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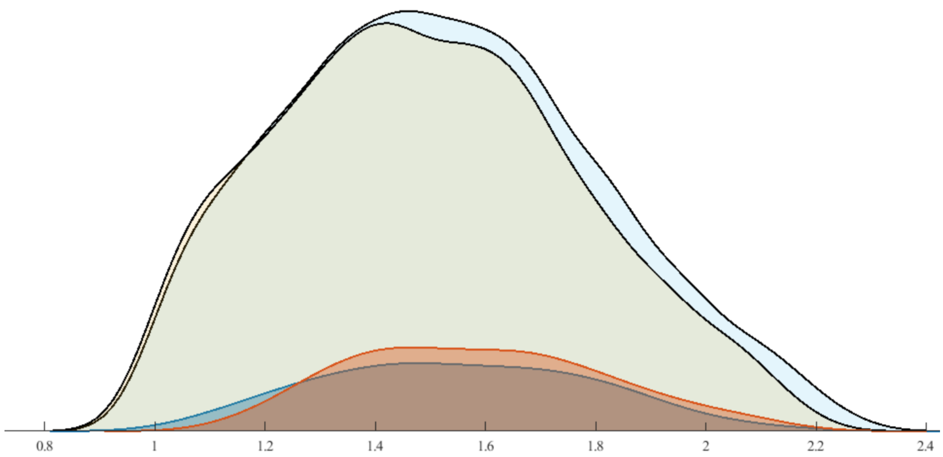


# Statistical comparison of second generation of Eurocode and Bro 2004 using Set-Based Design

## Preliminary design and comparison of design attributes using set-filtration of a concrete frame bridge

Master's Thesis at the Structural Engineering and Building Technology Programme

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Gothenburg, Sweden 2026
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# Abstract

The European commission has announced the complete enforcement of the revised Eurocodes by 2026, raising concerns regarding their influence on design complexity, material requirements, including the environmental impact. This transition toward the updated regulations creates a need to reassess existing design methodologies and evaluate the effects on structural engineering practice. This evolution, at the same time, challenges traditional Point-based design methodology where a single design is chosen early, increasing risk of costly redesigns when requirements change in later design phase. To address this limitation, this research applies a computational framework of Set-based design as an alternative design methodology in combination with the Finite Element Analysis. Unlike Point-based design, Set-Based design maintains a broad, parameterized space of feasible design solutions, improving adaptability in changing requirements while supporting optimization with respect to material use and environmental impact. The research performed a comparative analysis evaluating material efficiency and $CO_2$-equivalent emission regarding shifting regulatory frameworks.The two regulations compared was the second generation of Eurocodes and the former Swedish bridge design standard corresponding to using the general technical specification Bro 2004 for reinforced concrete frame bridges. The results indicated that from a broad perspective, no discernible difference was seen in material cost and $CO_2$-equivalent emissions between the two standards, both in terms of feasible and not feasible designs. However, the older Bro 2004 framework permitted higher optimization potential when studying the varying feasible designs.


Keywords: Set-Based Design, Eurocodes, Bro 2004, Concrete frame bridge, Preliminary design, Finite Element Analysis, Environmental impact.

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Wattana Khamporn, David Mårtensson, Gothenburg, 2026

# List of Acronyms

*Below is the list of acronyms that have been used throughout this thesis listed in alphabetical order.*

| | |
|---|---|
| BBK | Boverket Handbook for Concrete Structures |
| BKR | Swedish National Structural Design Regulations |
| BV2000 | Swedish Rail Administration Technical System 2000 |
| BV BRO | Swedish Rail Administration Bridge Regulations |
| CEN | European Committee for Standardization |
| $CO_2$-eq | Carbon-dioxide Equivalents |
| EC | Eurocodes |
| EKS | European Construction Standards |
| EN | European Standards |
| FE | Finite Element |
| FEA | Finite Element Analysis |
| GTS | General Technical Specification |
| KDE | Kernel Density Estimation |
| LM71 | Train Load Model 71 |
| NA | National Annex |
| NSB | National Standardization Bodies |
| NDP | National Determined Parameters |
| PBD | Point-Based Design |
| SBD | Set-Based Design |
| SBPD | Set-Based Parametric Design |
| SLS | Service Limit State |
| STA | Swedish Transport Administration |
| TDOK | Trafikverket Technical Document |
| TK Bro | Technical Requirements for Bridges |
| TR | Technical Reports |
| TRVINFRA | Swedish Transport Administration Infrastructure Regulations |
| TS | Technical Specification |
| TSFS | Regulations issued by the Swedish Transport Agency |
| ULS | Ultimate Limit State |

# Nomenclature

*Below is the nomenclature of parameters, variables and other letters that have been used throughout this thesis.*

## Greek Letters

| | |
|---|---|
| $\alpha$ | Classification factor for the train load model |
| $\varepsilon_{cs}$ | Shrinkage strain |
| $\gamma_{G,i}$ | Partial factor for permanent action $i$ |
| $\gamma_{inf,j}$ | Partial factor for variable action $j$ (lower/inferior value) |
| $\gamma_m$ | Partial factor for carrying capacity / material property |
| $\gamma_n$ | Partial factor for safety class |
| $\gamma_P$ | Partial factor for prestressing forces |
| $\gamma_{Q,1}$ | Partial factor for the leading variable action 1 |
| $\gamma_{Q,j}$ | Partial factor for variable action $j$ |
| $\gamma_{soil}$ | Soil unit weight |
| $\gamma_{sup,j}$ | Partial factor for variable action $j$ (upper/superior value) |
| $\eta$ | Modification factor for concrete strength |
| $\xi$ | Reduction factor applied to unfavorable permanent actions only |
| $\rho_{concrete}$ | Concrete unit weight |
| $\rho_{m,i}$ | Unit price of material $i$ |
| $\rho_{steel}$ | Steel unit weight |
| $\rho_{w,min}$ | Minimum shear reinforcement ratio |
| $\sigma_c$ | Concrete stress |
| $\sigma_s$ | Steel stress |
| $\tau_{Ed}$ | Design shear stress from actions |
| $\tau_{Rd}$ | Design shear resistance |
| $\tau_{Rdc}$ | Design shear resistance of concrete without shear reinforcement |
| $\tau_{Rdc,min}$ | Minimum design shear resistance of concrete |
| $\upsilon_{Edx}$ | Shear force per unit width along the local x-axis |
| $\upsilon_{Edy}$ | Shear force per unit width along the local y-axis |
| $\phi$ | Creep coefficient of concrete |
| $\phi_2$ | Dynamic amplification factor |
| $\psi_{0,1}$ | Combination factor for the leading variable action 1 |
| $\psi_{0,j}$ | Combination factor for the accompanying variable action $j$ |

# Latin Letters

| | |
|---|---|
| $A_s$ | Design reinforcement area |
| $b$ | Width of the considered element |
| $E$ | Total $CO_2$-eq emissions |
| $E_c$ | Total $CO_2$-eq emissions of concrete |
| $E_{c,eff}$ | Effective modulus of elasticity of concrete defined in Eurocodes |
| $E_{ck}$ | Modulus of elasticity of concrete defined in BRO2004 |
| $E_{cm}$ | Mean modulus of elasticity of concrete defined in Eurocodes |
| $E_d$ | Design value of the effect of actions |
| $E_{eff}$ | Effective modulus of elasticity of concrete defined in BRO2004 |
| $E_r$ | Total $CO_2$-eq emissions of reinforcement |
| $E_s$ | Modulus of elasticity of steel |
| $eq_c$ | $CO_2$-eq emission factor per kilogram of concrete |
| $eq_r$ | $CO_2$-eq emission factor per kilogram of reinforcement |
| $f_{ck}$ | Characteristic compressive strength of concrete |
| $f_d$ | Design value of material strength |
| $f_k$ | Characteristic value of material strength |
| $f_{yd}$ | Design yield strength of reinforcement steel |
| $f_{yk}$ | Characteristic yield strength of reinforcement steel |
| $F_d$ | Design value of an action |
| $G_{k,i}$ | Characteristic value of permanent action $i$ |
| $INV$ | Total investment cost for the bridge |
| $INV_m$ | Investment Cost of materials |
| $k_E$ | Aggregate size constant |
| $M_{Ed}$ | Design bending moment |
| $m_{Edx}$ | Positive bending moment per unit width along the x-axis |
| $m'_{Edx}$ | Negative bending moment per unit width along the x-axis |
| $m_{Edy}$ | Positive bending moment per unit width along the y-axis |
| $m'_{Edy}$ | Negative bending moment per unit width along the y-axis |
| $m_{Edxy}$ | Twisting moment per unit width in the x-y plane |
| $m_{Rdx}$ | Positive design bending moment resistance per unit width along the x-axis |
| $m'_{Rdx}$ | Negative design bending moment resistance per unit width along the x-axis |
| $m_{Rdy}$ | Positive design bending moment resistance per unit width along the y-axis |
| $m'_{Rdy}$ | Negative design bending moment resistance per unit width along the y-axis |
| $O$ | Reinforcement bar size (diameter) |
| $P_k$ | Characteristic value of a prestressing force |
| $Q_c$ | Total quantity of concrete |
| $Q_{k,1}$ | Characteristic value of the leading variable action 1 |
| $Q_{k,j}$ | Characteristic value of an accompanying variable action $j$ |
| $Q_{m,i}$ | Quantity of material $i$ |
| $Q_r$ | Total quantity of reinforcement steel |
| $R_d$ | Design value of the structural resistance |
| $r$ | Radius beneath the bridge deck |
| $r_{rail}$ | Rail spacing |

| | |
|---|---|
| $s$ | Reinforcement spacing |
| $t_1$ | Bridge leg thickness |
| $t_2$ | Bridge deck thickness |
| $t_3$ | Foundation thickness |
| $V_{Ed}$ | Design shear force |
| $z$ | Lever arm between compression and tension forces |

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# 1
# Introduction

## 1.1 Background

In Sweden, when constructions such as bridge structures are planned and built, it is important that the design and execution follow the Planning and Building Act (PBA). The most common way of ensuring acceptable and approved solutions is by designing according to the Eurocode (EC) series in combination with the Swedish Transport Administration's Infrastructure Regulations (STA) (TRVINFRA). The Swedish National Board of Housing (Boverket) also provides the European construction standards that complement EC. Additional application standards also exist (Swedish Concrete Association, 2024).

The Eurocodes were introduced by the European Commission as a unified set of European standards used for the structural design of buildings and other civil engineering works (European Commission, 2024). These standards aimed to raise the unity of construction incorporation between European countries, including safety measures in the construction industry. The standard has been in use since the Eurocodes came into force in 2011. Before then, the PBA was followed using the Swedish Building Regulations (BBR) and the Swedish Construction Regulations (BKR) provided by Boverket (Boverket, 2024). However, in the near future, the European commission has revealed that a new evolved generation of the Eurocodes is set to come into force, replacing the previous one. Since 2022, the first revised EC has been accessible, and new EC:s have continuously been published. The new generation is expected to be distributed to all National Standard Organizations no later than March 30, 2026 (ALLPLAN, 2025).

With the revised standards being introduced since 2022, the regulatory environment has changed, raising the question of how these changes affect engineering practice. Although the new standards are intentionally proposed to improve safety and reliability in modern construction practice, this may introduce additional complexity into the design process. Therefore, it is significant to evaluate whether the revised regulations lead to genuinely improved engineering solutions or primarily result in more conservative and demanding requirements. Changes can directly affect structural products such as dimensions, strength class requirements, and material quantities lower than or higher than current or older regulations. Research based on interviews with entrepreneurs and engineering calculations indicates that the initial introduction of EC has increased construction costs (Byggföretagen, 2020), raising the question of what consequences additional refinement of EC causes. Ultimately, when newer regulations are applied, acceptable structures according to the older standards may appear insufficient or unfeasible in the updated frameworks. This leads to the importance of reassessing existing design methodologies and understanding the consequences of changing regulations in structural engineering practice.

At the same time, the traditional design approach in structural engineering is mainly Point-Based Design (PBD), where a single design concept is chosen early and developed in a step-by-step process (Rempling et al., 2019). This approach becomes problematic when new directives or demands appear during projects, causing the previous base design to be inappropriate, leading to necessary redesign and revision. Moreover, modern structural design requires both the interests of several stakeholders to be fulfilled and solutions to be sustainable (Mathern et al., 2018). In order to achieve a more efficient solution based on these evolving requirements, it is essential to investigate alternative methods that handle late-stage problems and eliminate iterative processes.

Set-based design (SBD) offers an alternative design methodology that develops a broad set of feasible solutions that are initially generated based on predefined requirements rather than committing to a single design throughout a project (Mathern et al., 2018; Rempling et al., 2019). In the infrastructure sector, the introduction of an approach similar to the SBD approach is increasingly important to achieve optimal outcomes, including economic conditions and sustainability. For example, the STA requires limiting the use of energy and the impact on the climate of the national transport system, including the construction, operation and maintenance of infrastructure (Toller, 2018). These requirements could be achieved through the implementation of SBD, providing optimization and a clear view of design alternatives that minimize environmental footprint while simultaneously determining the optimal construction costs.

To achieve optimization, Rempling et al. (2019) explored a Set-based parametric design approach (SBPD) in the context of bridge design during the preliminary design phase, fusing SBD and parametric modeling (Nahm & Ishikawa, 2006). Parameterizing the initial set of design allows the engineer to effectively evaluate a wide range of solutions simultaneously. As highlighted by Rempling et al. (2019) and Mathern et al. (2018), maintaining this diverse set of alternatives is beneficial in choosing a final solution in the preliminary phase, allowing the feasible design set to be narrowed down and refined as new decisions, limitations and details emerge through the project operation.

Against this background, while regulatory frameworks such as the Eurocodes undergo significant updates, structural engineering practice remains largely constrained by the traditional PBD approach. Beyond its conventional use for optimizing design solutions, the implementation of SBD offers a valuable comparative methodology. By applying SBD to evaluate designs across different regulations, it becomes possible to systematically compare historical standards with approaching codes, providing critical insights into their respective efficiency, adaptability, and economic implications.

## 1.2 Purpose and objectives

This master's thesis aims to conduct a comparative regulatory analysis between the second generation of the Eurocodes and the older Swedish bridge design regulation, Bro 2004. The study was performed by developing a computational SBD algorithm integrated with Finite Element Analysis (FEA). The work was built upon the foundational SBD framework developed by Lucas Hansson, which was a further development of his previous master's thesis (Hansson & Tacking, 2024), adapting and expanding it for the current comparative study. Through the integrated approach, the thesis investigated how different regulatory frameworks influence the feasibility and structural response of different bridge designs based on the used case study of concrete frame bridges during the preliminary design phase.

By integrating the SBD with the computational structural analysis, the study contributes to a deeper understanding of how future design standards affect the early stages of bridge design and engineering decisions compared to older design criteria. Furthermore, the study demonstrates how SBD can be utilized as a decision support for identifying sustainable and balanced design solutions that satisfy several design criteria simultaneously. The study also attempts to demonstrate a wider applicability of SBD. While earlier studies have focused on using SBD for optimization and decision support does this study also demonstrate it's possibility in comparisons of regulatory frameworks (Bergenram & Ulander, 2023; Fernández & Tarazona Ramos, 2014; Hansson & Tacking, 2024).

## 1.3 Limitations

The study is made up of several steps that create a consistent workflow to achieve intended outcomes. Implementations of regulatory frameworks need to be performed in similar and distinct environments for the case study to provide clear boundary conditions in order to be practically comparable. Apart from that, certain parameters may provide an excessive amount of data that is numerically difficult to analyze and needs to be limited or disregarded. The following text describes limitations for different aspects that the thesis consists of.

**Boundaries of used Set Based Design methodology**

- The SBD algorithm used is structured to evaluate solutions with the same reinforcement size and spacing in all studied bridge segments, combination of different spacing and reinforcement sizes within and between different segments is not used.
- The evaluation attributes used, investment cost and $CO_2$-equivalent (eq), are studied solely related to the amounts of concrete, reinforcement and formwork. The account of buildability, costs and $CO_2$-equivalent from the construction phase is not made. Similarly, maintenance is not accounted for.
- Only the bridge segments frame legs and slab deck are studied, the foundation of the bridge concept is modeled within the FEA, but not accounted for in the validation against regulations and results.

**Geometrical and conceptual scope**

- The study only explores the concept of a closed concrete frame bridge of 12 meter span. Alternative bridge concepts and spans are not studied.
- The geometries and properties studied are only the ones described in Table 3.1 and 3.2.
- Although concrete frame bridges typically comprise wing walls and edge beams, such components' impact on the structural behavior is not fully included in the present study.

**Loads and comparative scope**

- The comparative study does not perform the structural analysis with respect to fatigue, wind loading, traffic load on foundation slab, and geotechnical behavior.
- The comparative study is restricted to the second generation of Eurocode and Bro 2004. In addition, the evaluation incorporates their supporting documentation, including the General Technical Specification, National Annex, structural design rules and load-modeling guidelines.

**Structural and analytical constraints**

- Calculations in the SBD algorithm are performed using Finite Element Analysis. The FE model in Brigade/Plus is limited to linear elastic analysis. Nonlinear behavior of the concrete is not addressed directly within the FEA calculations.
- To provide representative results of the frame legs and slab deck, set boundary conditions were used. The model implemented fixed behavior towards displacement within the interaction between the foundation and soil. This

simulates a specific stiffness, but means that the interaction of the soil and the true behavior of the foundation slab are not fully captured. Since legs and deck slab are studied, it is considered sufficient.

## 1.4 Methods

This master's thesis was initially conducted with a literature study, focused on reviewing related research in the field of SBD application and development of Swedish regulatory frameworks. Additionally, methods of calculation and appropriate design criteria according to different construction design standards were studied. An increased emphasis was made for the two regulatory frameworks to be compared using SBD, second generation of EC in combination with the currently governing general technical specification (GTS) and the GTS Bro 2004 with associated documents in the context of the studied case.

The studied case of reinforced concrete frame bridges was modeled through a SBD Python script. An already made script by Lucas Hansson during the summer of 2024, a development of his previous master's thesis work, was provided and used as a framework for the development of the used Python script (Hansson & Tacking, 2024). With the framework of the script, modifications and implementation were made, adapting the script based on the separate regulatory standards. The new versions were then validated by analytical calculations to ensure the reliability and accuracy of the algorithm. Compared to the SBD-algorithm used by Hansson and Tacking (2024), the SBD procedure used was modified to provide a more general and simpler approach while still providing efficient and useful results in the context of preliminary design. The process aimed to obtain the entire design space of feasible- and not feasible designs by performing a filtration process with the resulting structural response from the BRIGADE/Plus software combined with validation criteria set by used regulatory frameworks.

The final phase of the study focused on evaluating the attributes of the resulting designs. The attributes studied were environmental impact, economic investment costs and magnitude of structural response. The properties were then compared using different regulations. Moreover, the influence of design standards that result in different outcomes was also explored. The study's overall procedures are presented in Figure 1.1, which illustrates a clear overview of the workflow throughout the process.

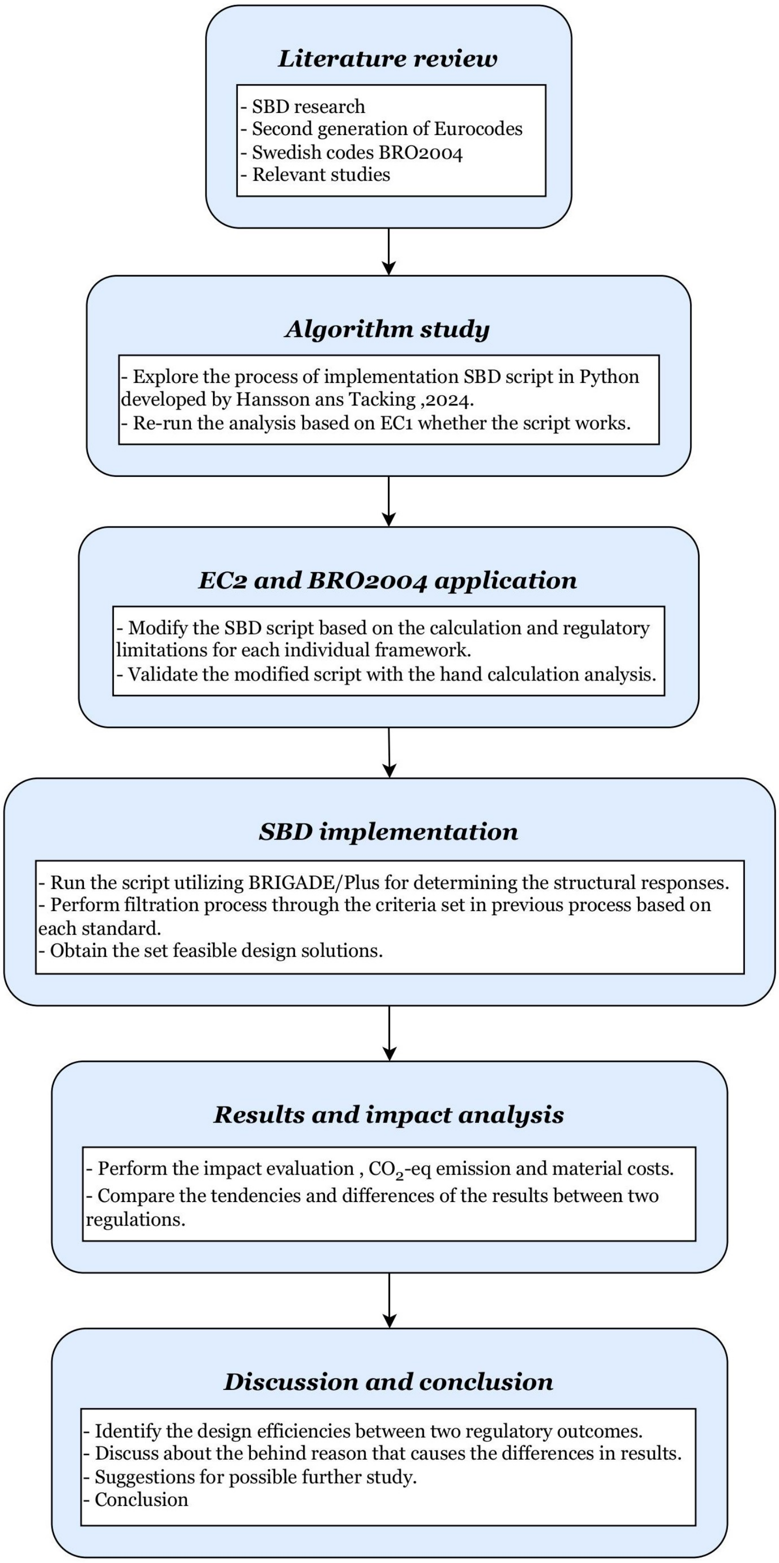



**Figure 1.1:** Overall methods implemented in the thesis.

## 1.5 Scientific approach

This study performs analytical research and quantitative methodology to investigate the differences between designing concrete bridges according to the second generation of Eurocodes and the older regulations surrounding the GTS Bro 2004. The study follows the approach where several bridge design alternatives are generated and evaluated through the method of computational modeling, utilizing the SBD approach in combination with FEA to assess structural responses under the defined loading conditions and parameters specified by the respective standards.

