Report

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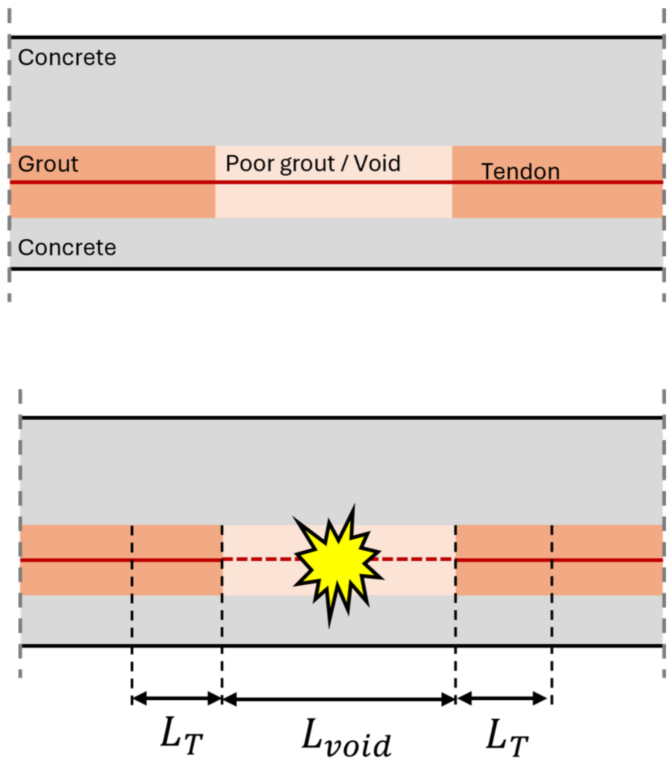
Capacity assessment procedures considering damaged post-tensioning systems

Herøy FoU: WP3 activities report

Trondheim – January – 2025



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ABSTRACT

This document reports the activities related to work package (WP3) of the Herøy FoU project. WP3 focuses on the structural assessment of bridges with damaged post-tension (PT) systems. This work package developed and validated refined models of the Herøysund Bridge. The condition of the post-tension system was modelled based on the inspection results from WP2. The updated model was used to evaluate the effect of different damage scenarios on the capacity of the structure. This work addresses key research questions, including the structural consequences of damaged PT systems and the application of robustness principles to structural assessment. The goal is to provide a comprehensive structural evaluation of the bridge under different damage scenarios using advanced modelling techniques, as well as to propose a framework to include PT damages in future assessment of existing bridges.

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Preface and acknowledgements

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1. Introduction

WP3 focuses on the structural assessment of bridges with damaged post-tension (PT) systems. This work package developed and validated refined models of the Herøysund Bridge. The condition of the post-tension system was modelled based on the inspection results from WP2. The updated model was used to evaluate the effect of different damage scenarios on the capacity of the structure. This work addresses key research questions, including the structural consequences of damaged PT systems and the application of robustness principles to structural assessment. The ultimate goal is to provide a comprehensive structural evaluation of the bridge under different damage scenarios using advanced modelling techniques.

The activities reported here are part of work package WP3, which focuses on structural assessment with damaged post-tensioning systems for the Herøy FoU project. The task within this work package has been divided into two distinct activities:

- WP3.A1 Modelling and assessment by master thesis works. Perform capacity assessments for various bridge conditions (design, current, additional damage) with focus on the damaged PT system.
- WP3.A3 Reporting. Summarize in this document all results from master thesis works together with additional discussions.

Note that another activity, called WP3.A2, was planned in the original project description but was eventually cancelled during the course of the project. This was agreed by the project's steering committee. However, the naming of the activities was kept as described in the original project description for consistency.

2. Master thesis works

The evaluation of the structural integrity of the Herøysund Bridge considering damaged post-tensioned system was proposed as a potential topic for master thesis work to students at NTNU. Following initial discussions with their supervisor, the students explored possible directions for their work. Ultimately, the final topic was tailored to align with the students' interests and was mutually agreed upon in collaboration with their supervisor.

Considering this context, four master theses were developed during the project's duration. These included:

- Thesis 1 – Title: "*Modellering i DIANA FEA og kapasitetskontroll av Herøysund bru*" (WP3.A1.NTNU.Thesis1)
- Thesis 2 – Title: "*Capacity analysis of Herøysund Bridge with a damaged post-tensioned system*" (WP3.A1.NTNU.Thesis2)
- Thesis 3 – Title: "*Modellering og kapasitetsvurdering av Herøysundbruen i DIANA FEA: Kapasitetsvurdering ved ulike skader i etterspennigssystem*" (WP3.A1.NTNU.Thesis3)
- Thesis 4 – Title: "*Robusthet av brukonstruksjoner - Kasusstudie av Herøysund bru*" (WP3.A1.NTNU.Thesis4)

This section describes each these thesis in separate subsections, offering a comprehensive summary of each.

2.1. Assessment for different properties (WP3.A1.NTNU.Thesis1)

The work presented in this section corresponds to the master thesis [1], designated under the codename WP3.A1.NTNU.Thesis1. Table 1 provides a summary of the bibliographical details for this document, outlining essential information for reference.

Table 1: Bibliographical information for WP3.A1.NTNU.Thesis1

Title:	Modellering i DIANA FEA og kapasitetskontroll av Herøysund bru
Author(s):	Brage Sikveland, Therese Steffensen
Date:	June 2023
Language:	Norwegian
Codename:	WP3.A1.NTNU.Thesis1
Link:	https://ntnuopen.ntnu.no/ntnu-xmlui/handle/11250/3093186

A capacity assessment of the Herøysund Bridge was performed in accordance with Eurocode 2, focusing on moment capacity checks at three specific cross-sections. The modelling was conducted using DIANA, incorporating simplifications such as symmetry to reduce calculation time. The study considered various loads, including self-weight, prestressing, traffic, and wind, while applying linear analysis to evaluate the bridge's performance. Additionally, a range of load combinations was explored to ensure a comprehensive assessment of the structure's capacity.

The geometry of the bridge was modelled based on the available drawings, with reinforcement and prestressing incorporated programmatically. DIANA facilitated Python-scripted inputs for defining some or all aspects of the model, enabling the passive reinforcement to be specified according to the spacings detailed in the drawings. Prestressing was integrated using a table-based geometry description of the tendons. This approach proved to be an efficient method for defining complex geometries such as the Herøysund Bridge.

The purpose of this thesis was to assess the moment capacity of the bridge's superstructure in its intact condition using the program DIANA, aiming to verify that the structure is sufficiently dimensioned and to establish an accurate and effective modelling method. This work was complemented by a sensitivity analysis focusing on two structural properties: the friction coefficient of the prestressing system and the compressive strength of the concrete.

The numerical model of the bridge was used to obtain the design moments based on the design loads and appropriate load combinations, which were subsequently utilised to perform moment capacity control for selected sections through simple calculations supported by the Mathcad tool. This analysis compared the new capacity calculations with previous results referenced in [2], revealing that the bridge likely possesses a greater moment capacity than previously estimated.

This worked focused on three particular cross-sections shown in Figure 1:

- Cross-Section 1 (CR#1): Section closest to the support, where the maximum moment value occurs.
- Cross-Section 2 (CR#2): Section along the main span, where some of the prestressing tendons stopped. This section might have lower moment capacity.
- Cross-Section 3 (CR#1): Section at the middle of the main span

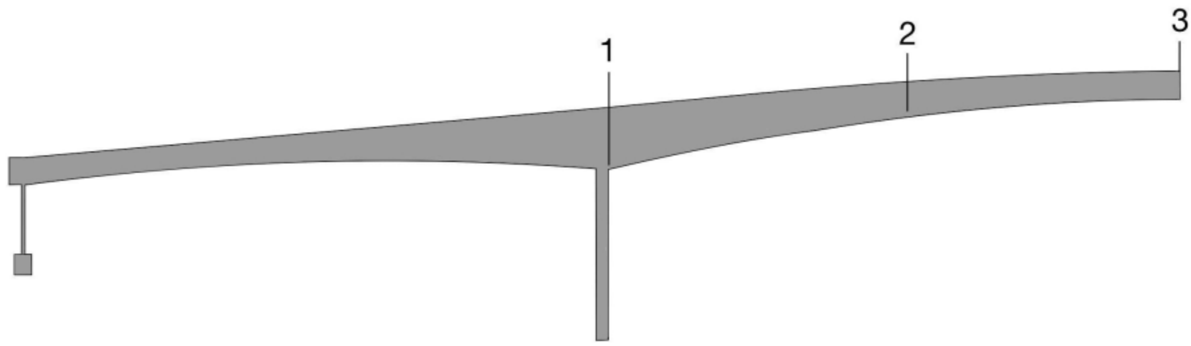

Figure 1: Schematic representation of half Herøysund Bridge indicating the 3 cross-sections studied in this work (Source: [1])

According to the original design, the bridge's concrete strength was classified as C35. However, recent tests determined the strength to be between 44 MPa and 58 MPa. As a result, the numerical model was developed for concrete class C45. Due to this variation, the concrete strength was selected as one of the parameters in the sensitivity analysis.

The prestressing tendons are of type 644, consisting of 44 strands, each with a diameter of 6 mm, bundled together in a parallel configuration rather than the helicoidal configuration commonly used in modern practice. The elastic modulus of the prestressing steel was set at 206 000 MPa, as specified in the original design documentation. The proof stress $f_{p0,1k}$ is 1 520 MPa, with a total tendon area of 1 244 mm² and a prestress force of 1 370 kN.

The analysis with the numerical model included the relevant loads as well as time-dependent effects such as creep, shrinkage, and relaxation, according to Eurocode 2 [3]. Prestress losses were also accounted for, with particular attention given to friction loss, which was evaluated in detail considering the tendon layout and construction stages. It was determined that the parameters influencing friction loss significantly affect the prestressing force across different cross-sections. Consequently, the effect of the friction loss was studied further through a sensitivity analysis of the friction coefficient in the final capacity assessment. Due to the length of the tendons, prestress force losses due to friction were quantified for each of the three sections under consideration, with losses reported between 9% and 12%.

This work validated the post-tension modelling using DIANA FEM software by comparing the modelling of concrete, reinforcement, and active steel, along with the prestressing force, against hand calculations for various simple structural configurations. These included checks for simply supported cases, the symmetry assumption, and the effect of secondary moments in statically indeterminate configurations, as illustrated in Figure 2. The modelling of prestress force losses due to friction was also explored in detail during this validation. Additionally, to obtain the cross-sectional moments required for the moment capacity calculations under specific loads, an auxiliary tool provided by the software, known as the composed line element, was utilised.

