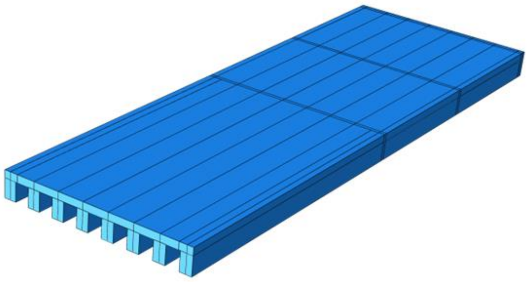

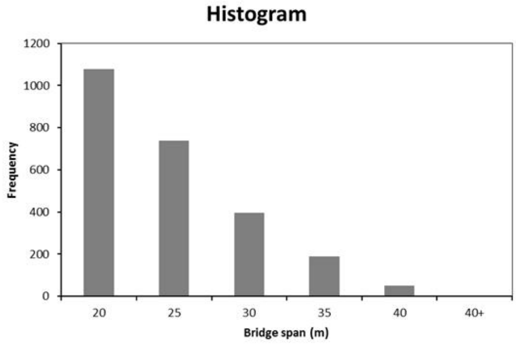

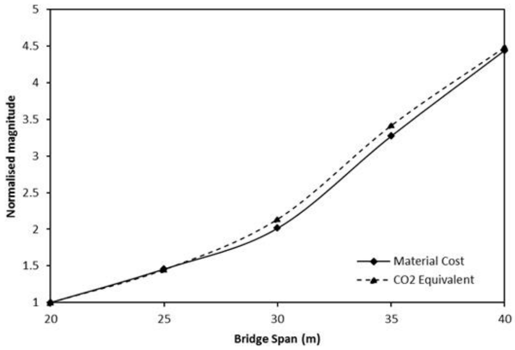

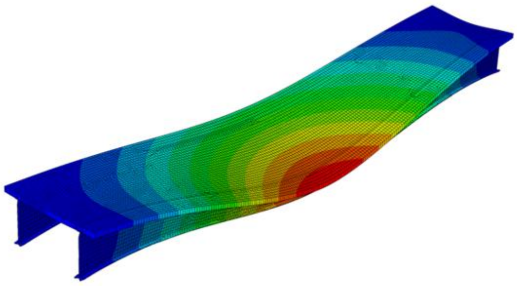

Applicability of Set-Based Design on Structural Engineering

Master's Thesis in the Master's Programme in Structural Engineering and Building Technology and Master's Programme in Infrastructures and Environmental Engineering

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Cover: NCC Montagebro FEM model, histogram of feasible alternatives over span for NCC composite concept, optimal alternative of NCC Montagebro over span, FEA deflection results for NCC composite concept.

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ABSTRACT

Traditional design practice in engineering has been based for decades on Point-Based Design, in which the design stage follows a stepwise process, going towards the next chain step after some design parameters have been chosen. However, this practice can reach a point, where a feasible solution cannot be found regarding some criteria involved in the design process, and hence, backtracking and rework are required. In order to solve the aforementioned drawbacks, in mid-1990s, a new design approach, Set-Based Design (SBD), was developed. SBD is based on considering sets of alternatives, which are narrowed according to the criteria of the design stakeholders, until an optimal solution remains. In this study, an assessment of the suitability of SBD in structural engineering is performed, motivated by the lack of application of this design approach in a field, where avoiding rework can be particularly beneficial due to its long and costly design process. This assessment is carried out by applying a computational tool developed for this purpose, which allows performing preliminary structural analysis on sets of bridge alternatives specified by the designer as ranges of different design parameters. These sets are narrowed not only regarding technical feasibility from the structural analysis, but also considering economic and environmental issues for selecting the best alternative. Two different bridge concepts are assessed as case studies in order to prove the potential of the methodology and for its validation. First, it has been demonstrated by different means that the methodology simulates properly the pre-design stage as performed by the traditional design practice. Secondly, the implementation of the methodology on two different already existing bridges has resulted in more optimal design alternatives than the ones were built, reducing the cost significantly. Furthermore, the methodology can be used for other purposes rather than direct design, such as suitability assessment of different concepts depending on the span length or for performing parametric studies. In summary, the methodology here developed adjusts very well to the principles of Set-Based Design and show the great potential of this design approach on structural engineering.

Key words: Set-Based Design, Robust Design, Integrated Design, Bridge Pre-Design, Computational Engineering, FEM analysis, Python , Automation

Contents

Preface

This Master’s Thesis has been carried out at the division of Structural Engineering of the Department of Civil and Environmental Engineering at Chalmers University of Technology and in collaboration with NCC Teknik.

The project would have not been possible without the help and supervision of Rasmus Rempling at the division of Structural Engineering who has been willing to give us support at any time and any day. Additionally, he has taught us how to organise the work load and structure the thesis in a proper way throughout his personal advice and organised workshops for thesis students.

Secondly, but not less important we would like to thank Alexandre Mathern from NCC Teknik Göteborg, who supervised and gave valuable help not only from an engineer perspective but from a company point of view, creating some new interesting challenges that will help us in our future careers.

We cannot forget all those people that have shared this wonderful period at Chalmers with us, including friends, flatmates, classmates, professors and our families. For your love and support, thank you.

Finally, we would like to acknowledge Scanscot Technology for providing us with a Brigade Plus license that has been really useful for the development of the project.

Notations

In the notation table, all variables occurring in the report are listed alphabetically.

Abbreviations

CFD	Computer Fluid Dynamics
CP	Car Placement
EC	Eurocode
FBC	Free Body Cut
FE	Finite Element
FEA	Finite Element Analysis
FEM	Finite Element Method
FRP	Fibre Reinforced Polymer
HVAC	Heat Ventilation and Air Conditioning
IPD	Integrated Project Delivery
LHS	Latin Hypercube Sampling
LM1	Load Model 1
MDO	Multidisciplinary Design Optimisation
PBD	Point-Based Design
SBD	Set-Based Design
S-BIM	Structural Building Information Modelling
SBPD	Set-Based Parametric Design
SF	Sectional Force
SLS	Serviceability Limit State
SM	Sectional Moment
TS	Tandem System
UDL	Uniformly Distributed Load
ULS	Ultimate Limit State
WSM	Weighted Sum Model
3D-CAD	Three Dimensional Computer Assisted Design

1 INTRODUCTION

1.1 Background

Traditional engineering design is based on single solutions for each step of the creation chain. When one of these intermediate decisions is not feasible anymore, the process needs to step back and some modifications of the initial design have to be performed. This principle is the most used design approach but some drawbacks such as wasted time due to reworking, motivated the development of alternative design approaches. Toyota was one of the first companies which started working and using a new concept, based on a parallel and delayed decision making process called Set-Based Design (Ward et al. 1995).

In Set-Based Design, the decisions involved in the design are not made over a single alternative, instead a set of alternatives is decided by the stakeholders and successively narrowed according to the limitations and decisions of those who are implicated in the project. This approach allows a better understanding between the different design stages, same level of implication of the different designers and the likeliness of reworking is reduced. Although this methodology has not been applied to many different fields yet, its suitability needs to be assessed in other areas such as structural engineering.

The design of any sort of structure requires a series of assumptions that limits the alternatives in the following design steps and can lead to unfeasible or suboptimal designs. Therefore, the Set-Based Design approach might be a suitable solution in the area of construction.

1.2 Purpose and Objectives

The Set-Based Design approach was developed in the 1990s in order to improve the design process in car manufacturing by reducing rework, but eventually led also to a better understanding of the procedure and wider collaboration between stakeholders. This approach has been widely applied in different areas but still has not been deeper assessed in structural engineering where it might be very useful due to the need of several assumptions at early stages in the design process.

Hence, the purpose of this project was to investigate the applicability of Set-Based Design on structural engineering, particularly in the field of bridge design. For achieving this purpose, three main objectives were established as well as several intermediate objectives related to them:

- Develop a well-defined methodology that allows iterative preliminary design of structures, by taking advantage of the principles of Set- Based Design.
    - Set a State of the art of Set-Based Design, focused on civil and structural engineering.
    - Perform a parameterisation of a given bridge regarding design variables and creation of a set of bridge alternatives according to industrial practice.

    - Develop a tool that allows iterative FEA based on the principles of SBD for the assessment of the alternatives

- Assess the applicability and potential of the methodology as well as its reliability under different situations or cases.
    - Present main advantages and disadvantages over the classic approach, as well as justify the adoption of the new one.
    - Study the possibilities that this new approach could provide regarding additional studies or assessment of results.
    - Assess the robustness of the results obtained with the developed methodology
- Present techniques showing how to apply said methodology in a feasible and reasonable way.
    - Propose strategies and techniques for the management of large amounts of raw data generated by the developed SBD tool and process these data in an efficient manner.
    - Apply parallel processing and computing power for reducing computational time and be able to expand further the design space.

1.3 Limitations

In order to assess the applicability of the tool developed according to the principles of SBD, checks from the construction standards need to be included. However, due to the limited time extension of this project, only the most common checks are considered and implemented. The checks regarding buckling of steel girders are based on simplified methods due to the difficulties of assessing buckling in non-prismatic members.

The selection of the set of optimal alternatives among the structurally-feasible bridge alternatives is performed according to a multi-criteria analysis which takes into account only two criteria. Besides, these two criteria are highly related to each other and little advantage of their combination can be taken.

Since this methodology is likely to be used as an aid in the design process but not in the analysis or verification stage, balance between level of detail and time consumption is needed. As intended for pre-design, only checks based on Linear Elastic Analysis will be performed, e.g. bending capacity or deflection; and specific analyses such as dynamic response, advanced cracking assessment, creep, aging, or fatigue, are outside the scope of this project, although they will be required to perform a detailed analysis of the selected alternative. The project is also focused only on one-span bridges in order to avoid too complex support behaviours.

1.4 Methodology

First, in order to understand the principles and assess the previous applications of Set-Based Design, a literature review was done. At this stage, a review of related theories and tools such as Robust Design, Integrated Design and Computational Optimisation was performed.

Once the approach of how to apply this methodology on bridge design was decided, since an automatic and iterative Finite Element Analysis carried out in ABAQUS requires a Python script; the ABAQUS scripting module was studied.

In order to be able to specify a set of bridge alternatives, the parameterisation of two bridge concepts was done. Their modelling, loads and boundary conditions were developed within the script. Furthermore, the implementation of the traffic Load Model 1 from Eurocode was performed.

Parallel to this process, for including the required checks from standards that are needed in the preliminary stage of bridge design, an investigation of the checks themselves according to Eurocode and their implementation within the Python script was carried out.

Since the tool for carrying out the iterative Finite Element Analysis required much computational resources, the Chalmers Cluster Glenn was used for performing the analyses.

The tool was then checked against the case studies in order to verify its reliability.

Finally, in order to reduce and narrow the sets of alternatives, a simple multi-criteria optimisation was applied, and results were collected and discussed.

1.5 Structure of thesis

After this introductory chapter, the report includes different sections intended to cover all the objectives set from the beginning of the project. It is structured in a way that it is possible to go from the general principles of the Set-Based Design approach to the case specific details that had to be included for the correct performance of the methodology.

In *Chapter 2* an introduction of the Set-Based Design methodology, together with a literature study and an overview of other related theories are presented to establish the theoretical solid foundation of the developed methodology. The application of Set-Based Design to bridge design is carried out in *Chapter 3*.

Later in the report, the principles of bridge pre-design according to Eurocode are presented in *Chapter 4*. Afterwards, the tool developed in the course of the project is introduced and described in detail in *Chapter 5* and applied to case studies later in *Chapter 6*. Optimisation techniques used in the project are discussed in *Chapter 7* followed by a compilation of possible usages that can be given to the tool in *Chapter 8*.

Finally, discussion, conclusions and recommendations for further research are included in the last part of the report together with references.

2 SET-BASED DESIGN

2.1 General Information

The most widely used approach in structural design is known as Point-Based Design. This approach involves selecting a single structurally-feasible design alternative at each step of the design chain and then refining that single design, or point, while developing more details during the design process (Parrish et al. 2007). This approach may imply rework when at a certain stage of the design process there is not any suitable alternative to move towards the next step. Then, the design must be backtracked and modified in order to reach the final goal.

Moreover, the decision-making process is dependent on the judgement of the engineers involved in the process, which is normally based on rules-of-thumb, trial-and-error or experience of those designers, and does not take into account the opinion of the rest of stakeholders involved in the project. This may lead to solutions which are technically-feasible, but are not optimal in other aspects such as fabrication, cost or supplying.

Wrong selection of an alternative in the design process and wrong communication with the different stakeholders, might cause rework and therefore delay and extra-costs. These issues contributed to the development of a new design concept, in which these main drawbacks were meant to be solved. This new approach was called Set-Based Design.

By contrast of Point-Based Design, Set-Based Design is based on considering broad ranges of design possibilities, i.e., design space, from the outset, explicitly communicating and reasoning about these sets of design alternatives, and gradually narrowing the sets to eliminate inferior alternatives until a final solution remains (Nahm & Ishikawa 2005).