The analysis aims to conduct a comparative quantity study for the set of feasible designs in order to identify trends, design indicators including how the updated regulations influence the structural behavior, material usage and environmental impacts associated with the production of utilizing different regulatory systems.

To ensure reliability and consistent modeling assumptions, the boundary conditions are maintained and verified against analytical calculations throughout the study.

# 2

# Frame of reference

## 2.1 Set-based design

### 2.1.1 The overview of Set-Based Design

As stated in Section 1.1, SBD is an approach that enables the consideration of all possible design solutions and gradual eliminate unqualified solutions with explicit commitments until a final solution is obtained (Rempling et al., 2019). The initial step is to define the design space for the total amount of possible solutions within a range of parametric values (Toche et al., 2020). The set of feasible regions are next defined through criteria boundaries used as filters based on requirements from engineers and stakeholders. Subsequently, the filtration begins by finding the intersection of different filters, defining feasibility. Ultimately, the final feasible design set of qualified candidates is provided. An illustration of the procedure is presented in Figure 2.1.

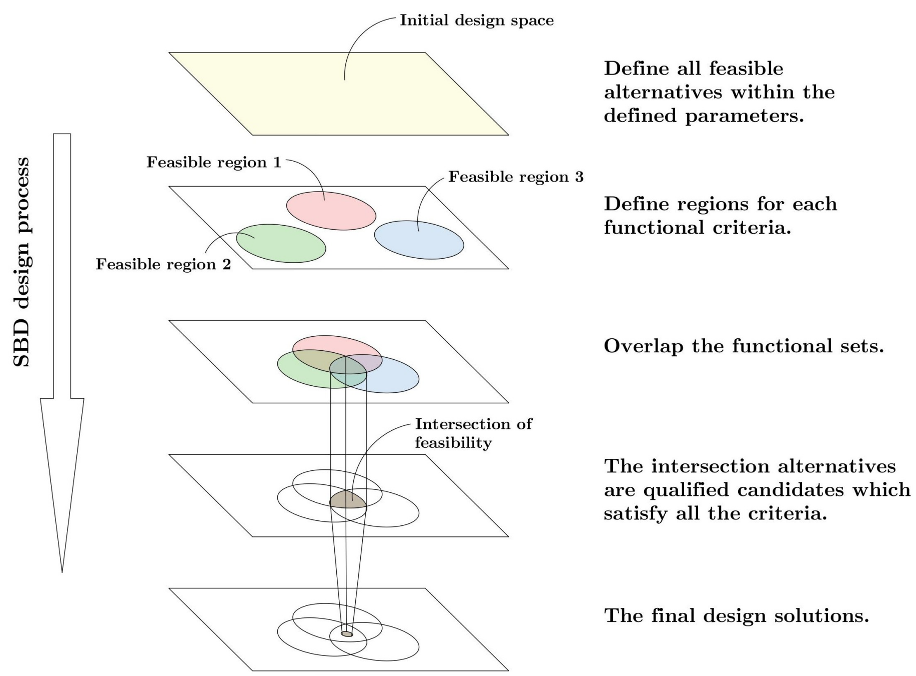



**Figure 2.1:** SBD process, Inspired by and modified from Hansson and Tacking, 2024 and Toche et al., 2020

Once the final sets of qualified solutions has been reached, the most optimal solutions from the eligible designs can be selected. In addition, the SBD provides the possibility of reprocessing and re-selection to obtain a fresh set of qualified design solutions when design requirements are changed or needed to be further studied which leads to new, optimal solutions can be achieved. Furthermore, discussion and reasoning with various design choices in order to obtain the best solutions lead to higher design efficiency than the development of single alternatives (Rempling et al., 2019). PBD, in contrast, entails the risk that further redesign from its prior design solution strays away from the possibility of reaching the best design alternative. This is avoided using an SBD, since it allows more demanding re-design while still providing optimal solutions.

### 2.1.2 Application of Set-Based Design in Bridge structures

The study by Rempling et al., 2019 is one of the earliest research attempts to investigate the applicability of Set-Based Parametric Design methodology to existing concrete bridge structures. The study intended to redesign existing bridges with the new approach of integrating SBD, parametric design, finite element analysis and multi-criteria decision analysis in order to evaluate the efficiency of the resulting design alternatives. The chosen set of parametric criteria was defined to generate the design options and was applied throughout the evaluation and comparison process. The investigation of SBD integration eventually resulted in higher efficiency in terms of material costs and carbon dioxide equivalent emission compared with the existing bridge designs (Rempling et al., 2019). Ultimately, Rempling et al. (2019) study provided both proof-of-concept and theoretical footing for SBD in concrete bridge design (Hansson & Tacking, 2024).

SBD in concrete bridge design was further explored in Löfgren, 2020, focused on evaluating the buildability and sustainability by performing the study on typical concrete frame bridge structures. The author concluded that there is great potential for SBD integration to obtain a higher sustainable and cost-effective construction in frame bridge structures. However, the results of the buildability tool turned out too difficult to identify in quantity (Löfgren, 2020).

Due to the difficulty in determining the buildability impacts in Löfgren, 2020, Bergenram and Ulander, 2023 continued examining the objective and further optimized the investment cost and the reduction of environmental impacts through SBD. The study demonstrates that the SBD effectively reduced the economic costs and decreased the environmental impacts, and buildability was further successfully quantified within the economic costs. Nonetheless, it introduced complexity as the optimized solution gives additional costs related to higher material strength, additional reinforcement and increased shear reinforcement. The authors finally suggest that the geometrically optimized solutions relates to the increased buildability costs (Bergenram & Ulander, 2023).

Hansson and Tacking, 2024 continued developing the study on the improvement of parametric requirements and uncertainties during the preliminary design and public procurement process. The concrete frame bridges were also studied to investigate the effects of input on the material costs, $CO_2$-eq emissions and buildability. Ultimately, the outcomes pointed that the adopting SBD can improve decision-making and result in more sustainable infrastructure development (Hansson & Tacking, 2024).

These studies provide a concrete basis for incorporating SBD into the process of optimization. Ultimately, the recent research has raised a strong theoretical foundation for the development of parametric design algorithms in the preliminary design process by obtaining significant requirements for the optimized solution while simultaneously considering economic costs, buildability and environmental impact. These aspects will be further examined in this thesis through the application of SBD in comparing feasibility between different design alternative and their corresponding structural responses under different regulations.

## 2.2 Construction design phases

The overview of infrastructure planning and construction phases, where the STA acts as a client, generally considers identifying needs, developing concepts and designing solutions (Hansson & Tacking, 2024). The operation can be briefly divided into several steps: Early stage, Pre-design, Preliminary design, Detailed design, Construction, Delivery and Asset assessment as presented in Figure 2.2 (Antonsson et al., 2022), which illustrates the detail of design and the possibility of change during the construction phases. As mentioned in Section 1.1, the implementation of SBD is highly relevant in the earlier stages of construction, providing a variety of designs. Hence, aiding in the case of uncertainty further down in the construction process.

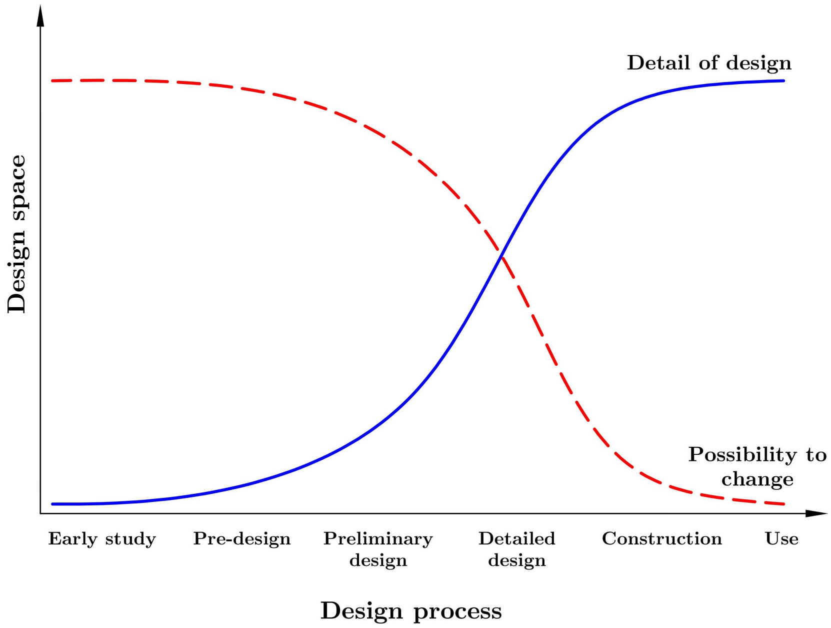



**Figure 2.2:** Illustration of the level of design detail allows for the possibility to change. Inspired and modified from Antonsson et al., 2022.

### 2.2.1 Early study

According to the STA, the early stage of construction (Åtgärdsvalsstudie) is structured with a long-term planning strategic framework that takes place before the design process and the physical construction begins (Trafikverket, 2023b). This stage is primarily focused on identifying problems, understanding the needs of the system and measuring the contribution towards sustainable social development and economic efficiency. With these regards in mind, wide exploration of different alternative solutions is made. By early planning, early communication is possible between different relevant stakeholders. Early communication generally leads to a common ground of consensus and shared responsibility within all possible alternatives. Typically, the procedure initiates with several actors such as the STA, municipalities, region or any other related stakeholders being involved.

A strategy that guides the early-stage work according to STA is the 'Four-step principle', which provides a work procedure ensuring good resource management and sustainable solutions in developing transport and infrastructure (Trafikverket, 2021). The steps are formulated in order as such described below to ensure optimal efficiency in management, resource usage and cost. The first steps within the procedure consists of measuring the ability to reduce the transport demands by means of speed limits, taxes, fees, etc. The second step is the alternative of reconstruction or modification of existing structures. Lastly, the alternative of new constructions is studied.

### 2.2.2 Pre-design

The pre-design phase aims to, in more detail, explore the preliminary technical considerations (Trafikverket, 2025b). The task also consists of deeper investigations surrounding the consequences of land claims for the planned construction. This consideration is made so that when roads or railways are built, the city and landscape as well as the natural and cultural assessments ensures as few inconveniences as possible for related individuals. Required preparation is to carry out a floor plan map, plan description and consultation report, including the environmental impact assessment and other relevant information. With these preparations laid out, the access of having the right to build the infrastructure on the land might be achieved. However, the level of detail remains relatively low compared to later stages with a flexible space of adjustment based on new requirements from relevant stakeholders.

### 2.2.3 Preliminary design

When the preliminary design phase begins, the purpose and the scope of the project will increasingly be clarified. Engineers begin to develop suitable technical solution. The project, however, remains limited in detailed boundaries. A significant action for this step in the design process is the preparation of the planing description (vägplan or järnvägsplan) which is a preliminary assessment of how the planning phase should be structured (Trafikverket, 2014). Many key parameters are translated from the conceptual ideas into defined solutions such as route layout, potential effects, environmental impact assessment, main structural system with the sufficient technical calculation to validate that the proposed solutions meet the standards including other legislation following the coordination with the Planing and Building Act (PBL) (Trafikverket, 2014). This guarantees that different design solutions is correctly determined, contributing to the legal approval process.

This significantly reduces the uncertainty due to the increased commitments compared to earlier processes which decreases the possibility of major change. However, after a review process from clients or stakeholders, a potential for modification remains in the design solution which may leads revision in the project's plan and design solution. While the limitations of requirements are changeable, instead of the development of a single design solution, the SBD can be applied in this process to perform the re-design steps for optimization and efficiency in the final design alternative. The process continues improving until receiving the statement from the country administrative board, the document is then assembled and sent to STA to be assessed and approved (Hansson & Tacking, 2024). After the approval from STA the detailed design phase are then be able to begin.

### 2.2.4 Detailed design

After the approval of the vägplan or järnvägsplan, the construction process enters the detailed design phase which no longer focuses on selecting appropriate design solutions, but instead focuses on developing the chosen alternatives to a higher level of precision. The implementations are based on the documents from the preliminary design phase where all the technical aspects will be fully defined to be ready for the construction without ambiguity (Hansson & Tacking, 2024).

During the implementation, the engineer produces a complete product of technical documents, including detailed drawings, specifications, descriptions of material of the detailed construction, as illustrated with increasingly detailed level in Figure 2.2. The design complies with the safety criteria stipulated by relevant standards, such as EC:s and STA:s regulations. The provided documents in the detailed design phase consists of refined communication in all aspect of designed parts. For example, geometric dimensions, types of structural system and the process of installment. This process aims to maximize the level of certainty where the contractor can implement the construction directly.

There is also an action to be conducted in this design step according to Trafikverket (2025a), which states that the designer must carry out measurements corresponding to defined reduction requirement that the STA has set. As a result, the engineer must perform optimization of material quantity, improved technical solutions including the integration of low-carbon alternatives in the design, ensuring the project's procedures reach the target of the climate-reduction level (Trafikverket, 2025a). Subsequently, after overall requirements in the detailed design are satisfied, the process can be continuously proceeded in the construction phase.

### 2.2.5 Construction, Delivery and Asset assessment

In the construction phase, the contractor takes full responsibility to carry out the work that corresponds to the technical documents established in earlier phases, with inspection and quality assurance from the STA as the client and the project's consultant. After the construction is completed, the actual construction cost will be calculated along with the assessment of the climate performance during the construction, operation and maintenance phases (Trafikverket, 2025a). This may imply to the importance of the preliminary design phase where the cost of modification is relatively low and the range of alternatives is still highly flexible. Therefore, integration of optimization approaches such as SBD during the preliminary design is interesting to look into. In order to suggest efficient decision-making from the initial designs where these choices are largely defined later on the overall products including economic costs and environmental impacts (Hansson & Tacking, 2024).

Following the completion of the construction, the project enters the delivery phase where the structure is officially handed over to the client or the operational organization which involves inspection, verification and testing, ensuring that the asset reaches the functional and technical requirements. Afterwards, the infrastructure becomes a part of the asset management such as the operation, annual maintenance including the long-term performance evaluation to maintain durability, safety and cost efficiency over the operational period.

## 2.3 Evolution of Eurocodes

Eurocodes are the series of 10 European technical standards (EN1990 - EN1999) that provides the approach of the structural design for buildings and other civil engineering works such as bridges, roads and geotechnical work, ensuring safety, reliability and consistency in engineering structures. The design standards cover several types of structural material calculation practice including concrete, timber, steel, masonry and aluminum structures (European Commission, 2024).

The European Standards is fundamentally delivered through the distinction between the European Standards (EN) and the National Annex (NA). While the EN provides general design principles and common regulations for the construction code of practice, the National Annex from each union member, on the other hand, localizes the Nationally Determined Parameters (NDP) such as partial safety factor, climate maps (wind, snow) or specific calculation methods corresponding to the individual local conditions (Eurocodes Tools, 2026). The specified values from the National Annex are essential in addition to the theoretical basis of EC:s implementation in order to achieve suitable construction practices for the unique regions.

### 2.3.1 The first generation of Eurocodes

In the early stage, the standards were primarily established by the European Commission in the 1970s under the rule of the experimental pre-standard (ENV) (Eurocodes Tools, 2026), serving as an alternative to the national regulations with the later purpose of abolishing obstacles to technical trade within the European Union (European Commission, Joint Research Centre, 2026a). Later in the mid-2000s, the standardized system became more necessary when the collaboration between European countries increased, the content were then converted into European Standards (EN) (Eurocodes Tools, 2026) and increasingly applied to the state members of the union.

The first generation of Eurocodes were gradually developed and implemented from 2002 to 2007 with international collaboration across European countries (Denton & Angelino, 2024), while the National Standards were still used in parallel. Later around 2010, the European Commission primary addressed to CEN (European Committee for Standardization) with the Mandate M/416 for the reference of structural design and issued the Construction Products Regulation in 2011 (European Commission, Joint Research Centre, 2026a). The EN were continuously maintained and developed including the updated Mandate M/515 in 2012 and finally marked the official declaration of Eurocodes performance for the Construction Products Regulation in 2013. (European Commission, Joint Research Centre, 2026a). In order to establish construction design safety and unified conditions within the community, enforcement was carried out to replace the National Annexes with the EN afterwards. This led to the achievement of seamless collaboration regarding the civil engineering products across European countries.

### 2.3.2 The second generation of Eurocodes

Although, in 2013, the first generation of Eurocodes was successfully launched and implemented within the entire Union, the standards needed updating to reflect the advances in engineering practice evolved through time. This led the European Commission formally introduced a revision process in 2015 by CEN/TC 250 through Mandate M/515 which aimed to enhance the ease of using the Eurocodes and to reach the exemplary level of international consensus (Denton & Angelino, 2024). While the first generation of Eurocodes focused on achieving consistency between national design rule, the new phase prioritizes improving practicality in the modern construction industry, including the extension of assessment for existing structures and strengthening the requirements for structures' robustness. The new generation also applies more consideration of environmental impacts on structures and geotechnical design. Furthermore, a new part in EC on structural glass will be introduced in the upcoming revision (Denton & Angelino, 2024).

The revised version of EC:s has been gradually published to National Standardization Bodies (NSB) since the early 2020s, which contains 11 EN standards in around 65 parts, 12 CEN Technical Specifications (TS) and 7 CEN Technical Reports (TR) (Denton & Angelino, 2024). This does not only provides the improvement of safety but also represents the ability of the structural EC:s evaluation to reflect the variety of new approaches, new materials, new regulatory requirements and new social demands developing, ensuring the involvement in a rapid development in modern social contexts and future engineering challenges for the upcoming decades of designing constructions (European Commission, Joint Research Centre, 2026b).

For the time frame of the enforcement, all countries are obligated to withdraw the entire first generation documents by 2028 to establish the full application of the second generation of EC:s across the European countries (European Commission, Joint Research Centre, 2026c). In the mean time, it allows the transition and replacement to the new regulation, which aims to improve and modernize the design implementation and progressively integrate the individual NA to the approaching standards. Moreover, it is also significant to understand the future direction and environmental consequences in the construction industry under the utilization of the second generation of EC:s.

## 2.4 Historiography of Swedish construction regulations

Throughout the modern era, there has always existed guidance and regulation when designing bridges, ensuring safety, reliability and structural integrity. To put the development of standards into perspective the following historiography is made, starting at 1976 and ending in the present day. Specific years have been chosen in which then-current directives and surrounding years' supplements has been identified. Relevant to this thesis, the main focus will surround the design of concrete structures and load modeling of railway bridges. As regulations has changed throughout the years, many detailed values in varying aspects has also changed. Going through and comparing the differences between every regulation that is covered in this historiography would be a considerably extensive task. Therefore, only sub-areas with substantial change will be highlighted.

### 2.4.1 1976 - Applicable design standards

In September 1976, a new bridge standard was established by the Swedish National Road Administration (Vägverket), Bronorm 76 - TB103 (Vägverket, 1976). Bronorm 76 was set to act as a central part of the existing national regulations for bridge design, acting as a governing basis in planning, analyses and execution. In Bronorm 76, descriptions and definition is made of how standards, requirements and structural design principles should be followed. Depending on what aspect is treated, the governing basis definition is also varying. Whereas sometimes only recommendations is given, other parts gives exact values used for calculations or limits that needs to be fulfilled.

To make correct load modeling Bronorm 76 references other documents that is needed (Vägverket, 1976). Two of these documents is Provisional Traffic Load Regulations (Vägverk, 1975) and State Load Regulations of 1960 for Structural Works (Kungl. Väg- och vattenbyggnadsverket et al., 1961). Both describe the previous method of modeling loads, where the first one was a later updated version of how traffic loads should be modeled. However, regarding railway-traffic loads, Load Regulations of 1960 for Structural Works was used. For concrete structures, Bronorm 76 references the usage of established concrete regulations. Published by the Swedish Concrete Committee, different versions of regulations for concrete was released. In 1965, volume one was released, presenting material and workmanship instructions/regulations for the concrete itself (Kungl. Byggnadsstyrelsen et al., 1965). In 1968, an additional part was released, presenting the regulations for reinforcement (Kungl. Byggnadsstyrelsen et al., 1968b). Lastly, an additional volume three was released 1968 (Kungl. Byggnadsstyrelsen et al., 1968a). Volume three purpose was to complement the previous two version as well as add additional clarification on certain parts, for example shear and plate structures. Similar to how EC provides calculations to determine active structural stresses and design resistances today, the Swedish Concrete Committee documents does the same. Except in these older version, classes, formulas, approaches, etc. vary compared to today's standards.

Comparing the older descriptions with today's standards significant differences in calculations can be seen regarding safety-margin approaches. Having coefficients on both the load and material properties coefficients that we use today was instead only applied to the material properties. Defined loads in previous documents was directly applied when calculating the structural response (Kungl. Byggnadsstyrelsen et al., 1968a; Kungl. Väg- och vattenbyggnadsverket et al., 1961).

### 2.4.2 1988 - Introduction of bronorm 88

At the first of March 1989, a new GTS named bronorm 88 was released, replacing the older bronorm 76 (Vägverket, 1988). Similar to the previous version the new generation would also act as a governing basis, incorporating revised values, new aspects, new references and additional refinements. With the new technical specification also came the introduction to a new method in order to take uncertainty within the regulations into consideration, the partial-coefficient method (SBUF & BBT, 2025). By using probabilistic methods, coefficients was calculated and added to the applied loads, increasing the loads used in dimensioning. Together with Bronorm 88, a dedicated document providing explanations of how train loads should be modeled was released, Tåglast 89 (Banverket, 1989). In Tåglast 89, the method of partial coefficients can be seen used.

Together with Bronorm 88, a new version of the applicable regulations belonging to the GTS was published (Vägverket, 1988). Pertinent to concrete, a new edition of the Swedish Concrete Committee's regulations was accompanied, regulations for concrete structures named BBK79 (Statens Planverk, 1979). BBK79 was released in 1979, with a second edition published around the same time as Bronorm 88. With the introduction of the second edition of BBK79, the older regulations was withdrawn. BBK79 highlights its usage defined by instructions, advisory guidelines and its examples of allowed solutions and methods. One interesting part is the statement of the upcoming planning and building act that is to be integrated.

### 2.4.3 1994 - New GTS and introduction of the Swedish National Rail Administration documents

On the first of October 1994, the new GTS named Bro 94 entered into force, replacing the old Bronorm 88 (Vägverket, 1994). Similar to the changes made between Bronorm 76 and Bronorm 88 the changes in Bro 94 focuses on revision, aspects and additional refinements to stay current with new modern science, development, challenges and regulations.

Bro 94 also came with a new version of regulations when dimensioning concrete structures due to the change of introducing the new plan and building act. Bro 94 states the new approach of using Boverket's Handbook of concrete structures (BBK 94) (Boverket, 1994) and Boverket's structural Design regulations. The usage of these documents was meant in tandem with one another, with BKR being regulations and general guidelines while BBK 94 being a handbook for calculating and craftsmanship when using concrete. BBK 94 consisted of several extracts and made comments on BKR, indicating the handbook's close connection to the regulations. An important change regarding the design of railway bridges was the introduction to the new BV BRO regulations provided by the Swedish National Rail Administration (Banverket) (Banverket, 1995). The new documents were meant acting as a development of BRO 94 when designing railway bridges. BV BRO was applied in conjunction with the current publication. The new BV BRO edition one consisted of adjustments, addendum and additional information based on the GTS and belonging documents stated by BRO 94.