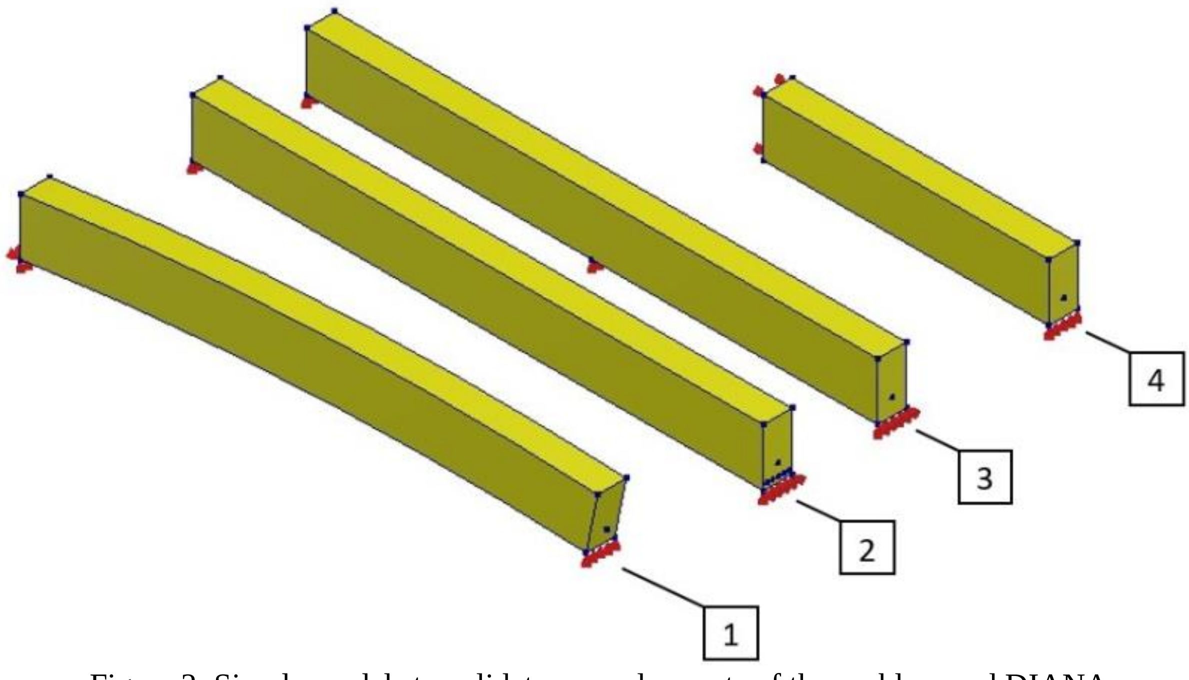

Figure 2: Simple models to validate several aspects of the problem and DIANA functionalities (Source: [1])

Once the tools were validated and all necessary information was gathered, a numerical model of half of the Herøysund Bridge was developed (see Figure 3) based on the reasonable assumption of symmetry. Although the bridge is not perfectly symmetric due to a slight horizontal curve in the span between columns 3 and 4, this deviation was considered negligible for the purpose of moment capacity calculations.

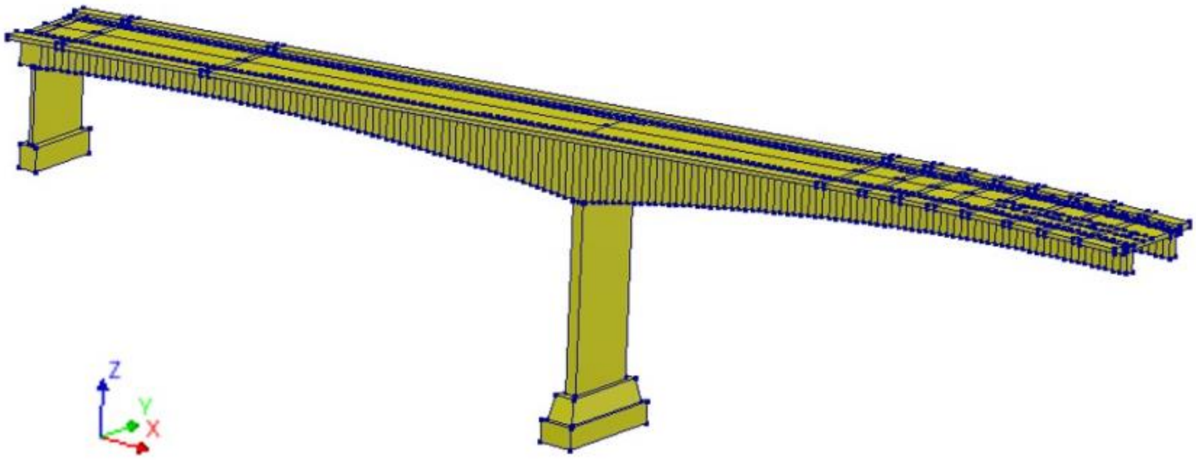

Figure 3: Numerical model of the Herøysund Bridge (Source: [1])

The final model of the bridge was generated using a combination of different element types and orders (linear and quadratic) with varying size requirements. Additionally, the automatic meshing procedure in DIANA further refined the mesh at points where concentrated loads from the vehicle load model were applied (see Figure 4).

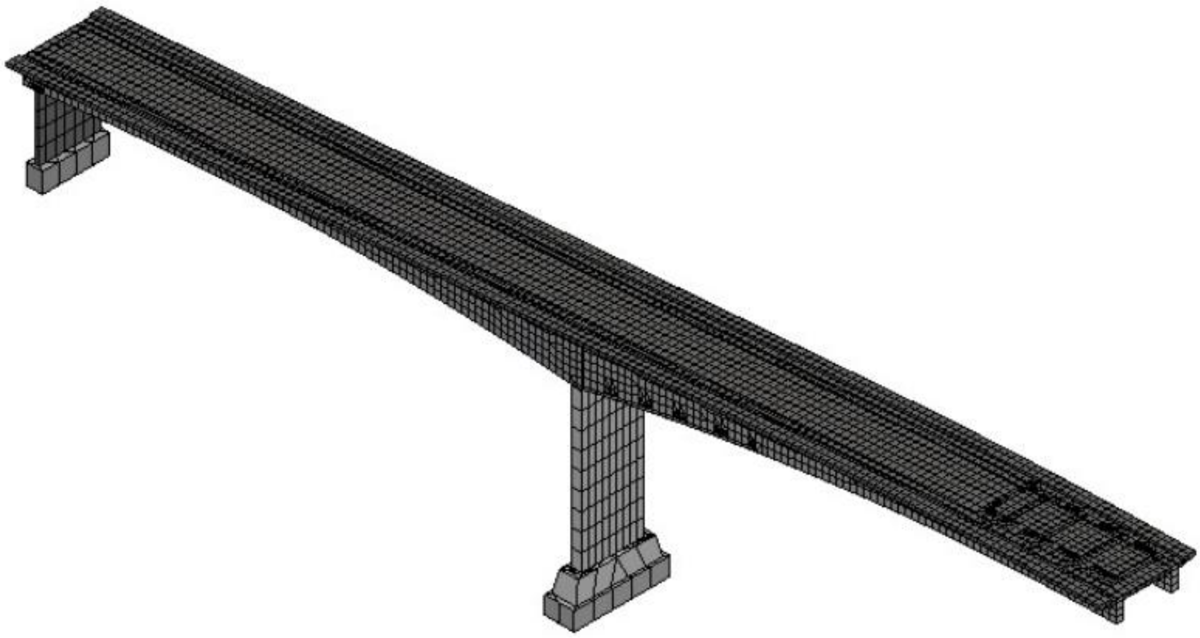

Figure 4: Visualization of the mesh of the model (Source: [1])

The developed numerical model was verified based on measurements reported in [2], where the vertical deformation was measured to be 33 mm when loaded by a 50-tonne truck. Using this information, the global model was fine-tuned by modifying the elastic modulus of concrete. The final adopted elastic modulus (36 000 MPa) differs significantly from the value used in the capacity assessment by Aas Jackobsen (27 174 MPa) in [2]. Figure 5 shows the vertical deformations of the bridge model with the updated elastic modulus value.

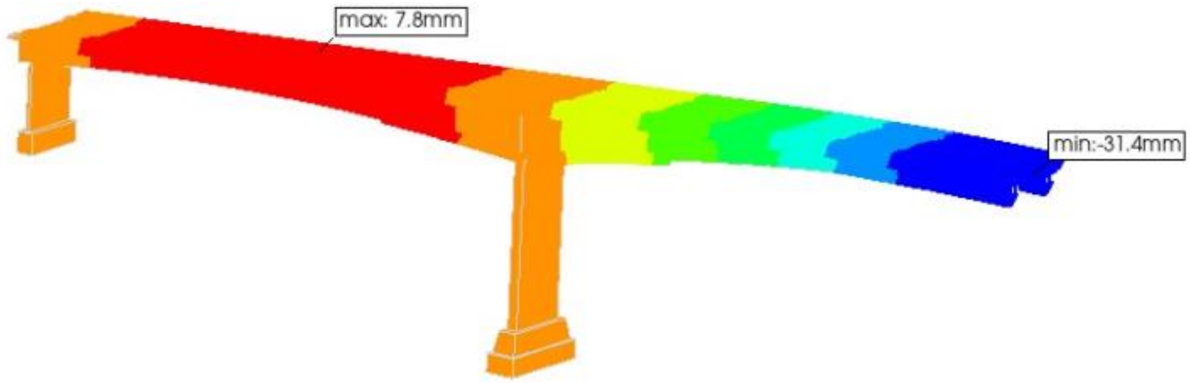

Figure 5: Validation of mid-span deformation due to a 50-tonne truck (Source: [1])

The application of the loads with different load combinations provided the required design bending moment values for the capacity assessment calculation. Figure 6 shows an example of the obtained bending moment results.

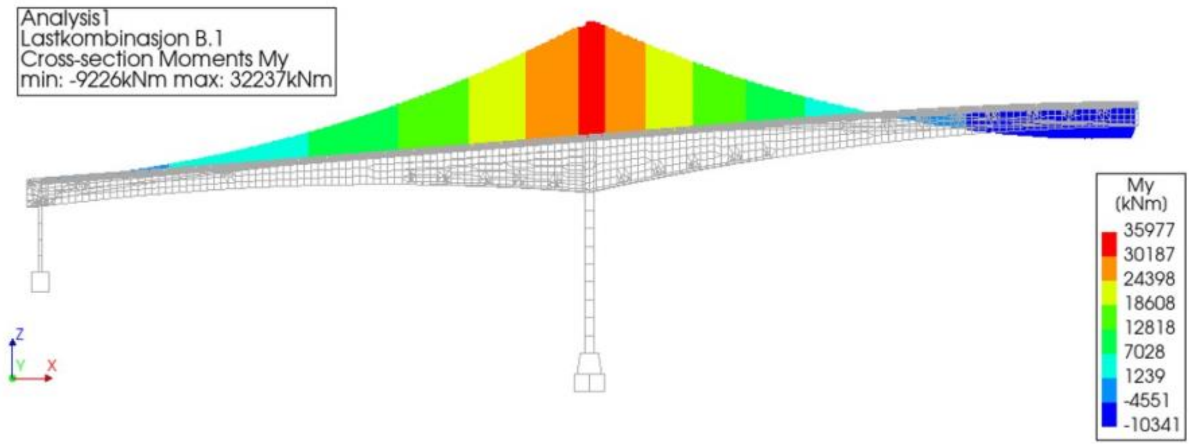

Figure 6: Example of bending moment diagram for certain load combination (Source: [1])

The moment capacity of each cross-section was calculated based on standard design assumptions, including a linear strain distribution, as shown in Figure 7.

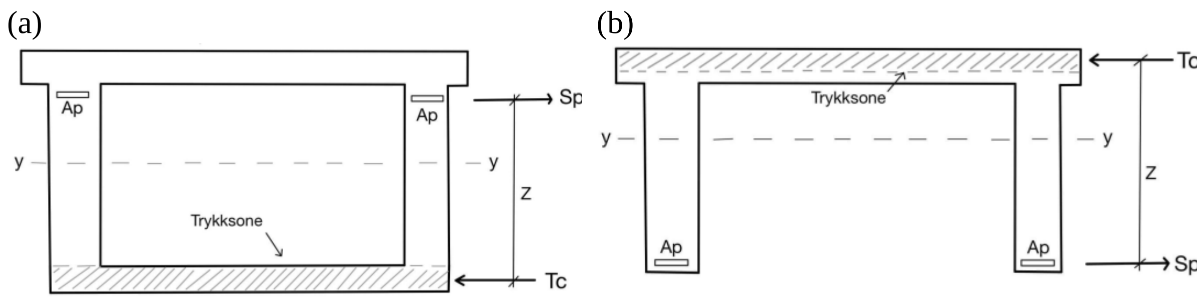

Figure 7: Moment capacity calculation model; (a) Cross-Section 1 (CR#1); (b) Cross-Sections 2 (CR#2) and 3 (CR#3)

The capacity calculations were then compared to previously reported results. In the original design calculations of the bridge in 1965, the capacities were fully utilized, which differs greatly from the results obtained in this revised analysis. This work argues that the primary cause of this difference is the variation in the considered concrete class. However, it is important to note that the available records from that time are difficult to read, and exact numerical values could not be extracted to perform a direct comparison.