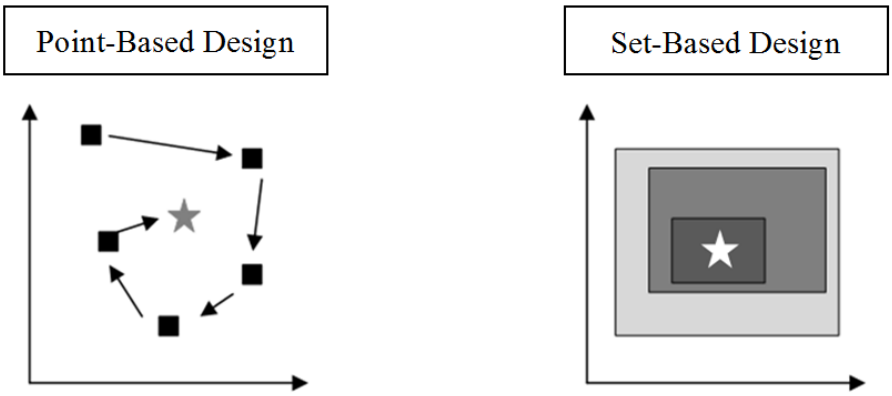

Figure 1 Point-Based Design vs Set-Based Design

Another characteristic of Set-Based Design, pointed out by Liker is that for the success of this design approach, also the different stakeholders must follow the same principles and cooperate through proper communication, e.g. the suppliers (Liker et al. 1996). Hence not only the designers are involved in the set-narrowing process but also the rest of stakeholders, finding a final solution which should be feasible in every aspect. Besides, a good job exploring solutions on one project can lead to a very focused search and much more rapid convergence on a design in later projects (Sobek II et al. 1999).

Set-Based Design assumes that reasoning and communicating about sets of ideas leads to more robust, optimised systems and greater overall efficiency than working with one idea at a time, even though the individual steps may look inefficient (Liker et al. 1996). Theoretically, by applying this approach meticulously enough, all the backtracking or reworking might be eliminated, although this could eventually increase delays and cost.

The main principles of this approach are (Sobek II et al. 1999):

1. Map the Design Space
    a. Define Feasible Regions: determine the primary design constraints on each subsystem, based on past experience, analysis, experimentation and testing, and outside information.
    b. Explore Trade-offs by designing Multiple Alternatives: creating trade-off curves about the relationship between two or more parameters. The other stakeholders (e.g. suppliers) also present multiple alternatives for each design alternative.
    c. Communicate Sets of Possibilities: an excellent solution from one perspective may be a poor solution from another. Use of design matrices, see *Figure* 2.

		Evaluation Criteria				
		Structural	Construction	Cost	CO_2	Etc.
Potential solutions	Bridge 1	O	O	Δ	♦	
	Bridge 2	O	♦	Δ	Δ	
	Bridge 3	♦	Δ	O	Δ	
	Bridge 4	Δ	♦	X	X	
	Bridge 5	O	Δ	♦	X	
	Bridge 6	Δ	Δ	♦	X	

O - Excellent ♦ - Acceptable **Δ** - Marginal **X** - Unacceptable

Figure 2 Example of a design matrix

2. Integrate by Intersection
    a. Look for Intersections of Feasible Sets.
    b. Impose Minimum Constraint: not only in the main design but also in the sub-designs carried out by the suppliers, “let them choose as well”.
    c. Seek Conceptual Robustness: Robustness = designs which are functional regardless of physical, manufacturing, weather and market variations. Conceptual robustness = create designs that work regardless

of physical, manufacturing, weather and market variations; but also of what the rest of the team decides to do.

3. Establish Feasibility before Commitment
    a. Narrow Sets gradually while increasing Detail: eliminate possibilities rather than picking the best. Narrow in different steps. Keep some back-up alternatives.
    b. Stay within Sets once committed.
    c. Control by Managing Uncertainty at Process Gates: design process as a continuous flow, with information exchanged as needed.

These principles are not steps, prescriptions, or recipes and they can be applied to each design project differently. Not only Toyota, from which it became known, has been using this approach but also other companies such as the aircraft engine division of General Electric. As a conclusion, any product development organisation that can master these principles and their application may be able to radically improve design and development processes (Sobek II et al. 1999).

2.2 History of Set-Based Design

Although it has not been established a specific date as starting point of this theory, the first documented reference about the practice of Set-Based Design appeared in the mid 1990s, when the description of this approach was included in a journal article by Ward , in which the existing and successful design system of Toyota was evaluated and discussed. This concept was developed to improve the car manufacturing process of this firm, and it was called the Toyota's Second Paradox by the authors, after the success of Toyota's production system (Ward et al. 1995). After this first introduction, the same group of authors developed the theory and established the principles of Set-Based Design as applied in Toyota (Sobek II et al. 1999). The principles of Set-based Design have also been discussed in other papers such as (Stephenson & Callander 1974), (Finger & Dixon 1989a), (Finger & Dixon 1989b) or (Aganovic et al. 2004).

During the last decade, some analytical formulations based on this approach were developed. These formulations dig deeper into the theory and try to connect its principles with other existing methods in order to develop the design process. First, (Wang & Terpenny 2003) described an interactive evolutionary approach to synthesize component-based preliminary engineering design problems, by combining Set-Based Design generation and fuzzy design trade-off strategy. Few years later, (Nahm & Ishikawa 2005) proposed a novel space-based methodology for preliminary engineering design, where the designer's preference structure is included in both the design space and the performance space by considering an aggregated preference and a robustness index. That methodology was based on finding a ranged set of design solutions that satisfied changing sets of performance requirements. By the late 2000s (Madhavan et al. 2008) proposed a Set-Based multi-scale and multidisciplinary design method in which distributed designers managed interdependencies by exchanging targets and Pareto sets of solutions, showing higher efficiency than the traditional design techniques. Two other formulations based on combination of existing theories were born at the end of the decade when (Shahan & Seepersad 2009) described a Set-Based approach to collaborative design, in which Bayesian networks are used to represent promising regions of the design space; and (Malak et al. 2009) combined the framework of multi-attribute utility theory, the perspective of Set-Based Design and the explicit mathematical representation of uncertainties into a single approach to conceptual design that handled inaccuracy. Finally, (Avigad & Moshaiov 2010) introduced a computational approach to support concept selection in multi-objective design with delayed decisions.

Regarding the area of application, the concept of Set-Based Design has been widely applied and assessed in the field where it was first developed, i.e. manufacturing and production development (Ulrich & Eppinger 2004) and (Ford & Sobek II 2005). However, Set-based design has also been studied for its applicability in the field of software engineering, by developing a new concept called Set-Based Parametric Design (SBPD) which combines the Set-Based Design practice with the parametric modelling technique widely used in most 3D-CAD systems (Nahm & Ishikawa 2006). It has also been studied in structural and HVAC system design (Lottaz et al. 1999), by suggesting the use of constraint solving to express possibly large families of acceptable solutions in order to facilitate and abbreviate the negotiation process; or in the field of ship design (Hannapel et al. 2012) where a novel multidisciplinary design optimisation (MDO) algorithm was developed, which assesses the variables in terms of sets.

Finally, there is need to mention that since this approach requires tools for narrowing the sets and choosing the best alternatives, optimisation techniques have been considered as suitable in order to evaluate those sets such as particle swarm optimisation (Eberhart & Shi 2001) and (Eberhart & Kennedy 1995).

2.3 Robust and Integrated Design

Two design concepts have to be mentioned in this report due to their implications and connections with Set-Based Design. On one hand, Robust Design is a widely used concept in many different fields which means designs that stand variations. As aforementioned, one of the principles of Set-Based is to seek for conceptual robustness which includes creating designs optimised regardless of physical, manufacturing, weather and market variations; but also of what the rest of the design team decides to do (Sobek II et al. 1999). On the other hand, Integrated Design is a more recent concept, applied to the construction industry, which looks for a better integration of the different construction phases and stakeholders. Again highly related with another characteristic of Set-Based Design, which is full collaboration between design team members and other stakeholder for better results (Liker et al. 1996).

As a brief description, Robust design searches for solutions which are immune with respect to production tolerances, parameter drifts during operation time, model sensitivities and others (Beyer & Sendhoff 2007). The name and methods became popular principally after Taguchi's methods were released in the late 1990s (Taguchi 1986). Robust Design has almost as many denotations as fields of application, e.g. Eurocode defines structural robustness as *"the ability of a structure to withstand events like fire, explosions, impact or the consequences of human error, without being damaged to an extent disproportionate to the original cause"*(SS - EN 1991-1-7: 2006). Nevertheless, when it comes to conceptual structural design, robustness can also be described as the ability of a structural concept to fit within different environments and situations, which is often referred to as flexibility. These environmental or situational variations are referred to as *Type I variations* (Chen et al. 1998) and flexible designs should be able to require little modifications in order to be applied to different contexts, i.e. a flexible bridge concept which can be used within a range of span lengths by changing its dimensions a little. For more information, an overview of Robust Design can be found in (Park et al. 2006).

Otherwise, Integrated Design is one of the phases of what is usually referred to as IPD (Integrated Project Delivery) which was conceived as an alternative construction project management approach to improve the known low profitability in the construction industry (Egan 1998). IPD motivates collaboration throughout the design and construction process and between stakeholders, tying stakeholder success to project success (AIA 2007). The principles and methodologies related to IPD can be extrapolated to the particular characteristics of the design phase. Successful use of Integrated Design involves changes over the traditional design practice. Example of these changes include: a team approach, support for innovation and tolerance of failure or strong lateral linkages and decentralised decision making (Owen 2009). One of the identified principles of integrated design of structures is a parametric design approach. Being the parametric design approach the most advantageous approach based on the theoretical frames of computer aided design, knowledge based engineering and generative design (Rempling et al. 2014).

2.4 Computational Optimisation

Modern engineering cannot be conceived without the power of computing. Computers are not only able to render immense amounts of calculations in short periods of time but they also permit automatic calculations. The first characteristic is directly related to computational structural analysis, e.g. Finite Element Analysis, and with optimisation techniques both used in performing full analysis or in sorting results. The latter, will be one of the pillars of the methodology developed in this project. A brief survey of Computational Optimisation is presented in this chapter in order to present tools and techniques which might be used in this or similar works.

According to (Shi 2007), optimisation is one of the most powerful tools to ensure efficient use of scarce resources. The first widely used method was Linear Optimisation in 1948 (Ben-Tal & Nemirovski 2013), and it is still very useful for the modelling of practical problems. This initial strategy was followed by other different new methods, such as convex quadratic optimisation and semi-definite optimisation (Ben-Tal & Nemirovski 2013) and (Wolkowicz et al. 2000) to model both economic and engineering problems.

However, new problems and applications showed the need to develop new techniques allowing the researcher to optimise other situations like discrete problems or finding explicit solution (Laier & Morales 2009).

When computers started to be available for research use, optimisation suffered some kind of transformation or adaptation to the new possibilities and resources. This is how Computational Optimisation was born. Additionally, since uncertainty is always present in real world systems, it is desirable to find not only an optimal solution but robust as well, in order to be able to overcome the existing uncertainty (Yang & Koziel 2011). As the noble prize winner Kenneth G. Wilson predicted in 1982, computational modelling (including Computational Optimisation) is becoming the third paradigm of modern sciences, and it is being widely used since the raw calculation power available is immense**.**

With the use of computing, new optimisation strategies suitable for these resources were developed successfully. Evolutionary algorithms have been especially important in this sense. The best known are evolutionary programming (Fogel et al. 1966), genetic algorithms (Goldberg 1989) and evolution strategies (Rechenberg 1989). Despite the fact that these techniques were initially developed to simulate the behaviour of natural processes, they have been widely applied to other fields like structural engineering. It can be used, for example, to assess the existing damage in structures (Laier & Morales 2009).

Finally, over the last twenty years, artificial intelligence techniques have been developed as a way to reduce the time consumption procedures of some scientific methods, such as Finite Element Analysis. The latter is usually done by the application of neural networks that are able to predict Finite Element outputs from given inputs (Adeli & Park 1995), (Berke et al. 1993) and (Papadopoulos & Eds 2013). With this technique, by testing a reduced amount of samples and then interpolating for intermediate values, a big amount of inputs can be evaluated in a more effective way, reducing computational cost. It is important to note that this technique is easily verifiable, since the only check that should be done is comparing real Finite Element outputs to the outputs obtained from the neural network.

2.5 Set-Based Design in Structures

There has not been much development in evaluating or implementing Set-Based Design in Structural Engineering until now. Only a few works such as a Set-Based methodology for reinforcement design (Parrish et al. 2007) and (Parrish 2009), a system to improve approval process for rebar estimation based on communication between the different stakeholders (Castro-Lacouture et al. 2006), or the evaluation of the capabilities of Set-Based Design using S-BIM (Structural- Building Information Modelling) (Lee et al. 2012).

Nevertheless, other design approaches strongly related with Set-Based Design have been developed in the field of structural engineering. A good example of this is (Dalton et al. 2013) which describes an optimisation-based methodology for structural design, considering safety, robustness and cost. In this approach, several designs are evaluated through Finite Element Analysis, designs that are defined upon variability of the design parameters. This methodology is described as based on Robust Design and it requires optimisation techniques and pre-selection tools for the initial set of alternatives, e.g. max-min Latin-Hypercube sampling (LHS), for reducing the computational cost of the calculation of all combinations, (van Dam et al. 2007) (Morris & Mitchell 1995).

3 SET-BASED BRIDGE DESIGN

This chapter aims to present the motivations and bases of the methodology which has been developed within this project.