### 2.4.4 2002 and 2004 - updates of the GTS

On the first of July 2002, the new version of GTS, Bro 2002 was put into use (Vägverket, 2002). Just like in the other changes of GTS, Bro 2002 consisted of revisions and refinements to meet current challenges. Similar to BRO 94, Bro 2002 states BBK 94 and BKR jointly in force. In parallel with updates of BRO 94 and the new introduction to Bro 2002, Banverket continuously provided new, updated versions of BV BRO. At the moment of Bro 2002 came into force, BV Bro 6th edition was released (Banverket, 2002).

Bro 2002 was discontinued on the first of August 2004, when Bro 2004 came into force (Vägverket, 2004). An updated version that was revised and refined. Belonging to Bro 2004 was the updated version of Boverket's Handbook of concrete structures (BBK 04), which as mentioned in previously was applied with the applicable BKR (Boverket, 2004). Bro 2004 also states the concurrent regulations by Vägverket (VVFS) (VKR) that handled load-bearing capacity, stability and durability of structures relevant to roads for road traffic. However, for railway bridges, BV BRO had continuously been released, valid for Bro 2004 in the 7th edition until the final 9th edition (Banverket, 2004, 2009).

### 2.4.5 2009 - significant change in structural design norms and introduction to Eurocodes

In 2009, a significant change happened in the system of regulating documents in bridge design. The new document stated as a technical description was implemented at the 1st of July 2009, named TK Bro 2009 (Vägverket, 2009). The new TK Bro 2009 document acted as a combined bridge standard for both road and railway bridges. This meant the previous format of Bro 2004 and a supplement of BV BRO was scratched. TK bro 2009 as a document stated the technical requirements in design. Meanwhile, in addition TR bro also existed, providing advice and explanatory notes associated with the demands set in TK bro 2009. Similar to the format before, additional documents is referred to as supplementing TK bro 2009. An additional supplement was the General Material and Workmanship Specifications documents, often referred to when describing civil engineering works.

With the new technical description a new supplement for calculation standards was added (Vägverket, 2009). Instead of the familiar BBK format the European calculation standards should be used, Eurocode. The newly integrated EC managed nationally by the Swedish Standards Institute is divided into different parts, handling different subjects of the calculation procedure. For example, SS-EN 1991 states how different Loads should be calculated/handled while SS-EN 1992 deals with concrete structures. In addition to the Eurocodes, national decisions made by Vägverket and Boverket were provided as aid in order to apply the EC:s to specific Swedish regulations. For railway bridges, a national decision was provided by Boverket's construction rules (BFS2008) (Boverket, 2008).

### 2.4.6 2016 and 2020 - New national authority, Trafikverket

At the first of April 2010, the two previous authorities Vägverket and Banverket were combined into a single national authority, the Swedish Transport Administartion (Trafikverket). The newly formed Trafikverket's responsibility is to control the long-term planning of the national transport system for road, rail, maritime and aviation sectors (Trafikverket, n.d.). Trafikverket builds, maintains and pursues state-owned roads and railways. On the third of October 2016, Trafikverket released their new requirement specifications document in the form of TDOK 2016 (Trafikverket, 2019). Similar to how TK Bro was laid out, TDOK 2016 refers to the Eurocode for structural behavior calculations. Also, with additional national decisions stated by Boverket's construction rules (BFS 2011:10) accompanying railway bridges (Boverket, 2011).

From the first of October 2020 to the present day, Trafikverkets infrastructure-regulation (TRVINFRA) has been the governing requirement specification documents (Trafikverket, 2025c). TRVINFRA replaced the previous TDOK 2016 documents. The regulatory framework of TRVINFRA is stated as consisting of 3 different parts for Bridge and Bridge-like constructions. Each with one certain work area, general requirements, construction and bridge maintenance. Structural behavior calculations have, just like in the two previous versions of the regulatory documents been ruled by Eurocodes acting as a supplement. However, in the new TRVINFRA national choices, regulations and advisory guidelines are defined by the Swedish Transport Agency's document (TSFS 2018:57) (Transportstyrelsen, 2018).

### 2.4.7 Conclusion of Historiography

Looking at the development of the mentioned changes in regulations an overall pattern of how the authorities and documents have changed throughout the years is seen. Starting with having Vägverket and construction agencies providing aid in 1976, which later created the expansion of the two separate authorities, Banverket and Vägverket. Then, in 2009, they were merged into a single authority named Trafikverket. Patterns in the layout of supplementing documents can also be seen being changed. In the beginning, all Load modeling was provided by one document in 1976 (not considering the appearance of provisional regulations). Later divided into two separate documents in 1988 and the following years with Tåglast 89 and BV BRO. Then in 2009 Load modeling and Structural calculations for concrete were combined into one single framework, the Eurocodes. For concrete calculations, similar to load modeling the regulations were provided by separate documents until 2009, when it was combined into the Eurocodes. Except for the change to using Eurocode in structural calculations in TK Bro 2009 the change to the partial-coefficient method was also identified in Bronorm 88.

Even though the structure of supplements has changed as time has passed, the recurring pattern of one general technical specification has always been there, from the Bronorm documents all the way to current TRVINFRA. However, the content of the general technical specifications is interesting to look at, reflecting the challenges that are faced in the modern Bridge construction sector. For example, since Bro 2002 separate documents have been used for bridge maintenance (Vägverket, 2002). This shows the needs, regulations and further demands of maintaining older bridges. As mentioned in Section 2.4, this historiography is only a brief explanation, time intervals in between the documents that have been addressed also have a large amount of revisions, extra supplements, aids and versions. Going through every change and difference between documents would be a considerably extensive task. But still interesting to understand what has changed and the reasoning behind it.

# 3

# Set-Based design algorithm

*This chapter interprets how the SBD algorithm is implemented and how the filtration criteria are measured.*

## 3.1 Design of railway bridges using SBD

SBD has previously been applied to the preliminary design of concrete railway bridges, demonstrating its potential and applicability in comparison of different design properties (Beischer & Furuhjelm, 2025; Bergenram & Ulander, 2023; Hansson & Tacking, 2024). For this specific thesis, the SBD approach is inspired by and a development of the SBPD algorithm made by Hansson and Tacking (2024) with the same approach of using a finite element software to achieve the structural response. In this thesis, a comparison of using different construction design regulations will be made by using and modifying the previous algorithm. The modifications consist of making the algorithm applicable to the adjusted regulations as well as new definitions of design alternatives and post-processing of the result. This chapter explains the workflow of the used algorithm and how the SBD tool executes the SBD approach in the context of bridge modeling.

### 3.1.1 Fundamental basis of design

When designing structural components the fundamental basis is to evaluate a capacity towards different structural responses, defining if a design is feasible or not. When designing modern bridges, it is necessary to follow the set regulatory framework provided by STA (Trafikverket). At present time the up-to-date version of regulations is provided by Trafikverkets infrastructure-regulation (TRVINFRA). Specifically TRVINFRA-00227, directing bridge and bridge-like constructions. TRVINFRA also has supplement providing instructions on methods of structural behavior calculations such as load modeling, capacity calculations etc, this is provided by the Eurocodes. As mentioned in the purpose of this thesis, the method of calculating the structural behavior will be made according to two separate regulations. One method to be used is the method of TRVINFRA-00227 in combination with the new generation of Eurocode. This will be compared to the previous regulatory framework Bro 2004 with its corresponding supplementing documents, BBK 04 and BV Bro 7th edition. Specific parts that will be investigated, changed and compared between the methods within the calculations are described in further detail in Chapter 4 and 5. Changes will be made based on the used SBD algorithms validation criteria explained below.

### 3.1.2 Applied SBD-method

Using an SBPD approach provides a method of parametric modeling. By using an initial dataset with varying geometry different element sizes and designs can be applied. These designs can then be checked if feasible or not. With the information of feasibility, an evaluation can be made, comparing the solutions to one another. For this thesis, the set-based parametric design algorithm made by Lucas Hansson during the summer of 2024 will be used. The Work Hansson did was a development of his previous master's thesis during the spring of 2024 (Hansson & Tacking, 2024). The algorithm can be seen as divided into three main steps, Generation, Verification and Evaluation.

In the generation step the initial design space is created by forming an input dataset that consists of all different combinations of values possible based on the initial parameters chosen. The dataset can be seen as a large matrix where each row represents one bridge design's properties. Each row's properties will be different from all the other designs with at least one set property differing, making one row/design inherently unique to all other rows/designs. The parameters used consist of variable geometric inputs that are defined as a range of values. Certain properties were also set as a single value that remained the same throughout every row/design. For the used ranges and values, see Table 3.1. The last part of the generation step is modeling of the designs. Each row of designs that is created calls upon generating a model according to the specific row's properties. This ensures that each design within the initial design space gets modeled based on the initial dataset's range of combinations. One important note is that the varying reinforcement sizes are not defined as a varying parameter for the initial set. Different reinforcement sizes and spacings are later applied in the verification step, this layout provides efficient computations as it reduces the amount of designs analyzed in the FE-software.

Secondly, the step of verification takes place. The base premise of the verification step could be seen as 2 parts, intertwined with one another. The first part is to induce hard- and soft constraints on the design space (Parrish et al., 2007). Hard constraints in the case of the used algorithm are conducting a structural analysis of each design provided in the initial dataset. With the structural behavior of all solutions defined, validation can be made based on the design standards the response is verified against. Soft constraints in the aspect of design preference, such as the number of reinforcement layers with corresponding spacing. This means that based on the result from the verification process, every single design from the initial space can be defined as feasible or not, depending on if it fulfills the requirements met by the used regulation. If a solution fulfills the requirement, the solution is stated as feasible and collected into a new dataset, becoming a part of the feasible design space and vice versa.
The second part of the verification can be seen as further expanding the initial design space by accounting for a variable reinforcement size. As mentioned in the generation

step, the reinforcement sizes are instead applied when the verification is made. The demonstrative explanation of rows in a matrix can be seen as further expanding within the verification process. With one certain row of properties expanding into a higher number of rows, depending on the amount of reinforcement sizes used parametrically. Combinations of different reinforcement sizes in different parts of the studied bridge are however not checked, the script is limited to only checking one reinforcement size in the entire bridge.
The last and third step of the algorithm's process is the evaluation. Data obtained throughout the verification step together with the initial input dataset is used to calculate relevant attributes which later will be used for comparison. Attributes studied in the studied case are material volumes, material costs and $CO_2$-eq performance. For further explanation of the attributes framework and how it is computed, see Section 3.5. The Evaluation will be made on both the feasible and total design space to make a comparison of their attributes relative to one another. A holistic illustrative explanation of the algorithm's process is shown in Appendix D, Figure D.1.

## 3.2 Implemented bridge model

With the general principles and workflow of the SBD algorithm presented, this section presents the case study in this thesis on which the SBPD approach is implemented. The case studied is the construction of a concrete frame bridge. This section highlights the scope of the design as well as the data used in the performed SBD analyses.

### 3.2.1 Bridge concept

The most common bridge type in Sweden is the concrete frame bridge (Trafikverket, 2008). The frame bridge is designed in either several spans or single-span, it is also used with either prestressed reinforcement or non-prestressed reinforcement. The studied construction in this thesis is a reinforced concrete single-span frame bridge. The frame bridge is defined by its bridge deck being connected to the frame legs by rigid connections. The used bridge design is based on a continuous structure that can transfer sectional forces between the different parts of the bridge (Hansson & Tacking, 2024). The type of bridge integrated into the algorithm is the solution of a closed frame bridge based upon the standardized designs used in the railway project Norrbotniabanan (Bruneby et al., 2023). Conceptualization is presented in Figure 3.1.

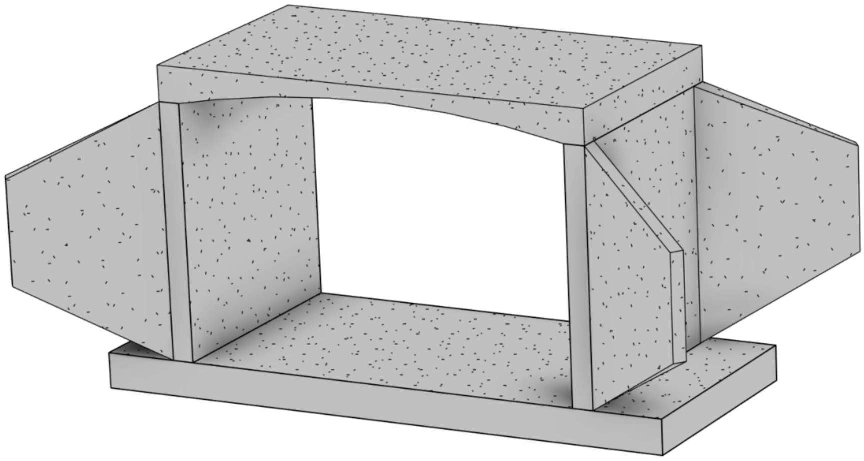


**Figure 3.1:** Conceptualization of the standard closed frame bridge.

The use of closed-frame bridges is often correlated to unfavorable ground conditions, as their closed and long foundation allows the forces to spread over a larger area (Trafikverket, 2008). This makes the closed frame advantageous in such situations. However, since more material is used due to a larger foundation slab, this bridge type is usually used for smaller spans, such as the 12-meter span examined in this study. In cases where larger spans are needed and the advantages of using a closed frame are unnecessary, an open-frame bridge is often used (Trafikverket, 2008).

### 3.2.2 Geometric layout and properties

As defined by the standardized designs (Bruneby et al., 2023), the frame legs and foundation slab have a constant cross-section while the bridge deck is formed with a radius, making the bridge deck thinner towards the center of the span and thicker closer to the connection with the frame legs as shown in Figure 3.2.

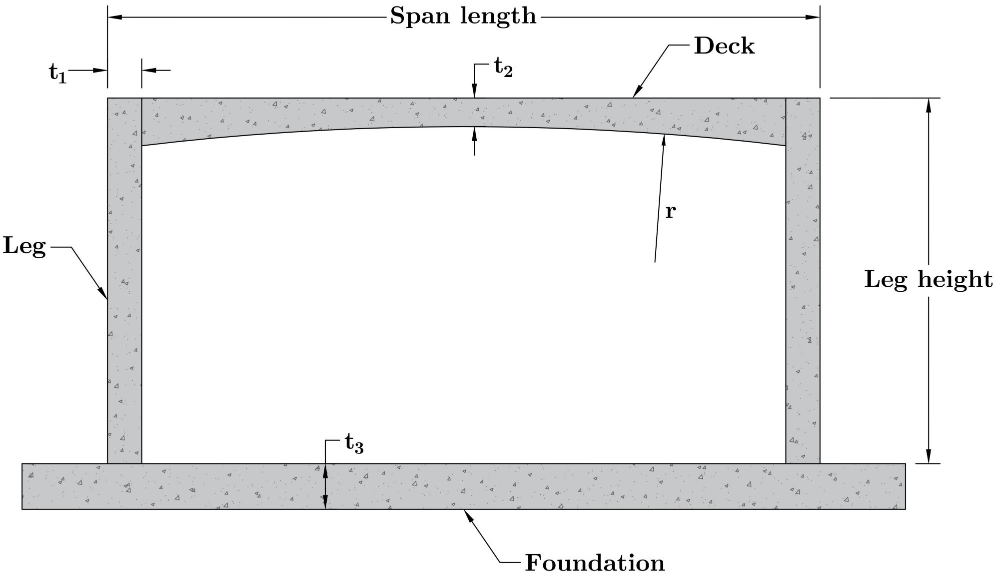



**Figure 3.2:** Section view of studied bridge

The parametrization of the SBD algorithm was made according to the following parameters, see Table 3.1. Properties such as the concrete class remain constant, indicating that it is not a parametric variable. For constant properties, see Table 3.2.

**Table 3.1:** Parametric variables

| **Bridge geometry** | **Input value** | |
|---|---|---|
| Span length | 12000 mm | |
| Width | 7200 mm | |
| Height | 7200 mm | |
| **Variable geometric parameter** | **Range** | **Increment** |
| Leg thickness, $t_1$ | [400,1400] mm | 200 mm |
| Deck thickness (Center), $t_2$ | [400,1400] mm | 200 mm |
| Foundation thickness, $t_3$ | [600,1200] mm | 600 mm |
| Radius beneath the deck, $r$ | [4,10,15]*Span length | Varying |
| Reinforcement size, $O$ | [16,20,25] | Varying |
| Spacing (longitudinal in elements) $s$ | [250,200,180,150,120] | Varying |

**Table 3.2:** Constant variables.

| **Constant parameters** | **Value** |
|---|---|
| Concrete class | C35/45 |
| $\rho_{concrete}$ | 2500 kg/m$^3$ |
| $\rho_{steel}$ | 7800 kg/m$^3$ |
| $\gamma_{soil}$ (Earth pressure at legs) | 20 kN/m$^3$ |
| Friction angle fill material (Only used in EC) | 45° |
| Expansion coefficient $\alpha_e$ | $1 \times 10^{-5}$ |
| Rail spacing, $r_{rail}$ | 1500 mm |
| E-modulus concrete $E_{cm}$ | Defined values according to regulatory framework (based on C35/45) |
| E-modulus steel, $E_s$ | 200 GPa |
| Yield strength reinforcement, $f_{yk}$ | 500 MPa |
| Stirrups size, $O$ | 12 mm |
| Spacing between reinforcement vertical | 40 mm |
| Concrete cover | 35 mm |

Typically, a closed frame bridge is formed with the mentioned parts together with wing walls and edge beams. Since the wing walls do not contribute to the overall structural capacity, they will be disregarded in this study (Hansson & Tacking, 2024). Edge beams will only be considered with the contributing self-weight, not being a parametrized property. For the evaluation described further in Section 3.5 and resulting properties presented in Chapter 6 the foundation slab is not considered, this is described further in Section 3.2.3 and 3.3.1.

### 3.2.3 Boundary and loading condition

Built on the implemented bridge model, boundary and loading conditions are set. The investigated scenario considers the bridge as a single-track railway structure. With a focus on remaining conceptual, the following loads will be applied and studied, see Table 3.3. As the loading conditions are closely related to used regulations, more detailed explanation of the implemented loads is provided in Chapter 4 and 5.

**Table 3.3:** Conceptual loads applied and studied in the bridge model.

| **Load Category** | **Applied Load** |
|---|---|
| **Permanent Loads** | Self-weight |
| | Resting earth pressure |
| | Shrinkage |
| **Environmental Loads** | Temperature loads |
| | Earth pressure due to temp. increase |
| **Live Loads** | Surcharge load |
| | Train load (LM71 or BV2000) |
| | Traction load |
| | Nosing force |

To create a representative model to obtain the structural response, boundary conditions are defined to account for the interaction with the soil. For the earth pressure acting on the frame leg, earth loads are modeled and modified accordingly to the used regulations. To provide a representative model for the frame legs and bridge slab deck, the foundation's boundary conditions with the soil underneath are modeled as strictly rigid in rotation and translation in any direction. This causes however the behavior of the foundation slab to be unrepresentative of its realistic behavior, hence not considering it as mentioned in Section 3.2.2.

## 3.3 Structural response

As mentioned in the general workflow of the SBD-tool, see Section 3.1.2, the structural response is calculated based on geometric properties and conditions. The following text explains the procedure of obtaining the structural response.

### 3.3.1 Finite element software BRIGADE PLUS

In order to calculate the structural behavior of the bridge models, an FE software was used. Brigade/Plus is a structural analysis software that uses the SIMULIA ABAQUS solver (TECHNIA, n.d.). Brigade/Plus can be seen as an extension of the ABAQUS software, specialized for the modeling of civil structures and bridges. The software has the ability to efficiently model live loads and has predefined load models from used design codes in regulations. As previously noted, each parametric design is modeled and the structural response calculations are performed. These calculations and modeling are made by Brigade/Plus, each design is modeled in Brigade and calculated using the ABAQUS solver. As mentioned in the limitations (Section 1.3) of this thesis, the performed FEA is simplified to only study the linear-elastic behavior. Since the study is focused on being applicable in the preliminary phase of bridge design, this approach is determined sufficient. Since the used generative FE-modeling was made based on the previous thesis work by Hansson and Tacking (2024), their made convergence study was determined reliable. Therefore, a mesh size of 200 millimeters was used, see Appendix B in the Master's thesis made by Hansson and Tacking (2024). From Brigade/Plus the following responses are obtained for each design, see Table 3.4. For the FEA shell elements were used with thicknesses that complied with the input geometries defined in the generation step of the SBD-algorithm.

**Table 3.4:** Description of output quantities in the local coordinate system.

| Denotation | Explanation |
|---|---|
| SM1 | Bending moment force per unit width about local 2-axis |
| SM2 | Bending moment force per unit width about local 1-axis |
| SM3 | Twisting moment force per unit width in the local 1–2 plane |
| SF4 | Transverse shear force per unit width in the local 1-direction |
| SF5 | Transverse shear force per unit width in the local 2-direction |
| U3 | Vertical displacement along the local 3-axis |
| U1 | Horizontal displacement along the local 1-axis (longitudinal along the bridge) |

### 3.3.2 Validation of the Finite Element Model

To ensure reliability in the FEA results, a validation process was established to verify the accuracy of the applied FE model. This was achieved by comparing the FEA outputs against simplified analytical calculations. The analytical calculations were made based on attempting to replicate similar modeling assumptions. The

analytical calculations were performed using vectorized calculations in MATLAB, a programming and numeric computing platform (MathWorks, 2026).
The validation process addresses two primary aspect of the model to be investigated. The first aspect evaluates the impact of considering effective cross sections of uncracked concrete (state I). By analytically calculating the increased moment of inertia $I$ contributed by the reinforcement, the impact of neglecting this additional reinforcement stiffness in the FE-model can be quantified. Fundamentally, this step is used to determine whether adopting a gross concrete section in the FEA constitutes a sufficiently accurate assumption for state I behavior within the scope of the thesis, preliminary design.

The second aspect validates the global structural response of the FE model. The global response studied is the maximum longitudinal displacement (along the bridge) of the bridge legs, along with the vertical deflection and maximum bending moment of the deck slab. The analytical baseline for this comparison relies on a simplified 2D rigid frame model, where the frame legs and deck slab are represented as vertical walls and a horizontal beam, respectively. Consistent with the established FE-modeling approach, this 2D frame assumes fully fixed boundary conditions at the base. To simplify, based on engineering assumptions, only specific loads were considered in the analytical calculations. Regarding longitudinal displacement, traction from the train was seen as the main load and shrinkage was also considered. The absolute value of the displacement was taken at the top of the frame legs. The longitudinal displacement from the FEA was extracted based on the maximum horizontal deformation occurring along the height of the legs, measured either at the centerline or 500 millimeters from the edge (Hansson & Tacking, 2024).

Regarding the vertical deflection, for Bro 2004, only the vertical train load was considered. For second generation of Eurocode, the earth load, permanent load and shrinkage were also considered. For both cases, the vertical train load was simplified by applying it as an equivalent distributed load over the entire deck. Additionally, an equivalent moment of inertia was applied across the entire slab, whereas the actual slab features an arching geometry that results in a continuously varying stiffness. The deflection from FEA was extracted based on the maximum deflection occurring either along the centerline of the slab or 500 millimeters from the edge. For the analytical calculations, the maximum vertical deflection was assumed occurring in the center of the span.