In 1987, the re-analysis of the bridge was conducted to accommodate a new layer of 20 mm of asphalt. Significant differences in design moments were found, particularly for the section near the support. It was discovered that in 1987, the structural model represented the bridge as a simply supported continuous bridge. However, in the revised calculation, the support conditions and columns were incorporated into the global model. The new analysis revealed that the secondary moments produced by the prestressing, due to the rotational restraints induced by the columns, greatly increased the design moments, reporting moments up to three times greater than those calculated in 1987.

Comparing the results to the more recent assessment by Aas Jackobsen [2], which was conducted after corrosion damages were detected, the assessment was performed in two ways: without considering the damages and considering the damages. The report used NovaFrame, a software based on beam theory, to obtain the loads, and NovaDesign for the capacity calculations. Although they did not report the capacity of the undamaged bridge, making a direct comparison with this work impossible, they concluded that the capacity was sufficient (when undamaged). This thesis compares the calculated moments for the two sections considered in both studies (namely CR#1 and CR#3), showing differences in results of approximately 7% and 3%, respectively, concluding that the results are approximately similar. Both studies indicate that the moment capacity utilization at the location of maximum moment (CR#1) was adequate (below 100%).

The thesis concludes with a sensitivity analysis to explore the effect of concrete strength and the friction coefficient on the final moment capacity. The results in Figure 8(a) show the design moment for ranges of possible values of these two variables. This analysis was carried out programmatically, where the bridge model was created and analysed systematically via a script. The properties were modified in each run, and the simulation results were extracted using Python scripts. The study demonstrates the potential of utilizing such scripting facilities to extend the capacity assessment to a probabilistic framework, allowing for the same analysis to be performed across a range of different material properties and model considerations.

Including the appropriate probability distributions for each aspect can yield the distribution of the structure's probability of failure. In this work, this was done for the moment capacity, as shown in Figure 8(b) for CR#1, which presents a probability distribution of the moment capacity. This is shown only as a proof of concept, but the procedure can be made more systematic to explore other limit states and repeated at several locations. The method for estimating failure probability can be further improved by generating a larger number of points to increase accuracy. Additionally, more parameters can be included in the study to provide a broader overview of the possibility space. The thesis includes examples of the necessary scripts in its appendix [1].

(a) (b)

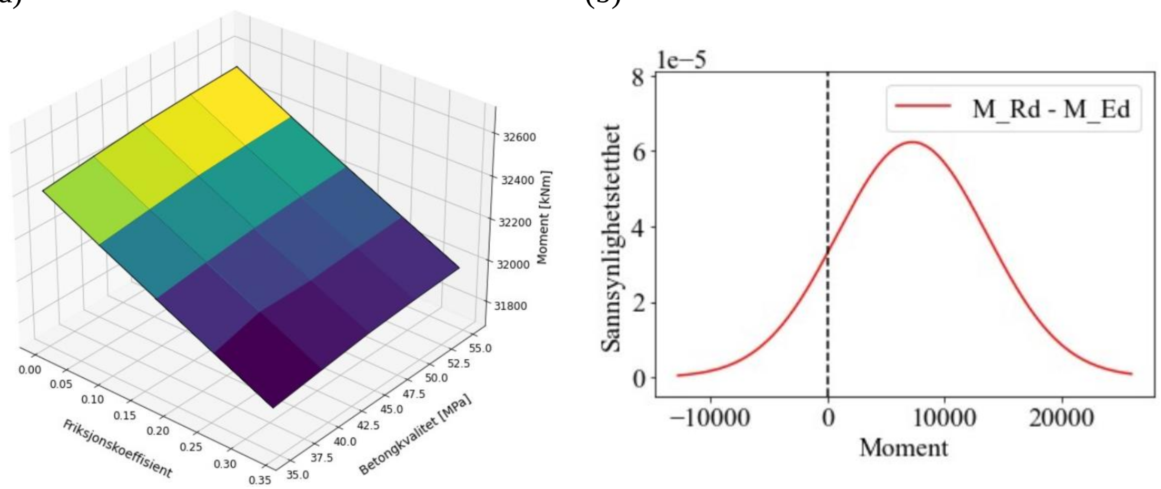

Figure 8: (a) Design moment at CS#1 for various friction coefficients and concrete strength classes. (b) Probability distribution of moment capacity as a result of the proof-of-concept.

This work has demonstrated that utilising a Python script has streamlined the modelling of the bridge. By assembling the entire model within a single Python script, it becomes easy to perform a parameter study, where a new model with adjusted parameters can be generated and analysed in just a few minutes.

The parametric study showed that the moment capacity is heavily influenced by variations in concrete quality. While, the design moment is only marginally affected by concrete quality, as it has minimal impact on long-term effects. This might explain the discrepancy in moment capacities, while the original design led to full utilization, the recent re-analysis showed that the moment capacities are much larger than the design moments.

2.2. Assessment for damage scenarios (WP3.A1.NTNU.Thesis2)

The work presented in this section corresponds to the master thesis [4], designated under the codename WP3.A1.NTNU.Thesis2. Table 2 provides a summary of the bibliographical details for this document, outlining essential information for reference.

Table 2: Bibliographical information for WP3.A1.NTNU.Thesis2

Title:	Capacity analysis of Herøysund Bridge with a damaged post-tensioned system
Author(s):	Amna Gonilovic, Simen Steinkjer Løken
Date:	June 2023
Language:	English
Codename:	WP3.A1.NTNU.Thesis2
Link:	https://ntnuopen.ntnu.no/ntnu-xmlui/handle/11250/3093190

Similar to previous theses, this study involves modelling the Herøysund Bridge in DIANA [5], but with a specific focus on studying the effects of prestress damage. In particular, it examines the consequences of tendon failure in terms of capacity assessments. The number of damaged tendons, the location of the damage, and its extent are systematically investigated.

The goal of the thesis was to evaluate the mid-span section moment capacity and investigate the impact of damage in the post-tensioning system on the design load effects and capacity of Herøysund Bridge, as well as how damage influences the bridge's structural behaviour, loadcarrying capacity, and overall safety. The capacity was initially calculated using hand calculations based on the cross-section properties obtained from existing documentation, incorporating considerations for damage where necessary. Additionally, the design moment was calculated using a FEM model of the bridge in DIANA.

This work extracted all relevant information from existing documentation, including drawings and reports, with particular attention paid to the prestressing configuration. The prestressing in the bridge can be divided into two types: one on the deck and one on the beams. The focus of this work is on the prestressing along the beams, as damage is only considered in this area, which is the only prestressing present at the mid-span section. The tendon configuration across the bridge is schematically represented in Figure 9. To gain a thorough understanding of this layout, a review of the construction process and an in-depth analysis of the existing drawings, design documents, and reports were necessary.

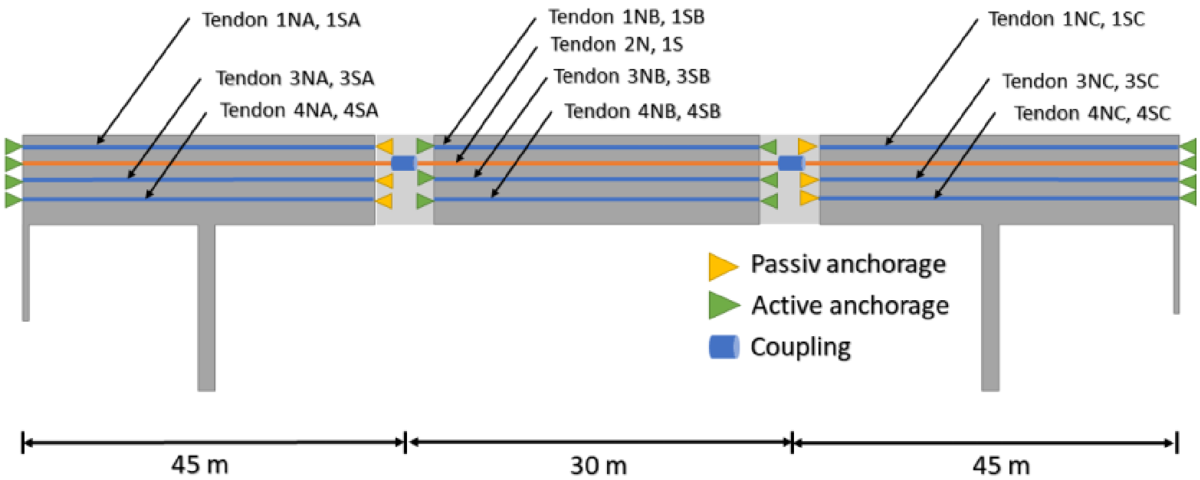

Figure 9: Schematic representation of the post-tension configuration in the beams (Source: [4])

The loads considered in the analysis include self-weight, traffic loading, and time-dependent effects such as creep and shrinkage. The prestressing force was also considered, including the effects of friction loss and relaxation. This work ensured that friction loss was correctly incorporated into the FEM software. For validation, a simplified beam model was explored with different modelling considerations. It was concluded that for a more accurate inclusion of friction loss into the model, linear elements should be used instead of quadratic ones (Figure 10). This approach was compared against hand calculations based on the theory and formulations available in Eurocode 2 [3].

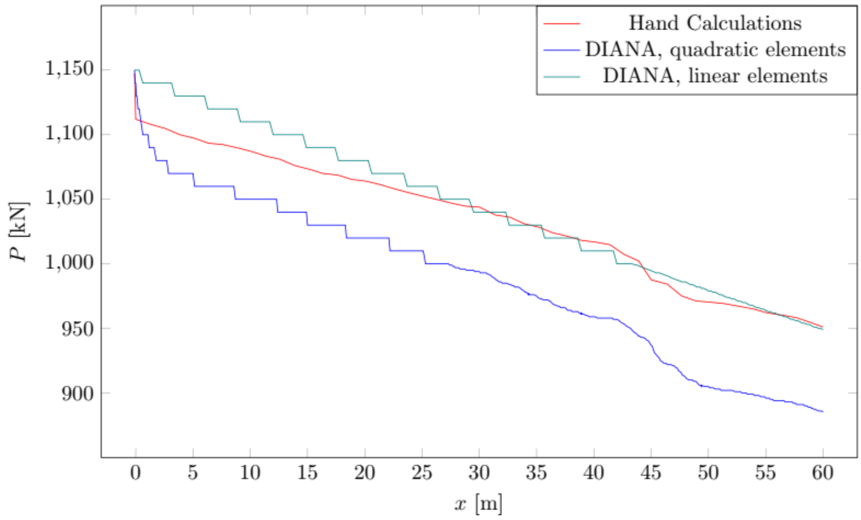

Figure 10: Validation of prestressing force variation with friction loss in DIANA.
(Source: [4])

The numerical model of the bridge was generated with great detail, incorporating passive reinforcement, detailed geometry, and exact tendon layout. The final developed model, along with some of its detailing, is depicted in Figure 11.

