, an alternative to section 8.2.2 may be used to calculate the design value of the shear resistance of members without shear reinforcement. Since the girder contains only prestressing tendons, ε_v shall be determined for the prestressing steel, taking into account the strain due to prestressing and the resulting design strain thereafter. The prestress stress and strain values are determined using RBK 1.2 (2022) and NEN-EN 1992-1-1 (2015). The effects covered in 8.2.2(3)–(5) shall also be taken into account, these concern a_{cs} , a_v , and k_{vp} .

$$\begin{aligned}
V_{Rd,c} &= (0.33 \cdot \frac{\gamma_{def}^{\frac{2}{3}}}{\gamma_V^2} \cdot \frac{\sqrt{f_{ck}}}{1 + 24 \cdot \gamma_{def} \cdot \varepsilon_v \cdot \frac{d}{d_{dg}}}) \cdot b_w \cdot d \\
f_{pd} &= 1472 \text{ MPa} \\
\varepsilon_{ud} &= \frac{\varepsilon_{uk}}{\gamma_s} = \frac{3.5}{1.15} = 3.043\% \\
\varepsilon_{pd} &= \frac{f_{pd}}{E_p} = \frac{1472}{190000} = 0.775\% \\
\sigma_p &= \frac{F_p}{A_p} = \frac{2507 \cdot 10^3}{28 \cdot 92.4} = 969 \text{ MPa} \\
\Delta\sigma_{p,rest} &= f_{pd} - \sigma_p = 1472 - 969 = 503 \text{ MPa} \\
\varepsilon_v &= \frac{\varepsilon_{pd}}{f_{pd}} \cdot \Delta\sigma_{p,rest} = \frac{0.775 \cdot 10^{-2}}{1472} \cdot 503 \cdot 100 = 0.265\% \\
V_{Rd,c} &= (0.33 \cdot \frac{1.33^{\frac{2}{3}}}{1.4^2} \cdot \frac{\sqrt{80}}{1 + 24 \cdot 1.33 \cdot 0.265 \cdot 10^{-2} \cdot \frac{0.1 \cdot 404.09}{34}}) \cdot 200 \cdot 791.73 = 262.04 \text{ kN}
\end{aligned}$$

Since this value is below the shear resistance calculated using the next-generation Eurocode (section 8.2.2), this formulation is not considered further in this study.

5.3. Girder resistance test

Acoustic emission (AE) measurements were used to interpret damage evolution during the load test. The combined AE hit rate (all sensors and hits counted for all sensors) increases with increasing load, indicating progressive damage activation (see figure 5.1). Around a load level of approximately 600 kN, the hit rate rises markedly, and the cumulative number of hits approaches 50,000. Thereafter, each load increment is accompanied by a distinct peak in AE hits.

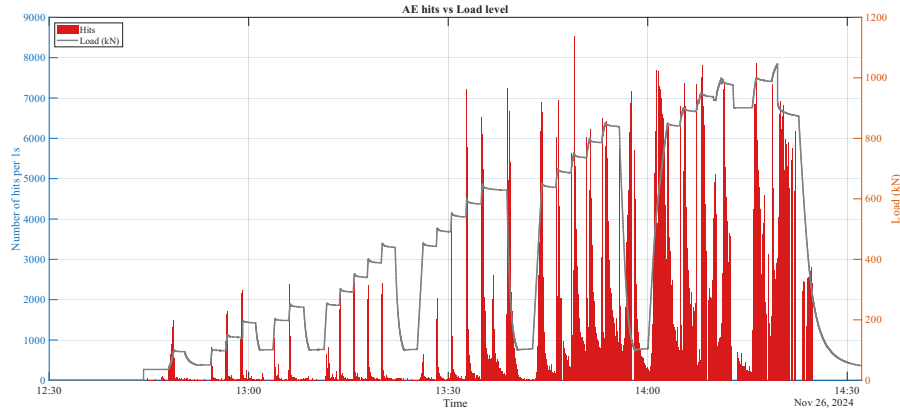


Figure 5.1: AE hits and load level versus time of beam 6

Several sensors showed peaks from the first load step, with similar peak heights repeating at each subsequent step. This pattern persisted until the load level at which the hit counts increased by roughly a factor of two, indicating the onset of cracking (Figures 5.2 and 5.3). Because these early, step-repeating peaks did not depend on the actual load level, they were interpreted as background activity and removed using a peak-frequency filter. A data-driven threshold of 27.5 kHz was selected after inspecting spectra across sensors. This suppresses low-frequency background while preserving load-correlated activity. After filtering, the large increases remain aligned with the same load steps, but most pre-peak activity is eliminated (Figures 5.4 and 5.7).

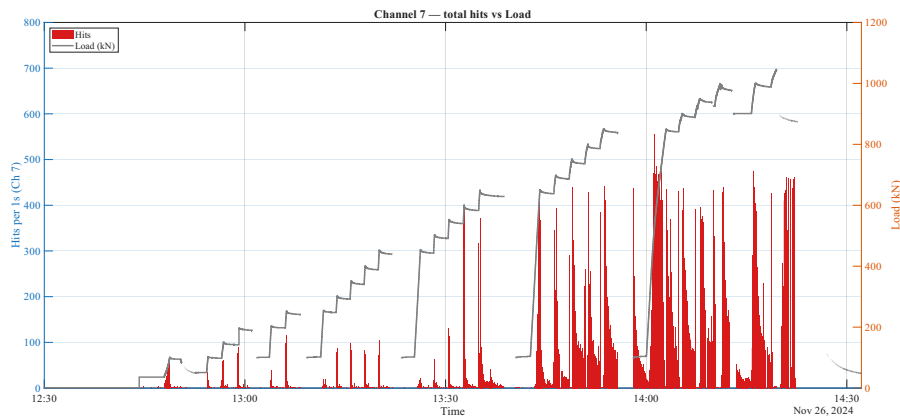


Figure 5.2: AE hits sensor 7 of beam 6 with no peak-frequency threshold

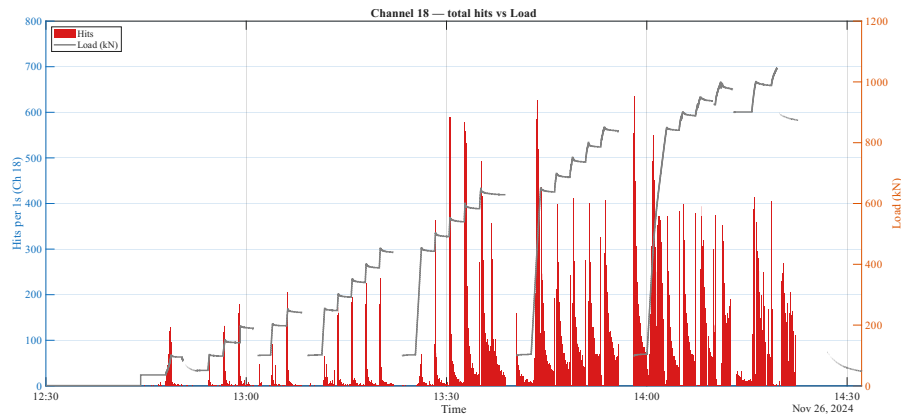


Figure 5.3: AE hits sensor 18 of beam 6 with no peak-frequency threshold

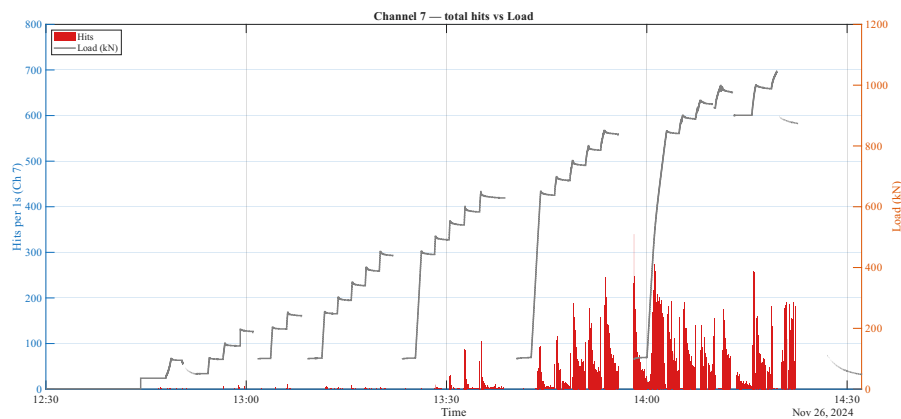


Figure 5.4: AE hits sensor 7 of beam 6 with peak-frequency threshold of 27.5 kHz

After identifying peaks in the combined signal, per-sensor timelines were examined to localise activity and relate it to the expected flexural and shear zones. The sensor locations are shown in figure 5.5. As visible in the figure, the bottom-flange and web sensors are mounted in the same plane on the precast girder, whereas the top-row sensors sit on the wider cast-in-place flange and are not in the same plane. To ensure consistent interpretation, the top-flange sensors were therefore excluded from the AE analysis presented in the results.

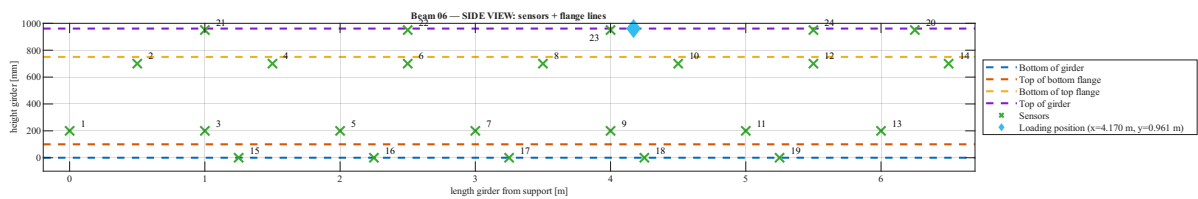


Figure 5.5: Sensor layout of beam 6

5.3.1. Bending resistance

The first flexural crack was expected in the bottom flange directly beneath the loading plate on the top flange, where the bending moment is highest. Based on the sensor layout, the corresponding sensors are 17, 18, and 19, and all sensors in this region were examined. Intervals with decreasing load were omitted from the figures for clarity, as different mechanisms are active during unloading. As anticipated, sensor 18 registered the earliest increase in AE activity (see figure 5.7). The first pronounced peak occurs during the load step from 443 to 503 kN and is clearly visible in the cumulative hits for sensor 18

(figure 5.9). This pre-peak activity indicates that the first microcrack initiated just before the main peak. Sensors 17 and 19 also show smaller peaks in the same step (figures 5.6 and 5.8), consistent with the formation of additional flexural cracks. Visual inspection confirmed flexural cracks from the 600 kN step onward, corresponding to a bending moment of 2222.6 kNm, including self-weight.

The first peak follows immediately after surpassing the preceding step (454 kN). The corresponding load at that instant is approximately 462 kN. Treating this as a point load excluding self-weight, the bending moment at first cracking is

$$M_{cr} = 1453.97 \text{ kNm}$$

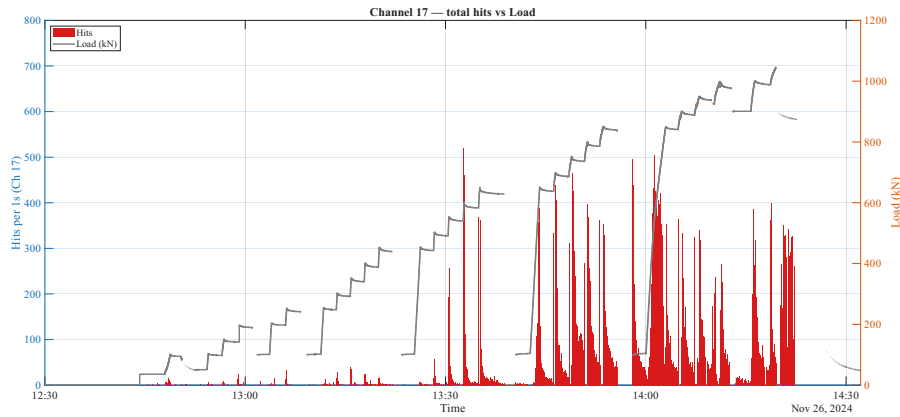


Figure 5.6: AE hits sensor 17 of beam 6

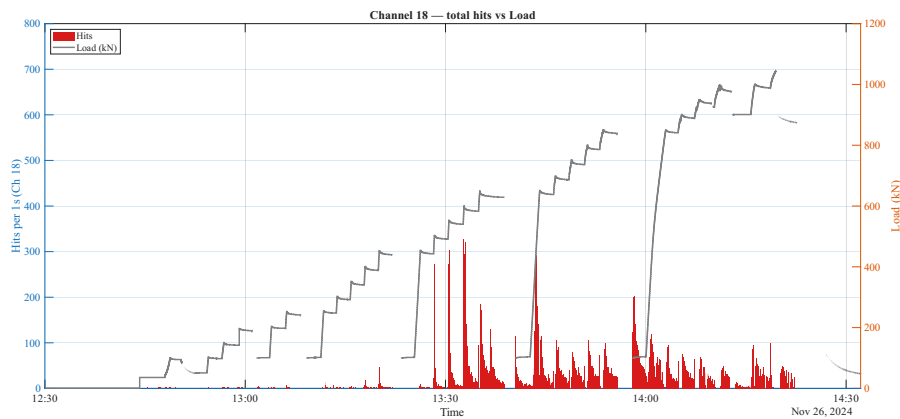


Figure 5.7: AE hits sensor 18 of beam 6

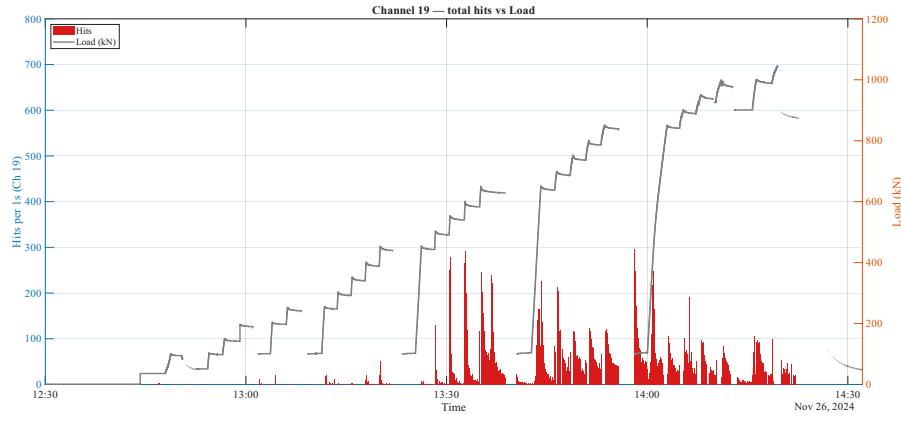


Figure 5.8: AE hits sensor 19 of beam 6

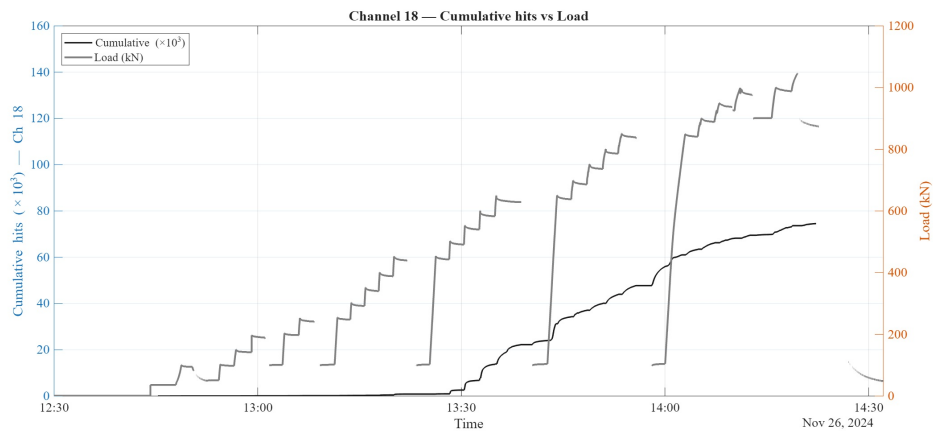


Figure 5.9: Cumulative AE hits sensor 18 of beam 6

5.3.2. Prestress level

The prestress force is estimated by enforcing that the tensile stress in the extreme fibre just reaches the concrete tensile strength at first cracking using the information from section 4.2.4. The mean flexural tensile strength is determined using the following way:

$$f_{ctm,fl} = \max \left\{ \left(1.6 - \frac{h}{1000} \right) f_{ctm}, f_{ctm} \right\}$$

$$f_{ctm,fl} = \max \left\{ \left(1.6 - \frac{961}{1000} \right) 4.8, 4.8 \right\} = 4.8 \text{ MPa}$$

The elastic section modulus is already given in table 4.1. The center of gravity is determined and has been found at 424.67 mm from the top fibre.

$$\begin{array}{lll}
 e_p = & 961 - 424.67 - 198.2 = & 338.13 \text{ mm} \\
 W_{b,thin} & = & 62828263 \text{ mm}^3 \\
 A_{c,thin} & = & 299000 \text{ mm}^2 \\
 f_{ctm,fl} & = & 4.8 \text{ MPa (Eurocode)}
 \end{array}$$

Imposing the stress equilibrium at the extreme tensile fibre, the following prestress estimate can be made:

$$\begin{aligned} \frac{-F_p \cdot e_p + M_{eq}}{W_{b,thin}} - \frac{F_p}{A_c} + \frac{M_{load}}{W_{b,thin}} &= f_{ctm,fl} \\ \frac{-F_p \cdot 338.13 + 320.36 \cdot 10^6}{62828263} - \frac{F_p}{299000} + \frac{1453.97 \cdot 10^6}{62828263} &= 4.8 \\ F_p &= 2686.24 \text{ kN} \end{aligned}$$

5.3.3. Shear resistance

The highest shear stresses and the first shear cracks are expected in the web, forming along a diagonal between the loading point and the support. Because the girder is an inverted T-beam, a shear-tension failure mode is anticipated. Based on the sensor layout, this zone corresponds to sensors 3, 4, 5, 6, 7, and 8. Intervals with decreasing load were omitted from the figures for clarity, as different mechanisms are active during unloading.