Lastly, the maximum moment in the bridge deck slab was calculated using the same assumptions as for the vertical deflection. However, the moment caused by temperature variation was also taken into account. The maximum moment defining the amount of reinforcement needed in the main tension cord in the center segment of the bridge deck was extracted from the FEA. The maximum moment from the analytical calculations was assumed to occur in the center of the span with the simplified equivalent load applied. Moment validation was only made for preformed SBD using the regulations of Bro2004.

### 3.3.3 Post-process of structural behavior

In order to design a reinforced slab element, the sectional performance will be assessed after the FE-analysis has been implemented. This involves determining the actual bending moment ($M_{Ed}$) in a specific direction which must not exceed the bending moment capacity ($M_{Rd}$) in order to be determined feasible. When designing concrete slabs, the torsional moment should be accounted for in determining the design moment in a specific direction (Engström, 2014). Equations 3.1 and 3.2 should be used when the slab gives tension in the bottom cord and the bending moment will be defined as positive.

$$m_{Rdx} \geq m_{Edx} + |m_{Edxy}| \tag{3.1}$$

$$m_{Rdy} \geq m_{Edy} + |m_{Edxy}| \tag{3.2}$$

On the other hand, for the slab regions where the tension occurs at the top cord, the Equations 3.3 and 3.4 should be used instead and the bending moment will be considered as negative moment.

$$m'_{Rdx} \geq -m'_{Edx} + |m_{Edxy}| \tag{3.3}$$

$$m'_{Rdy} \geq -m'_{Edy} + |m_{Edxy}| \tag{3.4}$$

To determine the shear force ($V_{Ed}$) in the slab design, the out-of-plane shear forces ($\upsilon_{Edx}$) and ($\upsilon_{Edy}$) which act on the cross section perpendicular to the x and y axis will be applied as shown the following Equation 3.5 (Plos et al., 2021).

$$V_{Ed} = \sqrt{\upsilon_{Edx}^2 + \upsilon_{Edy}^2} \tag{3.5}$$

### 3.3.4 Bridge model decomposition

The closed frame bridge is divided into six separate segments as presented in Figure 3.3. This is done in order to examine the variation in force and moment distribution in each segment of the bridge, the needed reinforcement will then be designed based on each segments sectional data. The bridge segments is be divided into deck span, deck support, leg upper, leg lower, foundation footing and foundation slab. Validation is however not made for the segments foundation footing and foundation slab as explained in Section 3.2.2 and 3.2.3. For the validation, each segment is checked separately. If one or several segments is not fulfilling the constraints validated upon, a design is determined not feasible.

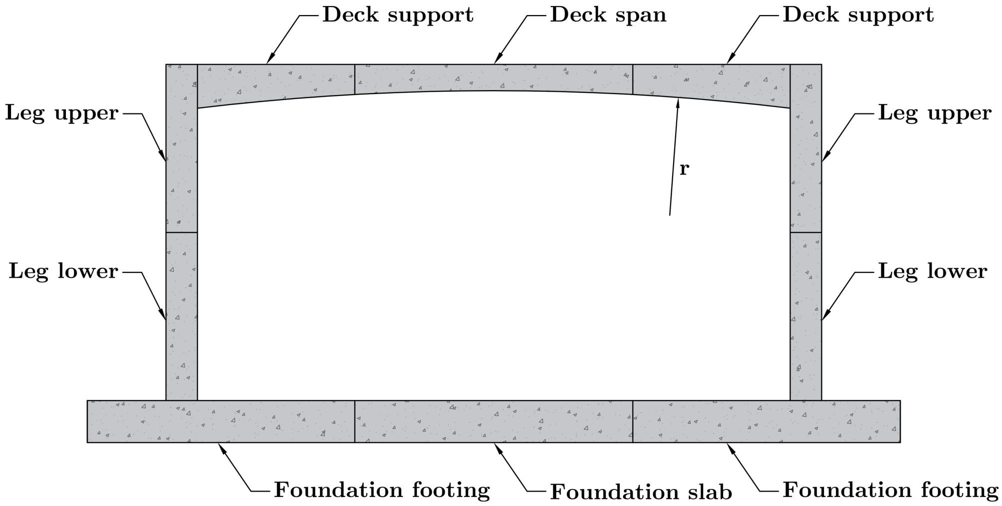



**Figure 3.3:** Frame bridge segments.

The sectional forces and displacement will be extracted from each part as stated in Table 3.4 and will be evaluated to obtain the design value as presented in Section 3.3.3.

## 3.4 Capacity calculations

To define if a design is feasible or not validations are made based on the constraints that are considered. The following section goes into detail about what different constraints are set and how they are used as filters in order to validate a design's feasibility. The constraints of feasibility are closely tied to the used regulations since they often define what absolute values the response is compared against. This section will describe what constraints are set and validated against, for more detail and identified variances between using different regulations, see Section 4 and 5.

### 3.4.1 Ultimate limit state

In ultimate limit state (ULS), the structural capacity is verified under failure conditions. The used failure conditions are obtained from the most unfavorable case generated based on the used load combinations and the live loads placement along the bridge.

#### 3.4.1.1 Moment resistance

Regarding moment capacity, the design is checked using a simplified approach. The moment resistance in the longitudinal (span) and transversal (width) direction is defined by estimating the amount of reinforcement needed to reach sufficient capacity, see Equation 3.6 (Al-Emrani et al., 2013). The amount of compression reinforcement is also estimated based on the maximum value of tension that can occur opposite to the main tension cord side of the cross section.

$$A_s = \frac{M_{\text{Ed}}}{f_{yd}\, z}\, b \tag{3.6}$$

Where

$$\begin{aligned} A_s &= \text{Design reinforcement area} \\ M_{\text{Ed}} &= \text{Design bending moment} \\ f_{yd} &= \text{Design yield strength of reinforcement} \\ z &= \text{Lever arm between compression and tension forces} \\ b &= 1 \text{ m width (to obtain reinforcement amount per meter)} \end{aligned}$$

In addition to the estimated amount of reinforcement, constraints are set based on the number of layers of reinforcement that are needed in order to fulfill capacity. Based on the reinforcement size, horizontal- and vertical spacing, the maximum amount of reinforcement area that is able to exist in one layer is calculated. If the amount of layers needed is higher than 2 in order to fulfill the resistance, the design is determined as not feasible. If 1 or 2 layers are only needed, the solution is determined as feasible. The check in amount of reinforcement layers used is not made for the compression reinforcement. For not feasible designs, the total reinforcement amount in order to fulfill resistance is obtained and used in the evaluation phase.
As described in Section 3.1, combinations of different reinforcement sizes and spacings in different parts of the bridge are not checked. The script checks the entire bridge for a certain setup of reinforcement size and spacing, determining that as one single solution, feasible or not. The script is also limited to not combining different reinforcement sizes and spacings in cross sections, the script checks cross sections using a single reinforcement size and a certain spacing, determining if it is feasible with that layout in 1 or 2 layers of reinforcement bars.

#### 3.4.1.2 Shear resistance

The shear resistance is compared to the subjected force that is active. Firstly, the shear capacity of a non-reinforced cross-section is checked, defining if shear reinforcement is needed or not. If reinforcement is needed, an optimization is made to obtain the necessary amount of shear reinforcement. Since By iterating the solution with a decreasing spacing of stirrups, the highest allowable spacing is provided for each design. The optimization is made based on the following set-up of stirrup spacing, see Equation 3.7.

$$\text{spacing stirrups} = \{\, x_i \,\}_{i=1}^{13}, \qquad x_i = 100 + \frac{i-1}{12}(400 - 100) \tag{3.7}$$

If a design is not able to fulfill enough capacity based on the minimum spacing available in the optimization, the design is seen as not feasible. For unfeasible designs, the amount of shear reinforcement is set to the value when the lowest spacing in the optimization was used. For detailed calculations to obtain the reinforcement amount and shear capacity. See Section 4 and 5.

### 3.4.2 Serviceability limit state

The service limit state (SLS) matters for the structural performance under normal service conditions. Similar to ULS, the most unfavorable case is obtained based on load combinations and live load placement. Based on the preliminary phase of bridge design, the following parts is investigated.

#### 3.4.2.1 Deflection limitations

Displacement checks are made horizontally (longitudinally) and vertically. For longitudinal displacement, the displacement of greatest magnitude along the height of the legs is obtained and compared against a limiting value, see Section 4.4 and 5.4. If the maximum displacement is lower than the value defined in regulations, it is defined as feasible. If not, it is defined as not feasible. For vertical deflection, the greatest magnitude of displacement vertically within the slab deck is checked. Similarly to longitudinal displacement, exceeding a set limit defines whether a design is feasible or not. An important note is the fact that deflections are obtained from linear elastic FE-analyses, meaning that non-linear behavior such as cracks is not considered (Al-Emrani et al., 2011). The linear model provides an overestimation in stiffness, providing an upper-bound approach. As mentioned in Section 3.3.1, the scope is focused on applicability in the preliminary phase of bridge design, making the approach deemed sufficient.

#### 3.4.2.2 Crack width

Restricting crack width is essential for assuring a structures longevity and durability ("In-service cracks", n.d.). Therefore, a limitation in maximum crack widths is set. Limiting values and simplified procedures of calculating crack widths is determined by used regulations, see Section 4.4 and 5.4. Crack width check is only made in the longitudinal direction of each segment. Paths are defined along each segment where data is obtained along each path. The leg data is obtained in steps of 200 millimeters and for the bridge deck data is obtained every 100 millimeters.
Additional to crack width constraints, a check of concrete and steel stress under characteristic load combination is made in the implication of the second generation EC, see Section 4.4.

Supplementary to the crack width and stress limitations, the SBD algorithm provides an adaptivity regarding these constraints. In the case of the stress or crack width limitations not being fulfilled the algorithm adds additional reinforcement to the cross section to check if it makes the design able to fulfill the requirements. In the case of the adaptivity making an unfeasible solution feasible the extra reinforcement is added to the total amount. The check of the allowed number of layers is not made on the extra reinforcement, but the design is commented that additional reinforcement is added. The algorithm adds the reinforcement in steps of 100 mm$^2$/m and the maximum amount of reinforcement the adaptivity is able to add is 1900 mm$^2$/m.

## 3.5 Evaluation parameters

Lastly in the SBD algorithms' procedure the evaluation step takes place. With the input dataset and computed reinforcement amounts the total volume of concrete and reinforcement needed for the different bridge designs is calculated. With the total volumes of material, the investment cost and environmental impact of each bridge design are estimated. The attributes are further explained in the following sections.

### 3.5.1 Investment cost

In prior studies on SBD, the cost definition has mainly been based on material quantities, which were directly converted into cost (Fernández & Tarazona Ramos, 2014; Löfgren, 2020). In the master's thesis made by Bergenram and Ulander (2023) the cost formulation was further expanded by including labor-related construction activities, building on the work of Yavari et al. (2016). The evaluation procedure in Hansson and Tacking (2024) thesis was inspired by Mathern et al. (2018) study of an optimization algorithm for structural designs. Mathern et al. (2018) work defines functions that act as the basis for evaluating design choices, which similarly can be used as evaluation criteria in SBD. In this thesis, only the investment cost component will be considered, particularly concrete, reinforcement and formwork, labor costs will not be included. For used functions, see Equations 3.8 and 3.9. The used function is based on the approach presented in the previous master's thesis (Hansson & Tacking, 2024). See Appendix B for values.

$$INV(x) = INV_m(x) \tag{3.8}$$

$$INV_m(x) = \sum_i \rho_{m,i} \cdot Q_{m,i}(x) \tag{3.9}$$

where

| | |
|---|---|
| $INV(x)$ | Investment cost for bridge cost of material for bridge design $x$. |
| $INV_m(x)$ | Cost of material for bridge design $x$. |
| $\rho_{m,i}$ | Unit price of material $i$. |
| $Q_{m,i}(x)$ | Quantity of material $i$ for bridge design $x$. |

### 3.5.2 Environmental Impact

As mentioned, the Environmental Impact of bridge designs will be used as a parameter of evaluation. Generated designs will be compared by calculating their $CO_2$-eq emissions. In earlier works, the $CO_2$-eq was established as the primary metric for evaluating material amounts (Bergenram & Ulander, 2023; Fernández & Tarazona Ramos, 2014). The possibility of expanding the criterion of environmental impact has also been studied, including a broader range of environmental factors by adapting the ReCiPe method from Yavari et al. (2016). However, since public road and railway projects use $CO_2$-eq to evaluate the performance of environmental impact (Miliutenko, 2022), it will be the property measured. The formulated function by

Yavari et al. (2016) used in the established evaluation is shown in Equations 3.10, 3.11 and 3.12, considering the reinforcement and concrete contribution.

$$E(x) = E_r(x) + E_c(x) \tag{3.10}$$

$$E_c(x) = \sum eq_c \cdot Q_c(x) \tag{3.11}$$

$$E_r(x) = \text{anchorage factor} \cdot \sum eq_r \cdot Q_r(x) \tag{3.12}$$

where:

$E(x)$ = Total $CO_2$-eq emissions for bridge design x
$E_c(x)$ = Total $CO_2$-eq emissions of concrete for bridge design x
$E_r(x)$ = Total $CO_2$-eq emissions of reinforcement for bridge design x
$Q_c(x)$ = Quantity of concrete for bridge design x
$Q_r(x)$ = Quantity of reinforcement for bridge design x
$eq_c$ = kg $CO_2$-eq/kg of concrete
$eq_r$ = kg $CO_2$-eq/kg of reinforcement
anchorage factor = 1.4

# 4

# Assessment According to Second Generation of Eurocodes

*This chapter presents the variable application including the definition of loads, load models, load combinations, key parameters and the assessment of bridge capacity control following the European technical standards (EN 1990 - EN 1999). The standard establishes principles and requirements for the resistance of the structure in relation to the ultimate limit state and the service limit state for the safety, serviceability, and durability of structures (SS-EN 1990:2023, 2023) which are supplemented by national requirements in relation to the local environment (e.g. TRVINFRA, TDOK and TSFS) to ensure the highest level of satisfaction in individual regions.*

## 4.1 Loads

According to SS-EN 1990:2023, 2023, the design of a concrete section must satisfy the capacity requirements of the load combination in both the ULS and SLS. ULS design limits the load of the structure under the collapse load of a section, while SLS design limits long-term consequences such as deflection of the deck slab and the width of cracks to ensure functional and comfortable experiences for users. ULS and SLS load combinations generally consist of permanent load ($G$) and variable load ($Q$) magnified with partial factors for different design situations.

Permanent actions are the loads that maintain the magnitude and position throughout the structure's lifetime. In contrast, the variable actions are characterized by the fluctuation of the load, which varies in magnitude and position during the operational service life. The loads are defined from EC and also the local Swedish regulations presented in Table 4.1 for permanent loads and Table 4.2 for variable loads.

**Table 4.1:** Permanent loads obtained from second generation of EC.

| Load | Description | Reference |
|---|---|---|
| Self weight | Weight of the concrete structure. | SS EN 1991-1-1:2025<br>Annex A.3 |
| Ballast weight | Weight of ballast on the structure. | SS EN 1991-1-1:2025<br>Annex A.3 |
| Earth pressure | Horizontal earth pressure at rest. | TDOK 2013:0668<br>Section 5.2.2.2.3 |
| Asphalt weight | Surface coating on foundation slab. | TRVINFRA-00227<br>Section 7.1.6.1.1 |

**Table 4.2:** Variable loads obtained from second generation of EC.

| Load | Description | Reference |
|---|---|---|
| Traffic load | Vertical load models from<br>the train, load model 71 (LM71). | SS EN 1991-2:2024<br>Section 8.3 |
| Nosing force | Concentrated force acting horizontal-<br>perpendicular at the top of the rail<br>due to train's snaking move. | SS EN 1991-2:2024<br>Section 8.5.2 |
| Traction and<br>Breaking load | Longitudinal load due to train's<br>accelerating and breaking. | SS EN 1991-2:2024<br>Section 8.5.3 |
| Counteracting<br>earth pressure | The reaction of earth pressure acts on<br>the wall due to traction and breaking. | TDOK 2016:0203<br>Section B.3.2.2.2 |
| Thermal effects | Thermal effects due to temperature<br>variations. | SS EN 1991-1-5:2025<br>Section 8.1 |
| Surcharge load | Additional earth pressure, converting<br>vertical train load into horizontal<br>pressure acting on frame legs. | SS EN 1991-2:2024<br>Section 8.10 |

The railway traffic loads in SS-EN 1991-2:2024, 2024 are defined to present the various types of vertical train traffic actions, covering the different loading scenarios which can be applied on standard rail track gauge and wider. The load model studied in this thesis are presented in Table 4.3 below.

**Table 4.3:** Vertical loads obtained from second generation of EC.

| Load Model | Description | Reference |
|---|---|---|
| LM71 | Vertical static loading due to normal<br>rail traffic. | SS EN 1991-2:2024<br>Section 8.3.2 |

In addition to the train load model, according to SS-EN 1991-2:2024, 2024, there is a classification factor $\alpha$ for the train load which represents cases where the rail traffic is heavier or lighter than the normal rail traffic load. The $\alpha$-factors' value is specified by the national authority. In Sweden, the factor is set by the NA to 1.33 according to Transportstyrelsen, 2018, for normal line rail traffic.

The factor $\phi$ is also an additional magnifying factor on the static train load analysis whose purpose is to increase stresses and vibration due to dynamic behavior. The used factor $\phi_2$ was obtained according to SS-EN 1991-2:2024, 2024, assuming a carefully maintained track, see Equation 4.1 below.

$$\phi_2 = \frac{1,44}{\sqrt{L_\phi} - 0,2} + 0,82 \tag{4.1}$$

The shrinkage strain ($\varepsilon_{cs}$), consisting of basic shrinkage and the drying shrinkage, is also considered and can be calculated following the equations in Annex B.6 in EC in SS-EN 1992-1-1:2023 (2023).

## 4.2 Load combinations

The combination of actions for persistent and transient (fundamental) design situations in ULS load combination is defined by formula (8.13) in SS-EN 1990:2023, 2023 which can be separated into two design of actions as shown in Equations 4.2 and 4.3. The Eurocodes recommend using the most adverse of those two equations in order to maximize the possible load cases. The expressions generally integrate the **permanent actions** with the **leading variable actions**, which is considered as a full design load action to represent the primary load. It is also necessary to take into account for the **accompanying variable actions** to reach their maximum intensity. However, actions defined as secondary loads need to be scaled down to represent realistic situations. By using the combination factor ($\psi$), reflecting the lower probability of peak load occurrence at the same time as the leading action, the intensity is scaled down.

$$\sum F_{\mathrm{d,ULS}} = \sum_i (\gamma_{\mathrm{inf},i}, \gamma_{\mathrm{sup},i})\gamma_{\mathrm{G},i} G_{\mathrm{k},i} + \gamma_{\mathrm{Q},1}\psi_{0,1} Q_{\mathrm{k},1} + \sum_{j>1} \gamma_{\mathrm{Q},j}\psi_{0,j} Q_{\mathrm{k},j} + (\gamma_{\mathrm{P}} P_{\mathrm{k}}) \tag{4.2}$$

$$\sum F_{\mathrm{d,ULS}} = \sum_i (\gamma_{\mathrm{inf},i}, \gamma_{\mathrm{sup},i})\xi_i \gamma_{\mathrm{G},i} G_{\mathrm{k},i} + \gamma_{\mathrm{Q},1} Q_{\mathrm{k},1} + \sum_{j>1} \gamma_{\mathrm{Q},j}\psi_{0,j} Q_{\mathrm{k},j} + (\gamma_{\mathrm{P}} P_{\mathrm{k}}) \tag{4.3}$$

NOTE: the value of partial factors can be found in Appendix A.
where

| | |
|---|---|
| $F_d$ | represents the design value of an action. |
| $\Sigma$ | denotes the combination of the enclosed variables. |
| $\gamma_{G,i}$ | is the partial factor for permanent action $i$. |

$G_{k,i}$ is the characteristic value of permanent action $i$.
$\gamma_{Q,1}$ is the partial factor for the leading variable action 1.
$\psi_{0,1}$ is the combination factor for the leading variable action 1 (if applied).
$Q_{k,1}$ is the characteristic value of the leading variable action 1.
$\gamma_{Q,j}$ is the partial factor for the variable action $j$.
$\psi_{0,j}$ is the combination factor for the variable action $j$.
$Q_{k,j}$ is the characteristic value of an accompanying variable action $j$.
$P_k$ is the characteristic value of a prestressing force.
$\gamma_P$ is the partial factor for the prestressing forces.
$\xi$ is a reduction factor applied to the unfavourable permanent actions only.
$\gamma_{sup,j}$ is the partial factor for the variable action $j$.

Furthermore, Transportstyrelsen, 2018 states that load cases considered in ultimate state design shall also account for the safety class with the partial factor ($\gamma_d$). In this study safety class 3 is considered, with a value of 1.0, stated for railway bridges.

It is also necessary to consider the structural behavior under the service state to evaluate the long-term effects such as slab deflection or the crack width responses. According to SS-EN 1990:2023, 2023, the design situation in SLS can be considered utilizing the characteristic, frequent and quasi-permanent load combinations for different loading situations as presented in Equations 4.4, 4.5 and 4.6 respectively.
The **characteristic load combination** can generally be calculated as the equation below for the specific rare events to verify the structural capacity when the action could cause irreversible damage.

$$\sum F_{\mathrm{d,SLS,char}} = \sum_i (\gamma_{\mathrm{inf},i}, \gamma_{\mathrm{sup},i}) G_{\mathrm{k},i} + Q_{\mathrm{k},1} + \sum_{j>1} \psi_{0,j} Q_{\mathrm{k},j} + (P_{\mathrm{k}}) \tag{4.4}$$

If the consideration is performed under a certain load situation that has a higher probability of occurrence, the **frequent load combination** is recommended according to the Eurocodes for such load case. This commonly requires lower load magnitudes in order to fulfill the reversible damage limit.

$$\sum F_{\mathrm{d,SLS,freq}} = \sum_i (\gamma_{\mathrm{inf},i}, \gamma_{\mathrm{sup},i}) G_{\mathrm{k},i} + \psi_{1,1} Q_{\mathrm{k},1} + \sum_{j>1} \psi_{2,j} Q_{\mathrm{k},j} + (P_{\mathrm{k}}) \tag{4.5}$$

where
$\psi_{1,1}$ is the combination factor for the leading variable action 1 to determine its frequent value.
$\psi_{2,j}$ is the combination factor for the accompanying variable action $j$ to determine its quasi-permanent value.
For the permanent loads and service actions that are almost always present on the structure (highest reoccurrence), which will be utilized to consider for the long-term structural behaviors such as creep effects or long-term crack widths, the **quasi-permanent load combination** shall be applied as follows.

$$\sum F_{\mathrm{d,SLS,quas}} = \sum_i (\gamma_{\mathrm{inf},i}, \gamma_{\mathrm{sup},i}) G_{\mathrm{k},i} + \sum_{j>1} \psi_{2,j} Q_{\mathrm{k},j} + (P_{\mathrm{k}}) \tag{4.6}$$

The partial factors according to the different loading situations can be found in Appendix A.