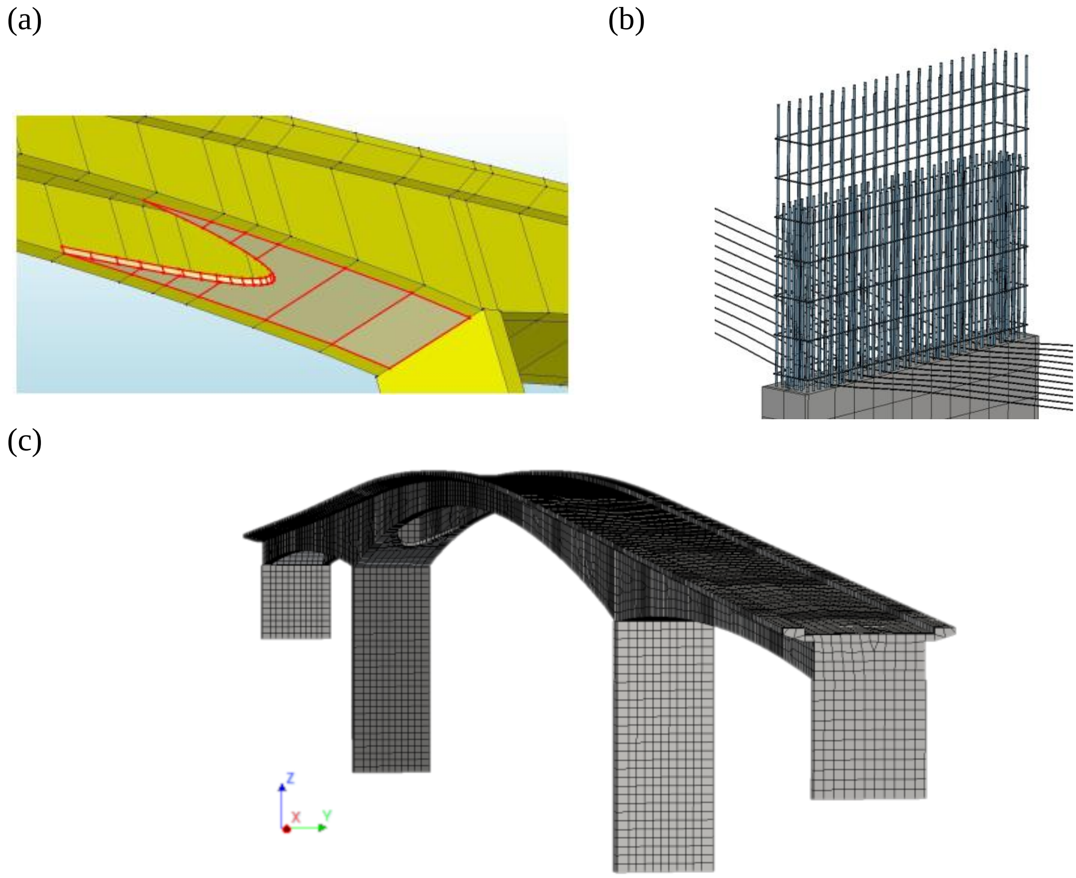

Figure 11: Numerical model of the Herøysund Bridge. (a) Detail of geometry near the columns. (b) Detail of the reinforcement of the columns. (c) Mesh of the full model
(Source: [4])

The traffic load model was included, consisting of a concentrated load and a distributed load, as specified in V412 [6]. The model was calibrated and validated for the measured 50-tonne event reported in [2], against horizontal and vertical displacement, and strain measurements.

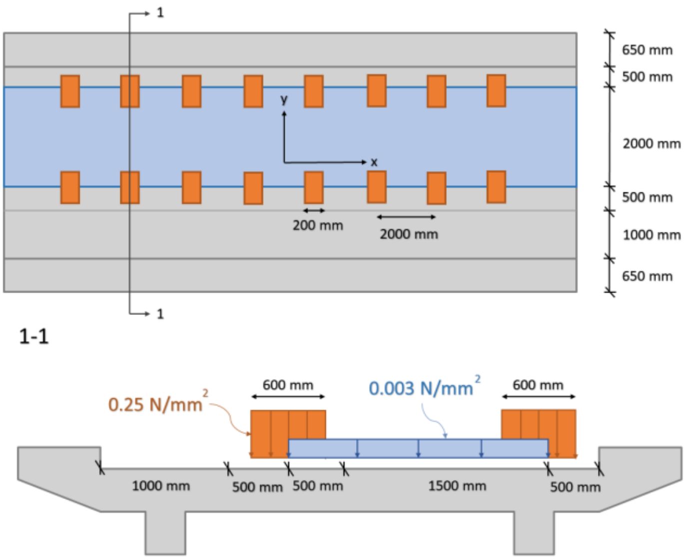

Figure 12: Vertical traffic load included in the analysis (Source: [4])

The active steel was included in the model via Python scripting, ensuring the correct and efficient inclusion of all tendons. This approach also enabled the modelling of broken tendons, with tendon breakage represented as a discontinuity in the active steel (Figure 13). The length of the discontinuity was defined in terms of the void length and the transmission length.

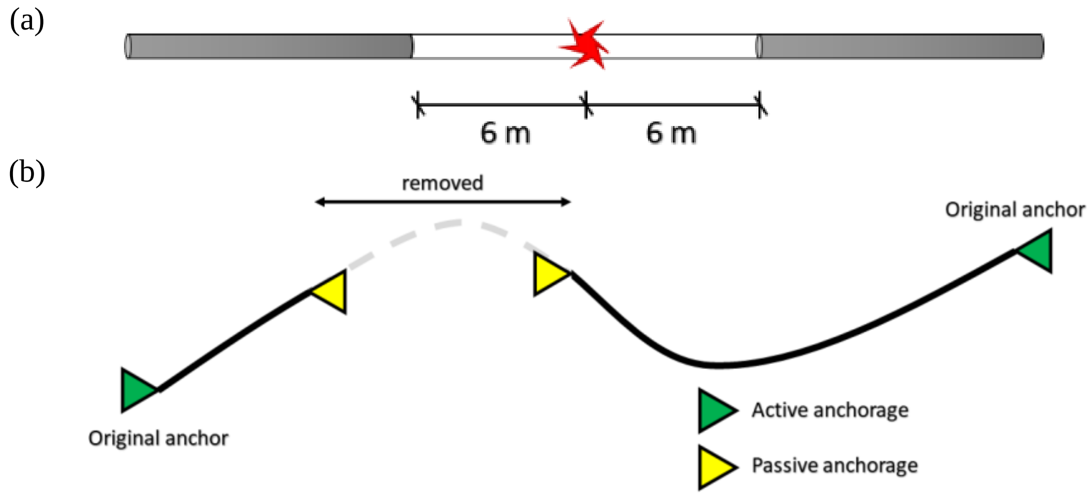

Figure 13: (a) Example representation of broken tendon. (b) Model of broken tendon included in the FEM via Python (Source: [4])

Moment capacity was calculated based on standard design assumptions and formulations. The cross-section of the bridge was studied as a T-section. The analysis of the moment capacity, compared against the design load obtained via the FEM simulation, resulted in a moment utilization at the mid-span section of approximately 30%. This is remarkably low, but can be explained by the historical context of when it was designed. The bridge was likely designed to achieve full prestressing, ensuring that the cross-section would be under full compression even when loaded by its ultimate load combination.

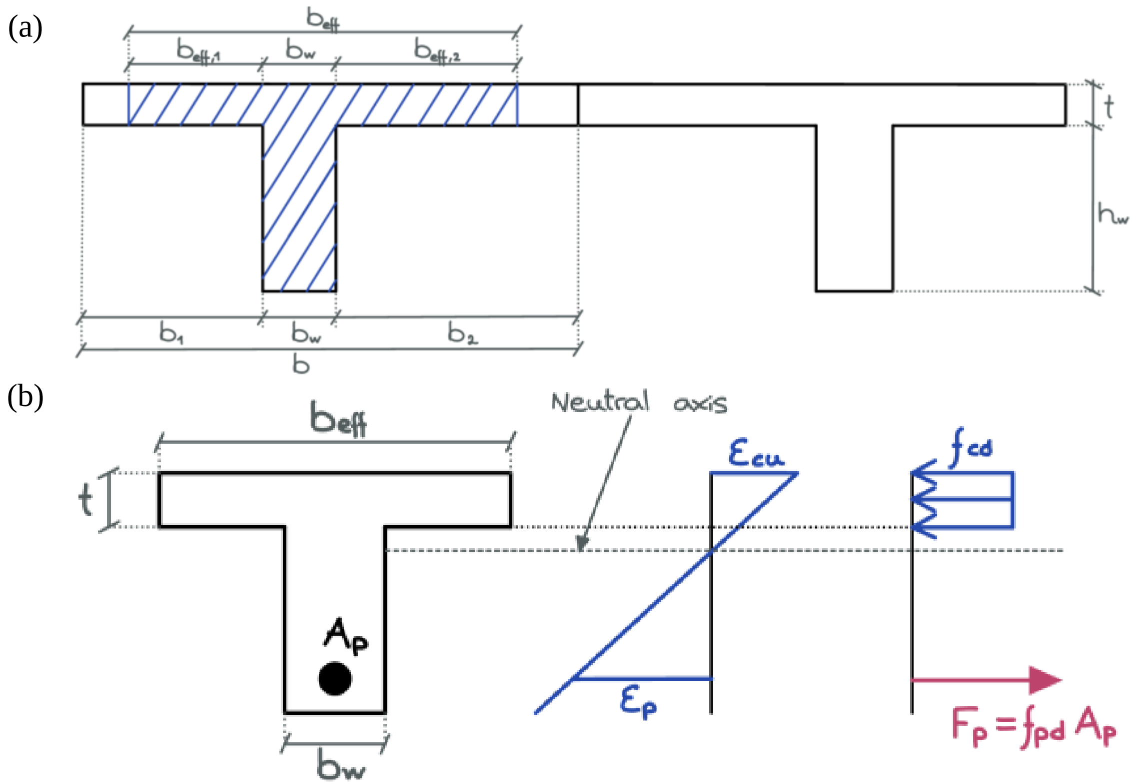

Figure 14: (a) Mid-span as a T-section. (b) Calculation model for moment capacity calculation (Source: [4])

Regarding the effect of tendon damage, an initial study compared the design moment to the capacity of the structure when whole tendons were removed. This assumed that tendon breakage effectively eliminated its contribution along the entire bridge. The study investigated the sequential removal of several tendons. The analysis results in DIANA demonstrated that four tendons could be removed while the bridge still maintained sufficient capacity at the mid-section. This finding aligns with the report by Aas Jackobsen [2], which states that all sections have sufficient capacity when 50% of the prestressing reinforcement area is incorporated in the capacity assessment.

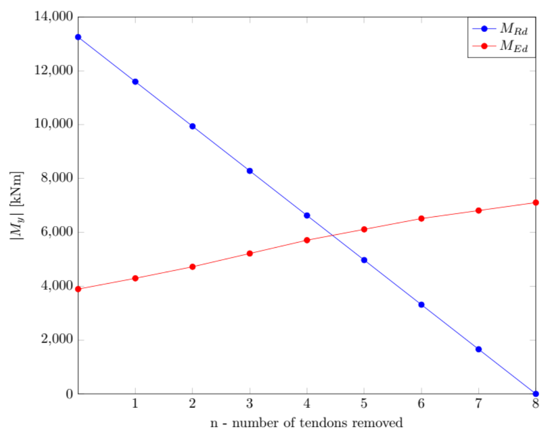

Figure 15: Design and capacity moments with varying number of tendons (Source: [4]).

The study now focuses on damage to a single tendon, investigating the effects of varying damage extents and the location of the damage. This investigation is prompted by the likely scenario of damage, as the ducts have voids at certain locations. Damage is expected to occur at these locations, and the eventual breakage will affect a region of the prestressing system, but not its full length. The exact location and extent of damage remain a point to be addressed in further studies. However, this work conducted a parametric study to investigate these two variables. The study below indicates the relative change in design moment (see Figure 16). It is important to note that capacity assessment is not relevant in this context, as the focus is solely on one tendon, and previous findings have already demonstrated sufficient capacity at the mid-section when one tendon is removed. The results do not show a clear or simple trend, highlighting that the effect of a damaged tendon cannot be inferred intuitively and requires precise calculation to quantify it. To try to interpret the results, the thesis explored the different effects of secondary moments in a statically indeterminate structure for various eccentricities and tendon configurations.