The recorded hits show that sensors 3 and 6 show pronounced increases during the load step from 583 to 650 kN. The preceding step already shows a smaller rise, which can be interpreted as an early indication of shear-crack initiation (see figures 5.13 and 5.10). Sensor 5 also shows a minor peak in the previous step (540 to 601 kN; see figure 5.12). Taking these first increases as the indicator, the corresponding load is approximately 589 kN. The associated jumps are also evident in the cumulative hit curves (for example, figures 5.16 and 5.17). This pattern is consistent with the onset of microcracking due to shear. The corresponding shear force at the critical section (1.73 times the effective depth from the support) is 537.50 kN, including self-weight. After this first peak, subsequent load increments produced additional peaks at the same sensors, indicating progressive cracking within the shear-critical region. Visual inspection during testing confirmed the appearance of shear cracks from the 750 kN step onward, corresponding to a shear force of approximately 651.71 kN at 1.73 times the effective depth from the support.

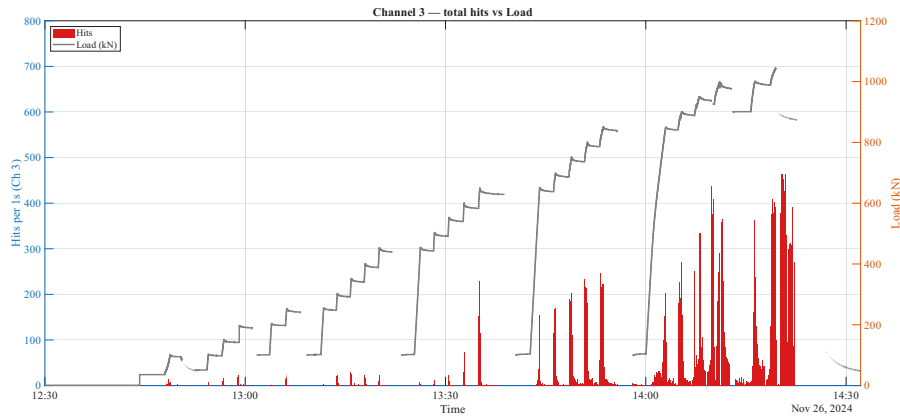


Figure 5.10: AE hits sensor 3 of beam 6

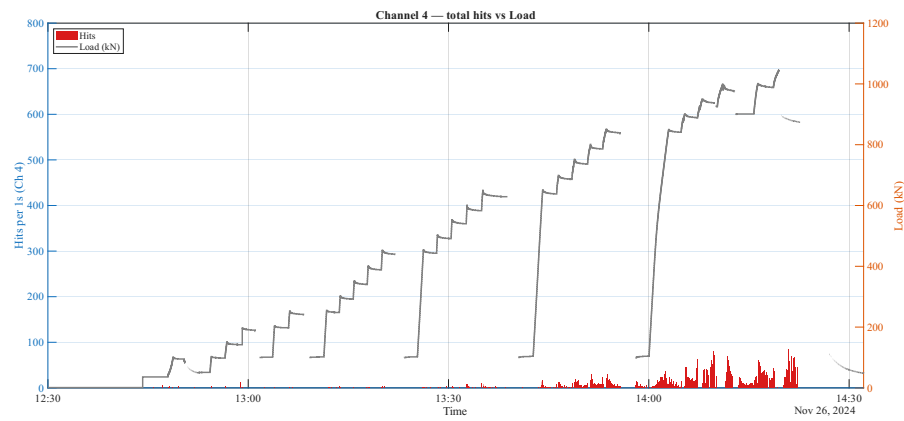


Figure 5.11: AE hits sensor 4 of beam 6

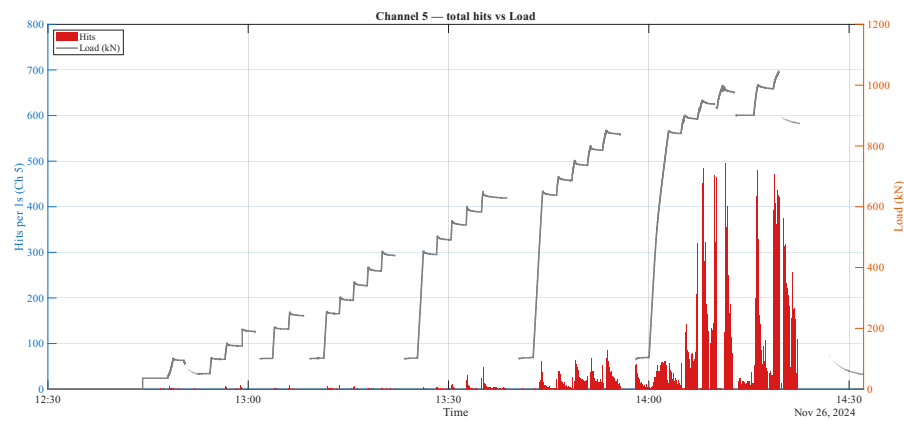


Figure 5.12: AE hits sensor 5 of beam 6

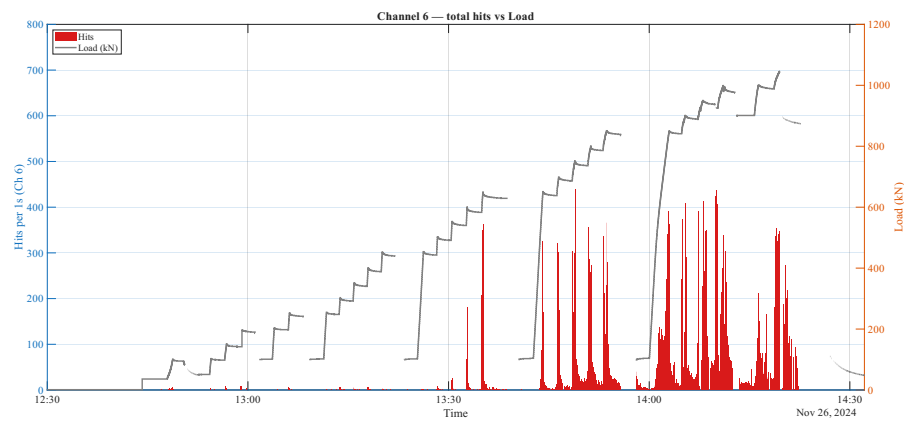


Figure 5.13: AE hits sensor 6 of beam 6

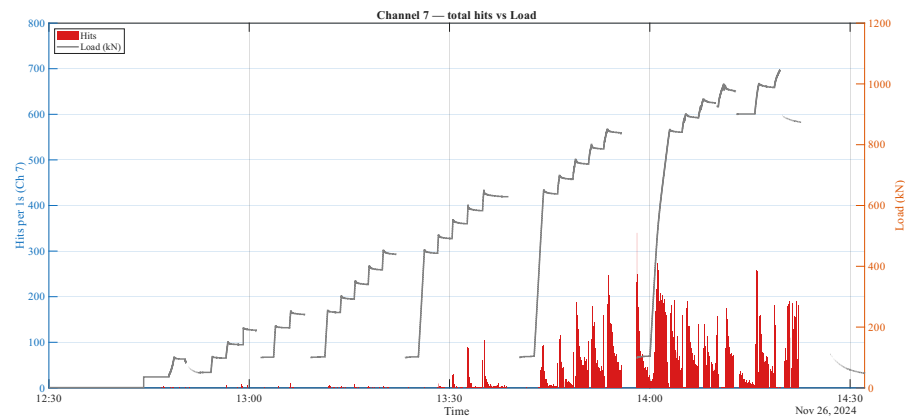


Figure 5.14: AE hits sensor 7 of beam 6

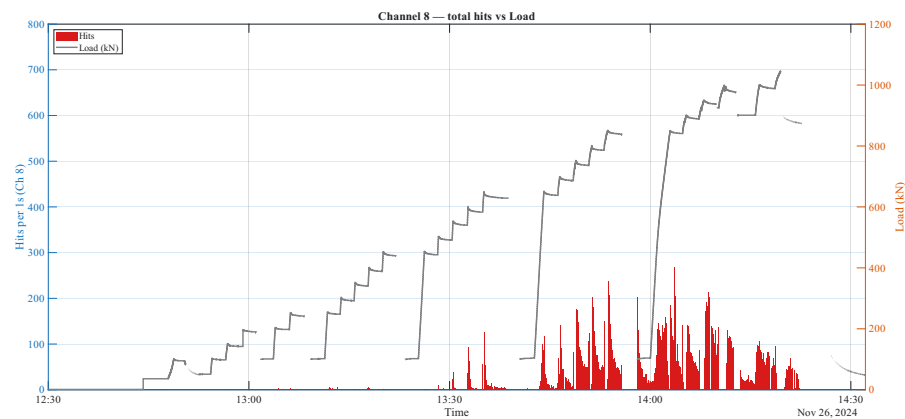


Figure 5.15: AE hits sensor 8 of beam 6

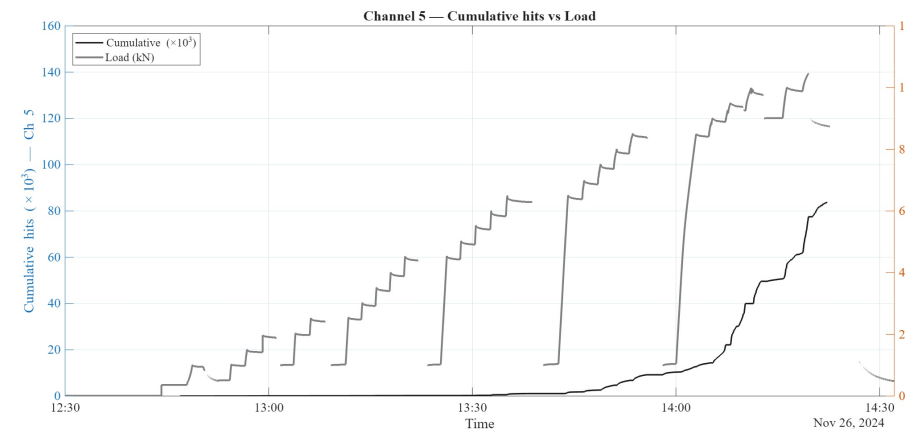


Figure 5.16: Cumulative AE hits sensor 5 of beam 6

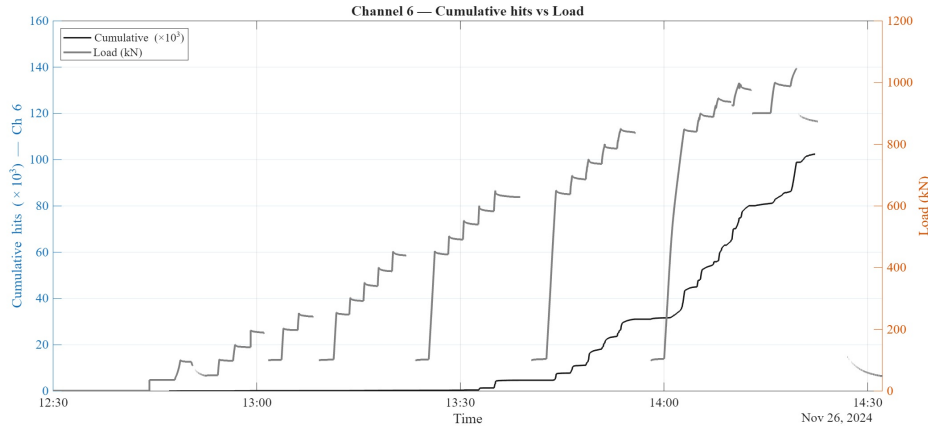


Figure 5.17: Cumulative AE hits sensor 6 of beam 6

5.3.4. Localisation of cracks

Crack locations were estimated from AE arrival times using an effective wave speed of 3200 m/s. Although values near 4100 m/s are commonly assumed for concrete, 3200 m/s yielded smaller localisation errors for this setup (sensors on the web and bottom flange), as verified with pencil-lead break calibrations near sensor 9 (Howse et al., 2025). Because the cross-section is not uniform in width and the bottom flange is the widest component, a source at the edge of the bottom flange must propagate first in the x -direction and then in the y -direction to reach the web sensors. This 3D path produces inconsistent travel times between the flange and web arrays. Therefore, only sources localised using the web sensor array are considered reliable, and results that combine web and bottom-flange sensors are not reported. Figure 5.18 shows the estimated source locations for all hits, and Figure 5.19 presents the corresponding probability density of hit occurrence.

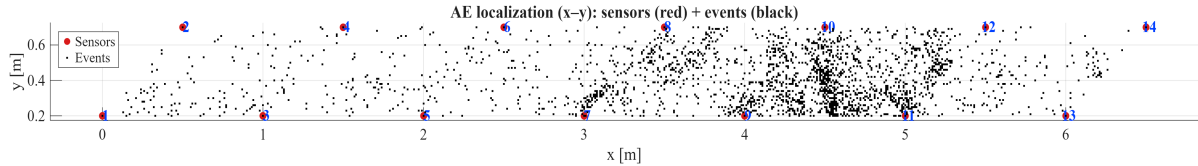


Figure 5.18: All estimated hit locations

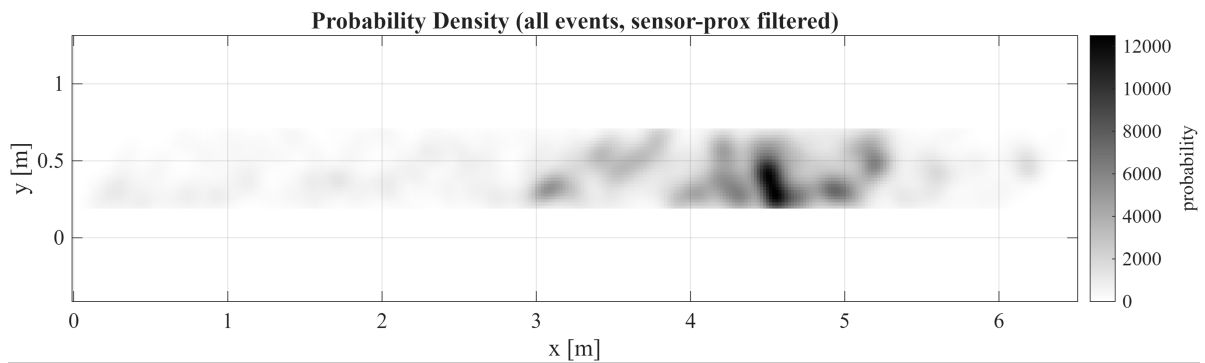


Figure 5.19: Probability density hit locations

Selected load levels were chosen to cross-check the AE-based identification of flexural and shear activity. In Figure 5.20, a large number of estimated hit locations is visible up to 450 kN. Some background activity remains despite applying the peak-frequency threshold. For example, Figure 5.10 shows a nearly constant number of hits from the first load step through 450 kN. Because this level does not

vary with the applied load, it is unlikely to indicate crack initiation. It was not possible to remove all background activity, so some artifacts remain visible. Consistently, the probability density of AE hits in figure 5.21 is very low, and spatial patterns are weak at this stage.

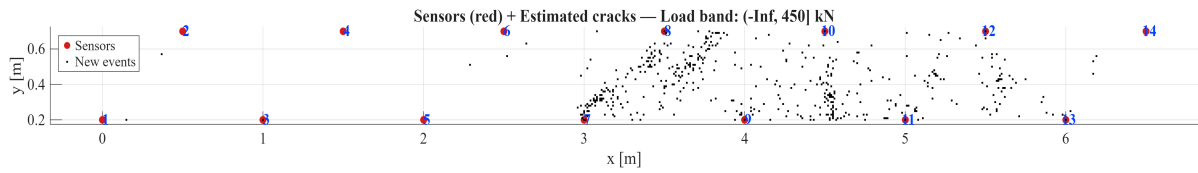


Figure 5.20: Estimated crack location from start load test to 450 kN

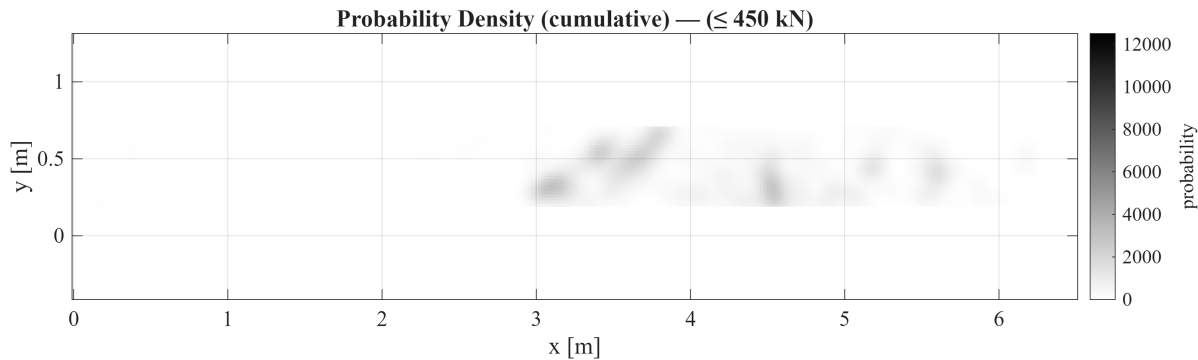


Figure 5.21: Probability density hit locations from load test to 450 kN

The first flexural crack is estimated at a load level of about 462 kN. The corresponding figure 5.22 shows localised sources between the load application point and the bottom, with notable activity near sensors 9 and 10. There are relatively few events directly at the bottom flange, so widespread new cracking is not indicated at this stage. Some activity between sensors 9 and 10 suggests microcracking at the web or potential initiation points for later crack growth. Additional hits between these sensors are consistent with bending-related activity in the web. The probability density up to this load level remains low. However, the region where flexural cracks are expected is beginning to darken, indicating an emerging cluster, see figure 5.23.