Some main issues that are related with the inefficiency of bridge design and were intended to be solved, at least partially, by this methodology are the following:

- Bridge design is based on Point-Based Design, which is prone to require rework and hence extra cost. This issue is even more critical as some of the assumptions at early stage of the design of bridges, which have high influence in the overall design, are made based on rules-of-thumb or trial-and-error.
- Bridge concepts are usually meant to meet specific requirements and fit within a specific environment, and therefore big changes are necessary if the same concept wants to be reused in a different but similar situation.
- Miscommunication between different phases within the design process and between stakeholders can lead to unfeasible solutions. Besides, there is scarce collaboration between designers, contractors and suppliers.
- The pre-design is normally made based on hand calculations which extremely limit the number of design alternatives that are assessed. Hence, an optimal solution is difficult to be found.

The impact of these four main issues can be avoided or reduced by applying some techniques and methodologies derived from the theories aforementioned: Set-Based Design, Robust Design, Integrated Design and Computational Optimisation.

First, in order to define the problem to be solved, some limitations have to be set. The prime step in a bridge construction project is to determine the location of the structure, depending mainly on traffic conditions. However, this stage is outside the scope of this project and hence, the decision for the bridge situation is considered to already be made.

Once the location of the bridge has been decided, there are some parameters which are already predefined, like the bridge span or width and number of lanes. Nevertheless, other parameters have to be selected by the designers such as the type of bridge, dimensions or reinforcement in case of concrete structures.

Although this work aims to develop a systematic methodology to optimise bridges and help in their design process, it does require some pre-decisions like the type and shape of the bridge or the structural elements which will compose the whole structure. For this reason, this thesis will start from the stage where the bridge has been defined in type, material, general shape and structural components. However, this does not mean that only one bridge model can be assessed at the time, but an evaluation of different models can be performed in order to obtain the most suitable one out of bridges of different materials, shapes or some other characteristics.

Another point that requires highlighting through this thesis is that the scope of this project focus on the predesign stage and it does not apply to the final design stage, where a full analysis as it normally performed or verified is always needed.

The Set-Based Design theory as it has been described before is based on considering sets of alternatives, as long as they can be considered as feasible, including also the different stakeholders in the decision-making process. However, the methodology of applying this theory can widely vary, depending on the problem that needs to be faced. The narrowing process from the Set-Based Design theory, wherein the different stakeholders are meant to have direct influence, can be enhanced by including some of the main features of Integrated Design, such as the collaboration between design phases and members. In order to make flexible designs, Parameterisation, one of the foundations of Robust Design, i.e. also referred as Parametric Robust Design, can be applied, increasing the adaptability of the bridge to meet new requirements. Finally, all this could not be applied without using the power of computing, and some of its potential is used such as computational Finite Element Analysis or Computational Optimisation techniques.

In this project, two different phases can be distinguished. The first phase is based on a combination of the concept *Set of Alternatives* and *Parameterisation*. In this phase, the parameters considered for characterising a bridge model are defined as wide ranges, whose combinations will define a set of bridge alternatives.

These alternatives are then evaluated by FE analysis carried out in powerful multi-node computers to reduce the computational time of the calculations. A first filter might be considered at this stage in order to narrow the set of alternatives, rejecting those which do not fulfil some criteria or thresholds. This method sorts out a set of alternatives containing the viable bridges among a huge number of them, regarding their structural behaviour and also some construction restrictions, e.g. structural capacity, deflection limitation or number of reinforcing layers that can be feasibly placed in a beam.

The second phase consists on the management of the outputs from the former phase in order to find out the optimal alternative or set of alternatives out of the whole set of bridges initially selected. This is carried out by multi-objective optimisation, where not only the structural response is considered but also other aspects such as material costs, time estimation of the construction process or supplying.

4 PREDESIGN OF BRIDGES ACCORDING TO EUROCODE

The methodology developed in this project was focused on pre-design of structures, specifically bridges, which would be further developed in later stages of the process on its way to the final design. Therefore, it was important to establish beforehand how accurate the structural checks should be in this early design phase.

Accuracy of the checks in this framework was an issue that had to be analysed very carefully due to the direct consequences it has for both the results and the performance of the process. Predesign should be complete enough to ensure correct results that would later lead to solid final design stages, but not as complex as final design since it would make the preliminary design stage too long and would probably narrow down the solution space too early.

As a consequence, this phase was considered as a compromise between accuracy of the results and conservation of the essence of Set-Based Design, where a lot of different alternatives should be considered and assessed. In the next sections both bridges considered will have its predesign phase checks described, referencing Eurocode as the main standard used for the design.

In the following chapters different rules and methods for the assessment of structures will be presented as general as possible, even though for the sake of precision they are specifically selected for the case of study. For this purpose, it was taken into account both characteristics of the study cases and the scope of the methodology. However, it is still applicable to any other case by selecting the proper checks from the standards.

4.1 Concrete bridges

4.1.1 Dimensional compatibility

As a first check, dimensional compatibility of the alternatives was assessed in order to ensure if the structure was feasible considering buildability aspects. As stated in previous chapters, Set-Based Design implies the consideration of multiple alternatives. If these alternatives are built by random combinations of dimensional parameters within specified ranges, it might lead to non-buildable designs. The main example of these incompatibilities might be considering a number of beams that cannot fit within the slab width provided, and hence, this alternative should be discarded before the analysis.

4.1.2 Deflection

Deflection is a limitation that should be taken into account even though it is not usually critical in short-mid span concrete bridges. Eurocode itself does not limit the overall maximum vertical deflection of bridges but refers to other issues such as dynamic impact of traffic, cracking or gaps in expansion joints. Hence, this limitation is normally up to the national road administrations. As an example, the Swedish standard limitation from section B.3.4.2.2 (TRVK BRO 11 2011) was taken. Consequently, L/400 was the limit value for the vertical deflection.

As a consequence of the designed methodology, the deflection was calculated by the Finite Element Analysis of the bridge, which means before the reinforcement is designed. Hence, the value obtained could not be considered as the final value, since only the stiffness from the concrete was taken into account, neglecting the additional stiffness provided by the reinforcement.

As a consequence, two approaches were considered to handle this issue. The first one stated that since the stiffness taken into account for deflection calculations was always lower than the real one, since reinforced concrete would always be stiffer than the same concrete with any kind of reinforcement, the results were always on the safe side. The second approach was the execution of a second analysis considering the stiffness of the reinforcement once it was designed, but it was discarded due to the amount of resources needed to perform the analysis.

Although oversizing of structural members might happen by considering the first approach, the unlikeliness of the deflection as a limiting parameter in this case, as well as the huge impact that the adoption of second approach would have, lead to the decision of using the first one. The predesign phase stage together with the Set-Based Design methodology supported this decision, since in case that this parameter could be limiting in any alternative, it would be easily discarded in later phases of the design.

4.1.3 Sectional Design

In concrete bridges with a high number of beams, as in the studied case, the quotient between the separation of beams and the width of the deck is expected to be fairly low. It was then considered that instead of designing the beams and the slab separately, the deck should be designed as a set of consecutive T-sections. Eurocode includes this approach and sets the limitations that must be fulfilled to be able to consider this design strategy. Therefore, these limitations were checked before the analysis in order to be able to ensure the integrity of the results.

According to EC2 the main requirement for the consideration of this approach is having a spanning distance between beams so that the member between beams cannot be considered as a slab according to (SS-EN 1992-1-1:2005, Section 5.3.1(4)), i.e. separation of beams lower than five times the thickness of the deck.

For the design of T sections, the effective flange width was considered as stated in (SS-EN 1992-1-1:2005, Eq. 5.7).

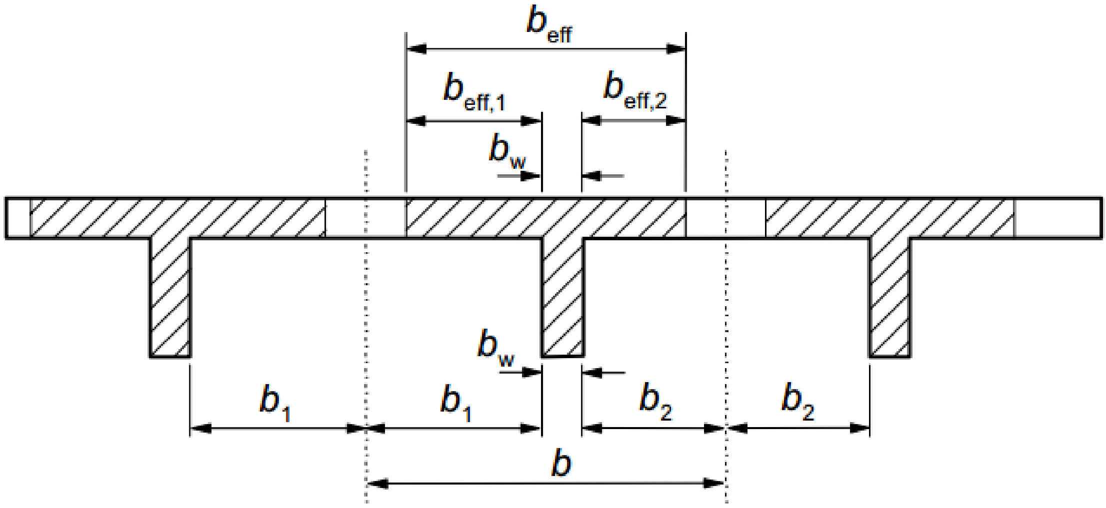

Figure 3 Effective flange width parameters (EN 1992-1-1:2005 5.3.2)

As previously discussed, the bridge was considered as a group of concrete T-beams subjected to a bending moment and shear force. Hence, those beams would need steel reinforcement in the cases where the capacity of the concrete is not enough to carry those load effects.

Reinforcement design was divided in two main categories, bending and shear reinforcement, following design procedures according to Eurocode 2.

4.1.3.1 Bending reinforcement

For the sectional design in ULS, linear elastic approach was used with no moment redistribution considered. For the equilibrium equations, yielding in the reinforcement bars was assumed. The value of the lever arm was approximated by:

$$z \approx 0.9d$$

Once the reinforcement was designed, ductility was checked in order to know if the design fulfilled the Eurocode limitation.

4.1.3.2 Shear reinforcement

Shear capacity was calculated according to (SS-EN 1992-1-1:2005, Section 6.2), checking the capacity of the concrete to shear and designing the reinforcement when needed. Once reinforcement was designed, it was assumed that the shear force was completely taken by the reinforcement, not taking into account concrete shear resistance in that case.

Concrete resistance was defined as stated in (SS-EN 1992-1-1:2005, Section 6.2).

In case shear force exceeded the concrete shear resistance, reinforcement was required and designed according to (SS-EN 1992-1-1:2005, Section 6.2.3). To keep buildability of the structure, shear reinforcement was restricted to 100 mm minimum spacing, and multiples of that value for other spacing values.

4.1.4 Slab design

Although the sectional design of the bridge is approached as described in the previous section, this is only a design approach that cannot be taken as literal. It is still necessary to reinforce the concrete slab to fulfil the code requirement. However, provided that the whole section was considered for the beam reinforcement design, the code allows reinforcing it according to the minimum value of reinforcement. Therefore, area of steel needed in the slab was established according to (SS-EN 1992-1-1:2005, Section 9.2.1.1.(9.1N)).

4.1.5 Crack width

Although it is not recommended to make an advanced cracking analysis in preliminary design, it was necessary to consider this in some way in order to ensure solid alternatives for subsequent stages of the design. As a result, the crack width was limited to the threshold value that Eurocode sets as maximum acceptable crack width w_{max}=0.3mm as stated in (SS-EN 1992-1-1:2005, Table 7.1N). By including crack analysis, even with this simple approach, a lot of poor designs were discarded and consequently a lot of calculations would be avoided in the next step of the design process. The crack width was estimated according to (SS-EN 1992-1-1:2005, Section 7.3.4).

4.2 Composite bridges

The term composite when it refers to bridges can be ambiguous, and often it is understood as Fibre Reinforced Polymer (FRP) bridges. In this project the term is the one considered in Eurocode 4, which means steel and concrete bridges, specifically bridges made as a concrete deck resting on steel girders.

4.2.1 Dimensional compatibility

As explained in the previous section, dimensional compatibility had to be checked, in order to ensure the geometric feasibility of the structure before checking its structural behaviour. As it happened with the concrete bridge, it was important to check if the beams were overlapping. Only extreme configurations would lead to overlapping, but it would be disastrous for the methodology if geometrically non-feasible bridges would reach later stages of design. It was therefore checked whether the flanges of the steel I-beams were overlapping.

Additionally, since the steel girders had variable section along the beam, it might happen that the section becomes zero because of an extreme combination of parameters. Since this issue turns the alternative into unfeasible, this was another check that was introduced in the predesign phase.

4.2.2 Deflection

Deflection was checked in the composite bridge as well, in order to ensure it is under the threshold set by the code. As stated previously for the concrete bridge, L/400 was taken from the Swedish standard from the road administration as maximum vertical deflection allowed. It is important to note that, while in the concrete bridge it was generally not a critical parameter, it is much more important when steel girders are present.

Additionally, in contrast with the concrete bridge case, deflection can be calculated together with the other variables since the stiffness of the steel is considered from the beginning of the process, avoiding one of the issues stated before for the other case, i.e. the concrete slab has very little influence in the overall deflection.

4.2.3 Stresses in steel

In traditional design, capacity of the steel sections is calculated according to classic mechanics by means of sectional forces or moments applied. In this case, since the methodology was using a Finite Element Analysis (FEA), the approach was different.