This study highlights the importance of accurately representing the location and extent of each damage. For example, if damage occurs at the start of the tendon, it may regain its prestress force before reaching the mid-section, leading to minimal impact on the design moment at that particular section.

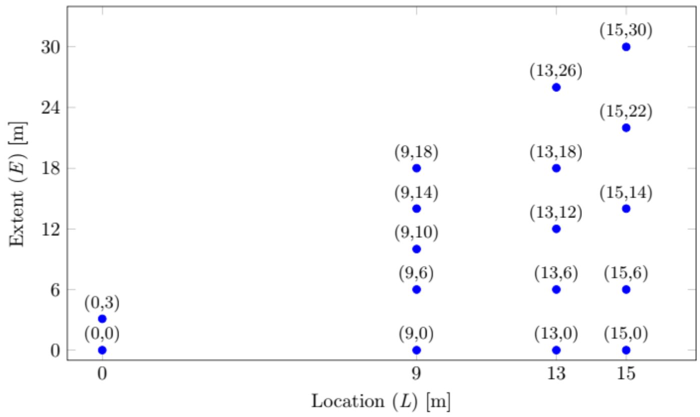

Figure 16: The studied damage location and extent scenarios. (Source: [4])

Therefore, this work concludes that, the capacity assessment of the mid-section of the undamaged bridge under ULS conditions showed a design moment of 3 892 kNm and an ultimate moment capacity of 13 254 kNm, resulting in a utilization ratio of 29.4%. Additionally, the analysis of the post-tensioning system with damage revealed that it is possible to remove four tendons passing through the mid-section without compromising its safety.

Therefore, it can be concluded that the bridge, in its undamaged state, can withstand service class Bk 10/50. Furthermore, the mid-section is found to be significantly over-dimensioned, as removing 50% of the tendons passing through it still maintains adequate capacity. It is also emphasized that developing a detailed model that incorporates damage is essential for accurately assessing the service life of an existing bridge. This study underscores the ease of including damage and the potential to streamline damage state assessments through systematic digitization of the modelling and assessment process, as demonstrated by the use of Python scripting in this project.

2.3. Exploration of programmatic procedures (WP3.A1.NTNU.Thesis3)

The work presented in this section corresponds to the master thesis [7], designated under the codename WP3.A1.NTNU.Thesis3. Table 3 provides a summary of the bibliographical details for this document, outlining essential information for reference.

Table 3: Bibliographical information for WP3.A1.NTNU.Thesis3

Title:	Modellering og kapasitetsvurdering av Herøysundbruen i DIANA FEA: Kapasitetsvurdering ved ulike skader i etterspennigssystem
Author(s):	Amjad Tayyem, Mahmoud Shaar
Date:	July 2024
Language:	Norwegian
Codename:	WP3.A1.NTNU.Thesis3
Link:	https://ntnuopen.ntnu.no/ntnu-xmlui/handle/11250/3153484

This thesis builds upon two previous master theses, continuing their work and following a similar methodology. Design values were obtained from a FEM model of the Herøysund Bridge developed in DIANA, while capacity assessments were performed using standard procedures.

In this case, the process was automated for the entire bridge, with the geometry and prestressing defined in table formats and the model generated programmatically. Additionally, this work investigated the moment capacity near the columns and the vertical displacement at the mid-span section, comparing the results to Eurocode requirements.

The creation of the bridge geometry involved initially defining several sections. All points within each section were imported from pre-prepared tables and connected into polygons (see Figure 17(a)). This process was programmatically repeated for multiple sections, which were subsequently joined together to form the main body of the numerical model (see Figure 17(b)).

(a)

(b)

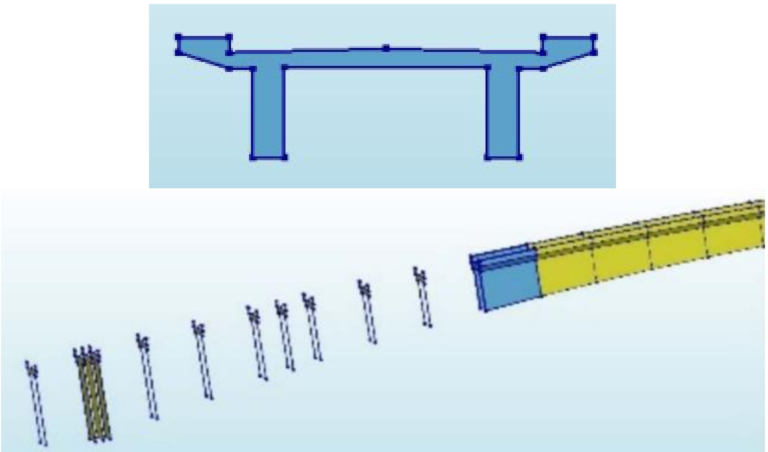

Figure 17: (a) Single cross-section example; (b) Multiple cross-sections to create complete model (Source: [7])

A similar programmatic procedure was implemented to define and model the passive and active steel. Due to the complex geometry of the bridge and layout, this process was not perfectly successful. The procedure could not be corrected during the limited time of work available to the students. It was decided to simplify the numerical model of the bridge and remove the changing elevation. Instead, the bridge was modelled with a road surfaced at constant height, this enabled the systematic definition of the steel layout. This simplification was deemed appropriate for the students work and as a proof-of-concept procedure for future bridge assessments.

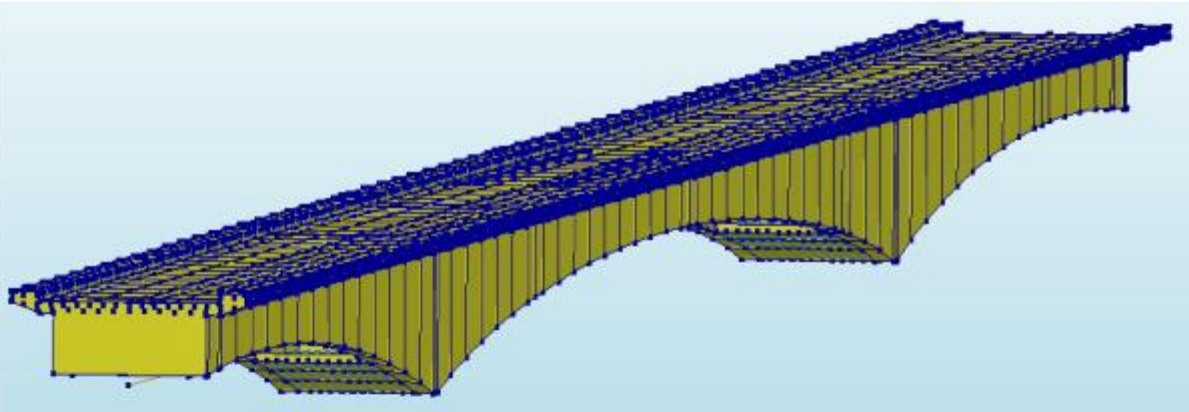

Figure 18: Simplified numerical model in DIANA (Source: [7])

As in previous examples, the moments were calculated using DIANA's composed line elements tool. The analysis incorporated various loads, including the self-weight of the structure, imposed loads from the pavement, traffic loading, and prestressing.

The bridge was modelled under four distinct damage scenarios, referred to as "Analyse" in the original document but renamed here as "Scenario":

- Scenario 1: Bridge without any damage.
- Scenario 2: 50% of the tendons are removed in the central span (4 broken tendons).
- Scenario 3: 50% of the tendons are removed along the entire bridge (4 broken tendons).
- Scenario 4: 50% reduction in the prestressing force with intact tendons.

The analysis of the results indicates that the undamaged bridge, represented by Scenario 1, demonstrates very low utilization of moments at mid-span, reflecting substantial reserve capacity. In Scenarios 2 and 3, which involve damage to the prestressing system, the results suggest that utilization levels remain well below 100%, indicating that the bridge retains sufficient capacity even with four broken tendons. However, these findings may lack precision due to simplifications in the model geometry and potential inaccuracies in the modelling process. For Scenario 4, where the prestressing force is reduced by 50%, the mid-span deflection increases significantly, exceeding the permissible limits set by the Eurocode, highlighting a critical impact on the bridge's serviceability.

In conclusion, this study explored the potential of generating the entire bridge model programmatically. While the approach was only partially successful due to unresolved issues within the limited timeframe of a master's thesis, it highlights the feasibility and promise of systematically creating complex models for structural analysis. Once developed, such models can be effectively utilised to evaluate the structural safety of various sections or verify compliance with code requirements, including assessments of vertical deformation.

2.4. Assessment of robustness (WP3.A1.NTNU.Thesis4)

The work presented in this section corresponds to the master thesis [8], designated under the codename WP3.A1.NTNU.Thesis4. Table 4 provides a summary of the bibliographical details for this document, outlining essential information for reference.

Table 4: Bibliographical information for WP3.A1.NTNU.Thesis4

Title:	Robusthet av brukonstruksjoner - Kasusstudie av Herøysund bru
Author(s):	Lars Gøran Farstad
Date:	October 2024
Language:	Norwegian
Codename:	WP3.A1.NTNU.Thesis4
Link:	https://ntnuopen.ntnu.no/ntnu-xmlui/handle/11250/3168382

This work investigates the concept of robustness and its application to structures with post-tension systems, using the Herøysund Bridge as a case study. The primary aim of the thesis is to explore how damage and loss of prestress force in bridges can be evaluated through calculations, employing both linear and non-linear analysis methods. Robustness is quantified using established deterministic formulas identified through a comprehensive literature review.

In the first part of the work, the document presents a review of various definitions of robustness. Robustness can be defined as the ability of structural systems to prevent collapse in the face of unforeseen or abnormal events. It refers to structures that have sufficient reserve capacity to function during or after such incidents. A robust system is designed to maintain its functionality throughout its intended lifespan, even when subjected to damage or extraordinary events.

Robustness encompasses multiple critical aspects of a structure's performance, including strength, deformation, ductility, stability, stiffness, loads, and weight. The literature offers various definitions of robustness, with the one presented above being just one of many possible interpretations. These definitions can be further subdivided into local or global forms.

Definitions related to robustness, such as redundancy, ductility, and reliability, share similar characteristics. The literature presents robustness in terms of different calculation paradigms, which can be classified into deterministic, probabilistic, and risk-based approaches. Figure 19 provides an overview of references that define robustness based on these different calculation paradigms.

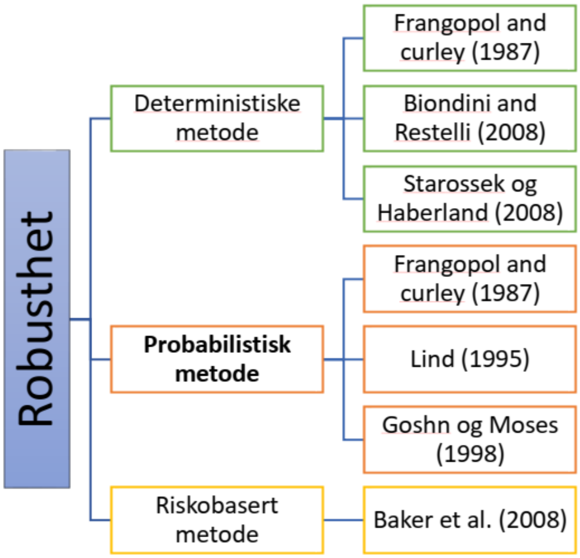

Figure 19: Overview robustness paradigms and relevant literature (Source: [8])

The deterministic method, which is used in this work, can be defined in terms of the ratio of structural stiffness between a damaged and an intact structure, the ratio between deformations, or the ratio between forces or stresses. All of these deterministic definitions are evaluated through a simplified example, as well as the case study involving the Herøysund Bridge.