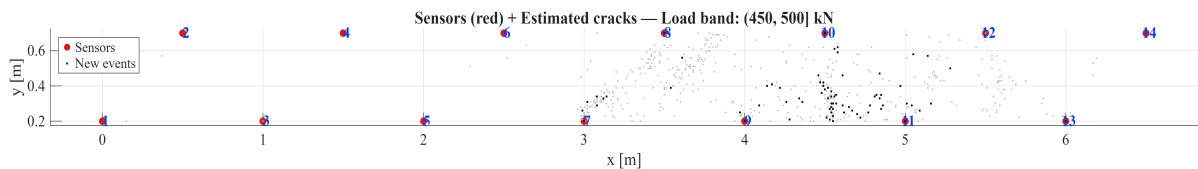


Figure 5.22: Estimated crack location from 450 kN to 500 kN

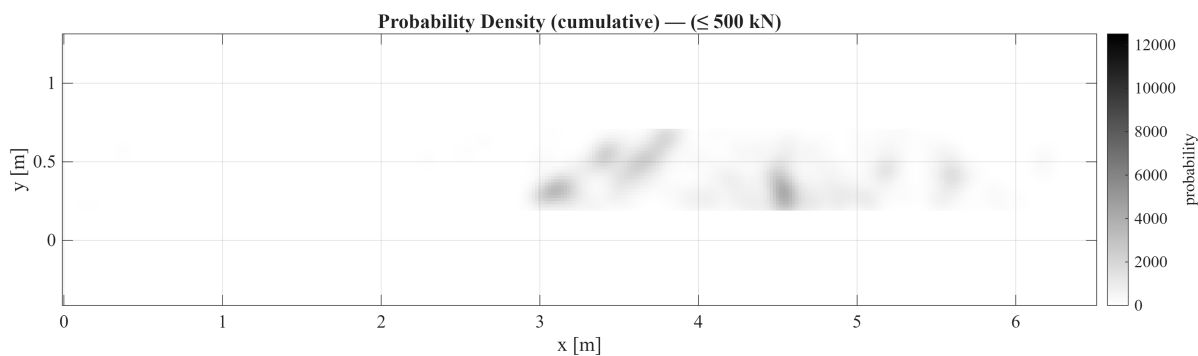


Figure 5.23: Probability density hit locations from load test to 500 kN

At 550 kN (figure 5.24) the density of localised events increases slightly in the flexural region. In the web to the right of the applied load a small increase is visible, consistent with flexural cracks propagating upward from the bottom flange. In addition, activity increases near sensors 7 and 8, which may indicate early shear-related mechanisms, even though the AE hit-rate for sensor 7 (figure 5.14) does not yet show a pronounced increase at this step. Figure 5.25 shows darker shades in the flexural area, indicating a higher local probability density and an overall increase in events there.

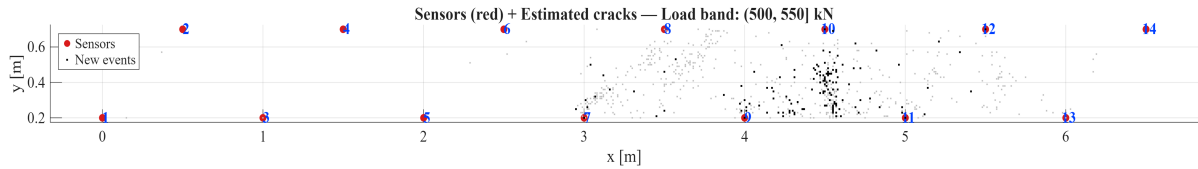


Figure 5.24: Estimated crack location from 500 kN to 550 kN

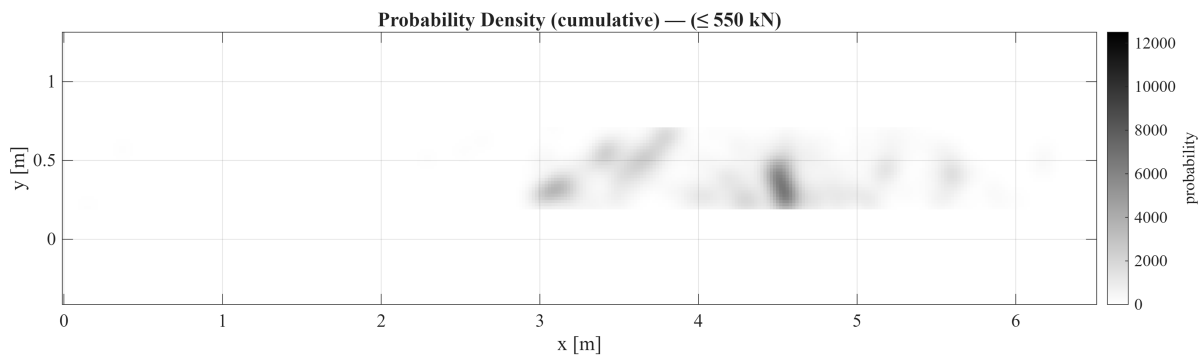


Figure 5.25: Probability density hit locations from load test to 550 kN

At 600 kN (figure 5.26) the density of localised sources increases in the flexural region, with lateral spread and slight inclination consistent with the development of multiple flexural cracks. Additional events are visible in the web between sensors 9, 10, and 11, suggesting further microcracking. The probability-density plot (figure 5.27) shows a modest darkening in the flexural zone, indicating more events, and a substantial increase compared with earlier steps.

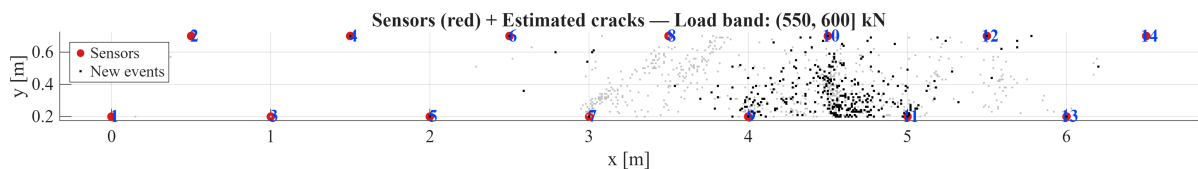


Figure 5.26: Estimated crack location from 550 kN to 600 kN

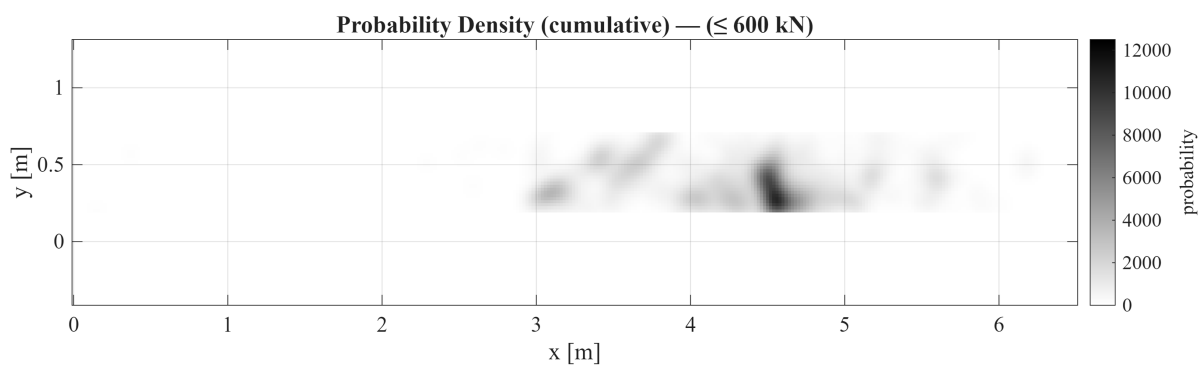


Figure 5.27: Probability density hit locations from load test to 600 kN

Comparing figure 5.26 with figure 5.28 shows a marked increase in localised sources at the web between sensors 9, 10, and 11. The concentration of events in this zone likely reflects the formation or widening of a dominant flexural crack, while the higher density of inclined events indicates additional inclined flexural cracking. The probability-density plot (figure 5.29) shows a distinct dark band at the bottom web, consistent with the probable crack location in the girder.

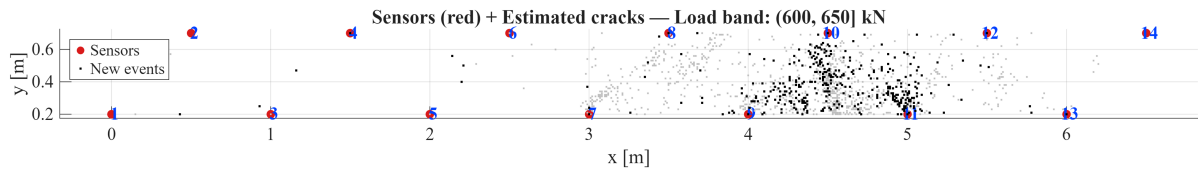


Figure 5.28: Estimated crack location from 600 kN to 650 kN

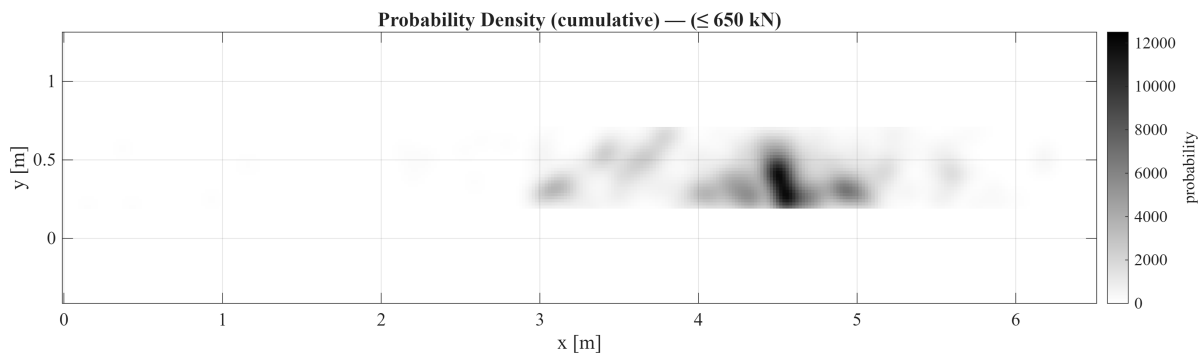


Figure 5.29: Probability density hit locations from load test to 650 kN

At higher loads up to 800 kN (figure 5.30) the concentration of estimated sources intensifies between the support and the loading plate, reflecting the development of inclined flexural and shear activities. Additional activity from the loading plate toward sensor 7 suggests additional flexural cracks. The probability-density map (figure 5.31) shows a darkening diagonal band from the loading point toward the support, reinforcing this interpretation.

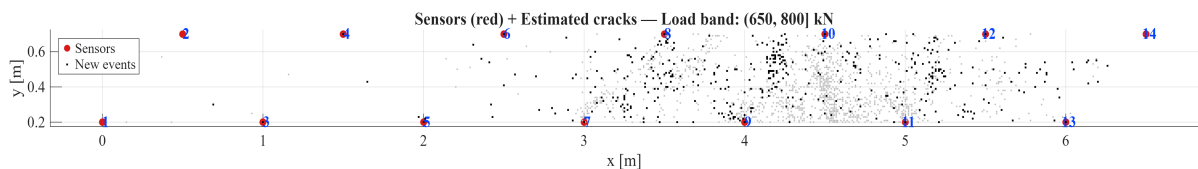


Figure 5.30: Estimated crack location from 650 kN to 800 kN

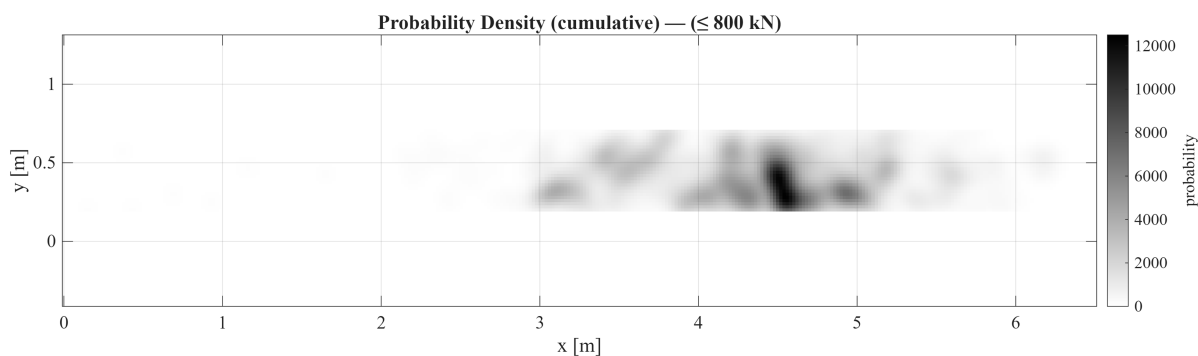


Figure 5.31: Probability density hit locations from load test to 800 kN

At the ultimate load (figure 5.32) the very high density of localised events across the girder is consistent with widespread damage at failure. Events between sensors 1–6 appear as the girder fails, indicating extensive shear cracking in that region. The test photograph (figure 5.34) shows a dominant shear crack. This feature is not clearly resolved in the probability-density map (figure 5.19), which may be because the critical fracture occurred within the instrumented zone with little precursor activity, producing limited warning hits immediately beforehand as expected with a shear-tension failure with a thin web.

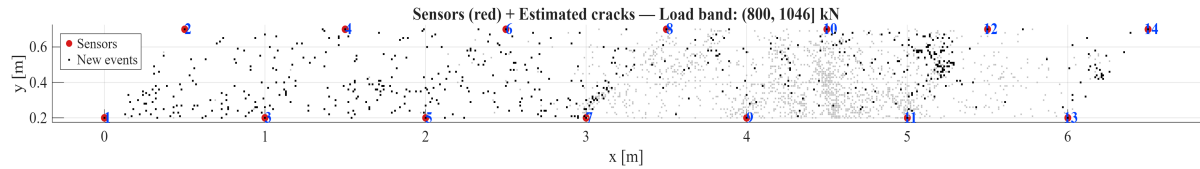


Figure 5.32: Estimated crack location from 800 kN to ultimate shear load 1046 kN

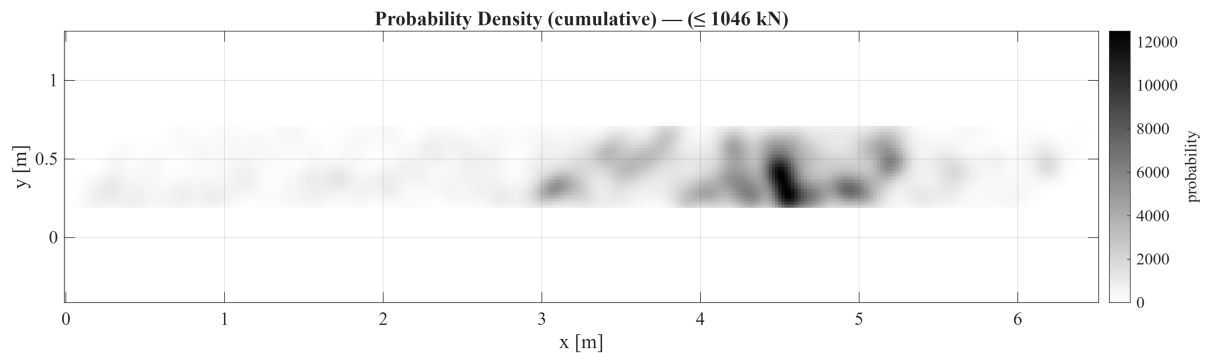


Figure 5.33: Probability density hit locations from load test to 1046 kN

5.4. Resistance girder failure test

The ultimate applied load reached 1046 kN. At the shear-critical section, located at 1.73 times the effective depth from the support, this corresponds to a shear force of 875.1 kN, including self-weight. The associated maximum bending moment was 3612.25 kNm, including self-weight.