## 4.3 Ultimate limit state verification

As stated in Section 5.1, ULS is a design condition where the load on a structure induces the collapse or other similar forms of structural failures. The ultimate state of a reinforced concrete slab begins when yielding in reinforcing bars occurs in one or several critical sections and continues while the load increases until the slab cannot carry any further load, at this point, the collapse mechanism is developed (Engström, 2014). The ultimate state analysis then requires the reinforcement detailing and arrangement in order to evaluate the ultimate moment capacity and shear force of a concrete section.
According to SS-EN 1990:2023, 2023, when a section in the ULS is determined, the design value of the effective actions ($E_d$) induced by ULS load combination according to the Equation 4.2 and 4.3 must be verified by the Equation 4.7 relates to the design value of the resistance ($R_d$) obtained from Section 3.4.1.1.

$$E_d \leq R_d \tag{4.7}$$

The shear detailed verification might be omitted if shear stress of actions ($\tau_{Ed}$) satisfies both the minimum concrete shear resistance ($\tau_{Rdc,min}$) and the concrete shear resistance ($\tau_{Rdc}$) in accordance with SS-EN 1992-1-1:2023, 2023 as presented in Equations 4.8 and 4.9 below.

$$\tau_{Ed} \leq \tau_{Rdc,min} \tag{4.8}$$

$$\tau_{Ed} \leq \tau_{Rdc} \tag{4.9}$$

However, the minimum shear reinforcement is still required to ensure distributed cracking and to handle the forces from the restrained deformations which should be calculated as the reinforcement ratio according to the following formula.

$$\rho_{w,min} \geq \frac{0.08\sqrt{f_{ck}}}{f_{yk}} \tag{4.10}$$

Otherwise, it is necessary to design the shear reinforcement to gain the shear resistance ($\tau_{Rd}$) that fulfills the applied shear stress ($\tau_{Ed}$) providing the shear stirrups with sufficient coverage stress over the excessive shear from the concrete capacity which must be verified by the Equation 4.11.

$$\tau_{Ed} \leq \tau_{Rd} \tag{4.11}$$

## 4.4 Service limit state verification

The behavior of concrete elements under the service state can generally be divided into uncracked and cracked states according to what behavior is active, based on the active stress. In the uncracked state (state I), the concrete stresses are small enough to practically predict the behavior under linear relationship of the stress-strain curve with relatively small contribution of reinforcement. After cracking (state II), the reinforcement will be fully utilized in order to handle the tensile stress in the cracked section (Engström, 2015). However, as stated in Section 3.3.1, the study is limited to linear analysis for the evaluation of deflections under the different service load scenarios. The crack width calculations, for long-term conditions, are considered under the cracked state based on stress conditions in the reinforcing steel with limiting values based on assumed exposure class XD3 and lifespan L100.

Under the serviceability assessment, the short-term heavy load effects are significant to be considered. The action occurs at a very short time and unlikely but results in high consequences on the bridge structures. The consideration is generally examined under the characteristic load combination to verify the short-term structural responses. According to SS-EN 1992-1-1:2023, 2023, the stresses in reinforcement and concrete shall be limited to avoid yielding in the steel bars during crack occurrence as well as to verify the durability verification for concrete stresses as presented in Equations 4.12 and 4.13 below.

$$\sigma_s \leq 0.8 f_{yk} \tag{4.12}$$

$$\sigma_c \leq 0.6 f_{ck} \tag{4.13}$$

For long-term service control, the slab's vertical deflection and longitudinal deformation will be limited according to Trafikverket, 2025c and SS-EN 1991-2:2024, 2024 under the frequent load combination ensuring users' comfort during the service life.

To evaluate sectional capacity beyond Stage I (post-cracking), crack control must be considered under the action of quasi-permanent load with the reference crack width control from The Swedish Transport Agency's regulations (Transportstyrelsen, 2018). However, since the sectional capacity is typically influenced by the creep effect after a long time of service actions, it is necessary to utilize the modulus of elasticity as the effective value ($E_{c,eff}$) found in Equation 9.1 in SS-EN 1992-1-1:2023 (2023), which accounts for creep effects in the calculations. However, the approximation for the primary concrete modulus of elasticity ($E_{cm}$) can be determined from Equation 5.1 in SS-EN 1992-1-1:2023, 2023. The calculation corresponds to the aggregate size constant ($k_E$) which can be taken between 5000 and 13000 based on the defined range, the median value of 9000 will be used.

The calculation for the crack control can be found in Equation S.6 in SS-EN 1992-1-1:2023, 2023, which gives the reinforcement size that satisfies the limited crack width specified in Transportstyrelsen, 2018. As stated in Section 3.4.2.2, the implementation begins with a selected bar diameter, checking whether the calculated bar diameter is larger than the current bar size. If not, additional reinforcement area gets added to the calculation (equation S.6). The adaptivity adds 100 $mm^2/m$ per iteration. If a sufficient bar size can not be calculated to surpass the current bar size after 19 iterations, the process terminates the calculation and results in an unfeasible design in crack control for the considered section. This means that a total of 1900 $mm^2/m$ can be added before terminating.

As presented in Table 4.4, the serviceability verification limitations are defined in accordance with the National Annexes and Eurocodes as follows.

**Table 4.4:** SLS verification limitations defined by documents relevant to second generation EC.

| **Verification** | **Requirement** | **Reference** |
|---|---|---|
| Vertical Deflection | L/800 | TRVINFRA-00227<br>Section 8.2.4 |
| Longitudinal Displacement | 5 mm | SS EN 1991-2 2024<br>Section 8.5.4.5.2 |
| Crack width | 0.15 mm | TSFS 2018:57<br>Kap. 12 |

# 5

# Assessment According to Previous Swedish Regulations

*This chapter describes the different applications of the regulations in Bro 2004 and supplementing documents. As mentioned before, only relevant regulations for preformed preliminary design covered in the scope of the thesis are studied, see Chapter 3. The following regulations studied are Bro 2004 (Vägverket, 2004), BV Bro 7th edition (Banverket, 2004) and BBK 04 (Boverket, 2004), laying out a simplified approach of attempting to replicate the procedure of calculations for the then-current era. The chapter will also highlight differences and similarities compared to second generation EC for the studied aspects.*

## 5.1 Loads

The definition of loads in regulation according to Bro 2004 (Vägverket, 2004) and BV Bro 7th edition (Banverket, 2004) is similar to how loads were identified in Section 4.1 and 4.2, as the method of partial coefficients is similarly used. However, the definition of SLS and ULS is not directly stated the same as in Eurocode with general formulas, but instead categorized into several load combinations directly. As mentioned in Section 3.2.3, the described loads in Table 3.3 are used in the scope of the thesis. The loads are defined from the following parts in the older regulations, see Table 5.1 and 5.3.

**Table 5.1:** Permanent loads obtained from Bro 2004.

| **Load** | **Reference** |
|---|---|
| Self weight | Bro 2004 21.12 |
| Ballast weight | Bro 2004 21.13 |
| Earth pressure | Bro 2004 21.13 |
| Asphalt weight | Bro 2004 21.12 |

**Table 5.2:** Variable loads obtained from Bro 2004.

| Load | Reference |
|---|---|
| Nosing force | BV Bro 7th edition Section 21.2222 |
| Traction and Breaking load | BV Bro 7th edition Section 21.2221 |
| Counteracting earth pressure | Bro 2004 Section 21.23 |
| Thermal effects | Bro 2004 Section 21.26 |
| Surcharge load | BV Bro 7th edition Section 21.224 |
| Train Load (load model BV 2000) | BV Bro 7th edition Section 21.22 |

In Addition to the defined loads, modifying factors are also presented in order to take account of dynamic effects. According to BV Bro 7th edition 21.2216 (Banverket, 2004), the dynamic coefficient added to the vertical train load is defined as shown in Equation 5.1. Noteworthy is also the fact that no factor similar to $\alpha$ in Eurocode is applied to variable loads.

$$D = 1 + \frac{4}{8 + L_{\text{best}}} \tag{5.1}$$

The account of shrinkage is made according to BBK04 chapter 2.4.6 where the end-shrinkage $\varepsilon_{cs}$ is set to $0.25 * 10^{-3}$ (Boverket, 2004), normal outside environment assumed.

## 5.2 Load combinations

Similarly to the approach described in Section 4.2, the partial coefficient method is also used in the application of the older regulations surrounding and within Bro 2004 (Vägverket, 2004). However, the load combinations are defined as individual load combinations instructed to be used depending on what behavior of the bridge that is looked upon. The combinations are ordered numerically from one to nine. Relevant combinations for the behaviors considered in this thesis are shown in Table 5.3.

**Table 5.3:** Structural responses and asserted load combinations according to BV Bro 7th edition.

| **Structural response** | **Load combination** |
|---|---|
| Ultimate limit state | IV:A |
| Ultimate limit state (Dominating permanent loads) | IV:B |
| **Serviceable limit state** | |
| Longitudinal displacement | V:A |
| Crack width | V:B |
| Deflection | V:C |

Consistent with the usage of partial- and combination factors explained in Section 4.2, a similar approach is described in the older regulation revolving around Bro 2004. Similarities between the regulations can be observed within the same usage of leading/accompanying variable action and reduction of permanent actions depending on whether favorable or not. However, the factors differ between the different regulations. For the older regulations, a simplified definition of pre-stated $\psi * \gamma$ values is used. The load coefficients used in Bro 2004 (Vägverket, 2004) and BV Bro 7th edition (Banverket, 2004) can be found in Appendix A.

## 5.3 Ultimate limit state verification

For the simplified approach described in Section 3.4.1.1 and 3.4.1.2, a resembling definition between the older regulations and Eurocode is described, with the base premise of effective actions verified against the design value of resistance, see Section 4.3. One major detail however, is the difference in definition of design strengths in BBK 04 (Boverket, 2004). The partial coefficient for safety class is applied to the design value of resistance. In Eurocode the coefficient is instead applied to the loads, contributing towards the effective action instead. The definition of reinforcement design strength and value of safety class coefficient is described in BBK04 Section 2.3.1 and 2:115 respectively (Boverket, 2004), see Equation 5.2.

$$f_d = \frac{f_k}{\eta\,\gamma_m\,\gamma_n} \tag{5.2}$$

| | |
|---|---|
| $f_d$ | design value of strength |
| $f_k$ | characteristic value of strength |
| $\eta = 1.2$ | Modification factor for concrete to consider differences in properties between test specimen and construction |
| $\gamma_m$ | partial factor for carrying capacity |
| $\gamma_n = 1.2$ | partial factor for safety class, class 3 |

For determining the shear resistance, the procedure described in BBK04 3.7.3 and 3.7.4 is used (Boverket, 2004). Likewise, as previously, the shear resistance definition is also determined by a resistance without shear reinforcement, a minimum shear reinforcement and resistance with shear reinforcement, see Section 4.3 and Equations 4.8, 4.9 and 4.11. The procedure in BBK04 Section 3.7.3 and 3.7.4 is also identified correspondingly to the equations according to SS-EN 1992-1-1:2005 Section 6.2 (Swedish Standards Institute, 2005). The equations are rewritten in other terms in BBK04, meaning that the method used in the regulations in BBK04 is the same for the current Eurocode.

## 5.4 Service limit state verification

As mentioned before, the longitudinal displacement of walls, the vertical deflection of the slab deck and simplified crack width calculations are made in SLS to determine designs feasible. For deflection and crack width limitations, the following limiting values are set with corresponding sources from BV bro 7th edition (Banverket, 2004) and Swedish standard (Swedish Standards Institute, 2002), see Table 5.4. For the crack width limitation, an exposure class of XD3 and lifespan L100 is assumed.

**Table 5.4:** SLS verification limitations defined by documents relevant to Bro 2004.

| **Verification** | **Requirement** | **Reference** |
|---|---|---|
| Vertical Deflection | L/800 | BV Bro 7th edition Section BV 12.421 |
| Longitudinal displacement | 5 mm | BV Bro 7th edition Section BV 12.427 |
| Crack width | 0.10 mm | SS 137010 chapter. 7 |

For crack width calculations, fully cracked concrete sections are taken into account by considering state II concrete. For long term effects taken into consideration shrinkage is considered (see Section 5.1) and creep. As stated in Bro 2004 section 42.14 (Vägverket, 2004), the creep coefficient is set to a value $\phi = 2$. Creep is taken into account by an effective elasticity modulus, see Equation 5.3.

$$E_{eff} = \frac{E_{ck}}{\phi} \tag{5.3}$$

Worth mentioning is the difference in definition of Young's modulus, while Eurocode determines the usage of a mean elasticity modulus, Bro 2004 and BBK04 determine a characteristic value. According to Svenska Betongföreningen (2012), it is stated that these can be seen as equivalent.

Implementation of crack width calculations was made according to BBK04 chapter 4.5.5 (Boverket, 2004). For estimation of effective concrete area for plates in bending, the definitions of effective tension areas in SS-EN1992-1-1:2005 Figure 7.1 were used (Swedish Standards Institute, 2005). The adaptation of the SBD crack width calculation was made similarly to how it was explained in Section 4.4. However, instead of a corresponding bar diameter, the limiting crack width is directly compared to a calculated crack width, where reinforcement area is used and added if calculations provide too high a crack width.

# 6

# Resulting design space and feasible designs.

*The following chapter presents the resulting generation of the design space as well as feasible designs provided by the SBD algorithm. The chapter also presents exploratory data analysis charts in order to visualize differences in the feasible designs between the use of the different regulations (Tandiallo, 2024).*

## 6.1 Design spaces

With the described properties and geometries of the concrete frame bridge according to Section 3.2, 216 different bridge designs were generated and run through Brigade to obtain the structural behavior. As described in Section 3.1, the initial set was expanded based on reinforcement size and spacing, resulting in a total of 3240 designs. After the verification procedure, the resulting feasible space consisted of 587 designs for the second generation of EC and 482 designs for the regulations according to Bro 2004.

## 6.2 Comparison - Material cost

From the evaluation, the material cost including framework, reinforcement and concrete cost was calculated. Figure 6.1 shows the smoothed probability density of normalized cost. The normalization is made based on the design with the lowest cost from either Eurocodes design space or Bro 2004:s design space. To approximate the continuous probability distribution of solutions kernel density estimation (KDE) was used (Diallo et al., 2018). Additionally, Figure 6.2 and 6.3 show detailed views of each regulatory framework's distribution. The detailed views provide normalized histograms, KDE curves as well as a rug plot showing the location of each design's cost. The feasible design curves are scaled down based on the total number of solutions to visualize the lesser amount of solutions.

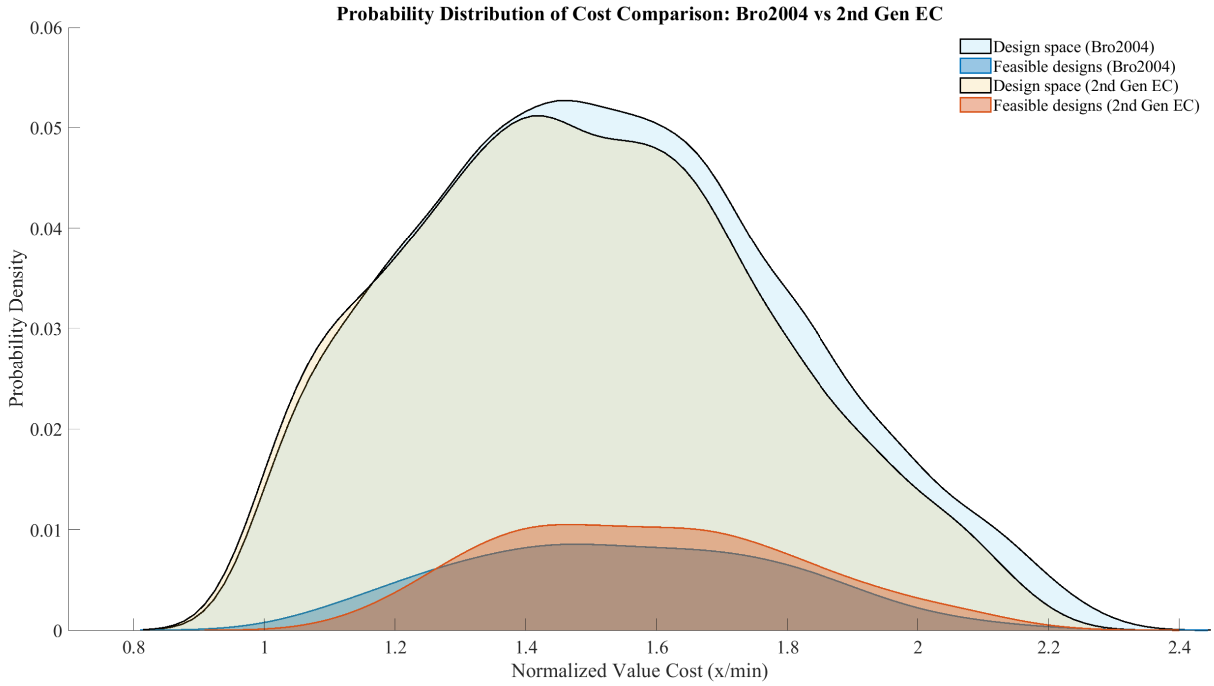



**Figure 6.1:** Probability density comparison of regulations, normalized material cost of initial and feasible design space.

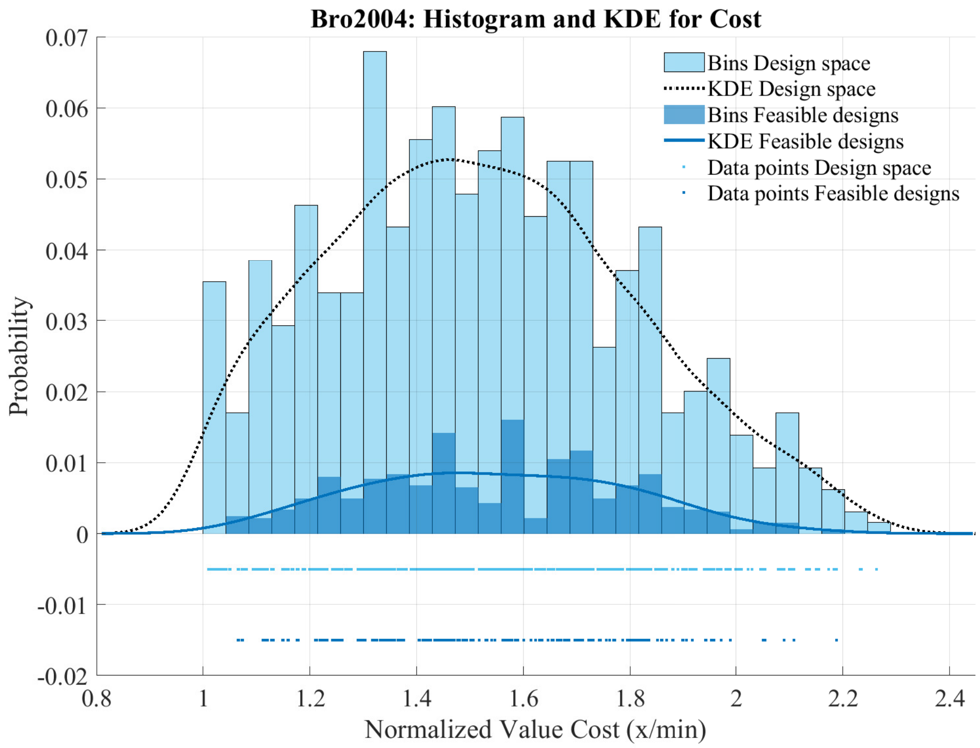



**Figure 6.2:** Detailed view of probability density for cost using Bro 2004.

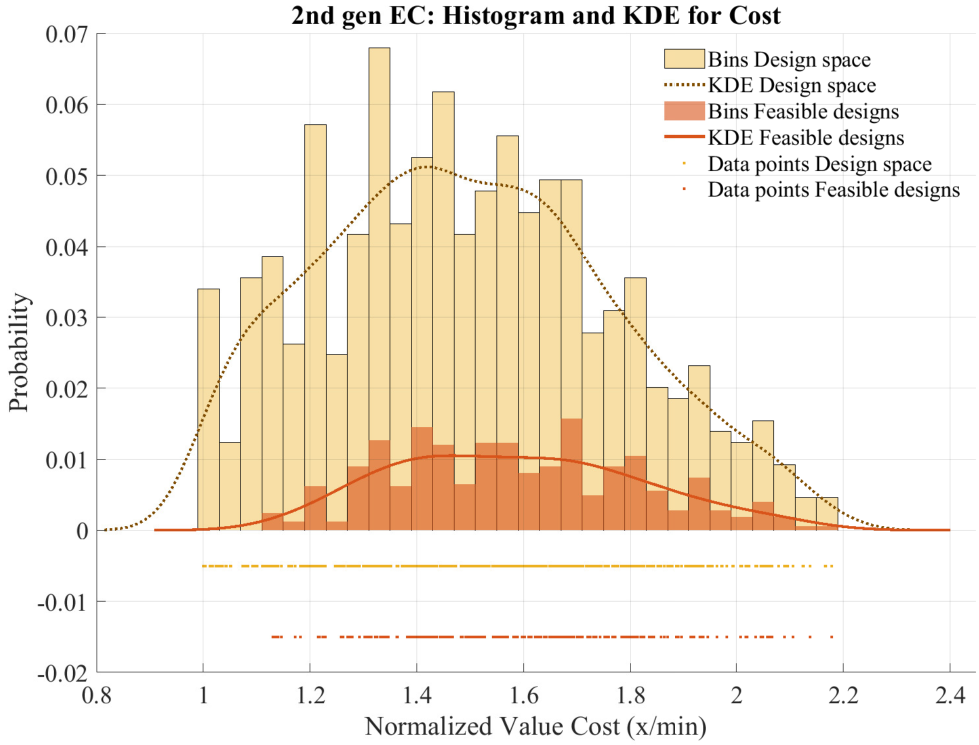



**Figure 6.3:** Detailed view of probability density for cost using second Generation EC.

The result shown in Figure 6.1 displays that the feasible designs using the regulations from Bro 2004 provide a greater number of solutions with lower cost than the second generation of Eurocode. The result also shows that the initial design space for Bro 2004 provides a higher density of more costly solutions. However, from a more general view, the result indicates a similar result between the different regulations. The feasible design space and total design space show no discernible difference.

## 6.3 Comparison - $CO_2$-equivalent

Derived from the evaluation as explained in Section 3.5.2, concrete and reinforcement volumes were calculated and converted into $CO_2$-eq values. The smoothed probability density of normalized cost of the design spaces is shown in Figure 6.4. Normalization and continuous probability are derived similarly as described in Section 6.2. Figure 6.5 and 6.6 are also provided with a detailed view of each regulatory framework's distribution. Shown data is the same as described in Section 6.2.