In FEA deflections and stresses are calculated at the beginning, and by means of integration moments and forces are obtained. It was considered unnecessary to calculate steel capacity by using moments and forces obtained from stresses, since the actual limitation of steel strength is the stress.

Consequently, the approach for the calculation of the capacity of steel was the comparison of the stresses obtained from FEA (Von Mises criterion) with the design yield stress of steel (SS-EN1993-1-1:2005, Annex C.8). According to EC3, the steel element has to fulfil at every point the equation included in (SS-EN 1993-1-1:2005 Section 6.1).

4.2.4 Buckling

As in every steel structure subjected to transversal load, buckling was checked in the bridge. Two different classes of buckling were considered, lateral torsional buckling and shear buckling.

4.2.4.1 Buckling of members in bending. Lateral Torsional Buckling during construction

Although the structural behaviour of the bridge is not prone to buckling in compression, it is important to check the members subjected to bending and prone to buckling, i.e. Lateral Torsional Buckling. This is important during the construction phase, while the concrete slab is being casted on the beams. In this phase, rigidity coming from concrete cannot be considered for the structure, but load coming from the self-weight is, resulting in a limit case regarding Lateral Torsional Buckling. If this limit case is not taken into account, it might happen that the bridge needs special construction considerations even if it is structurally feasible once constructed. If special measurements such as stiffeners or additional supports during constructions phase are not considered, the bridge should be tagged as non-feasible and discarded.

The check needed for lateral torsional buckling was made according to (SN003a-EN-EU).

The check needed for lateral torsional buckling was made according to (SN003a-EN-EU), where a critical moment is defined, which represents the maximum bending moment that a girder can carry without buckling.

4.2.4.2 Buckling of members in bending. Lateral Torsional Buckling under traffic load

In simply supported composite bridges the lower flange of the girder is always in tension, the upper flange is linked to the concrete slab and only a small length of the web might be in compression, which makes a lateral torsional buckling under traffic load a negligible check. Nevertheless, in bridges with fixed supports, a part of the lower flange near the supports is subjected to compression, and therefore, not only buckling during the construction phase is prone to happen but buckling under the traffic load might occur. The interaction between the deck and the upper flange makes not suitable the method proposed above and a different alternative has to be considered.

Even if the most suitable alternative should be the general method from (SS-EN1993-1-1:2005, Section 6.4.3), it implies a detailed analysis involving specific FEM buckling assessment which makes it unsuitable for pre-design considerations. Although several assumptions are not fulfilled, e.g. the lower flange is not a uniform member or the moment distribution is highly variable; the simplified method (SS-EN1993-1-1:2005, Section 6.3.2.4) which considers the part of the lower flange in compression as a strut has been used in order to give a rough estimation of the buckling resistance of the girder.

4.2.4.3 Shear buckling

Finally, shear buckling was checked to ensure the correct structural behaviour of the structure. EC3 procedure was followed to compare the shear force in the web with the shear buckling capacity of the girder. Additionally, some checks to know if shear buckling occurs were included to avoid pointless calculations.

According to EC3, the checks needed for shear buckling should be done by following (SS - EN 1993-1-5:2006, Sections 5.1,5.2,5.8).

4.2.5 Slab design

In contrast to the case study of the concrete bridge, where design approach using T-beams was assumed, the composite bridge deck fulfil the requirements of EC to be considered as a slab, and had therefore to be designed.

4.2.5.1 Bending reinforcement of the slab

Bending reinforcement of the slab was based on the recommendations found in (Johansson et al. 2012) coming from (Eibl 1995),(CEB-FIP 2008) and (Blaaunwendraad 2010). First assumption adopted was that longitudinal moment was carried by the girders, considering it as a one way slab. Therefore, sectional design of longitudinal and transversal reinforcement was performed in a different way.

As stated in (Johansson et al. 2012), transversal reinforcement of the slab had to be performed considering sectional moments in the appropriate direction as well as torsional moments by using

$$m_{rx} = m_x \pm \mu |m_{xy}|$$

Equation 1 Sectional moments for the transversal reinforcement of the slab (Johansson et al. 2012).

where $m_x$corresponds to transversal sectional moment and m_{xy} corresponds to torsional moment.

For longitudinal reinforcement, moment was mainly taken by the girders but still some reinforcement was needed near the supports of the bridge due to restraint stresses in tension (both ends were fixed). Following (Johansson et al. 2012) recommendations, membrane forces were used for the calculation of said reinforcement as

$$n_{rx} = n_x \pm \mu |n_{xy}|$$

Equation 2 Membrane forces for the longitudinal reinforcement of the slab (Johansson et al. 2012).

where $n_x$corresponds to longitudinal membrane force and n_{xy}corresponds to the shear membrane force, both at the mid-surface of the slab. In both cases, μ is a factor that might be chosen due to practical considerations, but usually close to 1 as stated in (Johansson et al. 2012).

Together with the minimum steel reinforcement established by Eurocode and presented in the concrete bridge section, there was a maximum area of steel that set the limit to discard the alternative due to excessive amount of reinforcement. This limitation can be found in (SS-EN 1992-1-1:2005, Section 9.2.1.1(3)) and depends on the National Annex.

4.2.5.2 Shear reinforcement the of slab

In opposition to the concrete bridge, since the concrete slab in the steel bridge was heavily reinforced, shear capacity of the slab was checked to know if the reinforcement design could be avoided. After performing some checks, it was tested that shear capacity of the concrete is usually more than enough to withstand the shear force in the slab, so no shear reinforcement design was included in this predesign state. If a combination was assessed where shear reinforcement was needed, the bridge would be discarded to avoid non feasible bridges to go pass in the design process.

5 TOOL FOR ITERATIVE FINITE ELEMENT ANALYSIS

5.1 Motivation

As it has been presented in *Chapter 3*, predesign of bridges has traditionally been a process based on multiple assumptions which can lead to unfeasible designs. However, although those assumptions are supported mainly on the designer's experience, the checks to verify a design alternative follow some sort of standard, e.g. Eurocode. Regarding the assumptions, the fact of having to guess and may end up selecting a suboptimal alternative could be avoided if a large amount of alternatives could be taken into account by implementing an automatic analysis process, rather than the few alternatives that are nowadays handled by manual calculations. In this automatic analysis process, the checks aforementioned should be included in order to simulate as close as possible the designer's criteria, and be sure that the process is just widening the number of alternatives considered in pre-design without skipping any required verification considered in the traditional design process of bridges.

The checks from standards mentioned in *Chapter 4* can be normally used without having to introduce any sort of computational analysis, and hence can be performed by hand calculations. However, new computational techniques, especially Finite Element Analysis, have opened new possibilities in structural design, leading to faster and more accurate analysis. These standards already approve in some extension the use of this type of analysis and their combination is promising.

Hence, the basis of this methodology was to implement the required checks, commonly used in traditional design practice of bridges, within an automatic process able to analysed large numbers of bridge alternatives, supported when needed on the results of a Finite Element Analysis. Throughout the process, the different checks and standard requirements select only the structurally-feasible alternatives, leading to a final set of possible alternatives.

Since the Finite Element Analysis engine that will be used, i.e. ABAQUS, is based on the programming language Python, the most suitable way of automating the process was the development of a Python script which controlled the Finite Element Analysis and the checks from the specific standards. The structure and the different aspects concerning this script and its functioning are presented in the following sections.

5.2 Structure

As the script was intended to be a tool for the designer, the structure follows a pattern that can be easily isolated and identified with the different phases of the traditional pre-design. A general idea about it can be obtained by checking Figure 4, where a flow chart describing the script structure can be found.

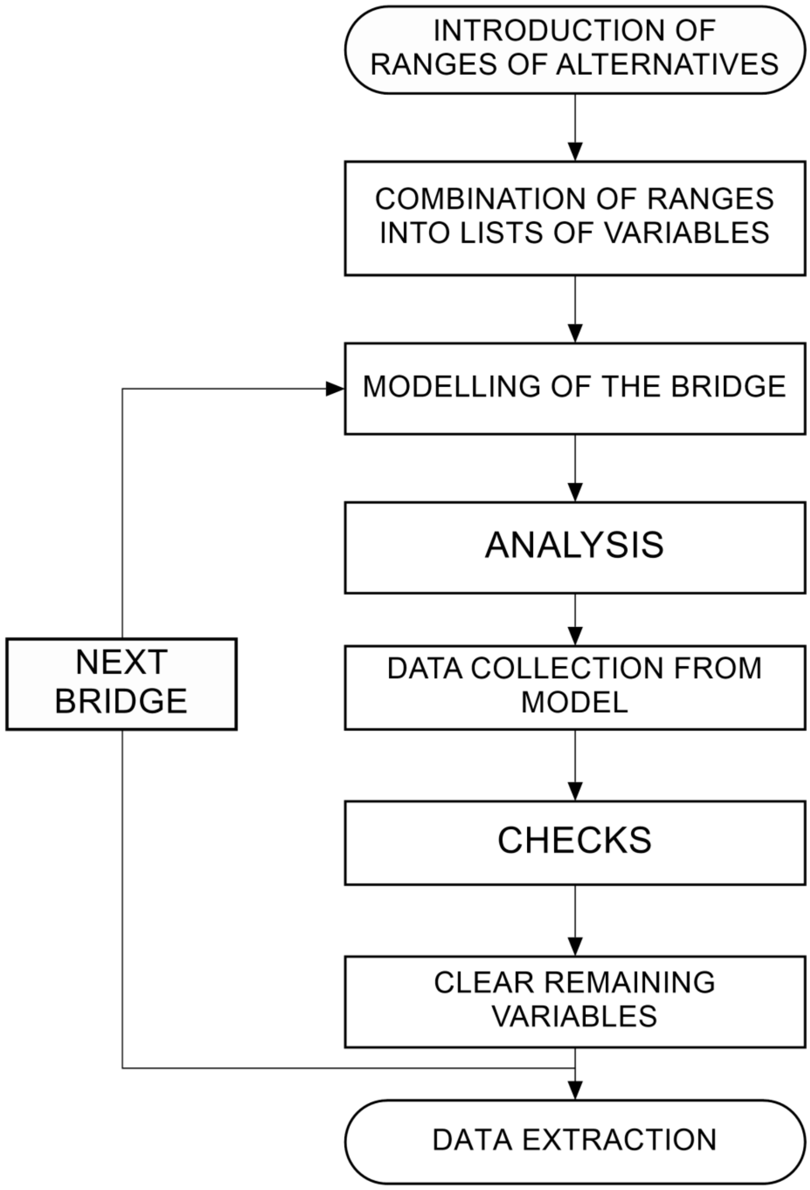

Figure 4 Flow chart describing the script structure

First, a module that creates a list of bridges by combining dimensional parameter from given ranges was implemented. It is important to notice that both variable parameters (those which vary during the loop) and previously fixed parameters are used for this purpose. More details about the parameterisation can be found in the case studies, *Chapter 6*.

Secondly, the script performs the evaluation of each bridge of the list. This process was carried out by a loop that performed the creation of the model of every bridge in the set, prepared and launched the analysis in ABAQUS sequentially. Details regarding the models can be found in *Chapter 6*, and details about the analysis can be found in *Subchapter 5.4*.

Following, the required data from the analysis that will be used in the standards' verifications was extracted from the model by means of three ABAQUS features: directly from the output fields of the whole model or parts of it, from paths that defined along which line the outputs were going to be extracted from, and from a feature called Free Body Cut, which computes the sectional moments and forces over an specified section of the model by means of the nodal forces in the section.

Next, some of these extracted data were used directly for checking some standards' requirements, e.g. maximum deflection or maximum stress; others were checked against resistances from the standards, e.g. lateral torsional buckling or shear buckling; and others needed to be further transformed in order to be used for verification, e.g. crack width.

In case there was need to design reinforcement of concrete members two methods were used regarding the type of structural element, an internal loop determined the reinforcement configuration for a range of bar diameters in concrete beams; and the sectional moments and membrane forces determined the reinforcement in case of concrete slabs.

Finally, once the bridge was modelled, analysed and verified, there were some instructions implemented to carry out the data extraction from the model, prepare the raw data for further analysis and clear the remaining information in the model. The latter was necessary to prepare the script to perform a new iteration for another bridge configuration, avoiding possible errors derived from old variables leftovers stored in the memory and an excessive required storage capacity. During the data extraction, the script checked if the bridge alternative fulfilled all the requirements. At this stage, it is important to notice that some of these requirements were *must criteria* and therefore they determined whether or not the alternative was considered within the final set of possible alternatives, e.g. maxim stress or deflection; or *want criteria* which did not discard the alternative but weighed the alternative negatively when the final set was assessed, e.g. buckling, avoided by including stiffeners. More information about the types of criteria can be found in *Subchapter 5.7*.

It is important to note that the whole process previously described was performed automatically, which means that the designer does not have to interact or introduce new information or parameters while the analysis is running. Therefore, the script was oriented to be able to work for as long time as needed, being possible to cover as many cases as desired.

5.3 Input Data

As it was presented earlier, the initial stage of the design process should be an assessment of the construction site of the bridge, taking important information such as total length of the bridge or required width, for example, according to the number of traffic lanes.

Depending on the intended use of the bridge, it is necessary to introduce the correspondent loads as stated in the codes. Typical loads are self-weight or traffic loads. It is also necessary to decide the material class that will be used for every structural element. This was done by introducing the mechanical properties of every material in use, such as Young's modulus, density or Poisson's ratio.