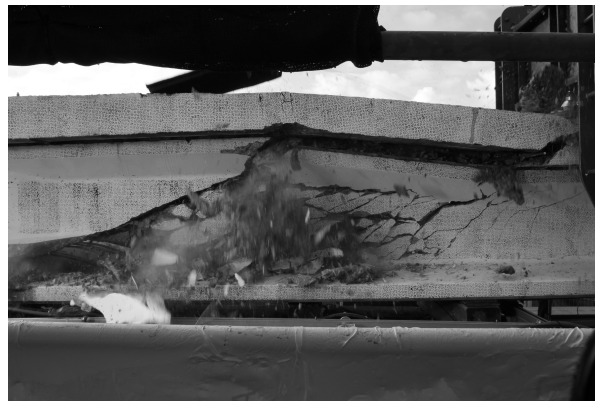


Figure 5.34: One of the reused girders failing during the load test (J. E. P. Pineda, 2024)

5.5. Probabilistic reliability girders

Taking the prescribed input distributions into account, the resulting load–resistance interference is shown in figure 5.35. The load effect and the shear resistance according to Eurocode 2:2004 and Eurocode 2:2023 overlap substantially. Both resistance distributions lie within the spread of the load-effect distribution, indicating non-negligible interference. Because the traffic action grows over time

(total increase of 13% over 100 years with a COV of 0.14), the load-effect distribution shifts to the right as the design life progresses. This shift is not shown in figure 5.35.

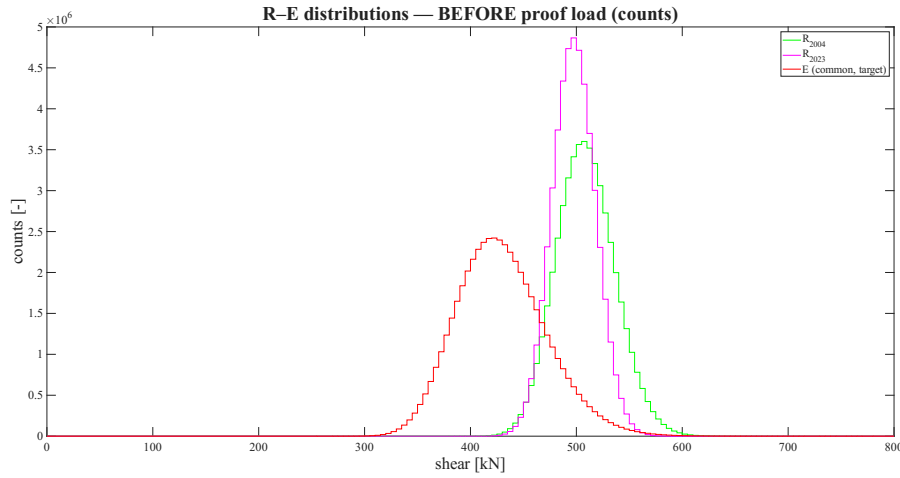


Figure 5.35: Load-resistance interference

Using the Monte Carlo procedure described in section 3.2.3, a simulation with $5 \cdot 10^7$ realisations was performed for each shear-resistance model. For a 100-year design life in consequence class 3, the target annual probability of failure is 10^{-5} , corresponding to a reliability index $\beta = 4.27$. Distributions for loads, materials, and geometric properties are listed in section 3.2.3. In the simulation, each realisation is assigned a stochastic resistance and, for each year, a corresponding annual load effect. If the load exceeds the resistance, that realisation fails in that year. The annual probability of failure is computed from the number of new failures among the surviving realisations, and the annual reliability index is reported as $\beta(t) = -\Phi^{-1}(p_f(t))$. Figures 5.38, 5.39 and 5.40 show $\beta(t)$ for three scenarios: no proof load, a proof load of 550 kN, and a proof load of 550 kN with an updated prestress level. The 550 kN proof load corresponds to a shear at the control section (1.73 d from the support) of 415.1 kN, excluding the girder's self-weight.

Because the girder satisfied the green-light criterion under the 550 kN proof load, realisations with resistance below the associated shear effect of 500.8 kN, including self-weight, are incompatible with the observation and are therefore removed (posterior conditioning). This truncation of the resistance distribution reduces the number of potential early-year failures, as illustrated in figure 5.36. In addition, proof loading led to flexural cracking, from which an updated prestress level is adopted. The updated prestress is modelled as normally distributed with a standard deviation taken as one quarter of the load-step size (12.5 kN). This assumption reflects the availability of a single tested girder; with multiple specimens, the standard deviation could be calibrated empirically.

The change in prestress modifies the resistance distributions, as shown in figure 5.37. For the Eurocode 2:2004-based model, the distribution shifts slightly to the right (increased resistance), because a higher prestress increases the average compressive stress term that enters directly in the concrete-shear expression. For the Eurocode 2:2023-based model, the distribution shifts slightly to the left (lower resistance). Although increasing prestress would reduce k_{vp} , the formulation already uses the lower bound $k_{vp} = 0.1$, so no additional effect arises through that pathway. Instead, the updated prestress influences the action ratio and increases the effective shear span a_{cs} , which in turn increases the mechanical shear span a_v . In the short-span formulation, where a_v replaces d in the resistance calculation, this substitution appears in a denominator term. The larger a_v therefore produces a small net decrease in the computed resistance, consistent with the leftward shift seen in figure 5.37.

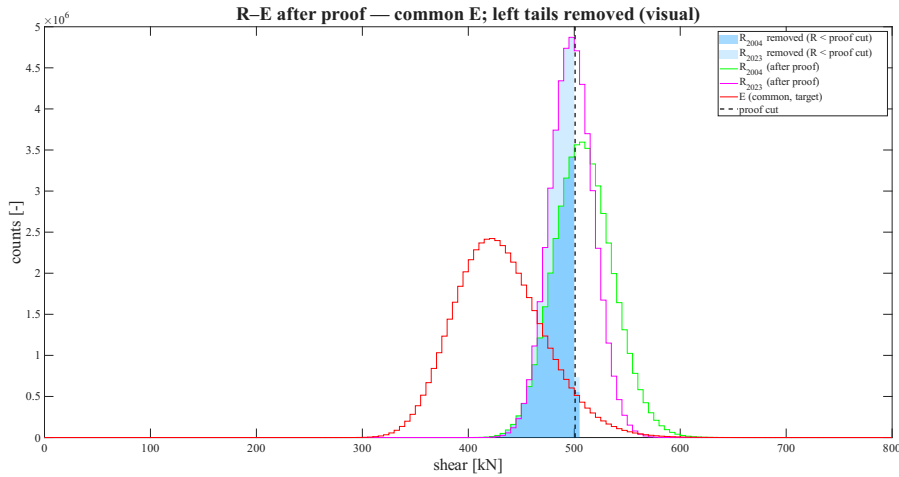


Figure 5.36: Load-resistance interference after proof loading

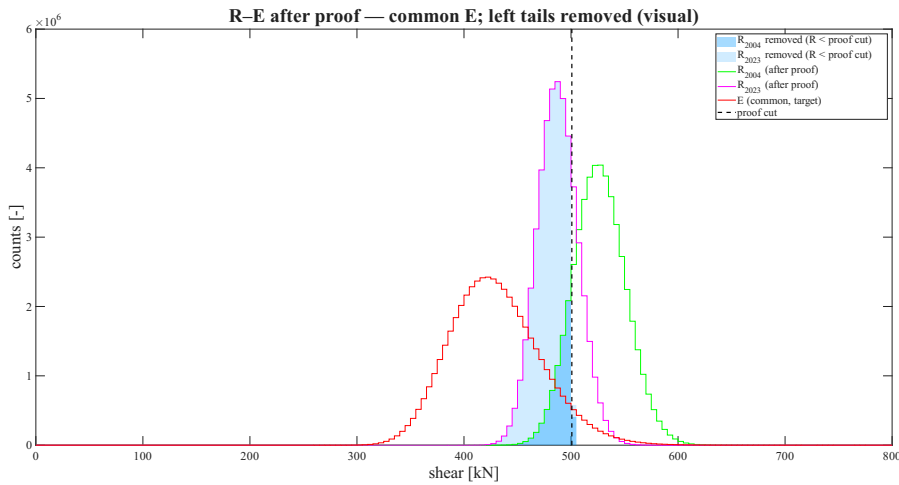


Figure 5.37: Load-resistance interference after updating the prestress level

For Eurocode 2:2004, the 100-year reliability index increases only modestly from $\beta_{100} = 2.81$ without proof loading to $\beta_{100} = 2.94 (+0.13)$ when a 550 kN proof load is applied, because survival at the proof load is implemented by truncating the resistance at the sectional proof demand at the control section, $V = 500.8$ kN including self-weight (figures 5.38 and 5.39). When the prestress level is additionally updated, the reliability index increases to $\beta_{100} = 2.98 (+0.17)$ (figure 5.40). Although these values are higher than in the no proof-load case, they remain well below the CC3 target of $\beta = 4.27$, so reuse would not meet the reliability objective under this resistance model.

For the Eurocode 2:2023 resistance model, proof loading likewise produces only a modest improvement. The sectional effect of the 550 kN proof load exceeds the resistance for many realisations, so conditioning removes many cases with $R < E$. As a result, the reliability index increases from $\beta_{100} = 2.76$ (no proof load) to $\beta_{100} = 2.90 (+0.14)$ (with proof load), and to $\beta_{100} = 2.88 (+0.12)$ when the prestress level is updated (figures 5.38, 5.39 and 5.40). These values also remain well below the CC3 target, consistent with the calculated shear resistance being substantially lower than the corresponding shear effects (see sections 5.1.2 and 5.2).

In both code models, the low reliability indices stem from substantial overlap between the load-effect and resistance distributions at the control section. Because the load effect overlaps a significant part of the resistance, a non-negligible fraction of realisations fails each year, keeping β low. Improving β would require reducing the load-effect distribution (for example, lower traffic actions or growth, reduced

dynamic amplification, shorter spans) and/or increasing resistance (for example, higher sectional resistance or strengthening), so that the load effect lies predominantly to the left of the resistance.

Table 5.1 summarizes the initial and 100-year reliability indices for the different scenarios and resistance models, compared with the CC3 target $\beta = 4.27$.

Table 5.1: Summary of reliability indices for different scenarios (CC3 target $\beta = 4.27$)

Scenario	Resistance model	β_{100}	Meets target?
No proof load	EC2:2004	2.81	No
	EC2:2023	2.76	No
Proof load 550 kN	EC2:2004	2.94 (+0.13)	No
	EC2:2023	2.90 (+0.14)	No
Proof load 550 kN + updated prestress	EC2:2004	2.98 (+0.17)	No
	EC2:2023	2.88 (+0.12)	No

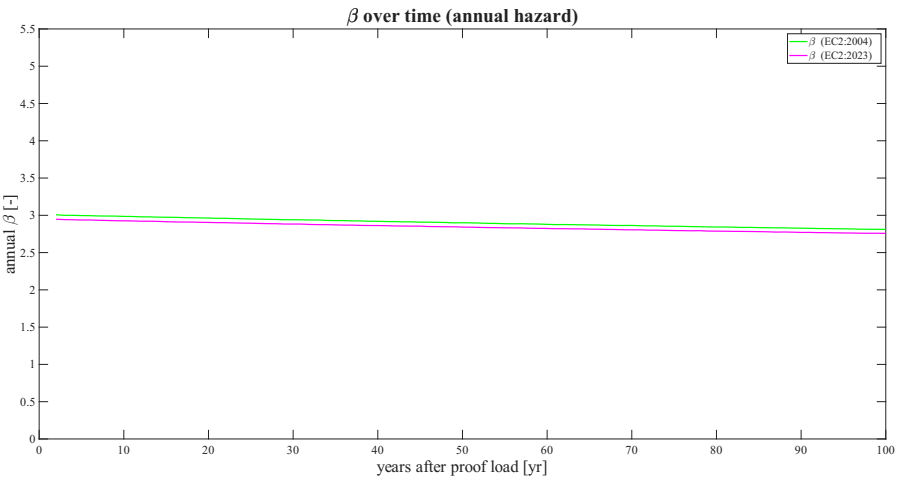


Figure 5.38: Reliability index with no proof load (R2004 $\beta(t) = 3.00 - 2.81$ and R2023 $\beta(t) = 2.95 - 2.76$)

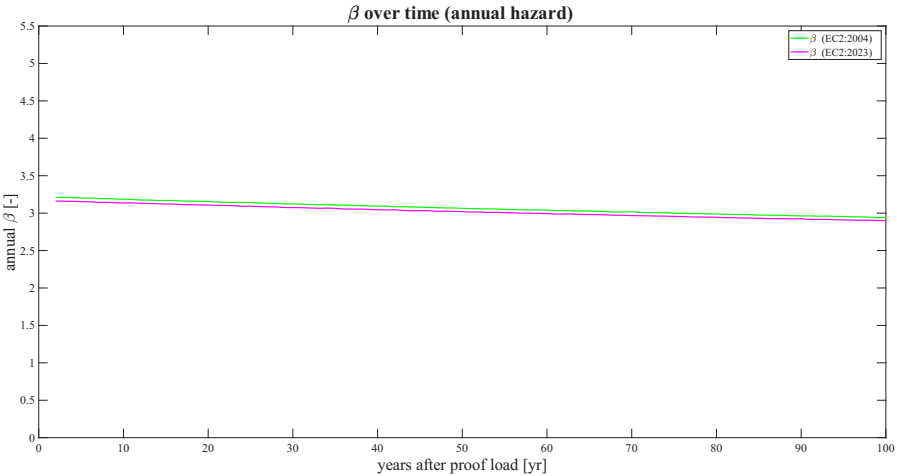


Figure 5.39: Reliability index with 550 kN load test (R2004 $\beta(t) = 3.21 - 2.94$ and R2023 $\beta(t) = 3.16 - 2.90$)

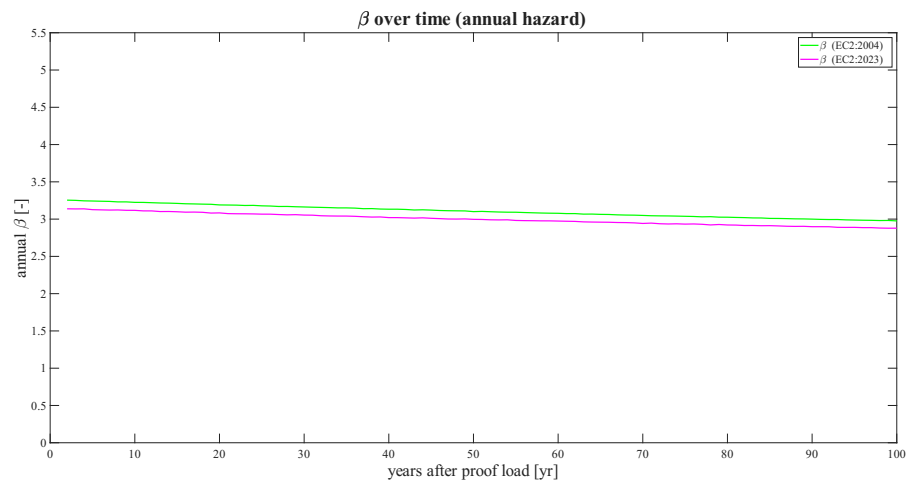


Figure 5.40: Reliability index with updated prestress level with 550 kN load test (R2004 $\beta(t) = 3.25 - 2.98$ and R2023 $\beta(t) = 3.14 - 2.88$)

6

Discussion

This chapter interprets the main findings of the study, connects them to the research objectives, and reflects on implications, and limitations. The discussion follows the structure of the results: the framework, acting loads and control section selection, code based resistance checks, observations from AE assisted proof testing, the subsequent failure test, and the probabilistic reliability analysis. Where helpful, it highlights how modelling choices and data processing decisions influenced the conclusions.

6.1. Framework

The assessment framework was applied almost in full in this study. The only step not executed as prescribed was the advance selection of a target proof load. Instead, the girders were taken to failure to establish shear resistance and to demonstrate whether untested girders could plausibly meet demand. Because the campaign included acoustic emission, this test was selected as the case study. Even before testing, running the framework with the adopted load and resistance models indicated that a CC3-level reliability was unlikely. The overlap between demand and resistance was too large for information updates alone to close the gap. In a routine application, the flowchart would therefore route to the branch stating that reuse at full capacity is not supported by the current model set and would advise against using proof loading as a path to the target reliability for this probabilistic reliability model. In that sense, the present work functions as a deliberate test bed to exercise and refine the workflow rather than as evidence that the framework guarantees a pass.

Operationally, the framework proved useful in two ways. First, it clarified where information has the most influence. Survival at a moderate proof level trims only the weakest part of the resistance tail, so measurements that update sensitive parameters (here, effective prestress) are more valuable than additional refinements of already well-known inputs. Second, it helped prioritise what to monitor in real time. Experience from this campaign suggests that using hit-rate peaks per sensor as the primary AE indicator provided sufficient resolution to manage step-and-hold decisions and to support parameter back-calculation, while dense localisation added little to acceptance decisions for using EC 2 based resistance distributions. In similar inverted T-beams, a leaner monitoring setup focused on critical regions could therefore reduce data volume and processing effort without undermining the framework's decision points, provided that conservative stop rules are maintained.