The thesis identifies the most suitable definition of robustness from FIB's report 110 (Management of Post-Tensioned Bridges [9]). This definition is expressed through a generic type indicator, as shown in Eq. (1). The formula provides a value greater than 1 when the capacity exceeds the load effect, enabling the assessment of robustness reduction. To ensure structural safety, Ultimate Limit State (ULS) values are used in the calculations. The reference report [9] defines a bridge as robust if the value is $I > 1.25$. This value can be assessed in each individual case and is considered a safety factor for robustness.

$$I = \frac{Resistance}{Load\ effects} \qquad \text{Eq. (1)}$$

Next, robustness is quantified through a simple numerical example of a reinforced rectangular beam, which is designed to withstand its own self-weight. The passive reinforcement is gradually reduced to evaluate its impact on robustness, as shown in Figure 20(a). Non-linear analyses of the beam are performed using SOFiSTiK and Response-2000 software. Applying the various deterministic definitions of robustness demonstrates that each definition focuses on a specific aspect of the problem. No single definition provides a complete and overarching quantification of robustness.

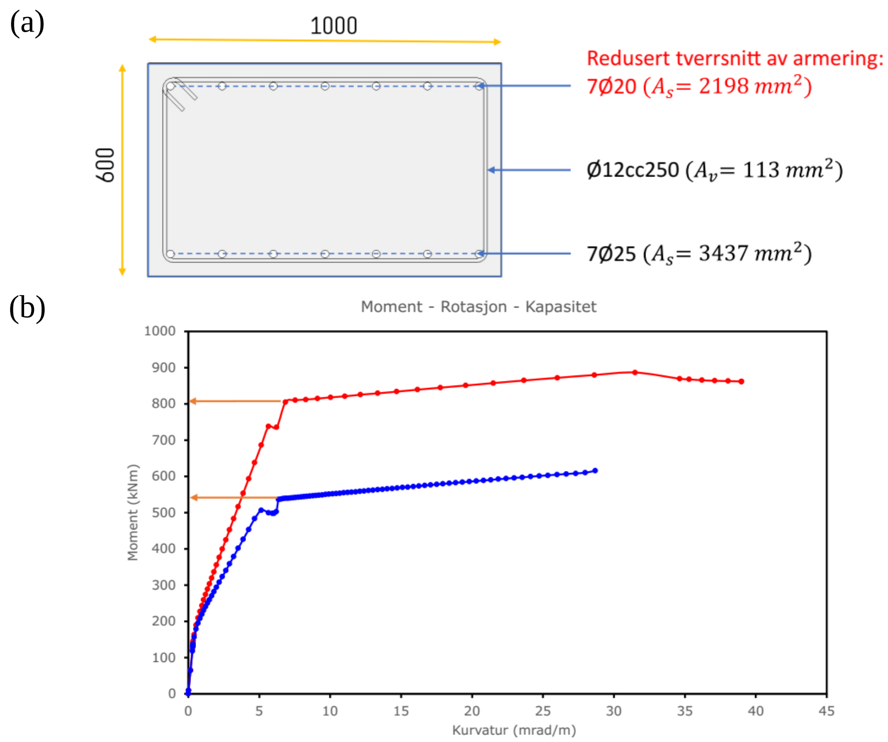

Figure 20: (a) Cross-section of beam and reduced reinforcement; (b) Moment to curvature relationships (Source: [8])

For post-tensioned bridges, the definition in Eq. (1) is considered the most appropriate, as it focuses on the effect of load magnitude. This is a crucial aspect of structural safety in bridges, often regulated by traffic and weight restrictions, as was the case of the Herøysund Bridge. The work then concentrates on the robustness of four specific cross-sections in the bridge, as shown in Figure 21.

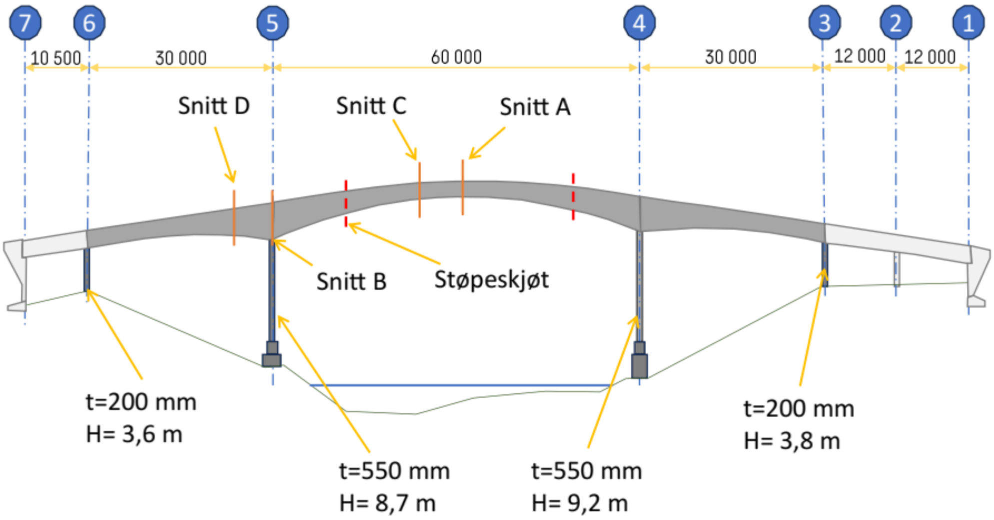

Figure 21: Overview of analysed cross-sections (Source: [8])

The bridge is modelled using SOFiSTiK and Response-2000 as a finite element representation with quadrilateral elements, as shown in Figure 22. Material non-linearities are incorporated into the simulation, and various loads, including self-weight, prestress, and traffic, as well as long-term effects such as creep, shrinkage, and relaxation, are considered in the appropriate load combinations according to the Eurocode [10].

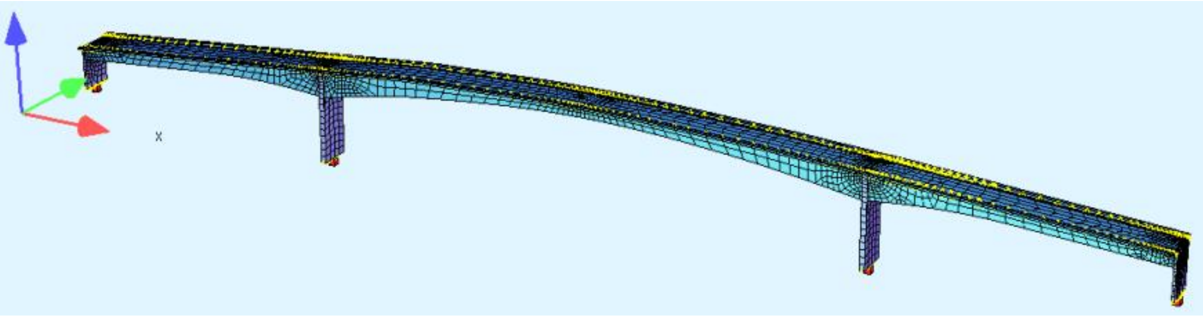

Figure 22: FEM of the Herøysund Bridge in SOFiSTiK. (Source: [8])

Equipped with these tools, the work analyses the effect of the successive removal of prestressing tendons and its impact on the capacity of different sections. For sections A and C, located near the middle of the central span, a rapid deterioration is observed, as shown in Figure 23. These results indicate that the moment capacity of the bridge remains sufficient even if two tendons are removed.

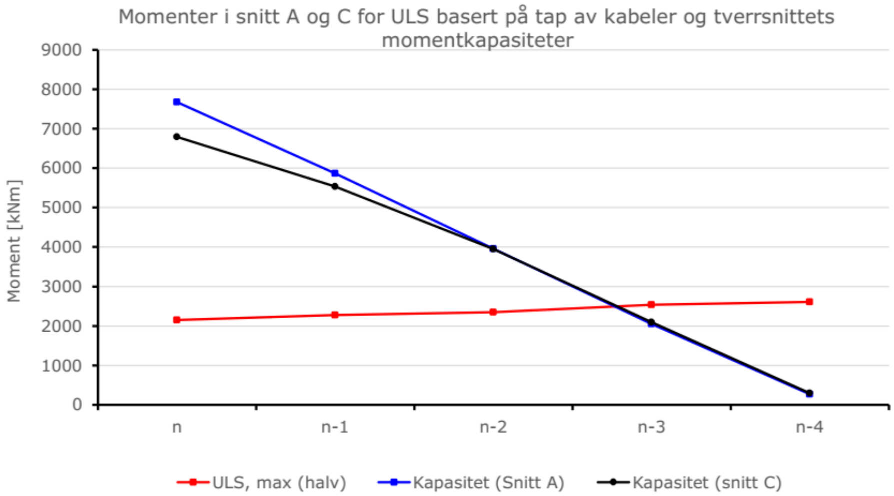

Figure 23: Design moments and capacities for sections A and C, with decreasing number of tendons. (Source: [8])

These analyses were quantified in terms of robustness, with the results presented in Table 5.

Table 5: Summary of results and corresponding robustness for sections A and C (Source: [8])

Snitt	**Motstand/kapasitet [kNm], snitt A**	**Motstand/kapasitet [kNm], snitt C**	**Lasteffekt [kNm]***	**Robusthet (I), snitt A**	**Robusthet (I), snitt C**
Intakt	7676	6791	2153	3.6	3.2
n-1	5867	5533	2272	2.6	2.4
n-2	3962	3949	2351	1.7	1.7
n-3	2053	2098	2527	0.8	0.8
n-4	270	300	2680	0.2	0.1

*basert på fordeling av de globale momentene til to bjelker (halv verdi)

A similar analysis was repeated for the sections near the columns (Sections B and D), shown in Figure 24. These sections are characterized by larger design moments, but also possess greater moment capacities due to the larger size of the cross-section and the increased number of tendons. These sections have two systems of prestressing cables, one in the beam and another in the deck. The analysis indicates that the sensitivity to tendon loss is significantly smaller in these areas. Both sections can withstand up to six tendon breakages without compromising their structural safety. A summary of the results and corresponding robustness values are given in Table 6.

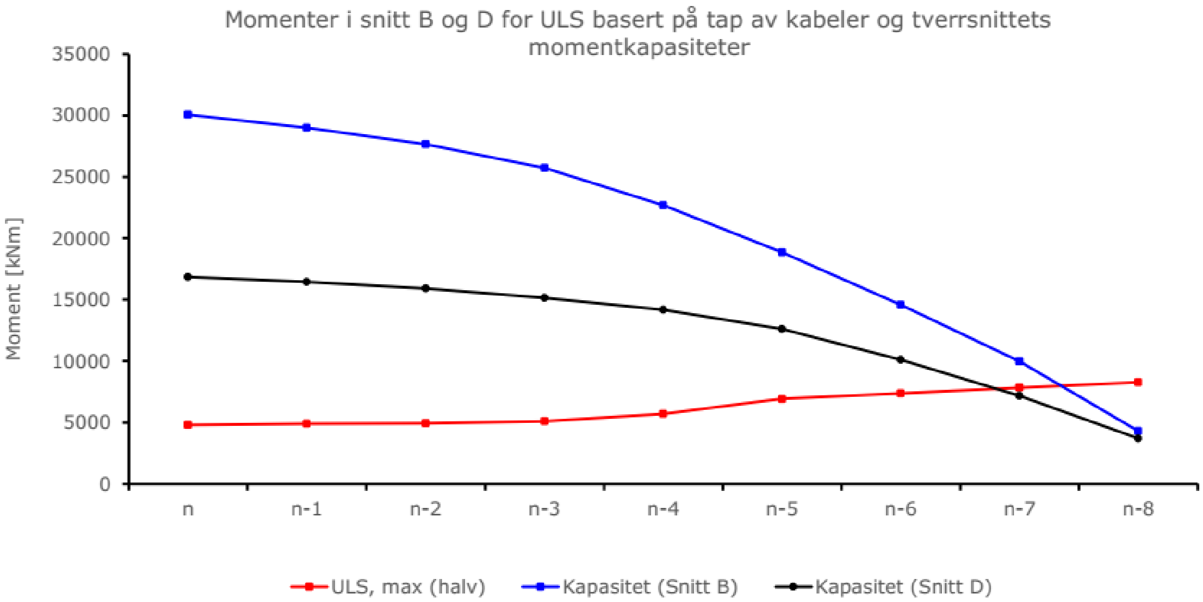

Figure 24: Design moments and capacities for Sections B and D, with decreasing number of tendons. (Source: [8])