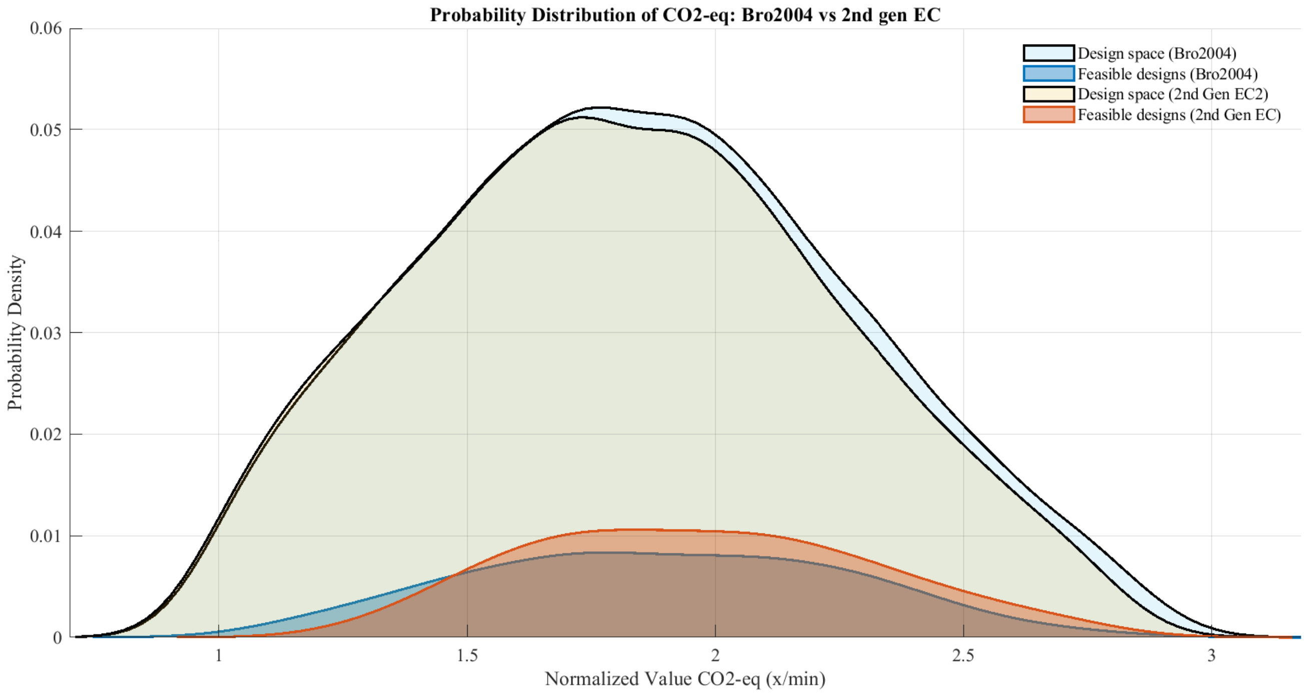



**Figure 6.4:** Probability density comparison of regulations, normalized $CO_2$-eq of initial and feasible design space.

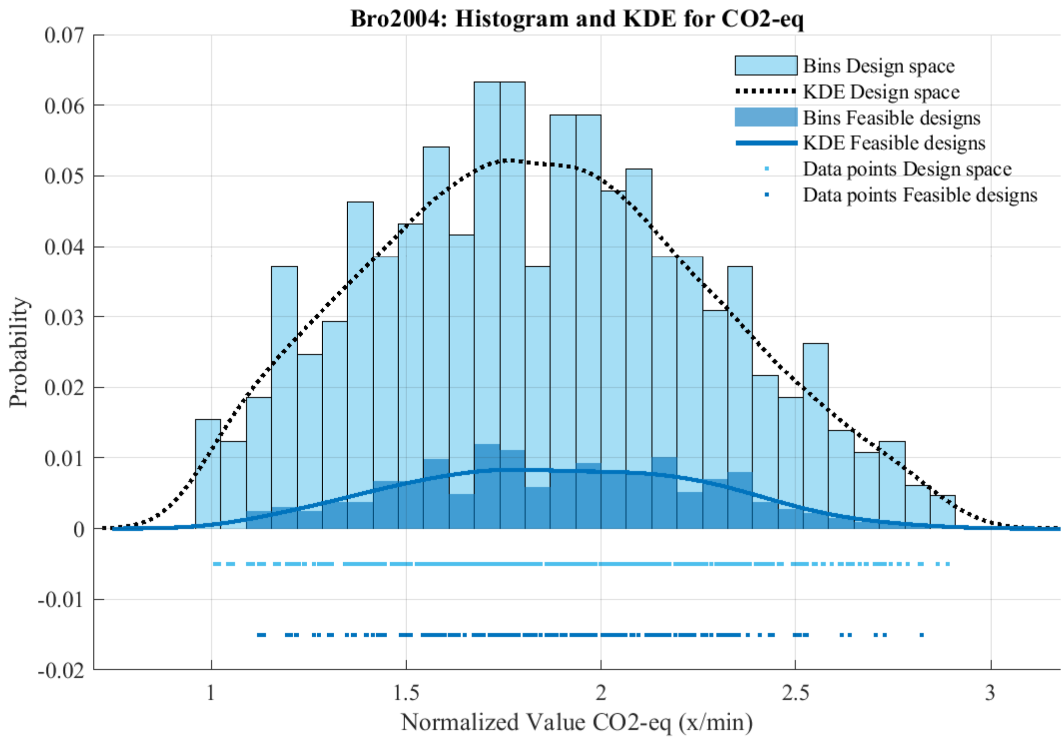



**Figure 6.5:** Detailed view of probability density for $CO_2$-eq using Bro 2004.

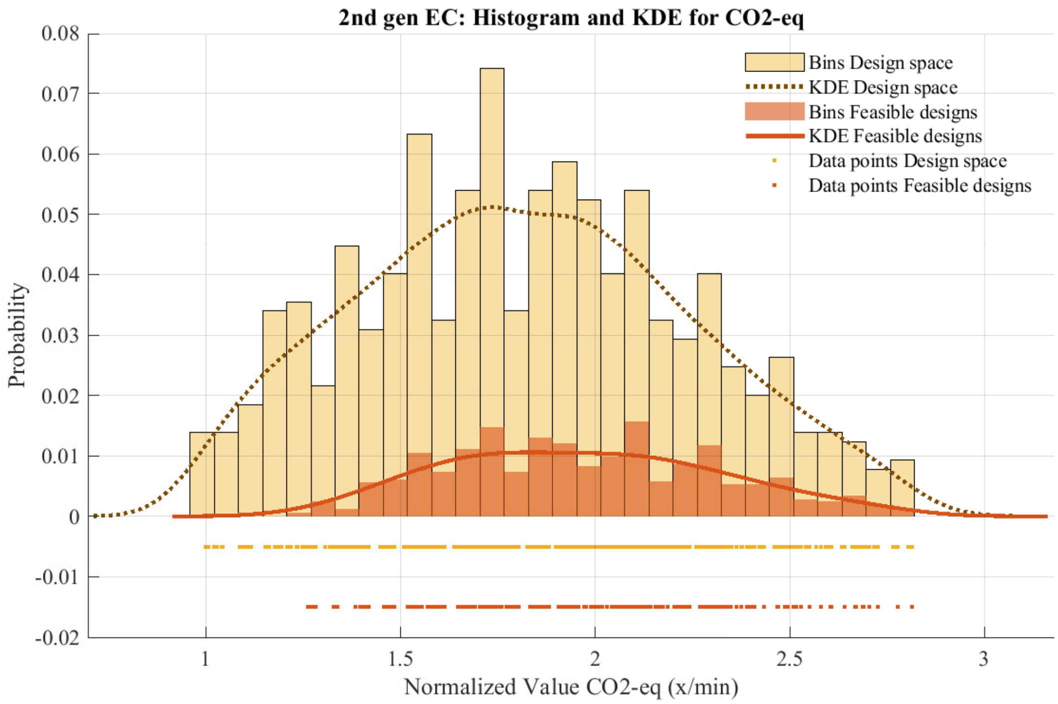



**Figure 6.6:** Detailed view of probability density for $CO_2$-eq using second Generation EC.

The resulting distribution is very similar to the tendencies shown in the cost distribution. For the feasible designs, the distribution is almost identical. However, the normalization of solutions does provide a larger range for the entire design space. Looking at the more detailed views in Figure 6.6 and 6.2, the distribution of solutions appears more distributed compared to cost, following the KDE better.

## 6.4 Structural response and resulting material properties

Throughout the validation process of the SBD algorithm data used along the way were gathered and stored. The behavior of each bridge design is present in the form of box plots, illustrating the median and value domain of different responses and properties that are calculated. The vertical deflection and longitudinal displacement obtained through the FEA are shown in Figure 6.7. The figure shows the results from all the designs run through the FEA. The vertical displacement is taken as the maximum value achieved in the concrete slab deck. The longitudinal displacement is taken as the maximum value achieved along the height of the frame legs.

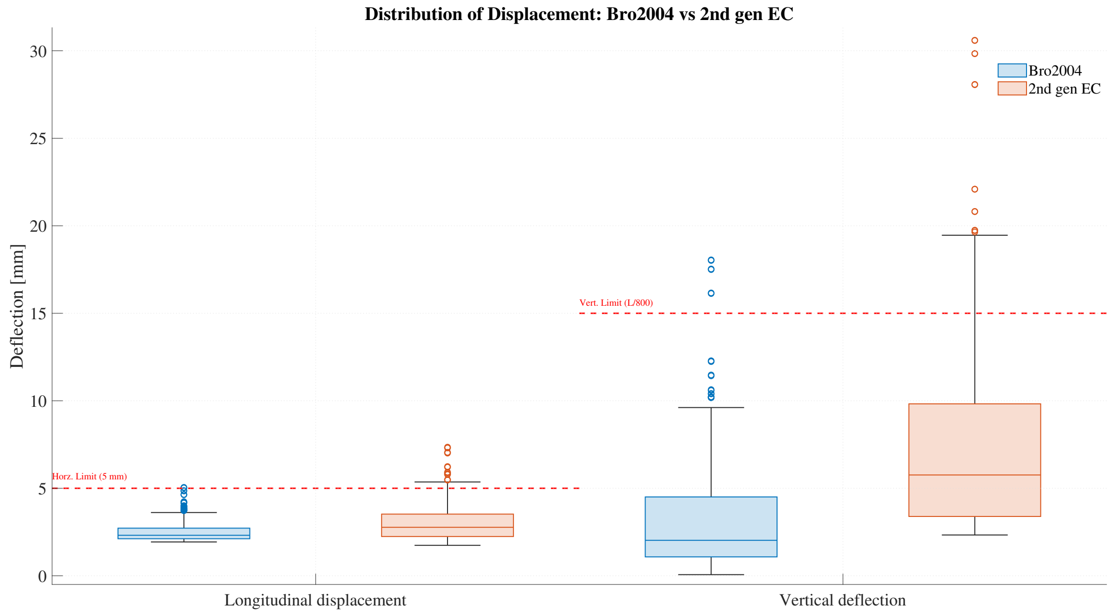



**Figure 6.7:** Box-plots of designs' vertical deflection and longitudinal displacement.

The resulting deflection indicates that the calculated longitudinal displacement and vertical deflection both have higher values when using the regulations of second generation of Eurocode.

The maximum moment in ULS for each studied segment is shown in Figure 6.8. The maximum moment is the response used in determining the amount of reinforcement on the tension side of each cross-section. The figure shows the distribution in feasible designs.

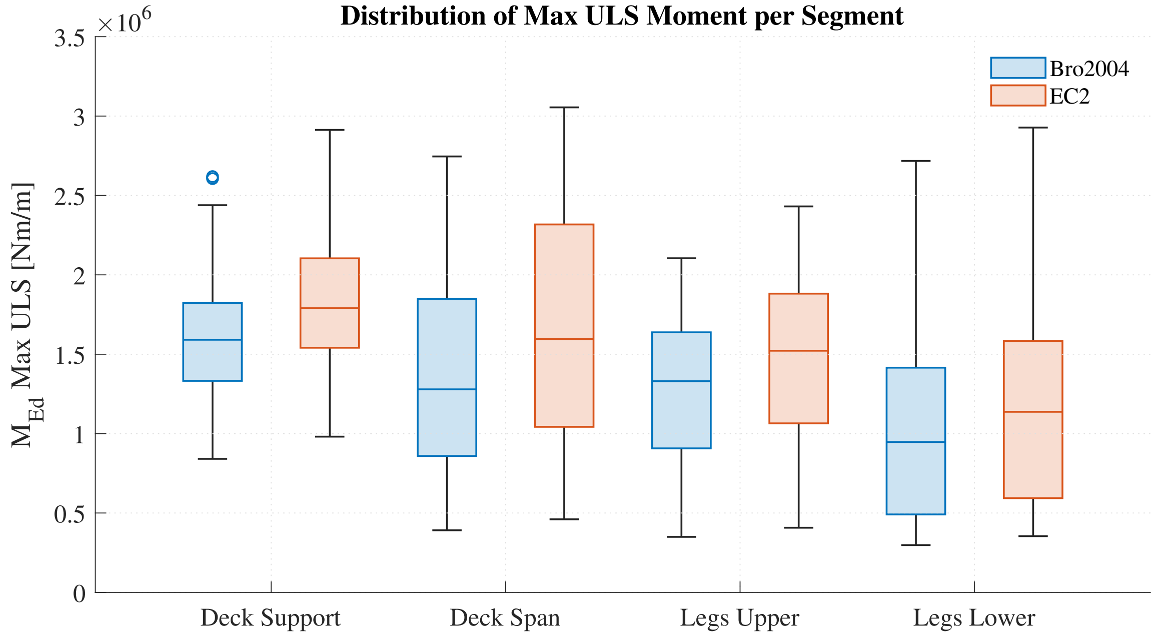



**Figure 6.8:** Box-plots of feasible designs' maximum moment in ULS in different segments.

The result shows that the moment for regulations according to Eurocode provides a slightly higher amount of moment in all segments. The amount of required reinforcement area on the tension side in the longitudinal direction for each section is provided in Figure 6.9. The reinforcement amount is estimated per meter width of the bridge, assuming that the response of the structure checked remains constant throughout the width of the bridge. The figure shows the distribution in feasible designs.

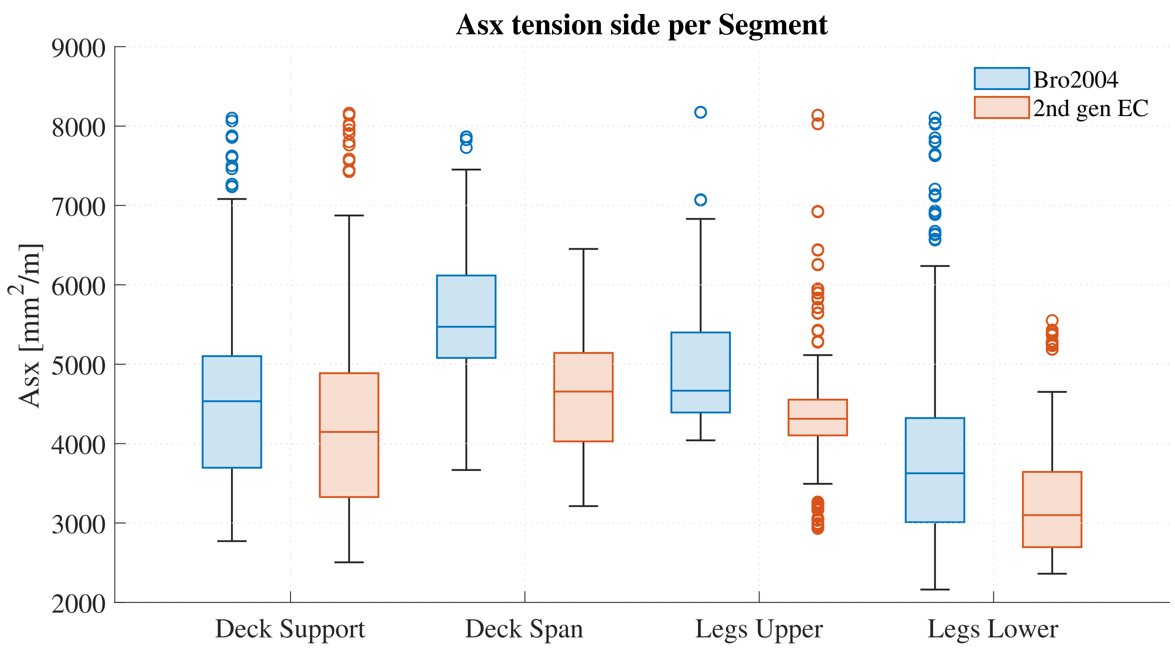



**Figure 6.9:** Box-plots of feasible designs reinforcement amounts in different segments.

For reinforcement amounts, a more deviated distribution can be seen. The amount of reinforcement is also higher for the regulations of Bro 2004 compared to the regulations of EC.
In close connection to the amount of reinforcement shown in Figure 6.9 the amount of added reinforcement from the adaptivity in the crack width calculations is shown in Figure 6.10. The data is visualized in a swarm chart, showing the number of designs with added reinforcement noted as n.

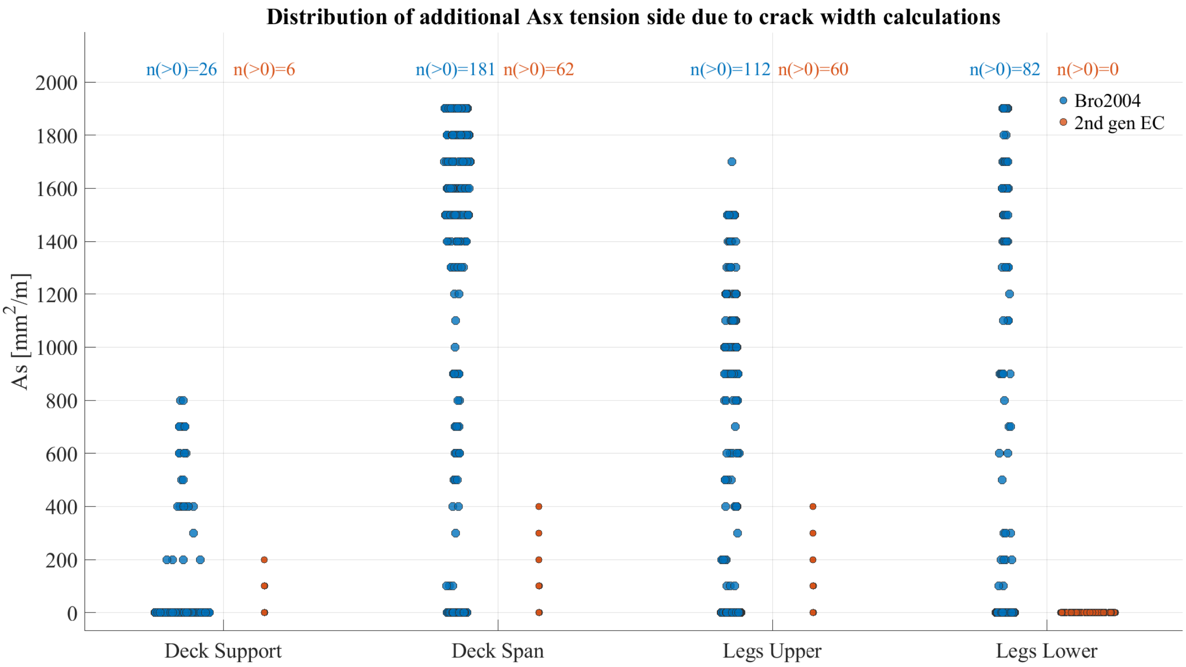
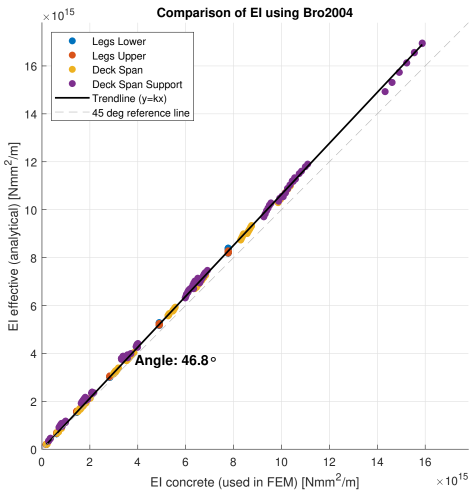



**Figure 6.10:** Swarm chart of the value of added reinforcement in crack width calculations as well as the quantity of solutions for which additional reinforcement is needed.

The swarm chart clearly indicates a difference in the amount of reinforcement added in crack width calculations. Using Bro 2004 regulations clearly shows more solutions needing additional reinforcement compared to using EC. Hence, contributing to the total reinforcement shown in Figure 6.10. The result in Figure 6.10 also shows solutions for Bro 2004 reaching the maximum amount of allowed added reinforcement in the adaptivity of crack width calculations, indicating that the limit of added reinforcement might be a reason for making designs unfeasible. This observation is not valid when looking at EC, where none of the feasible solutions reach the limit of the adaptivity.

## 6.5 Validation of results (maybe in Appendix?)

Lastly, the validation calculations were formed and computed. Analytical calculations for the effective moment of inertia were performed in accordance with Al-Emrani et al. (2013). An $\alpha$-factor was determined to account for the inserted reinforcement. Figure 6.11 illustrates the linear correlation between the effective bending stiffness ($EI_{eff}$) and the gross section stiffness ($EI$) for the studied segments. A trend line is included to highlight the relationship between the analytical values and those applied in the FEA. For bridge decks featuring a curved soffit, the support and span segments were simplified by calculating $EI$ based on the average segment height.



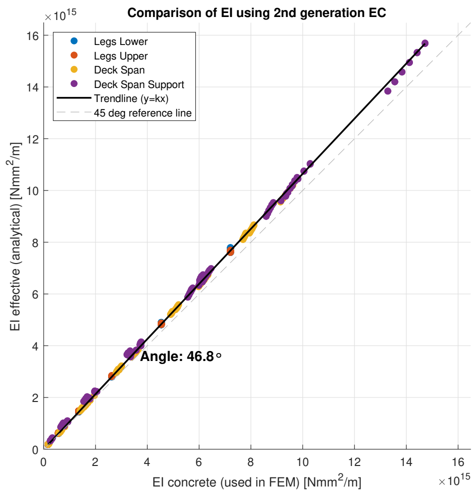




**Figure 6.11:** Comparison of EI and $EI_{eff}$ for the different regulations

The results indicate a strong correlation between the two approaches, with the bridge deck span exhibiting the largest deviation. Overall, the discrepancies are minimal, confirming that the use of gross concrete sections is a sufficiently accurate assumption for the FEA when used in the aspect of preliminary bridge design. However, the effect on the structural behavior is relevant, which will be further studied in the following validations. To verify that the modeling assumptions align with the expected structural behavior, the longitudinal displacement of the frame legs was compared with the analytical values. Figure 6.12 presents the comparison of the resulting displacement from the performed FEA and their corresponding analytical calculations.