The outcome also illustrates an important role for the framework as a screening tool. When a preliminary run indicates that even optimistic updates will not move the posterior reliability near the target, the framework can steer projects toward alternative strategies (demand reduction in the receiving structure, local strengthening near the control section, different resistance based distribution, or a different proof concept), rather than treating proof loading as a default. In short, the value of the framework is not just in post-test inference but in preventing low-yield tests and focusing resources where evidence can change decisions.

There are limitations to this application. The routing depends on the specific load model, control-section choice, and parameter priors used here. A different section, alternative mapping of global proof load to

sectional demand, or updated priors for parameters such as a_{cs} , a_v , d_{dg} , and effective prestress could shift the framework's branch point. The simplification to use AE hit-rate peaks in stead of robust localisation assumes that crack initiation occurs within instrumented hot spots and that thresholds remain stable across members and environments. Outside those conditions the reduced instrumentation may miss early signals. The framework's proof-level logic relies on a consistent translation between global test force and sectional effects. If system action during testing mobilises alternate load paths not captured by the section model, the inferred informational value of survival may be optimistic. Finally, the framework was exercised on a single girder type and setup. Generalising the monitoring simplifications or the pre-screening thresholds to other geometries should be done cautiously and, ideally, validated on a broader set of specimens.

6.2. Acting loads and control section selection

The acting shear at the control section governs the assessment logic. Shear verification was performed at 1.73 times effective depth from the support, with the tandem positioned to maximise design shear. The resulting ultimate shear effect is $V_{ULS} = 613.13$ kN, while the service shear at that section is $V_{SLS} = 430.94$ kN. These values set a demanding benchmark for any resistance model and frame the later reliability outcomes. The iterative procedure used to identify the control section is appropriate, since the effective depth depends on the local geometry and reinforcement layout. The placement of LM1 axles relative to the effective depth critically affects the shear peak at that section.

The interpretation is limited by modelling choices for the load side. Not all possible load cases and combinations were included. Only the main contributors were analysed because the goal here is not to produce a complete bridge design but to establish a framework for testing reclaimed girders for reuse. Consequently, the quoted V_{ULS} and V_{SLS} should be read as representative for the proof-testing framework rather than exhaustive worst-case envelopes for a specific bridge project.

6.3. Code based resistance: 2004 versus 2023 models

For the selected control section, EC2 2004 gives $V_{Rd,c} = 417.11$ kN, while EC2 2023 gives $V_{Rd,c} = 357.37$ kN. Both are substantially below V_{ULS} , so a conventional check alone would reject reuse for a CC3 design life. The 2004 result reflects the empirical formulation with size factor, longitudinal reinforcement ratio, and axial stress term. The 2023 result is lower because the CSCT based formulation reparameterises resistance through d_{dg}/d , the control section distance $a_{cs} = |M_{Ed}/V_{Ed}|$, the effective shear span a_v , and the axial interaction factor k_{vp} , linking shear resistance more closely to deformation demand and aggregate interlock.

In the next generation EC2 an additional verification is required when loading is predominantly distributed: a second control section is taken at the position of maximum bending moment, and then again at one times effective depth from that location. In such cases the effective depth can vary with the prestress layout, which should be accounted for when determining the governing section. In this study LM1 was treated as predominantly concentrated, so that verification is done.

The interpretation is limited by section selection and parameter sensitivity. Resistance was evaluated at one control section. Small shifts in the chosen section, in a_{cs} and a_v , or in inputs such as d_{dg} and effective prestress can change $V_{Rd,c}$ materially. These results therefore represent a consistent application of each code model to the chosen section, not an exhaustive search for a global lower bound across all admissible sections.

6.4. AE-assisted proof test

The AE processing choices strongly influence what is counted as meaningful activity and how those signals are interpreted against the mechanical response. Several channels exhibited repeating peaks that did not track the applied load. Treating these as background and suppressing them with a data-driven peak-frequency threshold of 27.5 kHz reduced spurious hits while preserving load-correlated trends in the remaining data. This filtering is defensible, as the time series show a clear reduction of pre-peak counts after its application, but it cannot guarantee that the very first physically meaningful peak is always captured for every sensor.

Source localisation used an effective wave speed of 3200 m/s, calibrated with pencil-lead breaks on the web and judged appropriate for the web sensor plane in this setup. Locations were accepted only when at least three sensors contributed within a plane and the solution lay inside the convex hull of those sensors. Because localisation is sensitive to both the assumed wave speed and the arrival-time picks, small spatial variations in effective wave speed or pick error can introduce position offsets. Moreover, flexural cracks that initiate near the edges of the bottom flange are relatively far from web sensors. The resulting lower signal-to-noise ratio and unfavourable geometry can yield clear peaks in hit counts per sensor without producing robust localisation clusters. This explains why peaks in AE hits are visible while corresponding source locations are sparse or absent until a larger, visually evident flexural crack develops.

Within this processing framework, the earliest flexural-related AE activity concentrated near the expected bending region at roughly 462 kN, preceding the visually dominant flexural cracking reported around 600 kN. Indications consistent with shear-related mechanisms appeared later, with first shear-type activity near 589 kN and visual shear cracks only close to 750 kN. From a reliability standpoint, this underscores the value of AE as an earlier indicator of internal damage progression than visual inspection alone, particularly where subsurface cracking is involved. Differences relative to potential DIC observations are unknown here, because the available DIC data were not used: image quality was affected by variable daylight conditions and a relatively coarse speckle pattern for correlation.

The interpretation is limited by residual background that remains after filtering, by the sensitivity of source localisation to the assumed wave speed and arrival-time picks, and by the lack of usable DIC to validate crack timing and locations. In light of these limitations, AE evidence should be used to flag incipient damage and inform proof-step decisions, while final acceptance remains grounded in measured mechanical response and the probabilistic reliability checks.

6.5. AE hit rate vs Historical Index

In addition to the hit rate used in this study, MISTRAS used the Historical Index (H.I.) in their report, a measure of how the cumulative AE signal strength evolves during a test. Golaski et al., 2002, the H.I. at time t is

$$H(t) = \frac{N}{N-K} \frac{\sum_{i=K+1}^N S_{oi}}{\sum_{i=1}^N S_{oi}},$$

where N is the number of AE hits up to t , S_{oi} is the signal strength of the i th event, and K is an empirically defined factor that increases with N . A rapid rise (e.g., a doubling) in $H(t)$, in the absence of noise, is typically interpreted as evidence of active damage.

In the information provided by MISTRAS, a doubling of the H.I. was identified as indicating active damage. The first doubling occurred between the 550 and 600 kN load steps and was estimated at 582 kN. This differs by about 120 kN from the load indicated by the hit rate, which is quite large. MISTRAS noted that this behaviour is consistent with the onset of flexural cracking. They further stated that the onset of shear cracking at around 750 kN is apparent in the AE location plots and correlates with an increase in H.I.. However, distinguishing flexural from shear cracking using H.I. alone is not possible.

It is also important to note that visible cracking observed during the test was broadly consistent with the AE indicators: bending (flexural) cracks appeared at about 600 kN, and shear cracks at about 750 kN. Thus, although the absolute threshold indicated by H.I. differed from the hit-rate threshold, the qualitative sequencing of damage mechanisms aligns with both AE measures and visual inspection. Relying on H.I. alone is riskier: it can suggest higher allowable levels even though different crack types tend to develop within a narrow load range. By contrast, the AE peak-hit threshold is more conservative, used together with DIC, it may allow operating closer to the limit with greater confidence.

6.6. Proof load outcome and failure test findings

The proof protocol reached 550 kN without irreversible damage under the adopted traffic-light criteria, implying a green light in the flexural reference zone and no observed shear cracking up to that step. This survival event is informative: conditioning the resistance on being greater than the mapped sectional demand at 550 kN removes weak realisations and raises the inferred reliability, provided the proof level

does not already sit to the right of most of the resistance distribution in the chosen model.

The subsequent failure test provides the complementary endpoint and a direct point of comparison with the code checks. The ultimate applied load reached 1046 kN. At the shear-critical control section located 1.73d from the support, this corresponds to a shear of 875.1 kN including self-weight. For the same section and load placement, the governing design shear effect is 613.13 kN. The code-based shear resistances for a member without transverse reinforcement are markedly lower: 417.11 kN under EC2:2004 and 357.37 kN under EC2:2023. Hence, a strict sectional check would reject reuse at the adopted ULS envelope (design shear exceeding resistance), whereas the tested girder demonstrated a global capacity well above these predictions (tested shear 875.1 kN). This divergence should not be read as a code error. It reflects the deliberate conservatism of the sectional models and the specific behaviour of inverted T-girders with thin webs and significant prestress, which are less common in international practice and not the primary “calibration class” for the basic shear design codes. Ongoing research at TU Delft targets this girder type. A forthcoming PhD thesis on approximately twenty inverted T-beams without shear reinforcement tested in the Stevin II laboratory is expected to provide a broader empirical basis. Such results could inform how inverted T-girders are treated in future assessments, particularly with respect to shear resistance. More broadly, the present test highlights a model discrepancy between an intentionally conservative sectional idealisation and the global capacity under the specific geometry, load path, and boundary conditions. In this setup, system action likely mobilised mechanisms, arching near the support, flange participation in load spreading, and crack closure under increasing axial compression, that are only partly represented in the baseline shear rules at the chosen control section.

The AE probability density near ultimate shows widespread activity, but the dominant shear crack visible in the test photograph is not strongly resolved in the density field, likely because the terminal fracture generated limited precursory hits in the instrumented zone. This indicates that AE is highly effective for progressive processes and for proof-step control, yet may provide less warning immediately before a brittle shear–tension failure if energy release concentrates locally or outside optimal sensor coverage. Within the assessment workflow, the proof-load survival trims the left tail of resistance. The failure result acts as an upper benchmark on what the system could carry under the given setup and the code checks provide a conservative lower bound at the control section. The practical task is to narrow the interval between these bounds with defensible refinements (for example, section choice within provisions, effective-width interpretations for existing structures, or calibrated project-specific shear modelling) and to propagate any changes through the reliability analysis rather than relying on point-capacity comparisons alone.

There are limitations that qualify these findings. The traffic-light judgement relies on AE and visual observation without a parallel DIC cross-check in the analysis, which adds uncertainty to the exact onset loads of cracking. The shear value at 1.73d reflects the chosen control section and test geometry. Small shifts in section location or in the effective shear span can change the comparison with the code-based resistance. AE indicators near ultimate are sensitive to sensor placement and the assumed wave speed, so sparse localisation close to the final fracture does not necessarily imply an absence of damage precursors. Finally, the failure test activates system effects beyond a single control section, so the observed global capacity should not be read as a direct contradiction of a conservative section check, but as evidence of model discrepancy that merits calibration in future work.

6.7. Probabilistic reliability

The Monte Carlo framework combines stochastic models for load effects with resistance models from EC2 2004 and EC2 2023 to obtain an annual reliability index over a 100 year horizon. The prior, desk based assessment of reliability yields $\beta_{100} = 2.81$ for EC2 2004 and $\beta_{100} = 2.76$ for EC2 2023. These values follow directly from the extent of overlap between the load effect distribution and the section resistance distribution at the chosen control section. With a large overlap, a nontrivial annual failure probability persists even when individual parameter means look reasonable. This framing is useful because it shifts the focus from point estimates of capacity to the geometry of two distributions and how modelling choices widen or narrow their overlap.

Conditioning on survival at the 550 kN proof load is implemented by truncating the resistance at the

sectional proof demand at the control section, $V = 500.8$ kN including self weight. Under EC2 2004 this raises $\beta_{100} = 2.94 (+0.13)$. Under EC2 2023 the index increases to $\beta_{100} = 2.90 (+0.14)$ (figures 5.38 and 5.39). The gains are modest because, although the weakest part of the resistance tail is removed, a substantial portion of the load effect distribution still exceeds the relevant post conditioning resistance quantiles, so many realisations continue to fail.

Parameter updating through AE based back calculation of effective prestress, from 2507 kN to 2686 kN, illustrates how measurement reduces epistemic uncertainty and shifts distributional mass. In EC2 2023, prestress influences the longitudinal strain entering the CSCT based terms and the axial interaction factor k_{vp} . In EC2 2004 it enters via the axial stress term with a different sensitivity. After combining the prestress update with survival at the 550 kN proof load, EC2 2004 shows an increase to $\beta_{100} = 2.98 (+0.17)$ over the horizon, while EC2 2023 gives $\beta_{100} = 2.88 (+0.12)$ (figure 5.40). The convergence of the two models near $\beta \approx 2.9$ suggests that once the weak tail is removed and prestress is increased, the governing part of the resistance distributions, namely the upper quantiles that control post conditioning failure, aligns more closely across models even though their unconditioned means differ. The values are also shown in table 6.2. This behaviour is consistent with reliability being controlled by tail geometry rather than by central tendencies.

Limitations specific to this analysis qualify the interpretation. The reliability calculation conditions on a single control section and on a specific mapping of the global proof load to sectional demand. The load model represents dominant cases for the proof testing framework rather than an exhaustive design envelope. Adopting different combination rules or spectra would alter the annual failure probability. Bayesian updating focuses on effective prestress inferred from AE, while other epistemic inputs that influence shear in EC2 2023 remain as modelled and may bias tail placement if misestimated. Finally, survival conditioning assumes that system effects during testing are represented consistently in the sectional resistance domain.

6.8. Use RBK 1.2 for next generation Eurocode

RBK 1.2 allows, for assessments of existing structures, the average web width to be taken over the effective-depth zone, capped at 1.25 times the smallest sectional width. Although RBK 1.2 is aligned with the previous generation of EC2 and this averaging is not explicitly included in the next generation EC2 shear model, it is instructive to examine the consequence of applying the same idea here. Interpreting the rule as an effective increase of the web width parameter in the shear check, the nominal shear resistance for the new generation EC2 rises by about 15%, from $V_{Rd,c} = 357.37$ kN to $V_{Rd,c} = 415.02$ kN.

On a deterministic level this remains below the acting shear $V_{Ed} = 613.13$ kN, so a purely code-based pass is still not achieved. Probabilistically, however, the rightward shift of the resistance distribution reduces its overlap with the demand distribution. The reliability index without proof loading in this situation for the EC2:2023 is increased to $\beta = 3.15 (+0.39)$ due to the average web width. As shown in figure 6.1, the resistance moves to the right while the load-effect distribution stays at the same position, so proof loading at 550 kN primarily trims the left tail of resistance without substantially altering the overlap in the upper quantiles that govern failure. The resulting reliability index decreases to $\beta = 3.12 (-0.03)$, see Figure 6.2, but still falls short of the CC3 target. Raising the proof load to 600 kN raises the index to $\beta = 3.21 (+0.09)$ for EC2:2023 towards 550 kN proof load and for EC2:2004 to $\beta = 3.13 (+0.15)$. Increasing even to 700 kN proof load increases for both $\beta = 3.58 (+0.60$ and $+0.46)$ towards EC2:2004 and EC2:2023 550 kN proof load, but at this level there is likely too much damage for reuse, since at 750 kN the shear cracks were visible for people attending the load test. The gain remains modest because the load tail is about the conditioned resistance, and traffic-growth effects keep the load side increasing over the 100-year horizon. In practical terms, adopting the RBK width averaging improves the statistical picture but does not, by itself or with a moderate proof level, close the gap to the target reliability for reuse at full capacity. All the reliability indices are summarized in table 6.1.

There are important limitations to this exploration. First, extending the RBK 1.2 web-width averaging to the new generation EC2 shear model is an assumption rather than an explicitly permitted rule, so the applicability is model-dependent. Second, effective width in shear is sensitive to the cracking pattern

and the actual load path near the support. Close to the control section the web dominates the shear-transfer mechanism, and a blanket 1.25 factor may overestimate aggregate-interlock contributions if flange participation is limited. Third, the change interacts with other parameters in the new model (for example a_{cs} , a_v , and d_{dg}). Without a full sensitivity analysis there is a risk of double counting favourable effects or masking adverse ones. Finally, the observed global capacity in the failure test reflects system action beyond a single control section, whereas the reliability model conditions on that section. Improvements inferred by section-level width adjustments should therefore be interpreted cautiously when transferred to whole-member behaviour.