It is important to note that everything described before were considered as fixed parameters, so they were established at the beginning of the design process and were not changed in the iterative process.

On the other hand, there are some parameters that were not considered as fixed values, but included within ranges of variables established by the designer. The suitability of the ranges would be decided and limited by the designer, according to his own experience and standards' recommendations. These parameters were the dimensional parameters which defined each bridge alternative. In case of the design of concrete members, diameters of reinforcement bars could be varied as well in order to find the most suitable reinforcement configuration for every case.

Finally, there were some parameters included in the simulation that were not directly related with structural behaviour but were important for the validity of the results as well as for the general performance of the process. These parameters are mainly related with the computational performance of the process, such as number of computational threads used in the simulation (logical and physical cores) or mesh density (size of elements). Additionally, it is also possible to decide the accuracy or complexity of the analysis by selecting which sections of the bridge would be individually analysed, e.g. sections where the sectional moment and forces are taken from. The more sections included, the more detailed the analysis would be but computational time might increase.

Therefore, it is interesting to select the right accuracy regarding the number of sections analysed, since exploiting this possibility might lead to very complex designs. For example, it could happen that the analysis of too many sections in a concrete bridge would result in a too complex reinforcement design, with different number of bars in each section that may difficult the construction of the bridge.

Since the aim of the script was assisting the engineer during the design process, it is necessary that the designer uses his own knowledge in order to help the script work better.

5.4 Type of analysis

Probably one of the most important features of the Finite Element simulation regards the type of analysis needed for each situation. While performing complex analyses may lead to more accurate results, it is important to take into account that these analyses may require too much information from the model as well as computational resources.

Since this methodology was intended for pre-design, advanced analyses are not required. Moreover, the faster the analysis is, the wider the set of analysed alternatives can be done. Therefore, linear elastic analysis was decided to be the most suitable, as it provided acceptable results for early phases and it was light enough to be iteratively performed.

However, it must not be forgotten that at least the selected alternative as final design will require a more complete analysis, i.e. non-linear analysis; for taking into account phenomena such as buckling or cracking.

Assuming this principle, it is possible to apply the methodology to the highest extent, by broadening the design space as much as possible and progressively narrowing it through the process. As a result, a lot of time and money would be saved by the utilisation of the tool, since early phases of the project can be carried out with a bigger number of estimations (by using a bigger design space) that would lead to better designs in later phases, since more possibilities were covered from the beginning.

5.5 Permanent loads

Permanent loads in the model are self-weight and an edge beam load.

The self-weight of the model was introduced as a gravity load by taking advantage of a previous definition of the materials' densities. Just with these densities, the software applies automatically the correspondent load to simulate gravity loading of the model.

The edge beam load was introduced in order to include in the model the additional size of the edge beams of the bridge. These beams are bigger than the other ones but its bigger size cannot be taken into account in the structural behaviour of the bridge. Hence, modelling of these beams had to be done by modelling an edge beam of the same size as the others, but applying an extra load on it to introduce extra weight that was not included in the previous load. The magnitude of the load was proportional to height and width of the beam depending on the case as well as concrete density. Load coming from the barrier expected to be placed on the edge beam was also introduced.

As seen in Figure 5, this load was applied on a surface equivalent to the edge beam in order to keep coherence between model and reality.

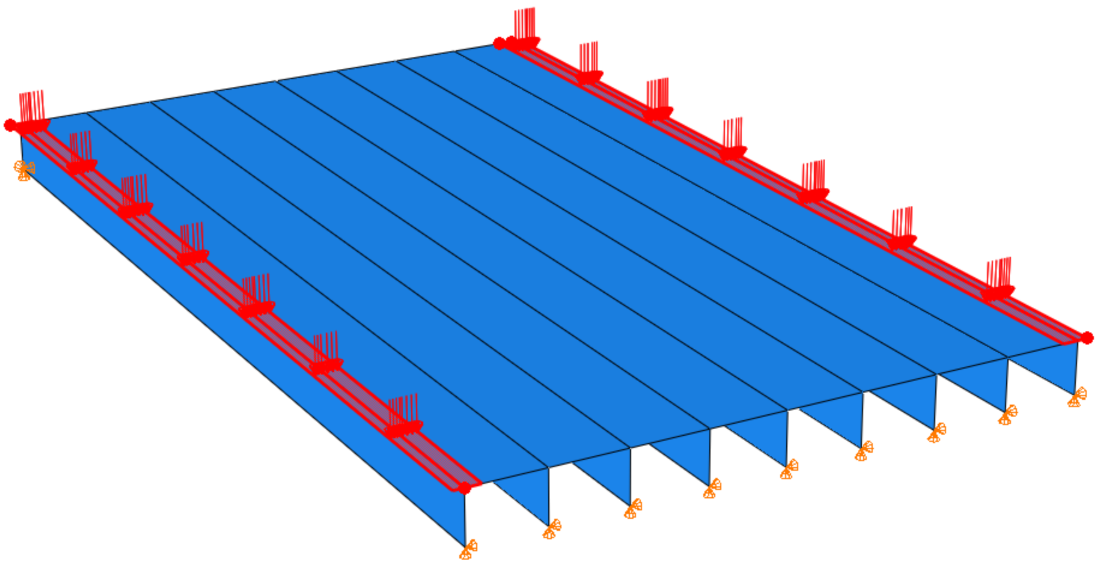

Figure 5 Edge beam load applied to a concrete bridge

5.6 Traffic Load

While performing the analysis and modelling of the bridges that were studied, it was decided that loading conditions were not realistic enough to produce valuable results and were not fulfilling Eurocode requirements. Consequently, an in-depth study was performed to solve this issue. It was concluded that some traffic load model should be integrated in the tool to ensure accurate results and consistent designs.

5.6.1 Load Model 1 from Eurocode

Once it was decided that a traffic loading model was necessary to be introduced in the tool, since traffic load is something that has to be definitely taken into account in bridge design, an in-depth study of the different Traffic Loading Models available in

Eurocode was carried out. As it was possible to introduce any available load model, Load Model 1 (LM1) was chosen because of its versatility as well as its general purpose that matched perfectly with the aim of the applied methodology, and more specifically with the tool developed for it.

Load model 1 is defined in Eurocode 1 (SS-EN 1991-2) as *"Concentrated and uniformly distributed loads, which cover most of the effects of the traffic of lorries and cars"*. Additionally, it is stated afterwards that *"this model should be used for general and local verifications"*.

The model consists in two partial loading systems that are intended to apply to the bridge loading conditions similar to the worst case scenario expected in reality.

First system is called Tandem System (TS), which is a double-axle concentrated load set with its specific loading stated in Eurocode. Values for the axle load can be found in Table 1.

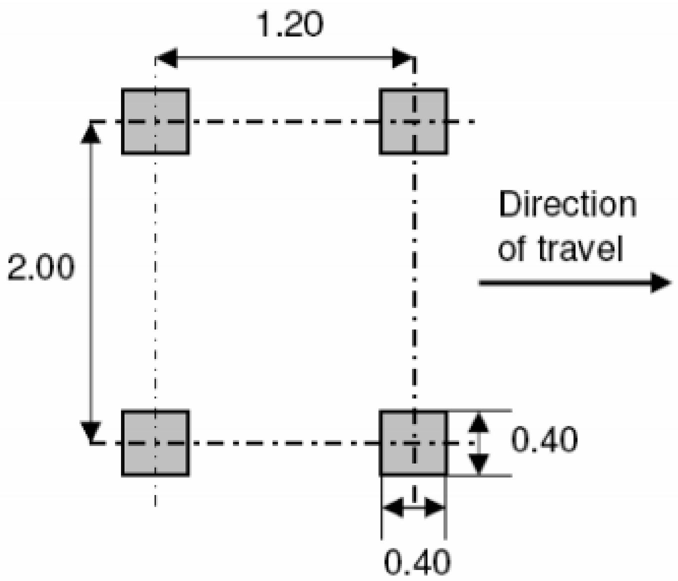

Figure 6 Tandem system from Load Model 1 (SS-EN 1991-2)

Second system is called Uniformly Distributed Loads System (UDL) and it is formed by a set of uniform distributed loads with pressure values as stated in Eurocode. These values can be found as well in Table 1.

The combination of both systems results in the loading model itself, as it can be seen in *Figure 7*. By the only application of these two systems it is possible to simulate the design conditions of a bridge subjected to heavy traffic conditions. However, to be able to reach worst case scenarios, i.e. design cases, it is important to place the different systems on the bridge deck in a correct way, by following Eurocode carefully.

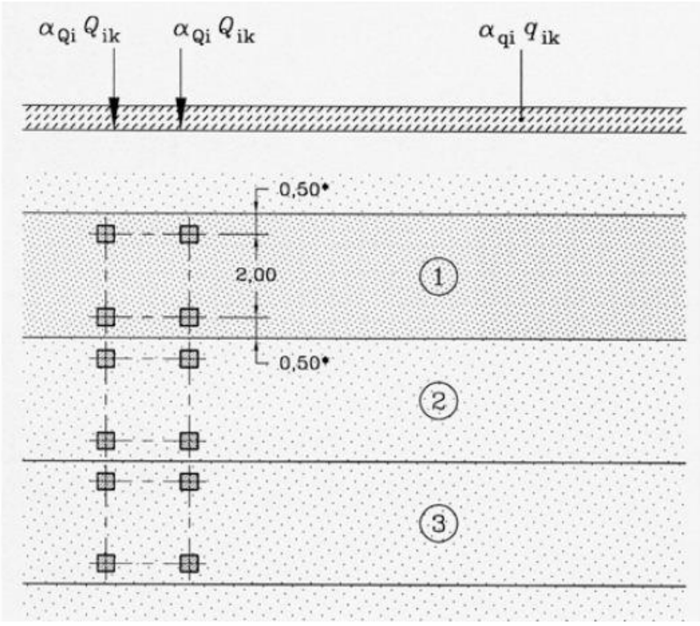

Figure 7 Load Model 1 from (SS-EN 1991-2)

Every loading model in Eurocode is applied individually to different surfaces called *Notional lanes*, whose width is perfectly established on beforehand.

Notional lanes are created to be able to impose different load conditions to different parts of the deck in a standard way, in order to have the same procedure for every case. They divide the carriageway part of the bridge deck in different surfaces with different loads applied, depending on the position. The remaining area that does not belong to any of the notional lanes is called *Remaining area* and it has its own loading conditions.

Notional lanes are very important in the model since they govern the amount of load placed in each part of the deck according to Table 1, by applying a different uniform distributed load in every lane. Moreover, they also have a huge impact in the tandem system since they define its location within the bridge deck.

Table 1 Load Model 1 Characteristic values (SS-EN 1992-1, Table 4.2)

Location	**Tandem system *TS***	***UDL* system**
	Axle loads Q_{ik} (kN)	q_{ik} **(or** q_{ik} **)** (kN/m^2)
Lane Number 1	300	9
Lane Number 2	200	2,5
Lane Number 3	100	2,5
Other lanes	0	2,5
Remaining area (q_{rk})	0	2,5

5.6.2 Implementation of the model

Once the loading model was decided to be LM1 and studied carefully, implementation phase started. It was necessary to make decisions about the basic implementation of the model as well as its performance and accuracy, trying to keep in mind the purpose of the applied methodology.

5.6.2.1 Basic modelling of the traffic load

In its very basis, loading model implemented in ABAQUS consisted in a very thin plate element with no modulus of elasticity and density, that would get applied the traffic load coming from the loading model.

The main advantage of this strategy was its high grade of versatility. Once integrated in the script, by adapting the dimensions of the plate it could be used in any previously modelled bridge. The connection between the bridge deck and this auxiliary element, referred as traffic plate, was solved with the *Tie function* from ABAQUS. By doing this, all the deformations applied to the nodes located in the traffic plate would be perfectly transferred to the bridge deck, giving the same results as if it was directly applied to the deck, as happens in reality.

Some other advantages found in having and independent instance for the traffic loading was the freedom to modify it with partitions, making easier the implementation of notional lanes and both load systems previously described.

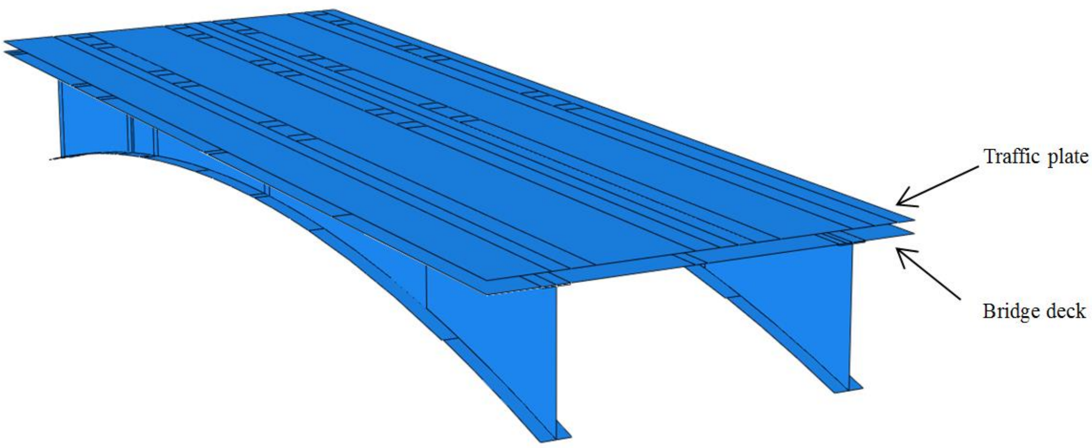

Figure 8 Traffic plate tied to the bridge geometry

5.6.2.2 Favourable loading

Another key aspect of the design was the management of favourable loads. According to Eurocode, when doing a global analysis Tandem Systems can be considered both in favourable and unfavourable cases for simplicity. On the other hand, it is stated in Eurocode that Uniform Distributed Load should not be applied in favourable cases (SS-EN 1991-2, Section 4.2), as this would produce more favourable results than expected. Therefore, it was necessary to assess which situations would produce favourable loading cases both in longitudinal and transversal direction.