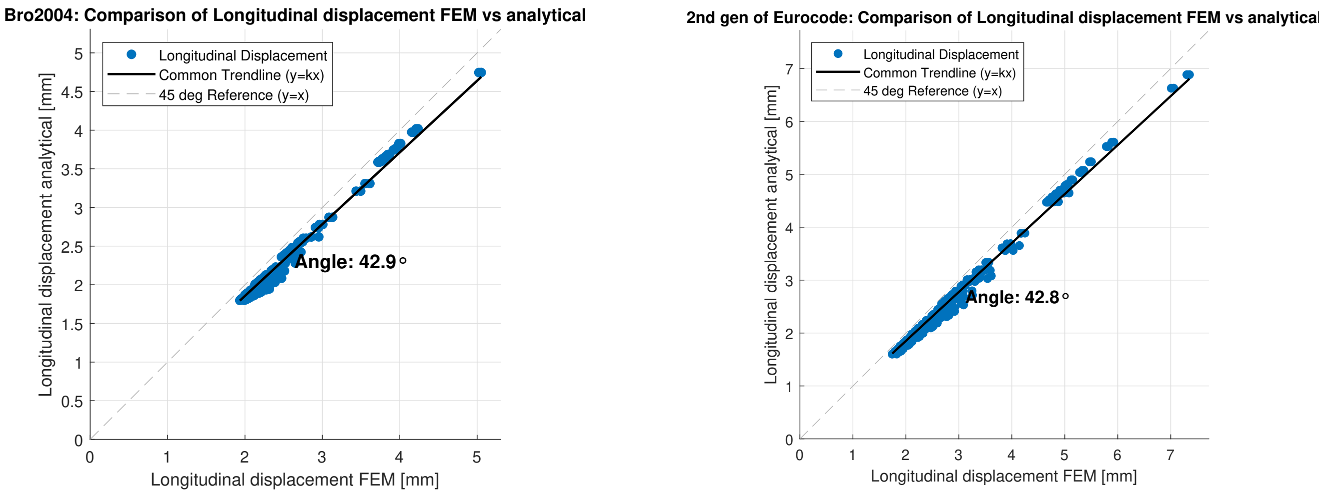



**Figure 6.12:** Comparison of longitudinal displacement between analytical and the FEA

The result in Figure 6.12 demonstrates a similar structural response between the FE model and analytical calculations. However, the FEA results gave slightly higher deflection values, suggesting that the analytical simplifications and assumptions do not fully reproduce the exact behavior. The reduction of a complicated 3D slab bridge to a simplified 2D frame model inevitably introduces inaccuracies. Nonetheless, the overall agreement in behavior validates the FE modeling approach and confirms that the model captures the intended structural response.

Additional validations were performed for the slab bridge deck to ensure accuracy of the result. The vertical deflection of the slab was calculated analytically under frequent load combinations for second generation EC, as well as load case V:A for Bro 2004. The comparison is presented in Figure 6.13.

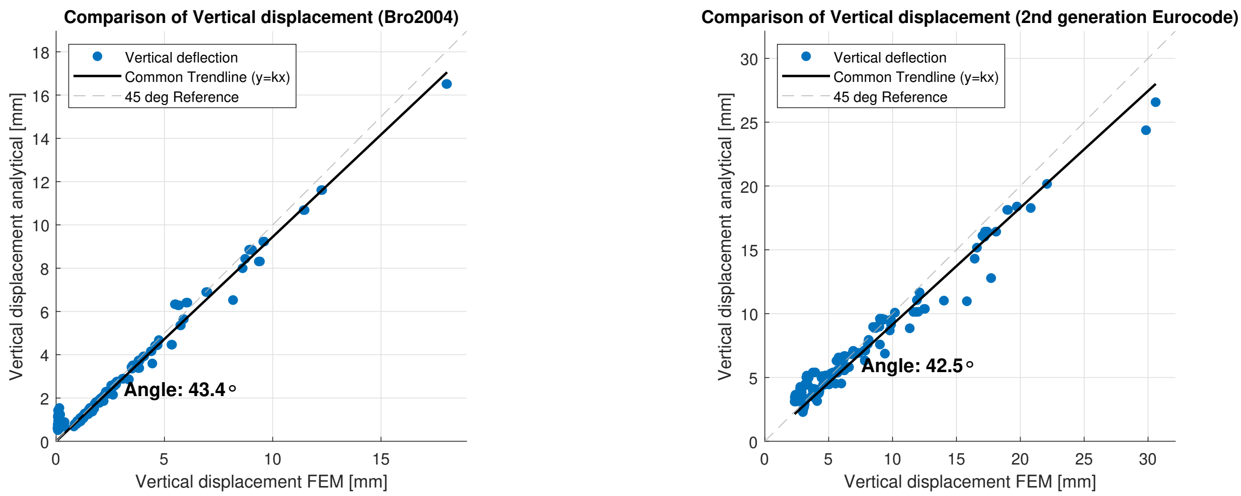



**Figure 6.13:** Comparison of vertical displacement between analytical and the FEA

Consistent with the longitudinal displacement findings, the vertical displacements

exhibit a comparable trend between the analytical and FEA models. However, a slightly higher deviation is observed in this case.
Lastly, a validation of the bending moments in the slab deck was performed. The maximum cross-sectional moments derived from the Bro2004 load combinations were calculated analytically and compared against the FEA results, as illustrated in Figure 6.8.

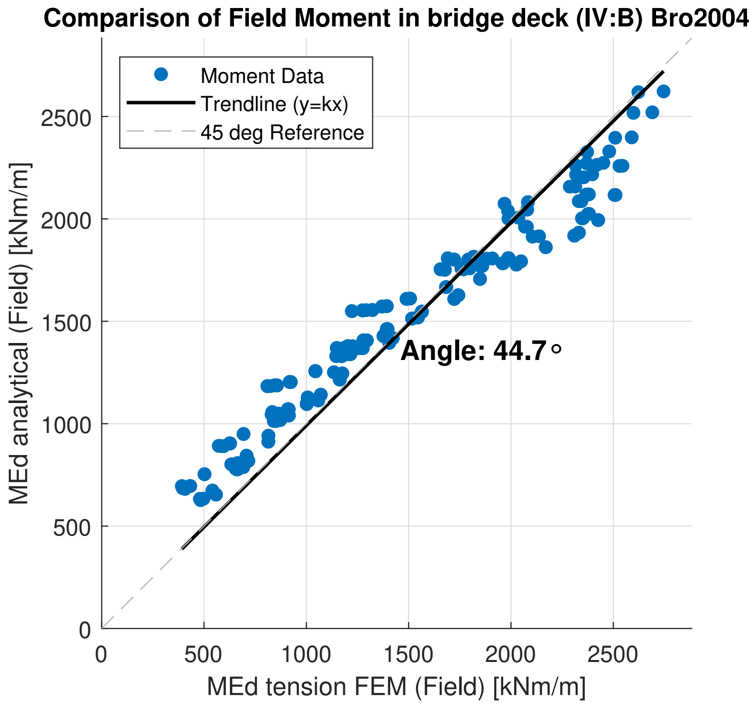



**Figure 6.14:** Comparison of maximum moment between analytical calculations and FEA

The results follow a pattern closely resembling the previous validations behavior. While some deviations are present, the overall behavioral trends align well.

# 7
# Discussion

*This section discusses the observed results from the comparison, as well as reviews important aspects further.*

## 7.1 Validation of the Finite Element Model

The performed validation indicates an overall agreement between the behavior seen in FEA and calculated analytical values. Summarizing the results from all the different structural response studies, all the three indicate on similar behavior. For lower loading effects, the behavior trends towards a lower deviation in difference between the calculations, sometimes even causing higher values analytically. However, for higher loads, the behavior can be seen deviating more, indicating on that the analytical approach is not sufficient where contribution from aspects not taken into account or inappropriate simplifications makes the deviation.
Higher values from the FEA also supposes that not accounting for an effective $EI_{eff}$ has contribution. This also makes sense considering that the effective moment of inertia is based on the applied load/Moment, since higher moments generates higher $EI_{eff}$ proposes a stiffer behavior in the analytical calculations, hence generating a lower magnitude in response.

The reduction of a complicated 3D slab bridge to a simplified 2D frame model inevitably introduces inaccuracies. For the analytical moment calculations, torsional moment $m_{xy}$ is not accounted for. This limitation further explains the margin of error between the analytical and numerical models seen in Figure 6.14.

## 7.2 Interpretation of comparative results

Comparing investment cost and $CO_2$-eq of concrete and reinforcement between the regulations of Bro 2004 and the second generation of EC shows no discernible difference overall. However, from the aspect of using the SBD procedure to obtain the most efficient feasible design clear differences arise. Bro 2004 allows for more cost-effective and $CO_2$-eq efficient solutions. Among the feasible designs, Bro2004 most efficient solution equals 0.41 MSEK, whereas the corresponding Eurocode solution reaches 0.43 MSEK, roughly 5% higher.
To further understand these differences, it is essential to examine the subset of designs that are feasible under Bro 2004 but become unfeasible when validated

according to second generation EC. The distribution of failure modes for 25 mm reinforcement is presented in Table 7.1.

**Table 7.1:** Summary of failures by regulation for 25 mm reinforcement.

| **Regulation** | **Failure in fitting reinforcement amounts** | **Failure in displacement** | **Failure in crack width** |
|---|---|---|---|
| Bro2004 | 538 | 30 | 100 |
| second gen EC | 531 | 220 | 0 |

The result shows that the most dominant cause of infeasibility is the inability to fit the required amounts of reinforcement. However, the behavior regarding displacement differs substantially, Eurocode produces roughly seven times more failures related to vertical deflection or longitudinal displacement than Bro 2004. Crack-width calculations also contribute to infeasibility in Bro 2004, indicating that the adaptive reinforcement procedure in the script did not always provide enough reinforcement to satisfy crack-width limits. This aligns with the results shown in Figure 6.10. In contrast, the crack width calculations do not contribute to any failures under the EC framework.
Although the number of failures related to reinforcement fitting is similar between the regulations, the slightly lower number for EC appears counterintuitive. Based on the result in Figure 6.8, this would be illogical, as Bro 2004 provides lower bending moments and should therefore require less reinforcement, not more. This deviant behavior is explained by the definition of design strength values in Section 5.3. In BBK04, the design strength is reduced by the factor $\gamma_n$ due to safety class 3 (Boverket, 2004). In EC, the safety class is instead applied to the loads, directly affecting the structural response shown in Figure 6.8 (Transportstyrelsen, 2018).

Based on the identified failure modes and the results presented in Chapter 6, the trends in structural behavior help explain the differences observed in the probability distributions for the two regulatory frameworks. The higher longitudinal displacement and vertical deflection in the EC lead to several designs becoming unfeasible, while the same designs remain feasible under Bro 2004. This explains why Bro 2004 produces a larger share of efficient feasible solutions.
The cases where Bro 2004 instead provides fewer feasible designs are primarily due to crack-width limitations or reinforcement fitting and do not eliminate more efficient solutions. Rather, they reduce the density of feasible designs around the mean cost and remove some of the more expensive alternatives. In the following Section 7.3, additional reasons for the greater structural response observed in the second generation EC will be examined.

The procedure of the simplified crack-width calculations produces trends that align with expected outcomes. Considering EC allows larger maximum crack widths compared to Bro 2004, it logically follows that less reinforcement should be required. However, directly ascribing the difference in reinforcement quantities solely to the limits requires caution. The regulations employ fundamentally different calculation methods and how these methodological variations influence the final design warrants further investigation. Nevertheless, empirical observation discussed in chapter 2.5 in SBUF and BBT (2025) indicates that bridges designed according to Bro 2004 exhibit the fewest cracks in practice. This suggests that the stricter requirements of that era did have greater measures to avoid cracks, such as higher reinforcement ratios, effectively corroborating the comparative finding in this thesis.

Despite the differences discussed, it is important to emphasize that the overall differences between the regulations are relatively minor in a broader context. Both the total design space and the subset of feasible designs return highly comparable values. Although the reinforcement amounts are higher under Bro2004, as shown in Figure 6.9, the overall costs (Figure 6.1) and $CO_2$-eq emissions (Figure 6.4) do not exhibit any substantial discrepancies. Given that the SBD methodology indicates that the regulatory frameworks themselves do not inherently dictate massive differences in material efficiency, this raises the question of what truly drives the historical increases in cost and material usage. Research indicates that this upward trend is largely attributable to changes in engineering practices rather than solely the structural code requirements themselves (SBUF & BBT, 2025). The transition from simplified traditional methods to complex 3D finite element analyses, coupled with an expansion of detailed code provisions, has led to a shift towards more conservative, compliance-oriented decisions in traditional Point Based Design. While PBD remains the conventional industry practice, the SBD approach utilized in this thesis effectively bypasses these methodological biases. Consequently, the SBD results offer a highly realistic and objective reflection of the regulations' inherent potential, confirming that the structural codes alone are not the primary cause of the increased material consumption observed in modern infrastructure.

## 7.3 Examination of Loads

When examining the application of the two regulatory frameworks, it becomes evident that the definition of loads differs between Bro 2004 and second generation EC. Although the overall modeling approach is similar, characteristic load values and associated partial and combination factors vary between regulations. Table C.2 in Appendix C provides a detailed view of load components, applied factors and resulting forces used in the structural response calculations.
To provide a relevant comparison categories A-E have been provided in Table C.1, matching the load combinations from each regulation with the corresponding verification step in the design procedure. For the slab deck, geometry-dependent loads are defined through a representative example case. The table also showcases a simplified uniformly distributed load, expressed as a distance-weighted mean that is used for subsequent conclusions.

From Table C.2, it can be observed that the characteristic point and distributed loads from train loads are generally higher in the older regulations. However, once load-combination factors and amplification parameters $\alpha$ and $\phi$ are applied, the resulting load combinations become consistently higher in the EC for all verification types.

The largest deviation between the regulations is the definition of load combinations used for evaluating vertical deflection of the slab deck of the bridge, denoted as category D in Table C.2. Second Generation of EC defines a frequent load combination, shown in Equation 4.5 together with Table A.2. Bro 2004 on the other hand specifies the load combination V:C, which (unlike EC) does not include the permanent loads of the bridge. Only the vertical train load and surcharge are considered. As a result, the total applied load differs, 194.5 kN/m in Bro2004 compared to 268.3 kN/m in EC. From an engineering perspective, this directly explains the larger vertical deflections obtained under the EC framework.

In the second generation of EC, partial factors exist that permeate all of the load combination, causing application of higher effective train loads. While a dynamic factor is used with similar values between the regulations, only differing by 0.07, the $\alpha$-factor introduces disparity. As described in 4.1 the NA specifies an $\alpha = 1.33$ (Transportstyrelsen, 2018). This factor is not defined and used in the case of Bro 2004, therefore equal to 1. This creates a large difference between the load combinations, with 33% higher train loads applied when using the EC regulations. Notably, when $\alpha$ is set to 1 (the default value in EC) the resulting loads become lower than Bro 2004 for all load combinations except D, as shown in Table C.3.
The influence of the $\alpha$-factor is also evident in the longitudinal displacement result, which is strongly affected by traction and braking loads, see Table C.4. When comparing the load combinations used for longitudinal displacement, V:A in Bro 2004 (categorized C) and frequent load combination (categorized D) in EC the amplification introduced by $\alpha$ becomes particularly significant. This leads to higher applied traction loads and, consequently, greater longitudinal displacements under the EC framework. With all of these resulting differences caused by the $\alpha$-factor, the question arises why the factor equals 1.33. Is there a specific reason?

## 7.4 uncertainties and future improvements

This section takes up potential parts of the SBD-algorithm procedure as well as the application of regulatory frameworks that have the possibility to be further investigated, adjusted or expanded upon.

### 7.4.1 Simplifications of implemented regulatory frameworks

Throughout the entire procedure of developing the SBD-tool, simplifications have been made. As stated before, the comparison is made in the aspect of preliminary design of bridges, therefore not deep diving into the design procedure, only general verifications have been compared between the different regulatory frameworks. For future work, a more detailed verification procedure could be made to explore the arising differences more.

Estimating the required reinforcement solely on its yield strength provides only a preliminary approximation. A more precise approach would be to evaluate the reinforcement area using strain compatibility (Al-Emrani et al., 2013). Furthermore, the SLS verifications, specifically regarding crack width calculations are limited to a 1D approach. In reality a more complex 2D behavior is likely active due to the presence of orthogonal reinforcement (both in the local x- and y-direction for each segment). Incorporating 2D crack propagation and distribution models would therefore significantly enhance the SBD algorithm's calculations, providing more realistic results. However, this consideration ties back to the underlying rationale mentioned previously. From the standpoint of preliminary design and computational efficiency, implementing complex crack width calculations would deviate from the intended scope of the tool (Engström, 2015). Finally, additional structural verifications, such as ductility checks could also be incorporated in future iterations.

### 7.4.2 Simplifications of the SBD algorithm and pathways for future research

A significant generalization in the current SBD algorithm is limitations of the generated design space. As stated in Section 3.4.1.1 the design is currently restricted to using one certain spacing and one reinforcement size in either one or two layers across all bridge segments. Future studies could explore varying reinforcement configurations both within individual cross-sections and between different segments. This would provide a more realistic evaluation of material, cost and $CO_2$-eq, enabling more efficient designs and higher resolution comparison between the regulations. Expanding the design space also opens up the possibility of additional filtering of the design space. For instance, the SBD algorithm developed by Hansson and Tacking (2024) implements a filter where the reinforcement spacing of adjacent segments should be divisible by one another to ensure manageable connections. This refines the algorithm's SBD approach even more and makes the resulting design spaces approach in realistic world applications even further.

Another Limitation lies in the FE-modeling of the interaction between soil and the structure. As discussed in Section 3.1.2, the connection between the soil and the foundation slab was modeled as rigid, preventing displacement in all directions. This made a representative structural response for the rest of the structure. While this serves the scope of the thesis, which focused on comparing the frame legs and deck slabs as mentioned in Section 1.3 it still yields an unrealistic representation of the foundation slab. Including a realistic foundation slab in the analysis could alter the structural behavior, potentially shifting the feasible design space, the probability density, and in the end affect the overall results of the comparison.

Furthermore, the current rigid boundary condition effectively simulates a foundation on solid bedrock, which corresponds to the behavior of a rigid frame according to Trafikverket (2011). However, this contradicts the typical application of closed portal frames. As noted by Trafikverket (2008), closed frames are generally utilized in unfavorable ground conditions, which is not the typical case when having Bedrock as foundation. Future work could therefore focus on implementing realistic boundary conditions, or alternatively, dividing the analysis into two separate models, one for the superstructure and one for the foundation. Applying representative soil conditions would harmonize the model with realistic world applications of closed-frame bridges, significantly increasing the practical relevance of the findings surrounding the most efficient bridge.

Additionally, a few aspects that are necessary to take into consideration when modeling and verifying the structural behavior have not been taken into consideration. The main parts disregarded are the account of wind load, fatigue analyses and traffic load on the foundation slab. These parts might heavily affect the amounts and properties of the resulting feasible designs obtained from the SBD algorithm, hence changing the results and conclusions of the comparison might be different.

Further simplification that has been made in the model is the exclusion of the wing walls. From a structural engineering perspective, this assumption is noteworthy because of the wing walls providing substantial stiffness in the longitudinal direction, thereby restricting longitudinal displacement. Therefore, neglecting the wing walls leads to an overestimation of the calculated longitudinal displacement compared to the actual behavior of a bridge that holds wing walls. Future development of the SBD algorithm and comparative analyses that incorporate wing walls would provide a more realistic and reduced longitudinal displacement which eventually influences the final results.

Lastly, an interesting area for future improvements is the implementation of the strip method for moment and reinforcement calculations. The strip method, originally developed by Hillerborg (1996), is a lower-bound approach based on plasticity theory that provides an alternative framework for calculating the moment distribution in concrete slabs. In the context of SBD, which requires the evaluation of numerous design combinations, the strip method can account for two-dimensional structural behavior and yield a more realistic reinforcement layout while maintaining computational efficiency. The current SBD approach relies only on the highest sectional responses and provides a solution for those specific peak values. Integrating the strip method, especially for the slab deck, would provide safe design values through its lower-bound nature and generate a practical reinforcement layout that reflects actual construction practices. Even though a more detailed design is provided would this approach remain sufficiently versatile and appropriate for the preliminary design phase. However, it should be noted that since the strip method is fundamentally a ULS approach based on plasticity, it does not inherently evaluate deformations or crack widths. Therefore, implementations to verify SLS criteria are still needed, utilizing either the current SBD procedure or an improved version as mentioned previously in this discussion.

## 7.5 Applicability of the SBD approach to regulatory comparison

The used SBD methodology provides a comprehensive view of utilizing different regulatory frameworks. Instead of comparing a single bridge solution between the regulations does the SBD methodology allow for mapping the entire feasible domain. The viewpoint provided is highly beneficial when comparing regulatory frameworks, as it reveals the density, distribution and overall robustness of both feasible and unfeasible solutions. Not only the properties of a single, optimized solution. By capturing the entire spectrum of these design spaces makes it possible to identify exactly which specific constraints eliminate certain solutions, and where within the design space these eliminations occur. A great example of this is the mentioned placements of crack-width- and displacement failures, where evaluating the resulting designs allows for understanding why certain solutions fail, and the reasoning behind it.

Despite these major advantages, the SBD methodology comes at the cost of limitations when applied to comparative analyses. The most notable constraint is the computational expense. Evaluating greater amounts of design alternatives the outcome becomes a time-consuming procedure, both creating the method and computing it. To maintain computational feasibility, structural verifications must be generalized and parameter resolution, such as reinforcement layout, element sizes and properties must be constrained. Consequently, while the SBD is beneficial in identifying broad, systemic trends between regulations during the preliminary design phase, it cannot currently accommodate the highly detailed steps required to provide a final bridge design. Examples of this include detailed capacity checks and analyses of non-linear behavior, which are vital components that characterize the final stages of practical bridge detailing.

Even though SBD has limits in detailed design, does the approach contain potential by adapting progressive refinement. One viable strategy is to implement a multi-tiered SBD framework. By initially employing a generalized approach, such as the one utilized in this thesis, the vast initial design space can be effectively narrowed down. The generated subset of feasible designs can then be processed through more detailed, computationally intensive SBD algorithms. Filtering the design space in steps would allow for reducing the overall computational time, preventing the execution of heavy structural analyses on design alternatives that already fail under generalized criteria.

# 8 Conclusion

This thesis has utilized a set-based design algorithm to conduct a comparative analysis between the previous Swedish bridge regulations surrounding and consisting of Bro 2004 and the second generation of Eurocodes for a concrete frame bridge. By evaluating the full feasible space, rather than a single point-based solution, enables a more precise assessment of how the regulatory frameworks shape material usage, cost, environmental impact and structural response during preliminary design.

The implementation of the Set-Based Design methodology has led to the following primary conclusions.

**Overall Impact on cost and emissions**
From a broad perspective, there was no discernible difference in the overall investment cost and $CO_2$-equivalent emissions of the material between the two regulatory frameworks.

**Optimization potential**
Despite the similarities across the total design space, Bro 2004 permitted the most highly optimized and efficient solutions. The most cost-effective feasible design under Bro 2004 was approximately 5% cheaper than the corresponding Eurocode solution.

**Governing constraints and failures**
The reasons the alternate regulatory frameworks eliminated different design alternatives varied somewhat. While the main reason for both regulations was the failure to fit the required amount of reinforcement in two layers, residual failures varied. Bro 2004 had a combination of both failures in deflection as well as crack width. Second generation of Eurocode had only failures in deflection, none due to crack width being limiting. Observing the structural results differences was shown. While the ULS dimensional moment did not vary to an extent, the vertical deflection and longitudinal displacement did. It was also observed that the reinforcement amounts trend towards higher values for Bro 2004.