Table 6.1: summary of reliability indices for use of average width of the web

resistance model	scenario	modelling option	β_{100}
EC2:2004	no proof load	reference	2.81
EC2:2023	no proof load	reference and $b_{w,gem} = 250$ mm	3.15
EC2:2004	proof 550 kN	updated prestress	2.98 (+0.17)
EC2:2023	proof 550 kN	updated prestress and $b_{w,gem} = 250$ mm	3.12 (-0.03)
EC2:2004	proof 600 kN	$b_{w,gem} = 250$ mm	3.13 (+0.32)
EC2:2023	proof 600 kN	$b_{w,gem} = 250$ mm	3.21 (+0.06)
EC2:2004	proof 700 kN	$b_{w,gem} = 250$ mm	3.58 (+0.77)
EC2:2023	proof 700 kN	$b_{w,gem} = 250$ mm	3.58 (+0.43)
CC3 target	—	—	4.27
CC2 target	—	—	3.8

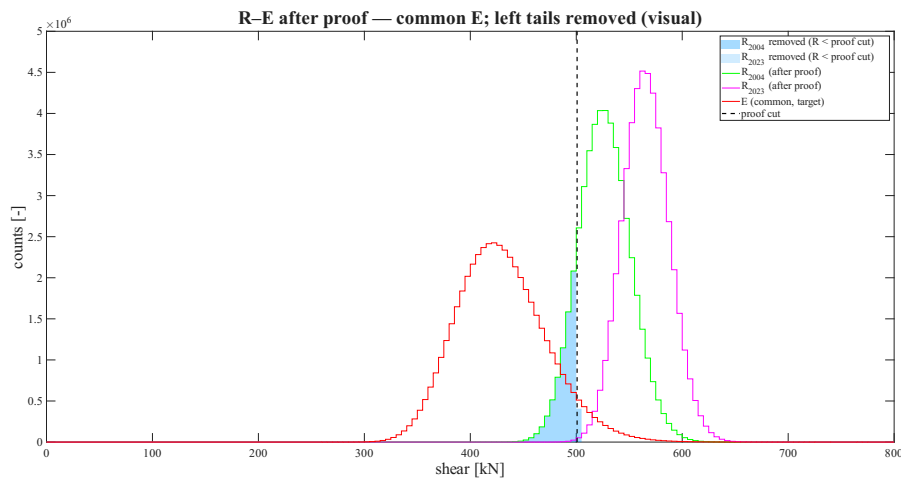


Figure 6.1: Load-resistance interference after updating the prestress level and using average width

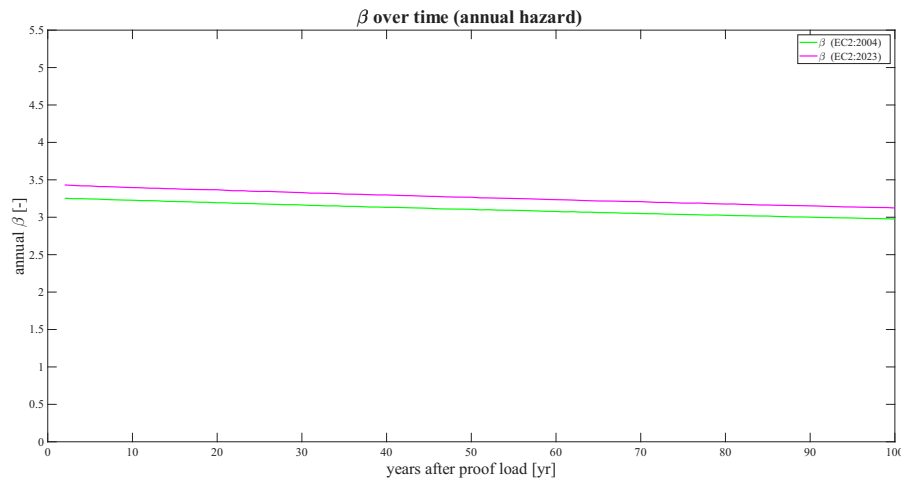


Figure 6.2: Reliability index with updated prestress level with 550 kN load test and using average width R2004 $\beta(t) = 3.25 - 2.98$ and R2023 $\beta(t) = 3.43 - 3.12$

6.9. Reduce load effect

Reducing the load effect can increase the reliability index. In this study the shear effect derived from WIM traffic was reduced by applying a transverse distribution over an effective width taken as the lane width plus twice the distance to the neutral axis, consistent with the LM1 tandem treatment in NEN-EN 1991-2, 2015. This gives a lower load effect from the traffic of around 51 kN. This lowers the sectional action noticeably (see figure 6.3).

For this case without proof loading the reliability increases already quite a lot for EC2:2004 $\beta = 3.50 (+0.55)$ and for EC2:2023 $\beta = 3.19 (+0.50)$. With a proof load of 550 kN and the updated prestress level, the resulting reliability indices are $\beta = 3.68 (+0.32)$ for EC2:2004 and $\beta = 3.58 (+0.32)$ for EC2:2023. These values remain below the CC3 target of 4.27, but approach levels consistent with a lower consequence class (e.g., CC2). Increasing the proof load to 600 kN, while still remaining within the green-light criterion so that reuse remains feasible, yields $\beta = 3.94 (+0.58)$ and $\beta = 3.88 (+0.62)$ for EC2:2004 and EC2:2023, respectively, which would be acceptable for CC2. The possible options that are discussed and the reliability indices of those are given in table 6.2

The improvement depends on (i) the chosen effective-width model for transverse distribution, (ii) the WIM-based load spectra and combination rules adopted, and (iii) the assumption that the green-light criterion is not exceeded at the higher proof level. These choices should be aligned with the applicable code provisions and project-specific traffic data before using the results for design decisions.

Table 6.2: summary of reliability indices for reduced load effect and original web width

resistance model	scenario	modelling option	β_{100}
EC2:2004	no proof load	reference	2.81
EC2:2023	no proof load	reference	2.76
EC2:2004	no proof load	reduced WIM	3.36 (+0.55)
EC2:2023	no proof load	reduced WIM	3.26 (+0.50)
EC2:2004	proof 550 kN	updated prestress and reduced WIM load	3.68 (+0.32)
EC2:2023	proof 550 kN	updated prestress and reduced WIM load	3.58 (+0.32)
EC2:2004	proof 600 kN	updated prestress and reduced WIM load	3.94 (+0.58)
EC2:2023	proof 600 kN	updated prestress and reduced WIM load	3.88 (+0.62)
CC3 target	—	—	4.27
CC2 target	—	—	3.8

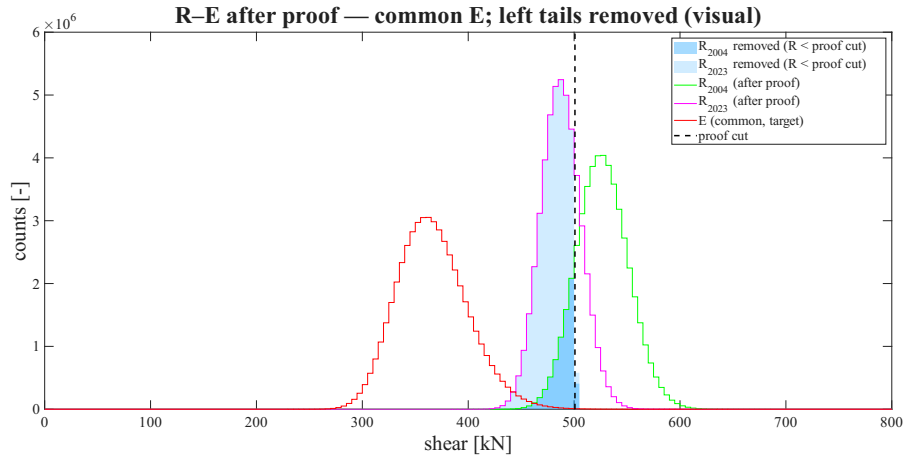


Figure 6.3: Load-resistance interference after updating the prestress level and using average width

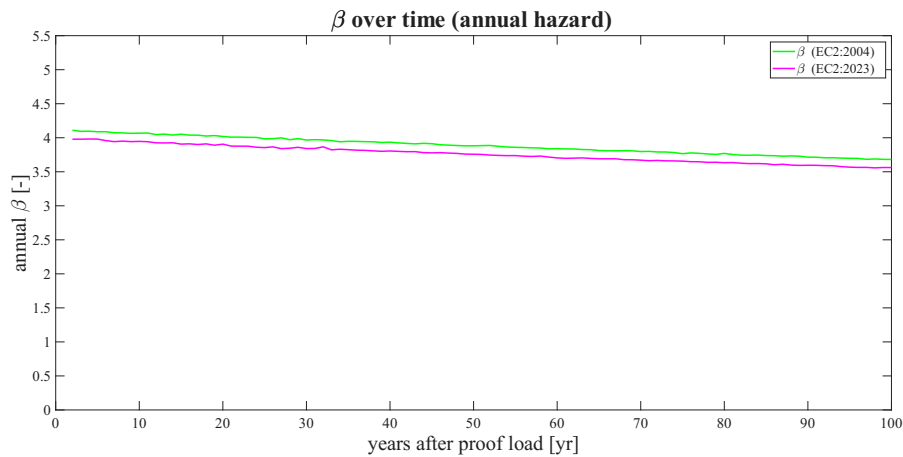


Figure 6.4: Reliability index with updated prestress level with 550 kN load test and using average width R2004
 $\beta(t) = 4.11 - 3.68$ and R2023 $\beta(t) = 3.98 - 3.56$

6.10. AE contributions

The results indicate that for inverted T-beams the number of AE sensors required for decision support may be reduced if the analysis focuses on early load correlated activity rather than on dense source localisation. In this study, using first peaks in the hit rate per sensor was sufficient to flag the onset of flexural and shear cracking well before visually dominant cracks formed. Sensors in the expected critical regions detected these changes early. In the case study, a sensor near the tensile zone highlighted flexural activity, while sensors closer to the web captured the first signs consistent with shear. This suggests that a sparse layout targeted at the control section and the adjacent flexural and shear zones can still provide actionable information for proof step control and for parameter updating, for example by anchoring the first cracking calibration used to refine effective prestress.

There is a practical trade off. A sparse array reduces installation time, cable management, cost per test, and the volume of data to screen, which can make larger proof testing campaigns more feasible. It also focuses attention on the few channels that are most informative for the acceptance logic, which can improve operational clarity during step and hold sequences. However, the benefits rely on the assumption that cracking initiates where it is expected from analysis and that the chosen thresholds remain valid across members and setups. If initiation occurs outside the instrumented zones, early signals can be missed. Likewise, without overlapping coverage, transient noise or a temporary loss of coupling on a single sensor can mask the early stages of damage, leading to late warning relative to a denser layout.

Limitations specific to this interpretation are important. First, the apparent adequacy of a sparse layout is conditioned on the geometry and load path of the tested girder. Different flange thicknesses, web widths, or prestress layouts can shift the zones of first activity. Second, relying on first peaks per sensor presumes stable thresholding and consistent coupling. Changes in surface condition, temperature, or sensor pressure can alter sensitivity and skew the timing of the first detected peak. Third, with fewer sensors, localisation becomes unreliable or unavailable, so distinguishing flexure dominated from shear dominated responses relies more on engineering judgement and less on mapped clusters. Fourth, without an independent modality such as DIC to confirm timing and location, there is a risk of attributing background to damage or, conversely, filtering out weak but meaningful precursors. Finally, the cost savings from fewer sensors must be weighed against the increased risk of missing an incipient crack that would have triggered an earlier stop under a denser array. For reuse at full capacity that risk should be explicitly assessed before standardising a sparse configuration.

6.11. AE-based reliability updating

Fengqiao Zhang (2025) proposes an AE-informed reliability update that converts localised acoustic-emission activity into a probability-density field to track damage regions and maps this information to a conservative traffic-light resistance factor that feeds a Bayesian proof-test update. In the paper, the reinforced concrete beams reached the target reliability at lower applied loads than a baseline without AE, because internal AE evidence anticipated crack-state changes before large global responses developed. For this thesis, the approach aligns with two core objectives: keeping the proof test strictly elastic while extracting quantitative, mechanics-relevant observables, and enabling earlier, defensible stop decisions once the reliability target is met.

Operationally, the method fits the workflow with limited changes. The step-and-hold protocol already creates natural plateaus at which AE information can be processed. The reliability framework already accommodates prior–survival–posterior updates and the acceptance logic already combines conservative green-light criteria with negligible residuals upon unloading. Within this structure, AE can supply an internal indicator that complements deflections and visual or DIC observations and may justify terminating a proof test before approaching damaging states, provided the updated reliability index has reached the project target (Fengqiao Zhang, 2025).

In Fengqiao Zhang (2025), the stochastic update during testing is driven by a stepwise AE-derived resistance multiplier (the traffic-light factor) coupled to survival at the current actuator load. The load model is essentially the laboratory test action, and the update targets reliability at that load level during the test. In this thesis, the annual reliability over a long horizon is evaluated and a wider set of variables is sampled in line with Eurocode-style sectional checks and traffic actions derived from WIM spectra and combination rules. The posterior resistance is obtained by truncating the sectional resistance at the mapped sectional demand of the proof level, updating key parameters such as effective prestress from measurements, and introducing a model-discrepancy factor that bridges sectional idealization and global behaviour. Dependencies between proof and service load-effect models are retained, and measurement errors in the back-calculation of parameters are propagated. Practically, the paper's Monte Carlo focuses on AE-informed resistance scaling at test increments, whereas the present Monte Carlo propagates code-model parameters, load tails, survival conditioning at the control section, and discrepancy so that the resulting reliability index reflects long-term traffic uncertainty as well as test evidence.

To implement the method here, several adaptations are required. The AE regions and thresholds should be calibrated for prestressed inverted T-girders rather than the reinforced-concrete specimens in the paper. This means defining tie and diagonal-strut windows around the control section and selecting conservative traffic-light limits from pilot data so that green reflects early flexural activity in the tensile zone and yellow reflects incipient diagonal action in the web under the present geometry and prestress level. Instrumentation and uncertainty treatment should be explicit: a sparse, targeted AE layout is consistent with the monitoring philosophy, but arrival-time picking accuracy, effective wave speed, and sensor-coupling variability need to be quantified and propagated through the AE probability field into the resistance factor used during reliability updating. Where DIC checks at key plateaus should validate transitions between traffic-light states. The probabilistic implementation should reflect the project choices: priors for loads and resistances consistent with the Eurocode models used here,

explicit correlations between proof-test and service load-effect models, and a model-discrepancy term separating sectional prediction from global behaviour.

Important caveats accompany the transfer. The published evidence is from laboratory RC beams. Extending thresholds to prestressed inverted T-girders requires careful mapping of crack kinematics and may shift the traffic-light loads. Field noise, boundary-condition variability, and access constraints can reduce signal quality relative to the lab and should be reflected in the likelihood and discrepancy terms. The traffic-light resistance factor is deliberately conservative between thresholds, so early green confirmations should be treated as supportive evidence rather than replacements for code checks and elastic stop rules. Despite these limitations, AE-based reliability updating remains well suited to this thesis: it strengthens the information content of an elastic proof test, offers a transparent route to earlier stop decisions, and integrates naturally with the probabilistic framework already developed.

6.12. Effect of testing additional girders on assessed reliability

Adding more proof-load tests can improve the assessed reliability, but the mechanism and the magnitude of the improvement depend on how the evidence is pooled and how representative the tested set is. Two effects are relevant. First, each tested girder that survives a specified proof load benefits individually from truncation of its resistance tail at the mapped sectional demand, which raises its own β . Second, if results are combined in a hierarchical population model, survival data from several members reduce epistemic uncertainty for the remaining untested stock, thereby improving the posterior reliability for the group.

In a hierarchical setting, survival at a common proof level from n representative girders tightens the population prior on the fraction of weak members. With no observed failures up to the proof load, a one-sided 95% upper bound on the per-girder failure probability at that level is

$$p_{0.95} = 1 - 0.05^{1/n},$$

which is well approximated by $-\ln(0.05)/n \approx 3/n$ for moderate n (and slightly conservative for small n). While this bound does not directly give β_{100} , it reduces the mass that must be reserved in the lower resistance tail for untested girders and therefore raises the population-level reliability. The gain is largest for the first few well-chosen tests and then shows diminishing returns: moving from $n = 2$ to $n = 6$ is far more informative than moving from $n = 20$ to $n = 24$.

The informational value also depends on where the proof load lies relative to the resistance distribution of the decision model. If the mapped sectional demand of the proof load sits near the lower tail, survival mainly trims a small portion of weak realisations and the improvement in β is modest. If it sits closer to central quantiles, survival is informative but the risk of inducing damage grows and the approach can conflict with the reuse objective. For the present geometry and models, raising the proof level above 550 kN would likely add information but also approaches load steps where visible flexural cracking occurs and initiating shear is detected, which reduces the margin for safe reuse at full capacity.

Beyond survival evidence, a small number of calibration tests with richer measurements can shift model parameters and reduce bias. For example, combining AE with targeted instrumentation and, where feasible, DIC can refine wave speed, arrival picks, and mapping of global force to sectional demand. In a hierarchical shear model this calibration step can move the population mean resistance and reduce model discrepancy, which often has a larger effect on β than further tail trimming from additional survivals. Consequently, a mixed campaign that includes a few proof-load survivals across key covariates together with one or two deeper calibration tests is generally more efficient than many identical survivals.

Representativeness is critical. If only the best-looking girders are tested, group updating will overstate the reliability of the untested stock. Tests should therefore be stratified over influential covariates such as effective depth, a_{cs} , a_v , aggregate size, and prestress level, and should reflect the range of manufacturing eras and exposure histories in the stock. The proof protocol and green-light stop rules should be kept consistent across tests to avoid confounding operational differences with structural behaviour.

There are limitations. The improvement achievable through group evidence is bounded by the demand–resistance overlap at the control section. If the modelled resistance sits far to the left of the load effect,

even a large number of survivals will not lift β to the CC3 target without addressing model bias or reducing demand. Benefits also depend on a consistent translation of global proof load to sectional effects. If system action differs across members, pooling survivals at a nominally common load can be misleading. Finally, uncertainty in AE thresholds and localisation, if not stabilised across campaigns, can blur the link between observed activity and the parameter updates that feed the reliability model.

In practical terms, testing more girders helps most when combined with a hierarchical Bayesian update, representative sampling over key covariates, and at least one well-instrumented calibration to address model discrepancy. Under those conditions, the assessed reliability for tested members increases through tail truncation, and the posterior for untested members improves through reduced epistemic uncertainty, albeit with diminishing returns as the sample grows.