In order to deal with favourable loads, it was decided to produce influence lines for reaction forces over the beams. For this purpose, every beam was considered separately, since one loading state might have favourable load areas when referring to one beam and do not have it when designing another one. This is because for the design of different beams, the influence line obtained comes from different reaction forces. As a result, multiple loading cases were derived from the original set, increasing the resources consumption but ensuring the assessment of the worst case scenario of the design.

As the main strategy to cancel favourable loads when needed, the approach used was the introduction of another load applied to the traffic plate in the region needed to be cancelled, featuring the same magnitude and opposite direction. By using this strategy, both implementation and behaviour of the model were smooth and versatile.

Since only single span bridges were considered in this project, no favourable part of the load was found due to longitudinal positioning of the load. In case of multiple span bridges, influence lines would have favourable loads zones and this should be taken into account.

Whilst in longitudinal direction there were no favourable loading situations, transversal direction loading had to be carefully considered and implemented in order to be sure that the design was being performed under the worst case scenario condition. As stated before, reaction forces over the beams were taken as a reference for the influence lines that decided the favourable zones for the design, and different cases were considered according to the beam that was being designed.

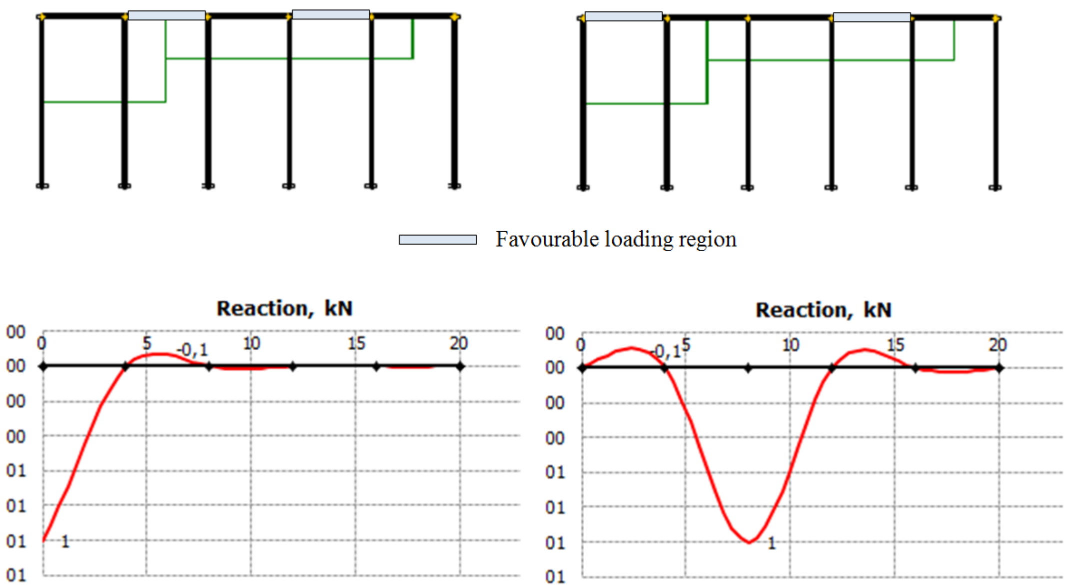

Figure 9 Comparison of influence lines between two design cases

In Figure 9, a comparison between two different design cases is shown. In this figure a bridge with six beams was considered, and two design situations are displayed: On the left, the design of the edge beam can be found while the design of the third beam (second internal beam) is showed on the right. As seen in the influence line diagram, in the first case, loading the span region between the first and the second interior beams would have a favourable effect on the reaction force over the edge beam that would lead to a wrong design of said beam if this was not addressed properly. Hence, it is necessary to unload that span region together with the fourth span, whose effect while smaller is favourable, as well. It was considered for simplicity that any favourable region further than two spans in any direction from the designed beam would be kept since the effect is negligible as can be seen in Figure 9.

Regarding the additional resources necessary for the inclusion of favourable unloading in the design, it is easily noticeable than one single tandem system position would lead to more than one load case, since the design of each beam demands unloading different favourable regions in each case. Therefore, the bigger number of beams included in the design, the bigger number of loading cases that will have to be taken into account. This will be derived later in this chapter, by showing the relation between different factors and the total number of load cases to be considered.

5.6.2.3 Management of the different load cases

Once the modelling phase of the traffic load was concluded, the management of raw data that this implementation generated was considered and optimised. Since the purpose of having the traffic model implemented in the tool was being able to assess the worst case scenario regarding traffic conditions, it was necessary to include different cases that would simulate the different positions of the only element that can be placed in different positions along the *Notional lane*: the Tandem system.

According to this, it would be necessary to have the possibility to simulate the bridge response while it is subjected to different loading conditions, depending on the positions of tandem systems. However, some limitations coming from Eurocode had to be taken into account:

- For global analysis, Tandem Systems should be assumed to travel centrally within the notional lanes.
- There is only one tandem system per notional lane.
- Only complete tandem systems should be considered.

Consequently, the only variation that was considered according to global analysis was the different positioning of the Tandem System along the lane. Moreover, taking into account the symmetry of the bridge, only positions between support and middle span were considered, in order to avoid pointless calculations of mirror cases from both sides of the bridge.

5.6.2.4 Performance of the loading model

At this stage one limitation that had to be handled arose. By considering too many different situations of the Tandem System the number of load cases might grow exponentially, increasing the computational resources required and harming the speed of the process. It was therefore decided to perform an in-depth study by considering possible consequences of any assumption taken within this issue as well as keeping in mind the scope of the methodology.

An important factor considered while implementing the traffic load was the resolution, i.e. the number of tandem systems in a lane, used for the movement of the loading combinations. However, it was decided to let the tool to have as much resolution as possible, giving the designer the possibility to adjust the accuracy/performance ratio by choosing the appropriate loading alternative for each situation. Therefore, resolution was defined as a function of the total length of the bridge, allowing as many tandem systems as possible while keeping a separation of

1.2m between them. This separation was set as threshold because it is the distance between axles in the tandem system model. According to this, resolution is defined as:

$$Res = \frac{L/2}{1.2} = \frac{L}{2.4}$$

Equation 3 Calculation of Resolution of Traffic Load Model

As stated before, while it was considered essential to be able to simulate and take as reference the worst case scenario, this should not compromise the performance of the whole process. It was therefore decided to implement three different sets of car placements in order to have different alternatives to choose from. By doing this, it was given to the designer the possibility to control how accurate the simulation should be depending on the design situation and/or the resources available.

Those three different Tandem System positioning combinations were implemented taking into account the expected behaviour of the model, in order not to have combinations that do not give new possibilities to the tool. Every combination was named as *Car Placement* followed by a number, from 0 to 2 in order to identify it.

Since these three alternatives were meant to assess the worst cases, some positions were common for all of them such as parallel tandem systems at mid-span or support regions. *Car Placement 0* (CP0) included every load combination where Tandem Systems travel parallel from support to mid-span. As can be seen in *Figure 10*, the tandem System moves at the same time from the support zone to the span. Although only six combinations are represented in the figure, this movement is done at its maximum resolution.

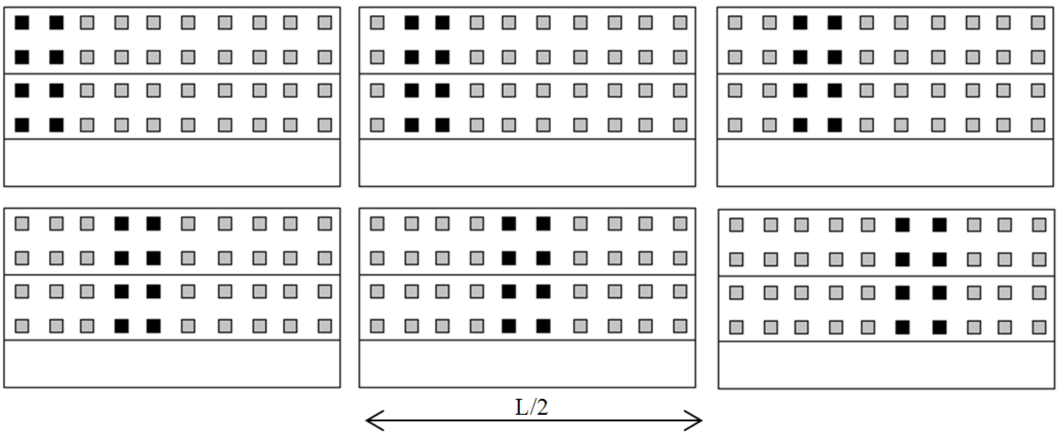

Figure 10 Car placement 0 sketch, showing possible tandem positions travelling parallel

Afterwards, in order to cover all the existing possibilities, *Car Placement 1* (CP1) was implemented. This alternative included all the possible combinations of the placement of Tandem Systems, including the parallel ones covered by the first case. As can be seen in

Figure 11, six random cases are represented to illustrate that there are no predefined combinations and all of them are taken into account. Moreover, maximum resolution is used in this case as well. As expected, this is the most accurate alternative but the most time consuming as well.

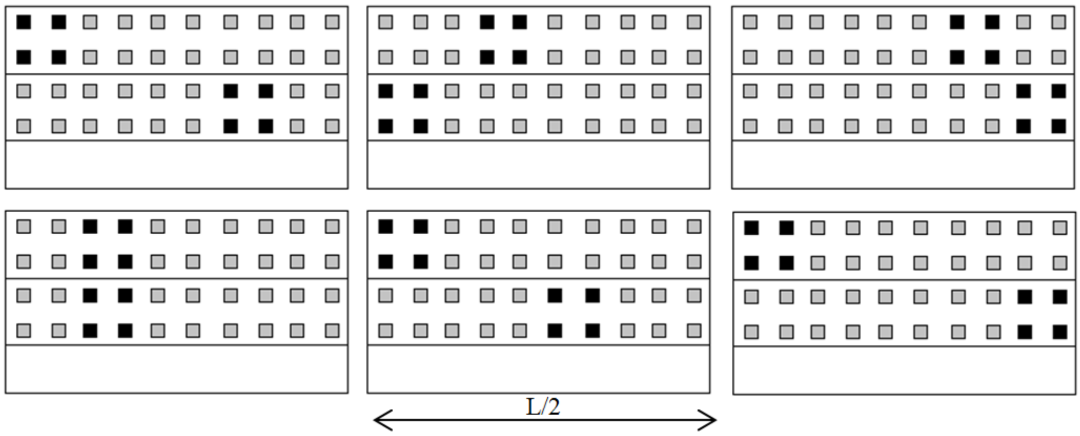

Figure 11 Car placement 1 sketch, showing some of the combinations of the tandem system.

In order to reduce the number of cases to only the most relevant ones, *Car placement 2* (CP2) was implemented by only considering the positioning of the Tandem systems on the regions over the support, the middle span and half way between them. As illustrated in *Figure 12*, all the combinations are considered in this car placement, but the resolution is reduced in order not to increase the time consumption as much as in the previous case.

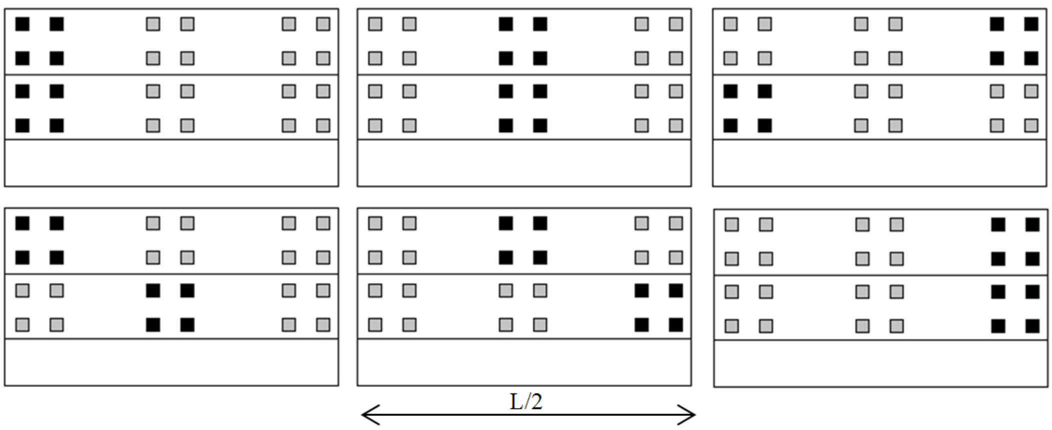

Figure 12 Car placement 2 sketch, showing the combinations of the reduced resolution model.