After investigation, comparing the load cases to one another suggested that the different values in deflection originated from two main causes. The main reason for vertical deflection varying came from Bro 2004 not considering self-weight, while Eurocode did. Both vertical deflection and longitudinal displacement along the bridge were affected by an $\alpha$-factor that accounts for irregularities appearing that contradict the used load modeling of trains. This $\alpha$-factor is absent in the case of Bro 2004, raising questions such as why it was added. What does it contribute? As shown in Table C.3 and C.4, the case of a non-existing $\alpha$-factor created differences, where Eurocode even had less total load in several cases. The $\alpha$-factor is something that could be further investigated, questioning its existence as well as its absolute value of 1.33 according to the national annex (Transportstyrelsen, 2018). The reason for higher amounts of reinforcement using Bro 2004 was also studied. The observed trend suggests that stricter crack-width limit and different calculation method cause the seen difference. The observed trend aligns with previous studies, where bridges dimensioned within a similar time period were observed with fewer cracks, suggesting a similar result (SBUF & BBT, 2025).

**Drivers of material consumption**
The findings suggest that the historical increase in material usage and costs in modern infrastructure is not solely dictated by the structural codes themselves. Instead, the case of conservative engineering practices are suggested as an underlying cause, as discussed in Section 7.2.

**Efficacy of Set-Based Design**
Set-based design proves to be a highly effective tool for comparing regulatory frameworks. By mapping the entire feasible domain and bypassing the methodological biases inherent in point-based design, it reveals the true potential, constraints robustness of the regulations.

**Future research**
Based on the uncertainty and model simplifications identified and discussed in Section 7.4, future research should focus on enhancing the applicability of the studied case to real-world scenarios. This involves implementing features such as the case of wind load and fatigue analyses, as well as accounting for wing walls and more accurate soil-structure interaction. The algorithm's design space also has potential of improving, providing more practical solutions. As discussed in Section 7.4, this includes allowing varying reinforcement layouts in different segments of the bridge and integrating the strip method (Hillerborg, 1996). Furthermore, the implementation of regulatory frameworks could be refined. While this study adopted a generalized approach suited for preliminary design, future studies could integrate additional aspects in regulations and more detailed verifications. This helps identify additional differences, providing an even more detailed comparison and generating exact differences between the frameworks.

Lastly, although a high resolution in Set-based design combined with Finite Element modeling is computationally intensive, its efficiency can be improved. Adopting a multi-tiered Set-Based Design Framework, where detailed calculations are exclusively performed on a narrowed subset of feasible solutions would reduce the overall computational time. This strategy aligns well with the goal of conducting deeper comparative analyses, allowing for higher-resolution detailing while keeping computational costs more manageable.

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# A

# Partial factors

This appendix contains the partial factors applied in the load combinations in different load scenarios obtained from Second generation EC (SS-EN 1990:2023, 2023; SS-EN 1992-1-1:2023, 2023) and Swedish GTS (Trafikverket, 2025c), see table A.1 and A.2. Appendix A also contains the load coefficients of the regulations in Bro 2004 and BV Bro 7th edition, see Table A.3.

**Table A.1:** Partial factors for ULS load combinations.

<table>
<tr><th colspan="3" rowspan="3">Load case</th><th colspan="8">Equation (4.2)</th><th colspan="8">Equation (4.3)</th></tr>
<tr><th colspan="6">Unfavourible</th><th colspan="2">Favourable</th><th colspan="5">Unfavourable</th><th colspan="3">Favourable</th></tr>
<tr><th colspan="3">$\gamma_G$</th><th colspan="3">$\gamma_{sup}$</th><th>$\gamma_G$</th><th>$\gamma_{inf}$</th><th colspan="2">$\gamma_G$</th><th colspan="2">$\gamma_{sup}$</th><th>$\xi$</th><th>$\gamma_G$</th><th>$\gamma_{inf}$</th><th>$\xi$</th></tr>
<tr><td rowspan="4">Permanent</td><td colspan="2">Self weight</td><td colspan="3">1.35</td><td colspan="3">1</td><td>1.35</td><td>1</td><td colspan="2">1.35</td><td colspan="2">1</td><td>0.89</td><td>1.35</td><td>1</td><td>0.89</td></tr>
<tr><td colspan="2">Ballast ±30%</td><td colspan="3">1.35</td><td colspan="3">1.3</td><td>1.35</td><td>0.7</td><td colspan="2">1.35</td><td colspan="2">1.3</td><td>0.89</td><td>1.35</td><td>0.7</td><td>0.89</td></tr>
<tr><td colspan="2">Shrinkage</td><td colspan="3">1.35</td><td colspan="3">1</td><td>1.35</td><td>0</td><td colspan="2">1.35</td><td colspan="2">1</td><td>0.89</td><td>1.35</td><td>0</td><td>0.89</td></tr>
<tr><td colspan="2">Earth pressure</td><td colspan="3">1.35</td><td colspan="3">1</td><td>1.35</td><td>1</td><td colspan="2">1.35</td><td colspan="2">1</td><td>0.89</td><td>1.35</td><td>1</td><td>0.89</td></tr>
<tr><td colspan="3" rowspan="2"></td><th colspan="3">Main</th><th colspan="3">Acc</th><td colspan="2" rowspan="10"></td><th colspan="2">Main</th><th colspan="3">Acc</th><td colspan="3" rowspan="10"></td></tr>
<tr><th>$\gamma_Q$</th><th>$\psi_0$</th><th>$\psi_{com}$</th><th>$\gamma_Q$</th><th>$\psi_0$</th><th>$\psi_{com}$</th><th>$\gamma_Q$</th><th>$\psi_{com}$</th><th>$\gamma_Q$</th><th>$\psi_0$</th><th>$\psi_{com}$</th></tr>
<tr><td rowspan="8">Variable</td><td rowspan="3">GR11</td><td>LM71</td><td>1.5</td><td>0.8</td><td>1</td><td>1.5</td><td>0.8</td><td>1</td><td>1.5</td><td>1</td><td>1.5</td><td>0.8</td><td>1</td></tr>
<tr><td>Traction</td><td>1.5</td><td>0.8</td><td>1</td><td>1.5</td><td>0.8</td><td>1</td><td>1.5</td><td>1</td><td>1.5</td><td>0.8</td><td>1</td></tr>
<tr><td>Nosing</td><td>1.5</td><td>0.8</td><td>0.5</td><td>1.5</td><td>0.8</td><td>0.5</td><td>1.5</td><td>0.5</td><td>1.5</td><td>0.8</td><td>0.5</td></tr>
<tr><td rowspan="3">GR12</td><td>LM71</td><td>1.5</td><td>0.8</td><td>1</td><td>1.5</td><td>0.8</td><td>1</td><td>1.5</td><td>1</td><td>1.5</td><td>0.8</td><td>1</td></tr>
<tr><td>Traction</td><td>1.5</td><td>0.8</td><td>0.5</td><td>1.5</td><td>0.8</td><td>0.5</td><td>1.5</td><td>0.5</td><td>1.5</td><td>0.8</td><td>0.5</td></tr>
<tr><td>Nosing</td><td>1.5</td><td>0.8</td><td>1</td><td>1.5</td><td>0.8</td><td>1</td><td>1.5</td><td>1</td><td>1.5</td><td>0.8</td><td>1</td></tr>
<tr><td colspan="2">Temperature</td><td>1.5</td><td>0.6</td><td>-</td><td>1.5</td><td>0.6</td><td>-</td><td>1.5</td><td>-</td><td>1.5</td><td>0.6</td><td>-</td></tr>
<tr><td colspan="2">Surcharge</td><td>1.5</td><td>0.8</td><td>-</td><td>1.5</td><td>0.8</td><td>-</td><td>1.5</td><td>-</td><td>1.5</td><td>0.8</td><td>-</td></tr>
</table>

Where the values are obtained from

| | |
|---|---|
| $\gamma_G$, $\gamma_Q$ | SS-EN-1990:2023 Table A.1.8(NDP) |
| $\gamma_{sup}$, $\gamma_{inf}$ | TRVINFRA-00227 6.2.6.7.3.1 and SS-EN-1990:2023 7.4 |
| $\xi$ | TSFS 2018:57 Table 4.4 |
| $\psi_{com}$ | SS-EN-1991-2:2024 Table 8.15(NDP) |
| $\psi_0$, $\psi_1$, $\psi_2$ | SS-EN-1990:2023 Table A.2.7(NDP) and Table A.2.9(NDP) |

In SLS load combination, the applied partial factors for the variable loads in the different combinations according to equation 4.3, 4.4, 4.5 and 4.6 can be found in the table A.2 below. While the permanent actions in SLS analysis can be used directly from the characteristic loads with no reductions needed.

**Table A.2:** Partial factors for SLS load combinations.

| Load case | | | $\psi_0$ | $\psi_1$ | $\psi_2$ | $\psi_{com}$ |
|---|---|---|---|---|---|---|
| Variable | GR11 | LM71 | 0.8 | 0.8 | 0 | 1 |
| | | Traction | 0.8 | 0.8 | 0 | 1 |
| | | Nosing | 0.8 | 0.8 | 0 | 0.5 |
| | GR12 | LM71 | 0.8 | 0.8 | 0 | 1 |
| | | Traction | 0.8 | 0.8 | 0 | 0.5 |
| | | Nosing | 0.8 | 0.8 | 0 | 1 |
| | Temperature | | 0.8 | 0.6 | 0.5 | - |
| | Surcharge | | 0.8 | 0.8 | 0 | - |

**Table A.3:** Load coefficients $\psi\gamma$ for respective load combination in Bro2004 and BV Bro edition 7.

| | **Ultimate limit state** | | | | **Serviceable limit state** | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | IV:A | | IV:B | | V:A | | V:B | | V:C |
| Permanent loads | Unfav. | Fav. | Unfav. | Fav. | Unfav. | Fav. | Unfav. | Fav. | |
| Self weight | 1.05 | 0.95 | 1.15 | - | 1.05 | 0.95 | 1 | - | |
| Ballast | 1.25 | 0.8 | 1.25 | - | 1.25 | 0.8 | 1.2 | 0.8 | |
| Shrinkage | 1 | 0 | 1 | 0 | 1 | 0 | 1 | 0 | |
| Earth pressure | 1.1 | 0.9 | 1.1 | 0.9 | 1.1 | 0.9 | 1 | - | |
| **Variable loads** | Main | Acc | Main | Acc | Variable load | | Variable load | | Variable load |
| Train Load BV2000 | 1.4 | 0.7 | 0.7 | - | 1 | | | | 1 |
| Traction forces | 1.4 | 0.8 | 0.8 | - | 0.8 | | | | |
| Nosing force | 1.4 | 0.6 | 0.6 | - | 0.6 | | | | |
| Temperature | 1.4 | 0.6 | 0.6 | - | 0.6 | | 0.6 | | |
| Surcharge | 1.4 | 0.7 | 0.7 | - | 1 | | | | 1 |

# B

# Variables for Evaluation Attributes

This appendix describes the values of cost and environmental impact that is used to determine different bridge designs investment cost and environmental impact in terms of material. Material costs where obtained from Yavari et al. (2016),see table B.1. The $CO_2$-eq emission related to mass for each material is shown in table B.2. Concrete and reinforcement emissions is obtained from Yavari et al. (2016) and Trafikverket (2023a) respectively.

**Table B.1:** Material costs.

| **Material costs** | $\rho_{m,\textbf{Bridgedeck}}$ | $\rho_{m,\textbf{Framelegs}}$ | $\rho_{m,\textbf{Foundation}}$ | **Unit** |
|---|---|---|---|---|
| Formwork for consistent thickness elements | 500 | 200 | 200 | SEK/$m^2$ |
| Formwork for variable thickness elements | 500 | | | SEK/$m^2$ |
| Concrete (C35/45) | 1800 | 1800 | 1800 | SEK/$m^3$ |
| Reinforcement | 9 | 9 | 9 | SEK/kg |

**Table B.2:** Environmental impact.

| **Material** | $\textbf{eq}_i$ | **Unit** |
|---|---|---|
| Concrete (C35/45) | 0.15 | kg$CO_2$eq/kg |
| Reinforcement | 0.7 | kg$CO_2$eq/kg |

# C
# Load comparison

The following table is used in chapter 7 for comparison of applied loads according to the different regulatory frameworks surrounding Bro 2004 and 2nd generation EC, see chapter 4 and 5 as well as tables A.2, A.1 and A.3 in appendix A.
In order to provide an relevant comparison categories A-E has been provided in table C.2, matching the load combinations from each regulation with the corresponding verification step in the design procedure. For the slab deck, geometry dependent loads are defined through a representative example case. The table also showcase a simplified uniformly distributed load, expressed as a distant-weighted mean that is used for subsequent conclusions. Table C.1 shows respective load combinations denotation-letter for the tables in C.2, C.3 and C.4. For the regulations following Bro2004 the instructed use of each load combination is found in Banverket (2004). instructed use of Load combinations following the 2nd generation of Eurocode is found in SS-EN 1990:2023 (2023), with used equations numbers shown in table C.1

**Table C.1:** Used letter for different load combinations in 2nd generation Eurocode and Bro2004 for load comparison.

| Loadcase | EC2 | Bro2004 |
|---|---|---|
| **A** | 8.13 (1) | IV:B |
| **B** | 8.13 (2) | IV:A |
| **C** | 8.29 | V:A |
| **D** | 8.30 | V:C |
| **E** | 8.31 | V:B |

**Table C.2:** Load combination table.

| Standard | Parameter | A | B | C | D | E |
|---|---|---|---|---|---|---|
| EC2 | *Unfactored values* | | | | | |
| | $\alpha$ | 1.33 | 1.33 | 1.33 | 1.33 | 1.33 |
| | $\phi_{dyn}$ | 1.28 | 1.28 | 1.28 | 1.28 | 1.28 |
| | $\psi\gamma\ (G_{slab}, G_{ball})$ | 1.35 | 1.20 | 1.00 | 1.00 | 1.00 |
| | $G_{slab}$ [kN/m] | 14.40 | 14.40 | 14.40 | 14.40 | 14.40 |
| | $G_{ball}$ [kN/m] | 90.48 | 90.48 | 90.48 | 90.48 | 90.48 |
| | $\psi\gamma\ (Q_{boggie}, q_{train}, q_{ave})$ | 1.20 | 1.50 | 1.00 | 0.80 | 0.00 |
| | $Q_{boggie}$ [kN] | 250.00 | 250.00 | 250.00 | 250.00 | 250.00 |
| | $q_{train}$ [kN/m] | 80.00 | 80.00 | 80.00 | 80.00 | 80.00 |
| | $q_{ave}$ [kN/m] | 120.67 | 120.67 | 120.67 | 120.67 | 120.67 |
| | *Factored values* | | | | | |
| | $G_{tot} = G_{slab} + G_{ball}$ [kN/m] | 141.59 | 126.01 | 104.88 | 104.88 | 104.88 |
| | $Q_{boggie}$ [kN] | 509.20 | 636.50 | 424.33 | 339.47 | 0.00 |
| | $q_{train}$ [kN/m] | 162.94 | 203.68 | 135.79 | 108.63 | 0.00 |
| | $q_{ave}$ [kN/m] | 245.77 | 307.22 | 204.81 | 163.85 | 0.00 |
| | $q_{tot}\,(q_{ave} + G_{tot})$ **[kN/m]** | **387.36** | **433.23** | **309.69** | **268.73** | **104.88** |
| Bro2004 | *Unfactored values* | | | | | |
| | $\alpha$ | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| | $\phi_{dyn}$ | 1.21 | 1.21 | 1.21 | 1.21 | 1.21 |
| | $\psi\gamma\ (G_{slab})$ | 1.15 | 1.05 | 1.05 | 0.00 | 1.00 |
| | $G_{slab}$ [kN/m] | 14.40 | 14.40 | 14.40 | 14.40 | 14.40 |
| | $\psi\gamma\ (G_{ball})$ | 1.25 | 1.25 | 1.25 | 0.00 | 1.20 |
| | $G_{ball}$ [kN/m] | 90.48 | 90.48 | 90.48 | 90.48 | 90.48 |
| | $\psi\gamma\ (Q_{boggie}, q_{train}, q_{ave})$ | 0.70 | 1.40 | 1.00 | 1.00 | 0.00 |
| | $Q_{boggie}$ [kN] | 330.00 | 330.00 | 330.00 | 330.00 | 330.00 |
| | $q_{train}$ [kN/m] | 110.00 | 110.00 | 110.00 | 110.00 | 110.00 |
| | $q_{ave}$ [kN/m] | 161.33 | 161.33 | 161.33 | 161.33 | 161.33 |
| | *Factored values* | | | | | |
| | $G_{tot} = G_{slab} + G_{ball}$ [kN/m] | 129.66 | 128.22 | 128.22 | 0.00 | 122.98 |
| | $Q_{boggie}$ [kN] | 278.53 | 557.06 | 397.90 | 397.90 | 0.00 |
| | $q_{train}$ [kN/m] | 92.84 | 185.69 | 132.63 | 132.63 | 0.00 |
| | $q_{ave}$ [kN/m] | 136.17 | 272.34 | 194.53 | 194.53 | 0.00 |
| | $q_{tot}\,(q_{ave} + G_{tot})$ **[kN/m]** | **265.83** | **400.56** | **322.75** | **194.53** | **122.98** |

**Table C.3:** Load combinations for EC ($\alpha = 1$).

| Standard | Parameter | A | B | C | D | E |
|---|---|---|---|---|---|---|
| | *Unfactored values* | | | | | |
| | $\alpha$ | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| | $\phi_{dyn}$ | 1.28 | 1.28 | 1.28 | 1.28 | 1.28 |
| | $\psi\gamma\ (G_{slab}, G_{ball})$ | 1.35 | 1.20 | 1.00 | 1.00 | 1.00 |
| | $G_{slab}$ [kN/m] | 14.40 | 14.40 | 14.40 | 14.40 | 14.40 |
| | $G_{ball}$ [kN/m] | 90.48 | 90.48 | 90.48 | 90.48 | 90.48 |
| | $\psi\gamma\ (Q_{boggie}, q_{train}, q_{ave})$ | 1.20 | 1.50 | 1.00 | 0.80 | 0.00 |
| **EC2** ($\alpha = 1$) | $Q_{boggie}$ [kN] | 250.00 | 250.00 | 250.00 | 250.00 | 250.00 |
| | $q_{train}$ [kN/m] | 80.00 | 80.00 | 80.00 | 80.00 | 80.00 |
| | $q_{ave}$ [kN/m] | 120.67 | 120.67 | 120.67 | 120.67 | 120.67 |
| | *Factored values* | | | | | |
| | $G_{tot} = G_{slab} + G_{ball}$ [kN/m] | 141.59 | 126.01 | 104.88 | 104.88 | 104.88 |
| | $Q_{boggie}$ [kN] | 382.86 | 478.57 | 319.05 | 255.24 | 0.00 |
| | $q_{train}$ [kN/m] | 122.51 | 153.14 | 102.10 | 81.68 | 0.00 |
| | $q_{ave}$ [kN/m] | 184.79 | 230.99 | 153.99 | 123.19 | 0.00 |
| | $q_{tot}\ (q_{ave} + G_{tot})$ **[kN/m]** | **326.38** | **357.00** | **258.87** | **228.07** | **104.88** |

**Table C.4:** Acceleration and braking loads (Only C and D).

| Standard | Parameter | C | D |
|---|---|---|---|
| | *Unfactored values* | | |
| | $\alpha$ | - | 1.33 |
| | $\psi\gamma$ (Acceleration, Brake) | - | 0.80 |
| **EC2** | Acceleration | - | 33.00 |
| | Brake | - | 20.00 |
| | *Factored values* | | |
| | Acceleration | - | 35.112 |
| | Brake | - | 21.28 |
| | *Unfactored values* | | |
| | $\alpha$ | 1.00 | - |
| | $\psi\gamma$ (Acceleration, Brake) | 0.80 | - |
| **Bro2004** | Acceleration | 30.00 | - |
| | Brake | 27.00 | - |
| | *Factored values* | | |
| | Acceleration | 24.00 | - |
| | Brake | 21.60 | - |

# D

# Illustrations

This chapter is dedicated for Illustrations made in addition to the report in order to create a better explanation of concepts by visualizing procedures and methods.

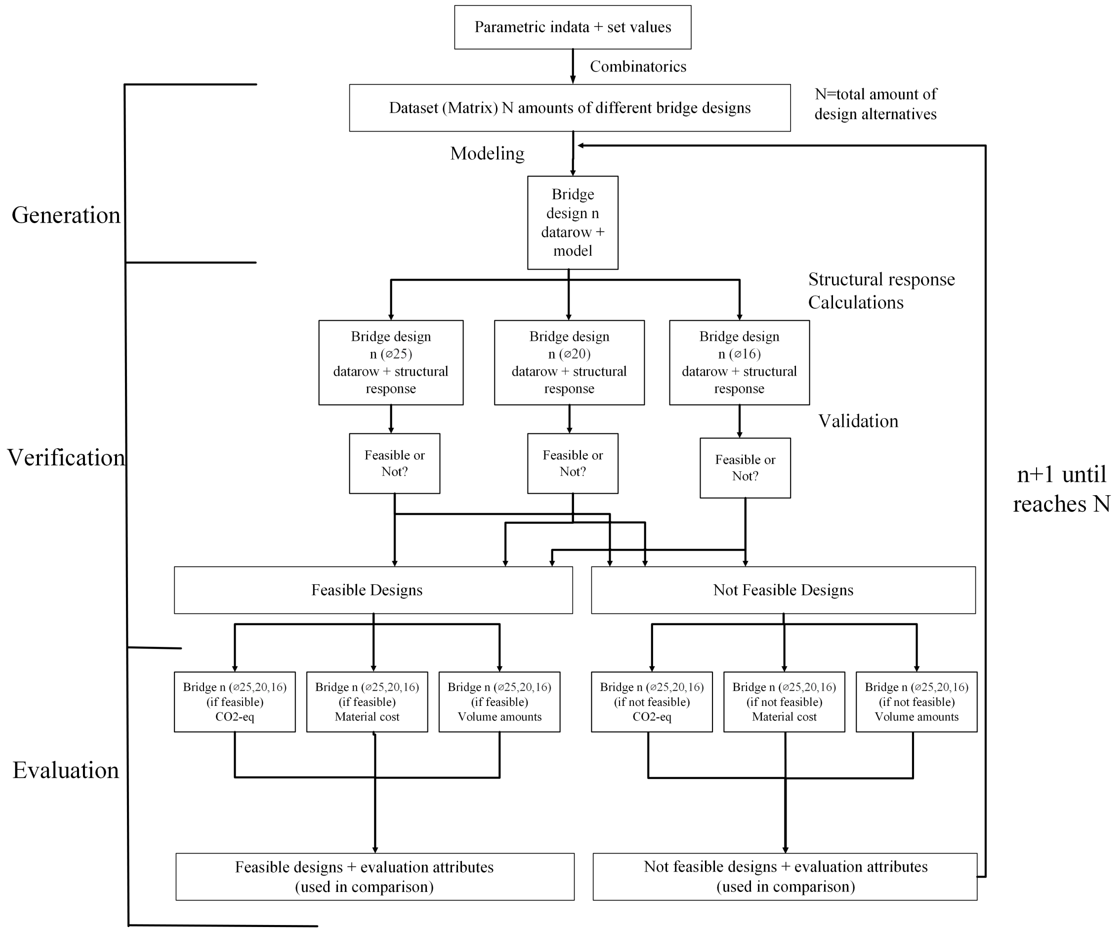


**Figure D.1:** Illustration of the SBD:s algorithms procedure. Example of using 16, 20 and 25 mm reinforcement sizes.

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