Table 6: Summary of results and corresponding robustness for Sections B and D (Source: [8])

Snitt	**Motstand/kapasitet [kNm], snitt B**	**Motstand/kapasitet [kNm], snitt D**	**Lasteffekt [kNm]***	**Robusthet (I), snitt B**	**Robusthet (I), snitt D**
Intakt	30060	16853	4039	7.4	3.5
n-1	28995	16459	4432	6.5	3.3
n-2	27670	15929	5284	5.2	3.2
n-3	25709	15149	5714	4.5	3.0
n-4	22694	14176	6104	3.7	2.5
n-5	18836	12606	6173	3.1	1.8
n-6	14567	10116	6626	2.2	1.4
n-7	9966	7187	7099	1.4	0.9
n-8	4311	3702	7585	0.6	0.4

*basert på fordeling av de globale momentene til to bjelker (halv verdi)

Furthermore, the study seeks to extend the analysis to a probabilistic definition of robustness. To achieve this, the probability of tendon failure is assumed to follow a normal distribution with a mean value of 1 and a standard deviation of 0.33. By integrating the results from the numerical model into a Monte Carlo simulation based on this probability distribution, the work produces a distribution of robustness values, as shown in Figure 25. However, this result relies on an arbitrary definition of the probability of tendon failure and assumes that tendon failure affects the entire length of the bridge. Therefore, these findings should be considered a proof-of-concept. Nonetheless, the results demonstrate the feasibility of conducting a probabilistic analysis to refine the safety assessment of existing bridges.

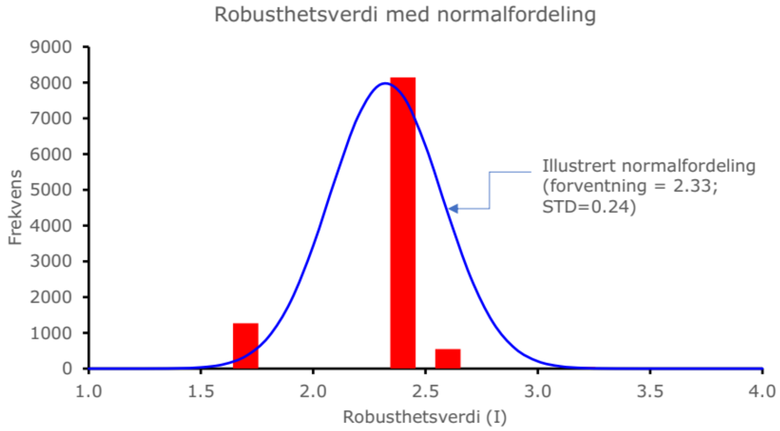

Figure 25: Probability distribution of the robustness (Source: [8])

This work concludes that the inclusion of material non-linear behaviour complicates the modelling process and increases the calculation time. However, its effect on the internal forces was found to be minimal. Therefore, it is suggested that considering linear analysis is appropriate for this study. Based on this, the subsequent analysis revealed that the Herøysund Bridge has a robustness value of 3.6 for an intact system and can withstand the loss of up to 2 tendons at mid-span. Near the columns, the robustness value is 7.4, and the bridge can accommodate the loss of up to 7 tendons.

3. Discussion

The students' works highlight the significant potential of modelling automation. Each study explored various methods to programmatically develop numerical models of the structure, steel layout, damage scenarios, and loads, integrating these elements into appropriate combinations to verify compliance with code requirements. These applications demonstrate that it is both feasible and practical to approach the re-analysis of existing structures through such systematic and programmatic methods.

This approach also offers the opportunity to evaluate the effects of damages systematically. In this work, the focus has been on damages within the post-tensioning system. Reported damages from on-site inspections can be seamlessly incorporated into the analysis framework. Furthermore, it is possible to explore a wide range of potential damage scenarios to assess their impact on the structure's integrity. These scenarios can be integrated into probabilistic analyses, allowing safety assessments to be expressed in terms of reliability.

Therefore, the sections below propose definitions and procedures to establish a plausible assessment framework. First, the systematic presentation of the effects of damage is addressed. Next, the potential for representing that damage as an external load is explored. Finally, the procedure for integrating these elements into the analysis framework is outlined.

3.1. Damage modelling

The type of damage investigated in this context is tendon breakage. Such breakage is most likely to occur in regions of the duct where grout is absent (void) or of poor quality. Consequently, the extent of the damage corresponds to the length of the prestressing steel that is no longer under prestress. This damaged length, (L_{damage}), is defined as the sum of the void length (L_{void}) or the extent of poor grouting, plus twice the transmission length (L_T), which is the length required for the tendon to regain its prestressing force.

To begin, the effect of tendon breakage must be clearly defined. As discussed in the report for WP1 (see [11]), this effect on the structure is influenced by several factors, including the length of voids within the duct, the condition of the grout, the geometry of the prestressing system, and the cross-sectional properties of the structure. A generic post-tensioned element is defined for this purpose, as illustrated in Figure 26(a). At the location of the tendon breakage, as shown in Figure 26(b), the area of active steel effectively becomes zero ($A_P = 0$). The primary consequence of this breakage is the loss of prestressing force along the affected length. The extent of this force loss is influenced by the void length, the condition of the grout and the transmission length.

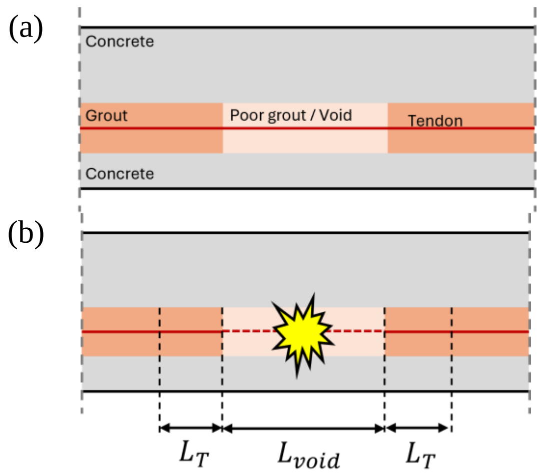

Figure 26: (a) Schematic representation of grouted post-tensioned element with poor grout/void; (b) Damage variables definition

The distribution of prestressing force along the structure can be schematically represented as illustrated in Figure 27(a). For practical purposes, this representation can be simplified to the model depicted in Figure 27(b), offering a more straightforward approach for analysing and calculating the structural impact of tendon breakage.

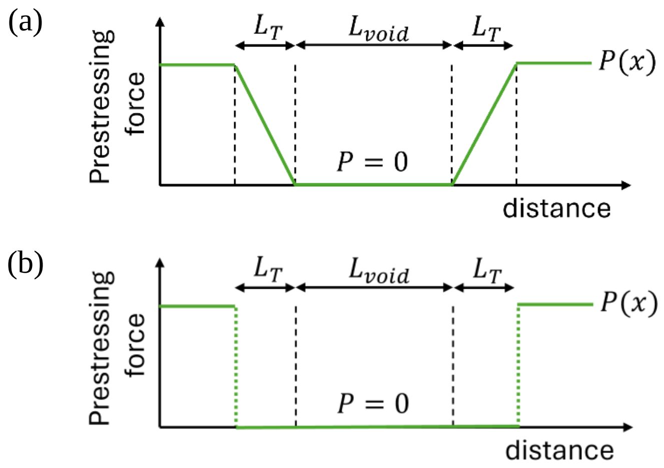

Figure 27: (a) Schematic representation of prestressing force magnitude around the tendon breakage; (b) Simplification of the prestressing force magnitude.

To model the damage caused by tendon breakage, it is useful to isolate and study the effects produced by the post-tensioning (PT) system separately. The first effect is the bending moment generated by the prestressing force. In each cross-section, the location of the prestressing force relative to the section's centroid defines the local eccentricity. When the prestressing force is applied with some eccentricity, it induces a bending moment. In the case of a straight tendon configuration with constant eccentricity, the bending moment caused by the prestressing force remains constant, as shown in Figure 28. If the tendon is broken and the prestressing force is absent along certain parts of the structure, the corresponding bending moment is also lost. This effect can be modelled in an existing structure by introducing an equivalent bending moment of opposite sign, as indicated in Figure 28.

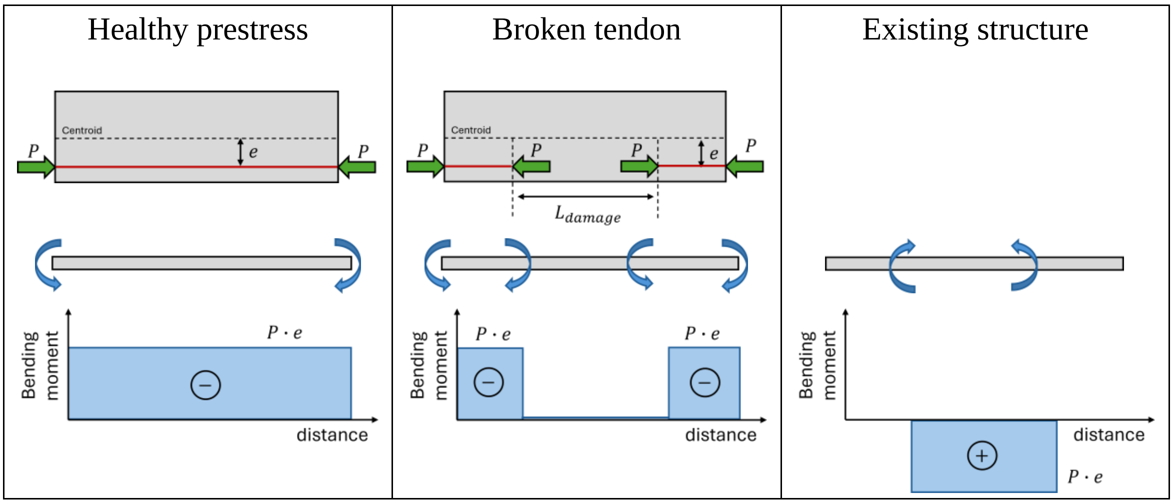

Figure 28: Representation of eccentric prestressing force in a damaged system as an external moment

The second effect of damage in the post-tensioning (PT) system is the alteration of the vertical loading. A prestressing system with a generic longitudinal geometry $y(x)$ can be represented as an equivalent vertical load ($q(x)$). This relationship is expressed as the prestressing force multiplied by the second spatial derivative of the tendon geometry, as shown in Eq. (2). For instance, Figure 29 illustrates the vertical equivalent load generated by a prestressing system with a parabolic geometry. When the system is damaged, creating a gap in the prestressing force, the corresponding equivalent vertical load also exhibits a gap. This damage can be modelled as an additional vertical force of opposite sign, as indicated in Figure 29.

$$q(x) = P(x) \cdot \frac{d^2y(x)}{dx^2} \qquad \text{Eq. (2)}$$

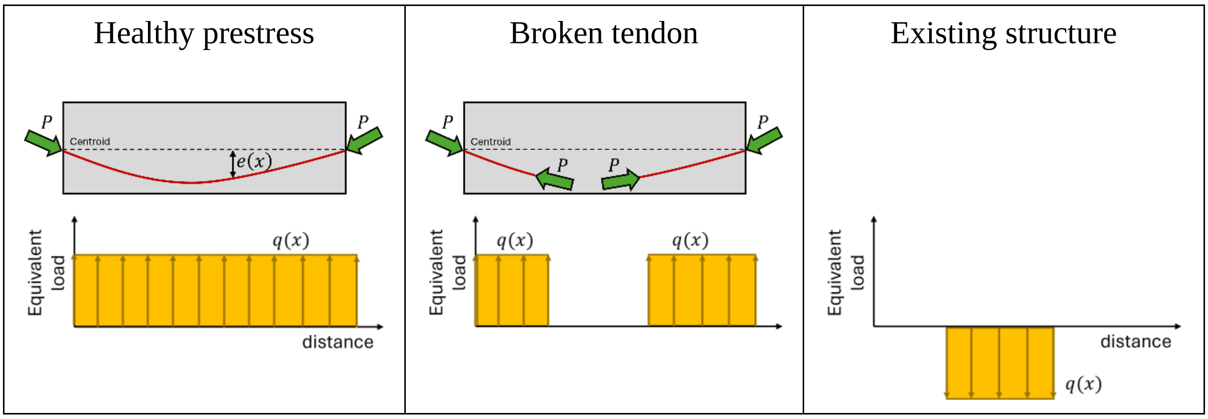

Figure 29: Representation of the equivalent load in a damaged prestressing system

Another consequence of tendon breakage is a slight reduction in the cross-sectional bending stiffness along the affected regions. This reduction is minimal because the tendon area is significantly smaller compared to the concrete area. Figure 30 schematically illustrates this concept, comparing the bending stiffness of a healthy beam (EI_H) with that of a damaged beam (EI_D). The impact of a broken tendon can be modeled as a variation in bending stiffness (ΔEI) over the damaged area.