5.6.2.5 Comparison of Car placement (Performance)

The performance of this module of the tool was a key aspect of the overall performance of the methodology. The reason was that if the number of different Tandem System positions considered, together with the number of loading cases within the same Tandem position and the number of lanes, the total amount of calculations that would be required for one bridge might be too high. Since this was not acceptable due to the iterative character of the methodology, an in-depth study of the different *Car Placement* options was performed in order to know how accurate and efficient each strategy was. This is something very important since it is the only parameter controlled by the designer when it comes to performance of the Traffic model.

The total number of load cases would be:

$$N_{Load\ cases} = 2 * \frac{n_{beams}}{2} * n_{Carplacement\ combinations}$$

Equation 4 - Number of load cases obtained from Traffic Load module

As the number of beams cannot be changed while assessing one alternative, the correct selection of the Car Placement alternative is a key choice performance wise. Note that the factor 2 at the beginning of the equation corresponds to the fact that each case is calculated for ULS and SLS, needed for other implemented modules in the tool.

In Table 2 the comparison of different deflections, maximum stresses, time and number of load cases generated for each Car Placement mode is shown.

Table 2 Comparison of Car placement modes

Car placement	Max. deflection (m)	Max. stress (MPa)	Time (s)	Number of load cases
CP 0	0,026251657	223,41	433	34
CP 1	0,026251657	223,54	3422	578
CP 2	0,026251657	214,54	365	18

From these values it is easy to conclude that deflection is not affected at all for the Car Placement choice, which means that maximum deflection will most likely happen when all the Tandem Systems are placed in the middle span. Regarding Maximum Stress, CP2 is slightly less accurate than CP0, considering CP1 as the reference since every different Tandem system combination is taken into account in it. While the error for taking CP0 is lower than 0.06%, taking CP2 would give around 4% of error.

If time consumption is compared, CP0 takes 18% more time than 2 with an 88% more of load cases calculated. It was therefore concluded that CP2 was good enough for preliminary design, while CP0 would be the best alternative for later phases of the design, when detailed analyses are performed only in previously assessed and filtered alternatives. CP1 was not considered as a good alternative since the cost of virtually having no error is too high computationally wise and the other alternatives' errors were fairly low. In Figure 13 a very visual comparison of the different alternatives is shown. As commented before, the huge impact in calculation time that CP1 has is not worth at all if resources are limited. Even in the case of having a very high amount of resources available, it would be more suitable to use them in the broadening of the design space rather than increasing the number of combinations of the traffic load.

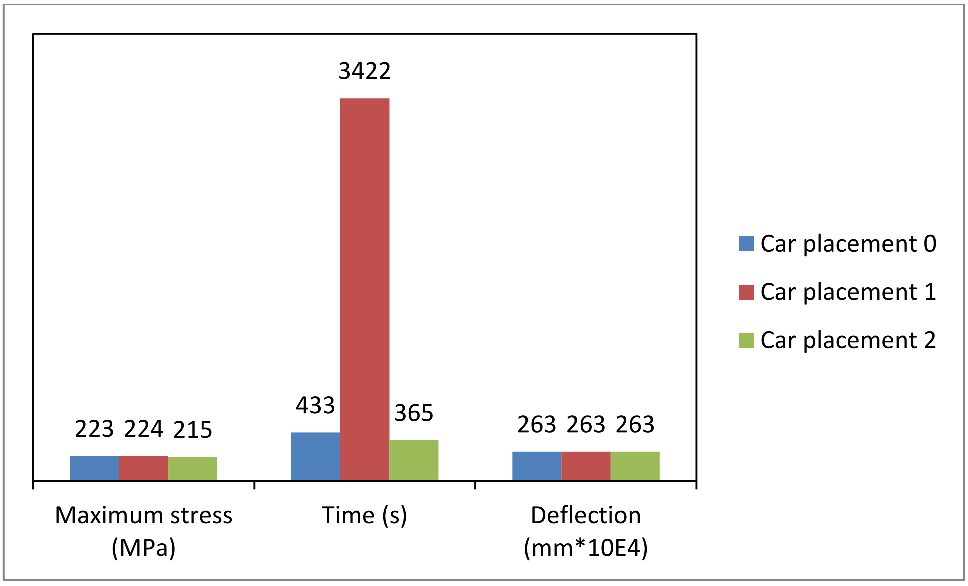

Figure 13 Comparison of different Car placement alternatives

Something that is important to consider from this study is that it was performed with a very fine mesh (ensuring high convergence, so the results are not altered for this reason). Other conditions pre-established were two notional lanes (the bridge deck width was lower than 9m) and two beams. By increasing any of these two parameters, number of load combinations would grow linearly in case of number of beams. Regarding the number of notional lanes, increasing from two to three notional lanes would have no impact in CP0, a huge impact in CP1 and a fairly high impact in CP2. The explanation for this can be extracted from the derivation of $n_{Carplacementcombinations}$ showed in Equation 5, with Res defined in Equation 3:

$$For\ CP0: n_{Carplacement\ combinations} = Res$$

$$For\ CP1: n_{Carplacement\ combinations} = Res^{n_{lanes}}$$

$$For\ CP2: n_{Carplamente\ combinations} = 3^{n_{lanes}}$$

Equation 5 Calculation of Car placement combinations for every alternative

In conclusion, a variation in the number of lanes or in the length of the bridge might vary significantly the ratio accuracy/resources of each alternative, being necessary to study each case thoroughly. As an example, if it is necessary to analyse a bridge with a deck wider that 9m (3 notional lanes at least), the number of combinations for CP0 would remain the same (assuming same length of the bridge), but the number of combinations for CP2 would grow from 18 to 54, being in this situation more suitable to use CP0.

5.6.2.6 Limitations

Limitations of the traffic model are mainly the ones included in the Load Model 1 from Eurocode. However, since this tool is meant to be used in pre-design phases, when global analyses are performed and no local verifications are required, there are no strong limitations of the traffic model that would restrain its applicability in this methodology.

5.7 Load combination

Since some modules of the tool and checks were meant to assess the Ultimate Limit State (ULS) of the bridge e.g. Ultimate Strength, while others were referred to Serviceability Limit State e.g. Crack Width, all the calculations performed in the model needed to be extended to both states in order to get proper results for all the checks performed. Hence, as presented in the previous subchapter, load cases were doubled and both ULS and SLS design load were obtained as stated in (SS-EN 1990), with partial combination factors according to Table 3.

Table 3 Load combination partial factors according to (SS-EN 1990)

Design action	**ULS**		**SLS (Quasi-permanent)**	
	Permanent action (γ_G)	Variable Action(γ_S)	Permanent action(γ_G)	Variable Action(γ_S)
Favourable	1	0	1	0
Unfavourable	1.35	1.5	1	0.3

These factors were introduced in Equation 6 to obtain design loads for both cases.

$$q_{D,ULS} = \gamma_{G,ULS} G_K + \gamma_{S,ULS} Q_k$$
$$q_{D,SLS} = \gamma_{G,SLS} G_K + \gamma_{S,SLS} Q_k$$

Equation 6 Load Combination equations according to (SS-EN 1990)

Regarding SLS, as recommended in (Beeby & Narayanan 1995) Quasi-permanent load was more restrictive than rare load in all the cases for this application. Therefore, crack width and maximum allowed deflection were checked with this load combination.

5.8 Functioning of the script

In order to have a clear idea of how the script works and which outputs are taken from the analysis to be included in the standards' verifications, in this chapter all the checks described in *Chapter 4* will be linked to their implementation within the script. In case the same check applies to both concepts, i.e. concrete and composite bridges; that check will be explained only once. For better understanding of the narrowing process of the set of alternatives carried out throughout the methodology, besides the implementation of the different checks, a classification of the different design criteria is presented according to the following definition:

Must criteria:

- ✓ A criterion within this class has to be fulfilled for the alternative to be considered as feasible and reach the set of possible alternatives.

Want criteria:

- ✓ A criterion within this class does not exclude the bridge alternative of the set of possible alternatives but weigh it negatively when the selection of the best alternatives among them is performed.

5.8.1 Dimensional compatibility

For the concrete concept, this dimensional compatibility check meant to verify that beams did not overlap, e.g. too many beams in a narrow deck. For the composite bridge, this overlapping issue had to be checked but also that the girder section did not become zero due to an extreme curvature defined at mid-span. These checks were assessed at the very beginning of the script, based only on the parameters that had been introduced for defining the different bridge alternatives, and carried out by simple geometric and algebraic calculations.

Must criteria:

- ✓ Overlapping of beams and extreme curvature leading to no web cross section at mid-span cannot occur.

5.8.2 Ultimate Limit State

5.8.2.1 Sectional Design of concrete beams

According to the concrete bridge concept presented before, the first step in the sectional design process was to define the effective width of each T-beam. This was done by simple calculations with the dimensional parameters defining the bridge, according to *Subchapter 4.1.3*.

As the bridge was considered a group of concrete T-beams subjected to bending moment and shear force, the beams could need steel reinforcement in the cases where the capacity of the concrete was not enough to carry those load effects. This design was included in the script and it followed strictly the Eurocode rules for designing concrete structures, *Subchapters 4.1.3.1* and *4.1.3.2*.

Since the beams are normally not homogeneously reinforced along the span, due to the variations on shear force and bending moment distributions, they were divided in the script into different regions, using defined positions as boundaries of these regions. For each region, the reinforcement was designed according to the maximum shear force and bending moment within the region, extracted from Free Body Cuts performed at the section boundaries, no matters if they took place in different positions along the region. Once the maximum load effects were determined, the reinforcement of the region was designed in two steps.

First, the amount of reinforcing steel and the layout of the rebars necessary to bear the maximum bending moment within the region were estimated. Due to the single-span and simple-supported configuration of the bridge, the T-beams were considered to carry only positive moments and consequently the rebars for bearing the bending moment were only needed at the bottom of the beams. Three layers were set as the maximum number of layers due to buildability limitations. Founded on the principles of Set-Based Design, different possibilities had to be maintained along the design process and therefore, different rebar diameters were considered. For each diameter and for each region, the script estimated the number of rebars that were necessary to reach enough bending moment capacity, according to the limitation of number of layers but also by fulfilling spacing, concrete cover or ductility restrictions as stated in Eurocode, *Subchapter 4.1.3.1*. If the number of bars for any diameter could not fit within the width of the beam, the region was defined as not possible to be reinforced.

As a result of this design, for each beam region, the script reports the different suitable rebar configuration in terms of:

- Number of rebars
- Rebar diameter
- Number of layers
- Check for yielding in the least stressed rebar (in order to verify equilibrium calculations)
- Bending moment resistance of the section
- Utilisation ratio of the section

Secondly, the script proceeded to estimate, if needed, the separation of the stirrups which prevents shear failure. In this case, since the diameter of the stirrups is not widely varied in practice, this parameter was defined as a designer's choice single value, being simple to include different diameters though. If the self shear resistance of the concrete was enough, the script suggested no reinforcement, *Subchapter 4.1.3.2*. Regarding feasibility reasons of stirrups placement, the minimum separation of stirrups needed to be limited and also converted into realistic values (a separation of 103.3mm is not realistic and must be turned to 100mm for instance).

For this case, the shear reinforcement design is defined by:

- Stirrups diameter
- Separation of stirrups
- Shear force resistance of the section
- Utilisation ratio of the section

Must criteria:

- ✓ Every beam can be designed for bending with rebars of the same diameter regardless the diameter of the other beams.
- ✓ The separation of stirrups is not lower than 100mm.

Want criteria:

- ✓ All beams in the bridge can be designed for bending with rebars of the same diameter.

5.8.2.2 Slab design

In the case of the concrete bridge and as stated in the *Subchapter 4.1.4*, the deck was reinforced only with the minimum area of steel required in Eurocode. However, the composite concept required more attention.

For the transversal reinforcement, the moment to be resisted by the reinforcement and the regions where the reinforcement is needed on top or bottom of the deck cross section was determined by assessing the sectional moment diagram of the transversal moment along a transversal path defined at mid-span of the bridge Figure 14. From the moment diagram for all load cases in ULS, the maximum positive and maximum negative moments were defined as design values, as well as the length of the regions corresponding to positive and negative moments, i.e. region between girders and over girders respectively. To elaborate, negative moment zones (over the girders) were reinforced according to the maximum negative moment previously obtained (top reinforcement), while positive moment regions were reinforced with the correspondent maximum positive moment value at the bottom. These moment values were combined with the corresponding torsional moment as stated in *Subchapter 4.2.5.1*, and the reinforcement area was estimated by means of sectional equilibrium as for the design of the concrete beams. Remaining areas were designed with the minimum reinforcement as stated in Eurocode.