6.13. Reflection on correlation between variables

In practice, several basis variables are not strictly independent. Fabrication tolerances can induce co-variation among geometric quantities (e.g. b_w , h , and the derived A_c), and concrete tensile and compressive properties tend to co-move. In the present model it captures the most material dependency by deriving A_c from the sampled widths and heights, but it retains independence elsewhere (including between $b_{w,2004}$ and $b_{w,2023}$), to avoid introducing weakly justified joint structures. The net impact of additional correlation on shear reliability is sign-ambiguous: positive co-variation in the product $b_w d$ broadens the resistance spread, while shared terms in $\rho_l = A_p/(b_w d)$ can partially cancel. In the end this can have some small influence on the outcome

6.14. Limitations

The load model reflects dominant cases for the proof-testing framework rather than a full bridge design envelope. Not all combinations and secondary effects were included. The quoted values for V_{ULS} and V_{SLS} should therefore be read as representative for the testing methodology, not as worst-case envelopes for a specific project.

Resistance checks were performed at selected control sections with parameters derived from available drawings and measurements. Small shifts in the section location or in parameters that shape the shear model, notably $a_{cs} = |M_{Ed}/V_{Ed}|$, the effective shear span a_v , the aggregate-related term d_{dg} , and the effective depth influenced by prestress layout, can change $V_{Rd,c}$. The discussion compares EC2 2004 and the next-generation EC2 at a consistent section rather than searching for a global lower bound across all admissible sections, so results are section-conditional.

Several assumptions affect the mapping between global test force and sectional demand. During proof and failure testing, system action and alternative load paths can be mobilised that are not captured by a single-section idealisation. If the experiment engages compression fields or arching outside the assessed section, survival at a given global proof level may overstate the truncation of the sectional resistance tail used in the reliability analysis.

Acoustic-emission evidence is subject to measurement and processing limits. Thresholding was tuned using a peak frequency of 27.5 kHz to remove step-related background, yet residual background remained and can blur the first-activity load. Source localisation relied on an effective wave speed of 3200 m/s and sufficient sensor geometry. Spatial variation in wave speed, arrival-time pick error, and unfavourable source–sensor geometry in the bottom flange can introduce location offsets or suppress localisation even when hit peaks are evident.

The probabilistic analysis adopts specific priors and modelling choices. Bayesian updating focuses on effective prestress inferred from AE. Other epistemic inputs in the EC2 2023 shear model, including d_{dg} and fracture-related parameters, remain as modelled and could bias tail placement if misestimated. The Monte Carlo horizon and traffic-growth representation influence the demand tail. Different spectra or combination rules would shift annual failure probabilities. Reported reliability indices are therefore conditional on these modelling selections.

Explorations that adapt provisions outside their original scope introduce additional uncertainty. Extending RBK 1.2 width averaging to the next-generation EC2 was treated as a what-if to examine sensitivity. It is not explicitly permitted by the new model and its interaction with a_{cs} , a_v , and aggregate terms is

not fully characterised. Gains observed under this assumption should be interpreted cautiously.

The “reduce load effect” scenario adds further limitations. The transverse distribution adopted for WIM-based actions (effective width taken as lane width plus twice the distance to the neutral axis) is a modelling choice for this framework context. Alternative effective-width models, combination rules, or WIM spectra could change the inferred demand and, with it, β . Any conclusions about meeting CC2 levels under reduced demand remain conditional on the chosen distribution model and on the project’s traffic data.

The “test more girders” discussion also carries assumptions. Group-level improvements rely on hierarchical pooling of evidence and on representative sampling across influential covariates (e.g. d , a_{cs} , a_v , prestress level, aggregate size). Selection bias towards better-looking members, dependence between specimens from the same structure, or inconsistent mapping from global proof load to sectional demand can distort the posterior and overstate reliability for the untested stock. The one-sided 95% upper bound with zero failures,

$$p_{0.95} = 1 - 0.05^{1/n} \approx 3/n \text{ for moderate } n,$$

is only a crude constraint on the fraction of sub-proof members and does not directly translate to β_{100} ; it should be embedded in a full probabilistic update.

Finally, external validity is limited. The discussion synthesises evidence from a single girder type, geometry, and instrumentation layout. The suggestion that a sparser AE array can suffice for decision support is contingent on crack initiation occurring in instrumented hot spots, on stable thresholds and coupling, and on conservative stop rules. Generalising instrumentation simplifications, pre-screening thresholds, proof-level choices, or group-inference gains to other girders should be done cautiously and, where possible, validated on a broader specimen set.

Conclusion & recommendations

This chapter revisits the research questions posed at the outset and answers them in light of the findings. It then addresses the main research question to bring the study to a close. Finally, it offers recommendations to guide future work on this topic.

7.1. Research question

How can load testing with acoustic emission be applied to assess the reliability of reused concrete girders subject to bending and shear?

This main research question will be addressed through the following subquestions:

1. How can acoustic emission techniques support load testing to provide useful information for the new Eurocode about the concrete girder?
2. How to assess the reliability of the concrete girders based on the information obtained by load testing and the new Eurocode?

7.1.1. Acoustic emission

How can acoustic emission techniques support load testing to provide useful information for the new Eurocode about the concrete girder?

In this research both eurocode 2:2004 and the next-generation eurocode 2:2023 are used. For bending, both generations rely on sectional strain analysis with a code-defined stress block. For shear without transverse reinforcement, however, eurocode 2:2023 introduces a more refined and generally more conservative model than eurocode 2:2004, especially for members with thin webs and prestress. For the case study this leads to shear resistances of $V_{Rd,c,2004} = 417.11$ kN and $V_{Rd,c,2023} = 357.37$ kN. The annex I verification gives $V_{Rd,c,2023,I} = 262.04$ kN and is therefore not governing. All of these resistances lie well below the acting shear at ult, $V_{Ed} = 613.13$ kN, so on a conventional code-check basis alone the girder would not have sufficient capacity for reuse in a new structure for 100 years (CC3).

Acoustic emission (AE) augments proof load testing by revealing when and where damage initiates and evolves. By filtering load-correlated hits with a peak-frequency threshold of 27.5 kHz and using a validated wave speed of 3200 m/s for source localisation, AE identifies flexural- and shear-related cracking during the test. This enables a defensible estimate of first cracking and subsequent crack growth rather than relying solely on visual inspection.

In this case study the first signs of flexural cracking were detected at a load level of 462 kN from the hit rate per sensor, well before the large flexural crack that formed between 600 and 650 kN. The first signs of shear-related activity were detected at 589 kN, while the first visual shear cracks appeared at about 750 kN. These results show that AE can indicate that the girder can carry substantially more load than predicted by eurocode-based sectional checks alone and can do so earlier than visual inspection.

The observations also enable back-calculation of uncertain parameters, in particular the effective prestress. Using the first AE peak associated with the initiating flexural crack gives a refined prestress consistent with the first activity in the flexural region. For this girder the reference point was a load of 462 kN and a cracking moment of 1454 kNm excluding self-weight. The effective prestress is updated from 2507 kN to 2686 kN, which directly influences the resistance obtained from both the current and the next-generation eurocode 2 shear models.

Overall, AE indicates whether cracks are developing during the test in real time and, in post-test analysis, provides estimated crack locations from clusters of hits. This supports predefined stop levels and acceptance criteria so that the test can be halted before irreversible damage occurs and the girder remains suitable for reuse at full capacity, while at the same time providing better-informed inputs to the eurocode-based reliability assessment.

7.1.2. Reliability girder

How to assess the reliability of the concrete girders based on the information obtained by load testing and the new eurocode?

Reliability was assessed by combining resistance models from eurocode 2:2004 and eurocode 2:2023 with probabilistic models for load effects over a 100 year horizon, with a target reliability index of $\beta = 4.27$ for consequence class 3.

Without using information from the proof test, the 100 year reliability indices are $\beta_{100} = 2.81$ for EC2:2004 and $\beta_{100} = 2.76$ for EC2:2023. Including survival at the 550 kN proof load, implemented by conditioning on the corresponding sectional shear at the control section, raises these values to $\beta_{100} = 2.94 (+0.13)$ and $\beta_{100} = 2.90 (+0.14)$, respectively. When the AE based update of the effective prestress (from 2507 kN to 2686 kN) is added on top of the proof-survival information, the indices increase further to $\beta_{100} = 2.98 (+0.17)$ for EC2:2004, and $\beta_{100} = 2.88 (+0.12)$ for EC2:2023. A full overview of the reliability indices for the different scenarios and resistance models is given in table 5.1. The pattern is clear: test information and parameter updating improve reliability, but even the most optimistic configuration remains well below the CC3 target.

Higher proof loads could, in principle, yield larger increases in β , but for this girder they would conflict with the reuse objective. From about 600 kN upward, visible flexural cracking was observed and AE indicated initiating shear activity around 589 kN, so pushing the proof level higher would likely cause unacceptable damage. Reducing on the load effect side also have a large influence. Reducing the load effect of traffic by 51 kN results in an increase of reliability index of respectively $\beta_{100} = 3.36 (+0.55)$ and $\beta_{100} = 3.26 (+0.50)$ without even doing a proof load test. In summary, combining proof-load survival at 550 kN with AE informed parameter updating raises the assessed reliability but does not close the gap to the target. Further improvement would require either demand reduction in the receiving structure, structural strengthening, or a different proof concept with carefully managed risk.

7.1.3. Assess reliability by load testing using acoustic emission

How can load testing with acoustic emission be applied to assess the reliability of reused concrete girders subject to bending and shear?

This study shows that AE-assisted proof-load testing can materially improve the evidence base for reliability assessment, but in the present case it does not by itself achieve the target reliability. Conventional EC2 checks indicated insufficient shear capacity at the control section, with $V_{Rd,c,2004} = 417.11$ kN and $V_{Rd,c,2023} = 357.37$ kN compared with $V_{Ed} = 613.13$ kN. During the proof test the girder survived 550 kN within the green-light criterion, while AE revealed early damage processes and locations: first flexural activity at about 462 kN and first shear activity at about 589 kN, well before visually evident cracking. The ultimate shear failure occurred at a sectional shear of about $V = 875.1$ kN including self-weight, which is much higher than the capacities calculated with EC2. AE-based back-calculation refined the effective prestress from 2507 kN to 2686 kN, reducing uncertainty in inputs to EC2 checks.

These data fed a probabilistic assessment in three steps. Without updating, the 100-year reliability indices are $\beta_{100} = 2.81$ for EC2:2004 and $\beta_{100} = 2.76$ for EC2:2023. Conditioning on survival at the 550 kN proof load, implemented by truncating the resistance at the corresponding sectional proof demand at the control section, $V = 500.8$ kN including self-weight, increases the indices to $\beta_{100} = 2.94 (+0.13)$ for

EC2:2004 and $\beta_{100} = 2.90 (+0.14)$ for EC2:2023. Combining survival at 550 kN with the AE-informed prestress update leads to an increase of $\beta_{100} = 2.98 (+0.17)$ for EC2:2004, and $\beta_{100} = 2.88 (+0.12)$ for EC2:2023. All values remain below the target of 4.27. Increasing the proof load might raise the updated reliability but would also raise the risk of inducing irreversible damage, as flexural cracking became clearly visible from 600 kN and the first signs of initiating shear occurred at 589 kN. Reducing the load effect also has a large influence on the reliability index, but that is depending on the type of structure that is designed if that is possible and desirable.

In sum, AE-assisted proof-load testing enhances observability, enables parameter updating, and produces better-justified reliability estimates for reused girders. For this girder, the approach improved the assessed reliability but did not meet the target, indicating that any reuse at full capacity would require additional measures such as demand reduction in the receiving structure, other resistance model, or an alternative proof strategy with carefully managed risk.

7.2. Recommendations

Based on the research, several recommendations can be made for future work and to enhance the reuse of reclaimed girders.

Validate acoustic emission with digital image correlation. Time-align AE hit peaks with strain and crack maps to confirm crack initiation and locations, then use the results to tune the peak-frequency threshold and the identified wave speed for source localisation. This reduces the risk of selecting a threshold that discards relevant hits and missing early crack initiation.

Refine the proof-load protocol for both safety and information gain. Define step and hold sequences, real-time stop rules tied to AE, and post-test checks to confirm the absence of irreversible damage, so that reuse at full capacity remains feasible when the target reliability can be met. Use Bayesian updating to predefine a maximum proof-load level at which the updated reliability is expected to reach the target.

Calibrate model discrepancy between EC2 predictions and observed behaviour. The ultimate shear failure at $V_{load} = 875.1$ kN relative to code resistance motivates evaluating alternative formulations better suited to this geometry. Results from the testing campaigns in the Stevin-II laboratory at TU Delft can provide relevant evidence for such calibration.

Optimise instrumentation layout for future campaigns. Use sensitivity studies to place AE sensors and DIC fields where they maximise detection of flexural and inclined cracks, reducing the number of sensors while improving the efficiency of the test protocol and post-processing. This can lower the cost per test and improve the business case for reuse.

Adopt AE-based reliability updating as demonstrated by Fengqiao Zhang (2025). Incorporate a traffic-light mapping from AE activity to a resistance modifier and perform Bayesian updating at each proof-load plateau to justify earlier stop decisions while remaining elastic. Begin with a laboratory calibration on a prestressed inverted T-girder to define project-specific AE regions, select thresholds, and quantify localisation uncertainty. Then run a small field pilot before wider rollout. Document a standard operating procedure that fixes sensor layout, unloading treatment, plateau durations, and acceptance/stop rules so that results are comparable across campaigns.

Consider life-cycle aspects of reuse. Combine the technical reliability outcomes with environmental and cost indicators to support decisions when several feasible reuse or strengthening options are available.

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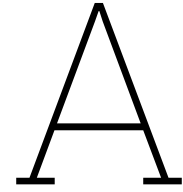
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Probabilistic reliability script

Down below is the script given that is used for the probabilistic reliability for this research.

```
% SHEAR reliability – EC2:2004 vs EC2:2023 handled SEPARATELY
% Figures:
% (1) R-E BEFORE proof (counts vs kN)
% (2) Reliability index over time
% (3) R-E AFTER proof (counts vs kN, common E; left tails for EC2004 & EC2023 shaded)

clc; close all; clf; clearvars;
set(0,'DefaultAxesFontName','Times New Roman'); set(0,'DefaultTextFontName','Times New Roman');
set(0,'DefaultAxesFontSize',20); set(0,'DefaultTextFontSize',20); set(0,'defaultlinelength',2);

%% ----- SETTINGS -----
ny = 101; % timeline points (0..100 years)
years = 0:100; % years since proof load
nr = 5e7; % NOTE: extremely large; reduce if memory-limited

% Common x-axis & binning for figs 1 & 3
XMIN = 0; XMAX = 800; NBINS = 160;
BIN_EDGES = linspace(XMIN, XMAX, NBINS+1);

%% ----- HELPERS -----
ln_mu = @(m,v) log(m.^2 ./ sqrt((m.*v).^2 + m.^2));
ln_sig = @(v) sqrt(log(v.^2 + 1));
logn = @(m,v,sz) exp( ln_mu(m,v) + ln_sig(v) .* randn(sz) );

normcdf_fast = @(z) 0.5 * erfc(-double(z) ./ sqrt(2));
norminv_fast_icdf = @(p) -sqrt(2).*erfcinv( 2.*max(min(double(p),1-1e-15),1e-15) );
logninv_fast = @(U, muL, sigL) exp( muL + sigL .* norminv_fast_icdf(U) );

EM = 0.5772156649015329;
gumbel_ms = @(m, s) deal( m - (sqrt(6)/pi)*s*EM, (sqrt(6)/pi)*s );
gumbel_mv = @(m, v) gumbel_ms(m, abs(v).*m);
gumbel_icdf = @(U, mu, beta) mu - beta .* log( -log( max(min(U,1-1e-12),1e-12) ) );

%% ----- INPUT MEANS & COVs -----
% Geometry & section (mm, mm^2)
d_mean = 791.73; cov_small=0.02;
bw2004_mean = 250.0; bw2023_mean=200.0; cov_geom=0.05;
h_top_flange = 250.0; bf=980.0; bw=200.0; hw=650.0;
Ap_mean=28.0*92.4;
h_tot_mean = 1000.0; z_bot_mean=567.54;

% Materials (MPa) – stochastic
fcm_mean = 88.0; cov_mat = 0.06; % f_cm ~ LN
fyd_mean = 650; % f_yd mean ~ LN
gamma_V = 1.0; % stochastic (no partial factor in resistance)

% Aggregates / geometry (EC2:2023)
D_lower_mean = 32.0; % mm

% Actions for EFFECTS only (E = GDL + GSDL + C0Q.Q)
M_Ed_mean = 492.17; % kNm
V_Ed_mean = 430.94; % kN

% Code constants (EC2:2004)
gamma_c_2004 = 1.0; C_Rd_c=0.18/gamma_c_2004; k1_2004=0.15; z_over_d=0.9;

% Loads for E
mGDL=113.04; mGDL_proof=85.69; mGSDL=28.07;

% Eccentricity used in a_cs & prestress equilibrium
e_p_mm = 338.13; % mm
```

```

e_p_m = e_p_mm/1000;      % m

%% ----- RANDOM VARIABLES -----
d_mm      = single(logn(d_mean,cov_small,[1,nr]));
bw2004_mm = single(logn(bw2004_mean,cov_geom,[1,nr]));
bw2023_mm = single(logn(bw2023_mean,cov_geom,[1,nr]));
Ap_mm2     = single(logn(Ap_mean,cov_geom,[1,nr]));
h_tot_mm   = single(logn(h_tot_mean,cov_geom,[1,nr]));
z_bot_mm   = single(logn(z_bot_mean,cov_small,[1,nr]));
h_top_mm   = single(logn(h_top_flange,cov_geom,[1,nr]));
bw_mm      = single(logn(bw,cov_geom,[1,nr]));
hw_mm      = single(logn(hw,cov_geom,[1,nr]));
bf_mm      = single(logn(bf,cov_geom,[1,nr]));

% Strengths (lognormal)
fcm_MPa = single(logn(fcm_mean,cov_mat,[1,nr]));
fyd_MPa = single(logn(fyd_mean,cov_mat,[1,nr]));

% Deterministic V, M for E plotting
V_Ed_kN = single(logn(V_Ed_mean,0.1,[1,nr]));
M_Ed_kNm = single(logn(M_Ed_mean, 0.1,[1,nr]));
z_mm     = single(z_over_d.*d_mm);

%Deterministic Ac
Ac_mm2_druk = 169000+(bw_mm .* hw_mm);
Ac_mm2      = Ac_mm2_druk+h_top_mm .* bf_mm;

%% ----- PRESTRESS FROM f_ctm,f_l -----
USE_UPDATED_PRESTRESS = true;      % updated method enabled
FP_UPDATED_SD_KN      = 12.5;      % requested SD for updated prestress level

% Constants for the prestress equilibrium (units: mm and N·mm)
W_mm3      = 62828263;
M_eq_Nmm   = 320.36e6;   % updated value
M_load_Nmm = 1453.97e6;

if USE_UPDATED_PRESTRESS
    % ----- UPDATED PRESTRESS: no COV; fixed SD = 12.5 kN -----
    % Use mean f_ctm,f_l = 4.8 MPa to get the mean F_p, then add N(0, (12.5 kN)^2)
    fctmf_l_mean_MPa = single(logn(4.8, cov_mat, [1,nr])); % stochastic with COV
    a = e_p_mm / W_mm3; b = 1 ./ Ac_mm2_druk; c = (M_eq_Nmm + M_load_Nmm) / W_mm3;
    Fp_base_N = (c - double(fctmf_l_mean_MPa)) ./ (a + b); % mean prestress force [N]

    sd_Fp_N = FP_UPDATED_SD_KN * 1e3; % [N]
    Fp_N = single( double(Fp_base_N) + sd_Fp_N .* randn(1,nr,'double') );

    sigma_cp_2004_MPa = single( double(Fp_N) / Ac_mm2_druk ); % [MPa]
    % Axial force for EC2:2023 taken from prestress (compression negative)
    N_Ed_kN = -single(double(Fp_N)/1000); % [kN]
else
    % ----- ORIGINAL PRESTRESS: keep COV on f_ctm,f_l (lognormal) -----
    Fp_N = single(logn(2507e3, cov_mat, [1,nr])); % [N]
    sigma_cp_2004_MPa = single( double(Fp_N) / Ac_mm2_druk ); % [MPa]
    N_Ed_kN = -single(double(Fp_N)/1000); % [kN]
end

%% ----- EC2:2004 RESISTANCE (kN) -----
rho_l = single(Ap_mm2./(bw2004_mm.*d_mm)); rho_l = min(rho_l,single(0.02));