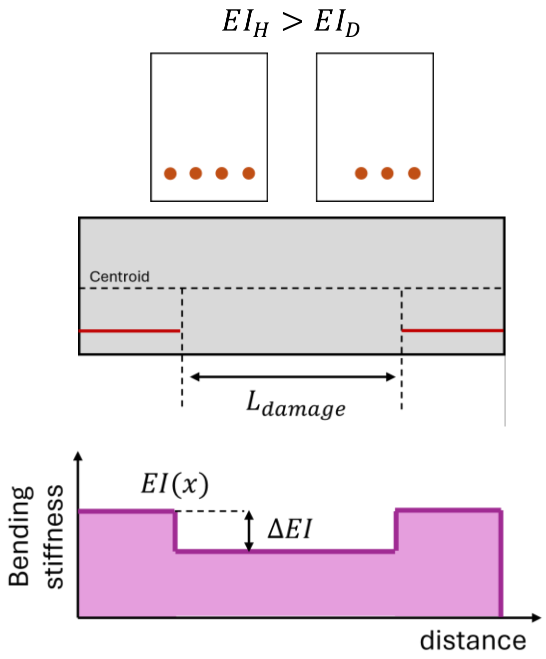

Figure 30: Schematic representation of bending stiffness variation due to a broken tendon

The reality of stiffness variation due to tendon breakage is much more complex than it appears. Tendons are likely to break in areas where little or no grout exists, meaning the active steel was never bonded at those locations. It can be argued that the bending stiffness in a section with prestressing steel depends on the quality of the bond between steel and concrete. Fully bonded steel contributes to the cross-sectional second moment of area. However, unbonded steel does not contribute directly. Instead, the spatial longitudinal relationship will determine the effective bending stiffness. When a structure is constructed with a PT with a void at a certain location in the duct, locally the bending stiffness cannot be simply calculated. It is believed that the distribution will be something like the one shown in Figure 31. First, the existence of a void, then the breakage of a tendon will affect the original distribution of EI. The change in bending stiffness (ΔEI) might be only effective in the transmission length areas, where the prestressing force builds up with the strength of the bond. Nevertheless, this is only speculative. More precise investigations are required to quantify the effective bending stiffness of PT elements with voids and its change with eventual damages.

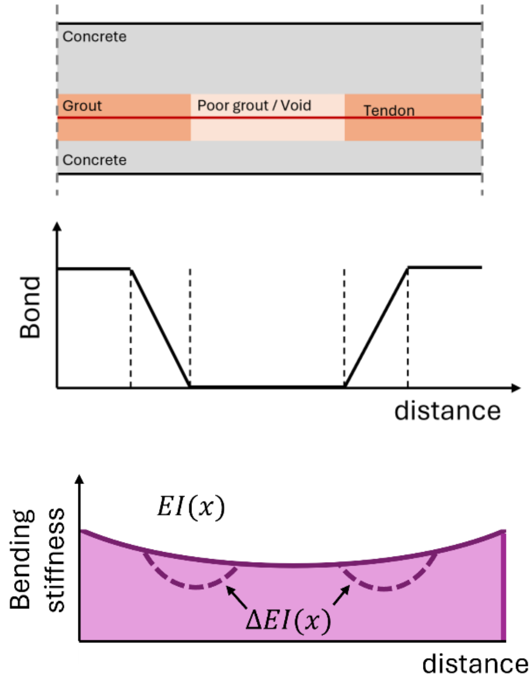

Figure 31: Schematic representation of possible bending stiffness variation due to a broken tendon in reality

3.2. Damage as a load

Based on the previous considerations of how damage affects a post-tensioned (PT) structure, it can be argued that damage can be represented as an additional load composed of two components: a distributed vertical load and additional moments. This approach allows for the treatment of damage as a load case characterized by the damage location (x) and damage length (L), as illustrated in Figure 32. Incorporating this load into existing structural analysis tools is relatively straightforward.

Furthermore, since structural analysis is generally performed using linear analysis, the principle of superposition can be utilized to easily analyse damages of any length. By defining a minimum unit length of damage (L_u) and analyzing all possible load combinations of (x, L_u), it becomes possible to study specific damage scenarios by combining the results through superposition. This approach offers a straightforward method to represent the effect of tendon breakage on the vertical loading of the structure.

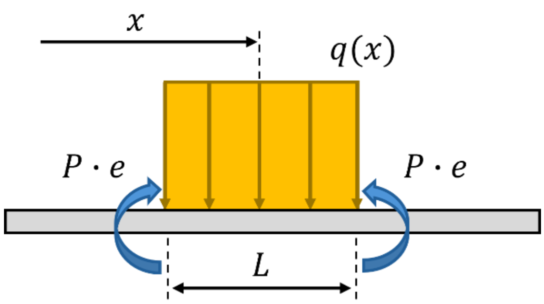

Figure 32: Modelling damage as a load

3.3. Procedure including damage

Therefore, based on all previous considerations, this document suggests an analysis framework for assessment of existing bridges, which includes the effects of possible damaged post-tensioned systems. The framework can conceptually be outlined as:

- Programmatic model generation. Create the model of the bridge with the help of a scripted methodology. Most current structural analysis programs allow some form of automation. The particularities on how to do it and what programming language to use vary depending on the software. Nevertheless, it should be possible to define programmatically the geometry and properties of the prestressing system.
- Damage as a load case. Define the minimum length of a damaged PT system (e.g.: 1 m). Divide each tendon into segments of that length and create a load case to represent a damaged tendon for each segment. Each load case consists of a distributed force and applied moments, where their magnitudes depend on the position and geometry of the PT segment. Define as many load cases as required to cover all tendons in the structure.
- Load combinations. Perform linear analysis for all the load cases. Combine the results from different load cases to represent the desired damage scenario. This can easily be achieved as the load combination of different load cases, a standard functionality in any structural analysis program.
- Include inspection results. If available, include in the assessment procedure any relevant information about the condition of the prestressing system.
- Update traffic load models. If available, use-site specific information about the traffic weights obtained form weight-in-motion station, to refine the traffic load model to be used in the assessment procedure.
- Probabilistic analysis. Define probability distributions of relevant problem variables, such us traffic loads, measured material properties or PT damage severity and location. It is desirable to define these distributions based on site-specific information and results from inspections. By factoring in these distributions in the analysis, it is possible to calculate the structural safety in terms or reliability.
- Refined analysis for critical cases. Based on previous analysis, identify what configuration of design load and PT damage is more critical. Update the cross-sectional stiffness according to the modelled damage. If further refinement is required, perform non-linear analysis to include material non-linearities.

4. Conclusion

This work package has investigated the capacity assessment procedures considering damaged post-tensioning systems, for structures in general and for the Herøysund Bridge in particular. This document reports the activities and summarizes the results from several student works in form of master thesis, complemented with some additional discussions on how damaged PT systems can be seamlessly be integrated into current assessment procedures.

The students' works consistently demonstrated that the capacity of the undamaged bridge exceeds initial expectations. This finding aligns with the re-analysis conducted by Aas Jakobsen [2]. Compared to the original design, which exhibited near full utilization, the higher capacity observed in these analyses may be attributed to several factors. First, the refined modelling approach employed in these studies likely yields more accurate results. Second, the use of updated concrete properties, particularly the revised concrete strength, contributes to an approximate 30% increase in moment capacity. Additionally, the bridge's design philosophy likely played a role, since it was probably designed to achieve full prestressing, ensuring the cross-section remained under full compression, even under its ultimate load combination.

Most of the studies explored scenarios where damage is represented by the loss of tendons and prestress along the entire bridge. They demonstrate that the bridge remains structurally safe even when some tendons fail. The number of tendons that can safely be removed depends on the cross-section under consideration. Nevertheless, the studies consistently show that the structure remains safe even if two tendons break, indicating that the moment capacity of the bridge is sufficient even with the removal of two tendons.

However, it was realised that this representation of damage is an overly conservative simplification. In reality, a tendon failure will affect only a region determined by the void size and transmission lengths. The results do not show a clear or simple trend, emphasising that the effect of a damaged tendon cannot be inferred intuitively and requires precise calculation to quantify. Refining capacity assessments with this improved representation of damage can arguably provide even greater structural safety margins. Therefore, developing a detailed model that incorporates damage is essential for accurately assessing the service life of an existing bridge.

Towards that end, all the students' works adopted various levels of automatization for model development. The works explored the potential of scripted procedures to: generate the entire bridge model, include the passive reinforcement, define the geometry and load of the prestressing, as well as to specify various load and damage scenarios. This approach enabled the modelling of broken tendons, with tendon breakage represented as a discontinuity in the active steel. The results highlight the feasibility of systematically creating complex models for structural analysis. Once developed, such models can be effectively utilised to evaluate the structural safety of various sections or verify compliance with code requirements. These applications demonstrate that it is both feasible and practical to approach the re-analysis of existing structures through such systematic and programmatic methods.

This document demonstrates the potential of utilizing such scripting facilities to extend the capacity assessment to a probabilistic framework, allowing for the same analysis to be performed across a range of different load assumption, material properties, damage scenarios and other model considerations. The results reported here constitute only an initial proof-of-concept. Nonetheless, the results demonstrate the feasibility of conducting a probabilistic analysis to refine the safety assessment of existing bridges.

Furthermore, this document outlines a possible assessment procedure, based on these conclusions, that include the consideration of damage PT systems. The proposed framework for assessing bridge integrity involves several key steps: using programmatic model generation to automate the creation of bridge models, defining damage as a load case by segmenting tendons and creating corresponding load cases, performing linear analysis and combining results to simulate damage scenarios, incorporating inspection results into the assessment, refining traffic load models using site-specific data, and conducting probabilistic analysis with relevant variable distributions, to ensure accurate safety evaluations and reliability assessments.

Future work should focus on improving the quality and quantity of site-specific information about the traffic loads, structural properties and condition of the PT systems. Arguably, the greatest improvement in the assessment procedure can be achieved by revising the traffic load models to reflect the actual traffic conditions on the bridge. On the other hand, the precision of structural assessments can be enhanced by obtaining a detailed map of actual tendon locations and potential voids using non-destructive techniques. Additionally, a better understanding of the impact of voids, poor grout, and tendon breakage is essential. Research should aim to clarify the extent to which these factors affect the overall integrity and safety of a structure.

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