```

```

k_2004 = single(1 + sqrt(200./d_mm)); k_2004 = min(k_2004,single(2.0));
v_min_2004 = single(0.035 .* (k_2004.^1.5) .* sqrt(fcm_MPa));
V_Rdc_2004_N = single((C_Rd_c.*k_2004.*(100.*rho_l.*fcm_MPa).^(1/3) +
k1_2004.*sigma_cp_2004_MPa).*bw2004_mm.*d_mm);
V_Rdcmin_2004_N = single((v_min_2004 + k1_2004.*sigma_cp_2004_MPa).*bw2004_mm.*d_mm);
V_R_2004_kN = single(max(V_Rdc_2004_N, V_Rdcmin_2004_N)./1000);

%% ----- EC2:2023 RESISTANCE (kN) -----
% Longitudinal ratio
rho_l_2023 = single(Ap_mm2./(bw2023_mm.*d_mm));
rho_l_2023 = min(rho_l_2023, single(0.02));

% Aggregate size term (capped at 40 mm)
d_dg_mm = single(16 + D_lower_mean.*(60./fcm_MPa).^2);
d_dg_mm = min(d_dg_mm, single(40));

% Critical section at one effective depth from support:
% a_cs = max( |(M_Ed + N_Ed*e_p)/V_Ed| , d )
M_for_kvp_kNm = M_Ed_kNm; % kNm
V_for_kvp_kN = V_Ed_kN; % kN
a_cs_m = abs( (M_for_kvp_kNm + double(N_Ed_kN).*e_p_m) ./ double(V_for_kvp_kN) ); % [m]
a_cs_mm = single( max( a_cs_m*1000, double(d_mm) ) ); % [mm]

% New Eurocode: use a_v in V_Rd,c with a_v = sqrt( (a_cs/4) * d )
a_v_mm = single( sqrt( max(a_cs_mm,0) .* max(double(d_mm),0) ./ 4.0 ) );
a_v_mm = max(a_v_mm, single(1e-6)); % numerical guard

% k_vp = 1 + (N/|V|)*d/(3*a_cs), EC2 min 0.1
k_vp_raw = single( 1 + (double(N_Ed_kN)./(max(abs(double(V_Ed_kN)),eps)) .* ...
( double(d_mm) ./ (3.0 .* double(a_cs_mm)) ) ) );
k_vp = max(k_vp_raw, single(0.1)); % EC2 lower bound
fprintf('k_vp: min=%.4f, max=%.4f, mean=%.4f\n', ...
min(double(k_vp)), max(double(k_vp)), mean(double(k_vp)));

% Shear resistance per EC2:2023
% Only V_Rdc uses a_v; V_Rdcmin uses d.
V_Rdc_2023_N = single((0.66/gamma_V) .* ...
(100 .* rho_l_2023 .* fcm_MPa .* (d_dg_mm ./ (k_vp .* a_v_mm))).^(1/3) .* ...
bw2023_mm .* d_mm);

V_Rdcmin_2023_N = single((11/gamma_V) .* ...
sqrt((fcm_MPa ./ fyd_MPa) .* (d_dg_mm ./ d_mm)) .* ...
bw2023_mm .* d_mm);

V_R_2023_kN = single(max(V_Rdc_2023_N, V_Rdcmin_2023_N) ./ 1000);

% Global factors for limit state (kept as-is)
mR=1; vR=0.05; mE=1; vE=0.10;

% E components
vGDL=0.04; rv_GDL = single(mGDL + mGDL*vGDL.*randn(1,nr));
vGDL_proof=0.04; rv_GDL_proof = single(mGDL_proof + mGDL_proof*vGDL_proof.*randn(1,nr));
vGSDL=0.25; rv_GSDL = single(mGSDL + mGSDL*vGSDL.*randn(1,nr));

% Traffic base distributions
mQ=(703/3.84); vQ=0.03; [mu_Q,beta_Q] = gumbel_mv(mQ,vQ);
mC0Q=1.2315; vC0Q=0.14; rv_C0Q = single(logn(mC0Q,vC0Q,[1,nr]));

%% ----- PROOF-LOAD SETTINGS -----
proof_year = 0; % year index of proof test (0..100)

```

```

Vp_shear = 452.82; % proof shear [kN]415.09
Vp_includes_selfweight = false; % set true if Vp already includes G

%% ----- COMMON E FOR FIGS 1 & 3 -----
E_common = single(rv_GDL + rv_GSDL + rv_C0Q .* rv_Q);

%% ----- R-E BEFORE PROOF (Figure 1) -----
figure(1); clf; hold on;
hist_stairs_weighted(R2004_vis, BIN_EDGES, w2004_vis, 'Color', [0 1 0], 'DisplayName', 'R_{2004}');
hist_stairs_weighted(R2023_vis, BIN_EDGES, w2023_vis, 'Color', [1 0 1], 'DisplayName', 'R_{2023}');
% Plot E with IS weights so it reflects the TARGET (not proposal bias)
hist_stairs_weighted(E_common, BIN_EDGES, w, 'Color', [1 0 0], 'DisplayName', 'E (common, target)');
box on; xlabel('shear [kN]'); ylabel('counts [-]');
legend('show', 'location', 'northeast'); xlim([XMIN XMAX]);
title('R-E distributions - BEFORE proof load (counts)');

%% ----- RELIABILITY OVER TIME (Figure 2) -----
mct0=1.0; vct0=0.10; ct0=single(logn(mct0,vct0,[1,nr]));
mdct=0.0013; vdct=0.14; dct=single(logn(mdct,vdct,[1,nr]));

nf_2004=zeros(1,ny); failed_2004=false(1,nr);
nf_2023=zeros(1,ny); failed_2023=false(1,nr);

S0 = sum(w); % initial total weight
thr_weight = S0/100; % 1/100 of original (freeze rule)

Sprev_2004 = S0; Sprev_2023 = S0; % survivor weights
pf_2004=zeros(1,ny); pf_2023=zeros(1,ny);
beta_2004 = nan(1,ny); beta_2023 = nan(1,ny);
Sprev_hist_2004 = zeros(1,ny); Sprev_hist_2023=zeros(1,ny);

capBeta = 5.5; % cap and axis limit

for jj=1:ny
    year=years(jj); growth=max(year-proof_year,0);
    Sprev_hist_2004(jj)=Sprev_2004; Sprev_hist_2023(jj)=Sprev_2023;

    scale=single(ct0 .* (1 + dct .* growth));
    Vt_kN=single(scale .* rv_C0Q .* rv_Q); % variable component

    if year==proof_year
        Vt_kN = max(Vt_kN, single(Vp_shear)*ones(1,nr,'single'));
        const_E_year = single(Vp_includes_selfweight * 0 + (~Vp_includes_selfweight) * rv_GDL_proof);
    else
        const_E_year = single(rv_GDL + rv_GSDL);
    end

    Rt2004=V_R_2004_kN; Rt2023=V_R_2023_kN;
    Z2004=rv_thetaR .* Rt2004 - 1.0 .* (const_E_year + Vt_kN);
    Z2023=rv_thetaR .* Rt2023 - 1.0 .* (const_E_year + Vt_kN);

    new_2004=(~failed_2004) & (Z2004<0);
    new_2023=(~failed_2023) & (Z2023<0);

    nf_2004(jj)=sum(w(new_2004));
    nf_2023(jj)=sum(w(new_2023));

    pf_2004(jj)=nf_2004(jj)/max(Sprev_2004,eps);
    pf_2023(jj)=nf_2023(jj)/max(Sprev_2023,eps);

```

```

b2004 = -norminv_fast_icdf(pf_2004(jj));
b2023 = -norminv_fast_icdf(pf_2023(jj));

if pf_2004(jj)==0 && Spreved_2004>0, b2004 = capBeta; end
if pf_2023(jj)==0 && Spreved_2023>0, b2023 = capBeta; end
if Spreved_2004<=0, b2004 = NaN; end
if Spreved_2023<=0, b2023 = NaN; end

% Freeze  $\beta$  when survivors <= 1% of original, unless new  $\beta$  is lower
if Spreved_2004 <= thr_weight && jj>1 && ~isnan(beta_2004(jj-1))
    b2004 = min(beta_2004(jj-1), b2004);
end
if Spreved_2023 <= thr_weight && jj>1 && ~isnan(beta_2023(jj-1))
    b2023 = min(beta_2023(jj-1), b2023);
end

beta_2004(jj) = b2004;
beta_2023(jj) = b2023;

Spreved_2004=Spreved_2004-nf_2004(jj);
Spreved_2023=Spreved_2023-nf_2023(jj);
failed_2004(new_2004)=true;
failed_2023(new_2023)=true;
end

mask=years>=2; yy=years(mask);
figure(2); clf; hold on;
plot(yy, beta_2004(mask), 'g', 'DisplayName', '\beta (EC2:2004)');
plot(yy, beta_2023(mask), 'm', 'DisplayName', '\beta (EC2:2023)');
xlabel('years after proof load [yr]'); ylabel('annual \beta [-]'); box on; ylim([0 5.5]);
legend('show');
title('\beta over time (annual hazard)');

%% ----- R-E AFTER PROOF (Figure 3 - shaded left tails aligned) -----
R2004_all = V_R_2004_kN;
R2023_all = V_R_2023_kN;

% Proof cut value for visualization
if Vp_includes_selfweight
    cut_value = Vp_shear;
else
    cut_value = Vp_shear + mean(double(rv_GDL_proof));
end

figure(3); clf; hold on;

% Compute FULL weighted counts used for outlines
c2004_full = histcounts_legacy_weighted(R2004_vis, BIN_EDGES, w2004_vis);
c2023_full = histcounts_legacy_weighted(R2023_vis, BIN_EDGES, w2023_vis);

% Derive left-tail counts from those (with partial-bin split at the cut)
c2004_left = truncate_counts_to_cut(BIN_EDGES, c2004_full, cut_value);
c2023_left = truncate_counts_to_cut(BIN_EDGES, c2023_full, cut_value);

% Draw shaded tails with EXACT same step geometry
fill_hist_from_counts(BIN_EDGES, c2004_left, [0 0.6 1], 0.35, 'R_{2004} removed (R < proof cut)');
fill_hist_from_counts(BIN_EDGES, c2023_left, [0 0.6 1], 0.18, 'R_{2023} removed (R < proof cut)');

% Draw outlines (target-correct)
hist_stairs_weighted(R2004_vis, BIN_EDGES, w2004_vis, 'Color', [0 1 0], 'DisplayName', 'R_{2004} (after proof)');

```

```

hist_stairs_weighted(R2023_vis, BIN_EDGES, w2023_vis, 'Color', [1 0 1], 'DisplayName', 'R_{2023}
(after proof)');
% Plot E with IS weights so oversampling isn't visible
hist_stairs_weighted(E_common, BIN_EDGES, w, 'Color', [1 0 0], 'DisplayName', 'E (common,
target)');

% Mark the cut
yl = ylim; plot([cut_value cut_value], yl, 'k--', 'DisplayName', 'proof cut'); ylim(yl);

xlabel('shear [kN]'); ylabel('counts [-]'); box on;
legend('show', 'location', 'northeast'); xlim([XMIN XMAX]);
title('R-E after proof - common E; left tails removed (visual)');

%% ----- LOCAL HELPERS -----
function q = quantile_fast_local(x, qprob)
    x = double(x(:)); n = numel(x);
    if n==0, q=NaN; return; end
    x = sort(x); k = max(1, min(n, ceil(qprob*n))); q = x(k);
end

function h = hist_stairs_weighted(x, edges, weights, varargin)
% Weighted histogram using legacy-compatible histc + accumarray, drawn as stairs.
    x = double(x(:));
    M = numel(edges);
    if nargin < 3 || isempty(weights)
        counts_full = histc(x, edges);
        counts = counts_full(1:M-1);
    else
        weights = double(weights(:));
        if numel(weights) ~= numel(x), error('weights length mismatch'); end
        [~, bin] = histc(x, edges); in = bin>0 & bin<=M;
        counts_full = accumarray(bin(in), weights(in), [M 1], @sum, 0);
        counts = counts_full(1:M-1);
    end
    y = [counts; 0]';
    h = stairs(edges, y, varargin{:});
end

function counts = histcounts_legacy_weighted(x, edges, weights)
% Legacy-compatible bin counts with optional weights (histc/accumarray).
    x = double(x(:)); M = numel(edges);
    if nargin < 3 || isempty(weights)
        counts_full = histc(x, edges);
        counts = counts_full(1:M-1);
    else
        weights = double(weights(:));
        if numel(weights) ~= numel(x), error('weights length mismatch'); end
        [~, bin] = histc(x, edges); in = bin>0 & bin<=M;
        counts_full = accumarray(bin(in), weights(in), [M 1], @sum, 0);
        counts = counts_full(1:M-1);
    end
end

function countsL = truncate_counts_to_cut(edges, counts, cutVal)
% Take counts per bin and return the portion to the LEFT of cutVal.
% Handles a partial bin if cut lies inside [edge_i, edge_{i+1}).
    counts = double(counts(:)); N = numel(counts);
    countsL = zeros(1, N);
    for i = 1:N
        eL = edges(i); eR = edges(i+1);
        if eR <= cutVal

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```

        countsL(i) = counts(i);
    elseif eL < cutVal && cutVal < eR
        frac = (cutVal - eL) / (eR - eL);
        countsL(i) = counts(i) * max(min(frac,1),0);
    else
        countsL(i) = 0;
    end
end
end

function h = fill_hist_from_counts(edges, counts, faceColor, faceAlpha, dispName)
% Draws a filled, step-shaped histogram area that matches 'stairs' binning.
counts = double(counts(:));
y = [counts, 0]; % step heights at each edge (right-closed)
N = numel(counts);
xs = [edges(1), reshape([edges(2:N); edges(2:N)], 1, []), edges(end)];
ys = [y(1), reshape([y(1:N-1); y(2:N)], 1, []), y(end)];
h = patch([xs, fliplr(xs)], [ys, zeros(size(ys))], faceColor, ...
    'FaceAlpha', faceAlpha, 'EdgeColor', 'none', 'DisplayName', dispName);
end

```