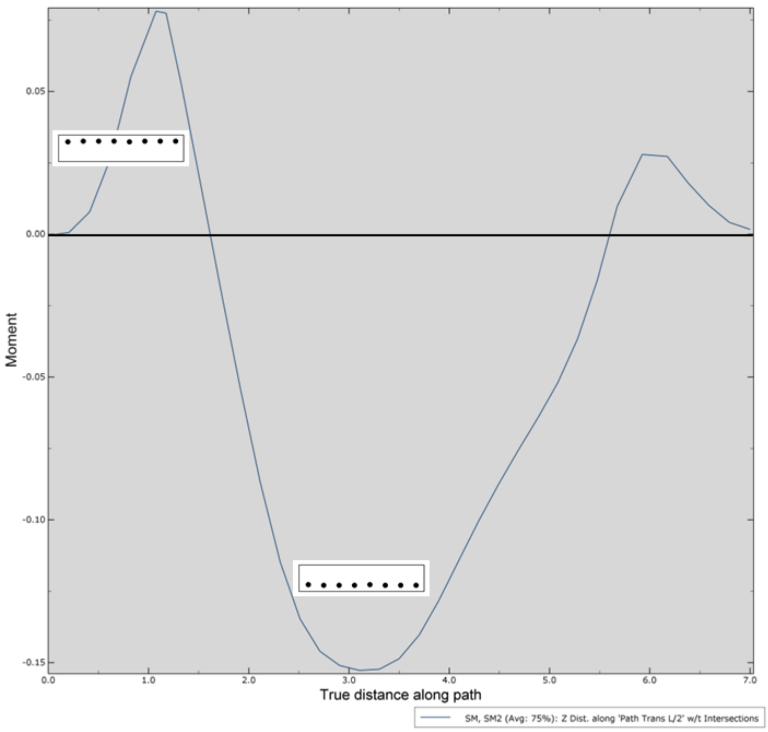

Figure 14 Transversal moment (SM2) in MNm at mid-span

Since the composite bridge was fixed at both ends, negative moments were present close to the support, and hence tensile stress were developed within the concrete slab. This caused need for reinforcement in longitudinal direction which could have been neglected in simply supported bridges. Assessing the effect of this negative moment on the concrete slab was complicated due to the sectional moments in the deck extracted from ABAQUS do not consider the effect of the girders in the neutral axis of the cross section, and these girders are the elements carrying manly the bending moment in longitudinal direction. Therefore the recommendations presented in *Subchapter 4.2.5.1* for longitudinal reinforcement were used. Similar to the design of the transversal reinforcement, in this case load effects come from a path drawn along the deck over the most loaded girder, where the membrane forces and their distribution were taken from Figure 15. The distribution determines the extension of the deck that has to be reinforced, i.e. the maximum length of the area next to the support with tensile forces considering all load cases; and the maximum value of that membrane force combined with the shear membrane force was considered as the force to be resisted by the longitudinal reinforcement. The area of steel required was estimated by equilibrium of forces. Zones on top corresponding to compressive stresses were reinforced with the minimum area of steel from Eurocode. It is important to note that, as a consequence of the assumption of considering the slab as a one way slab, bottom reinforcement in the longitudinal direction have minimum area of steel along its whole length.

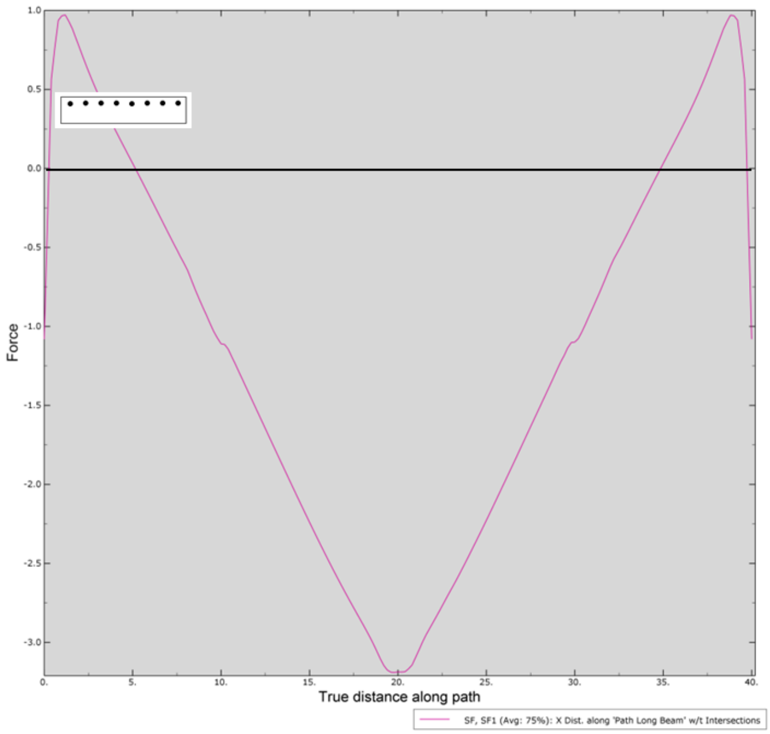

Figure 15 Longitudinal membrane forces (SF1) in MN over the most loaded girder

<u>*Must criteria*</u>:

- ✓ The area of steel required for carrying moment in transversal or longitudinal direction cannot exceed the maximum area permitted by Eurocode, as stated in *Subchapter 4.2.5.1*.

- ✓ The slab thickness needs to be thick enough for allowing the development of enough lever arm for the reinforcement to carry the bending moment or to place the required reinforcement in height.

5.8.2.3 Stresses in steel

As stated in 4.2.3, the failure of the steel girders due to bending, shear or axial force was performed by checking that the maximum stress in the girders did not exceed the design yield stress. This maximum stress was taken as the maximum Von Mises stress within the girders for all load cases, not considering areas very close to the supports to avoid singularities due to discrete boundary conditions.

Must criteria:

- ✓ The maximum stress in the steel girders cannot exceed the design yield stress of the steel.

5.8.2.4 Buckling

5.8.2.4.1 Lateral torsional buckling during construction

Since the critical moment used in assessing the lateral torsional buckling of steel girders presented in the *Subchapter 4.2.4.1* is meant for prismatic girders, for variable profiles as the one considered in the case study, the critical moment was estimated for the most critical sections, i.e. support and mid-span. This was done due to the specific cross section variability along the span of the case study, and therefore for other case studies the critical cross sections might be placed in different locations.

This critical moment was calculated by considering the whole cross section of the girder since during construction, the concrete is still fresh and carries no load. However, the effect on the bending moment of the concrete self-weight needs to be considered. This moment was taken as the maximum moment along the most loaded girder considering the corresponding part of the deck carried by that girder, and was estimated by using Free Body Cuts along the bridge length.

Want criteria:

- ✓ The maximum bending moment in the most loaded girder cannot exceed the critical moment for torsional buckling calculated in the most critical section of that girder, otherwise stiffeners or additional support during construction might be needed.

5.8.2.4.2 Lateral torsional buckling under traffic load

As described in *Subchapter 4.2.4.2*, for assessing the lateral torsional buckling under traffic load, when the concrete deck restricts the upper flange, the simplified method from Eurocode which considers only the lower flange in compression was used. Most of the parameters required for the verification of this check came from Eurocode itself or from geometrical properties of the cross section. Nevertheless, the maximum moment within the part where the lower flange is in compression and the length of that part were extracted from the Finite Element Analysis.

For the length of the lower flange in compression, the maximum length of the lower flange in compression for all load cases was extracted by assessing the longitudinal stress distribution along a path that followed the flange profile Figure 16. The maximum moment was taken from the maximum bending moment along the specified length by means of using Free Body Cuts.

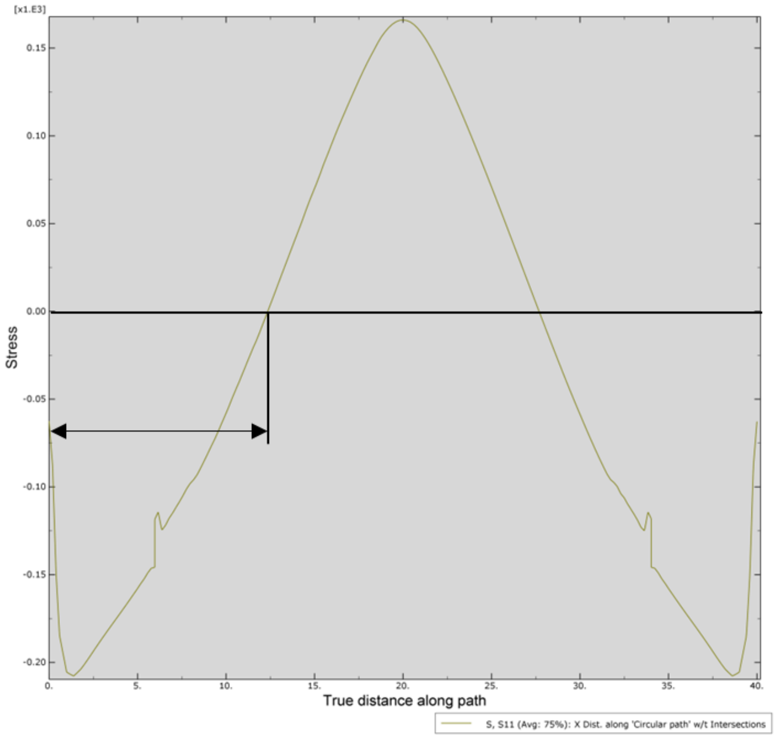

Figure 16 Length of the lower flange prone to Lateral Torsional Buckling. Stress in MPa.

Once both parameters were calculated, i.e. length of the lower flange in compression and maximum bending moment within that length; the length obtained had to be compared with the critical length estimated by following the Eurocode calculations.

It needs to be mentioned that for the sake of simplicity and for considering a scenario close to the worst one, i.e. class 4, the cross section of the web was always considered in class 3, and an approximate moment resistance was estimated based on the plastic moment resistance of the section reduced by considering an effective web in the part of the web in compression below the plastic neutral axis.

Want criteria:

- ✓ The maximum length of the flange in compression cannot exceed the critical length as stated in the simplified method for assessing lateral torsional buckling in Eurocode, otherwise stiffeners might be needed.

5.8.2.4.3 Shear buckling

As described before for assessing the lateral torsional buckling capacity of the steel girders during the construction phase, the shear buckling resistance of the girder was also checked at the most critical sections of the girder due to the variable section of the girder. Since the most critical parameter in shear buckling is the slenderness of the web, the highest and thinnest sections were considered, i.e. support section and section where the web becomes thinner due to the design criteria of the girder profile.

In these sections, the geometrical parameters were extracted directly, and the bending moment and shear force were extracted from the analysis by Free Body Cuts. All these parameters were then used to verify the shear buckling capacity of the girder according to *Subchapter 4.2.4.3*.

Want criteria:

- ✓ The maximum shear force in the girders cannot exceed the shear buckling capacity of the most critical section otherwise stiffeners might be needed.

5.8.3 Serviceability Limit State

5.8.3.1 Deflection

In both bridge concepts the deflection was measured as the maximum vertical displacement for all load cases in SLS searched within the whole model. This value was then compared with the standard limitation and characterised the bridge as feasible or not. Among the different load combination factors in SLS discussed in Eurocode, the quasi-permanent combination of loads was selected in maximum deflection verification for being the one often recommended (Beeby & Narayanan 1995).

Must criteria:

- ✓ The deflection cannot exceed the standards' limitation.

5.8.3.2 Crack width

For solving the equations presented in the *Subchapter 4.1.5*, two different variables were needed from the analysis in order to estimate the maximum crack width. On one hand the stress in the tension reinforcement assuming a cracked section which was determined through the stress produced at reinforcement level by the maximum bending moment in SLS, calculated by means of a Free Body Cut.

On the other hand, the area of longitudinal reinforcement was directly taken from the previously performed reinforcement design.

Must criteria:

- ✓ The crack width cannot exceed the standards' limitation.

5.9 Output data

After all the verifications are done, in order to sort and seek for the optimal alternative, the script has to be able to extract all necessary data. First, the script generally discards all the alternatives with at least one *Must Criterion* not fulfilled and it does not consider them in the set of possible alternatives. However in case the cause of discarding alternatives wants to be assessed, the script can be set up for keeping the discarded alternatives, indicating all the criteria were not met.

The type of data from the script that is stored can be classified into different categories:

- TAG of the bridge: this parameter defines a single alternative and can be used for tracking all data related to it. It can be seen as an identification number of the bridge.
- Input data: these parameters describe the alternative dimensionally, e.g. number of beams, deck thickness or length of the bridge. They are set beforehand through parameter ranges and picked randomly to build up each alternative. These parameters are important for estimating the material amount or cost of an alternative.
- Load effects: they include all the load effects considered in the design and verification of every alternative and can be used for checking the results, e.g. bending moments, shear forces, stresses or deflection.
- Reinforcement design data: these data correspond to those parameters used in the design of the reinforcement. Some of them are specified by the designer, e.g. the regions where different reinforcement can be placed or the stirrups diameter; and some others are the result of the reinforcement design itself, e.g. number of layers of bending reinforcement or stirrups separation.
- Checks data: in this group the parameters regarding the verification of some of the standards' requirements can be found. Most of them are numerical values, e.g. crack width, ductility or lateral torsional buckling resistance; whilst others describe whether the checks have been fulfilled or not.

5.10 Script Performance Optimisation

5.10.1 Convergence study

As every time that a Finite Element Analysis calculation is performed, a convergence study of the mesh size needs to be carried out in order to define the lowest amount of elements that led to acceptable results. For the assessment of the obtained results, deflection and stresses were taken into account due to their different degree of convergence. This is because convergence of displacements will always be one order higher than convergence of stresses as they are proportional to strains (Ottosen & Petersson 1992). Depending on the type of element chosen, convergence rates will be higher or lower, but they will always keep the aforementioned relation.

As a consequence, both models were probed regarding stresses and displacements at different element sizes, in order to estimate the optimal point where accuracy and performance would meet. It is important to note that the objective of the convergence study was both ensuring convergence of stresses and displacements and deciding a mesh element size that would give good enough results without excessively harming the performance.

As it can be seen in Figure 17, displacements converged fast enough for an acceptable mesh density regarding time/resources consumption, and faster than stresses as expected.

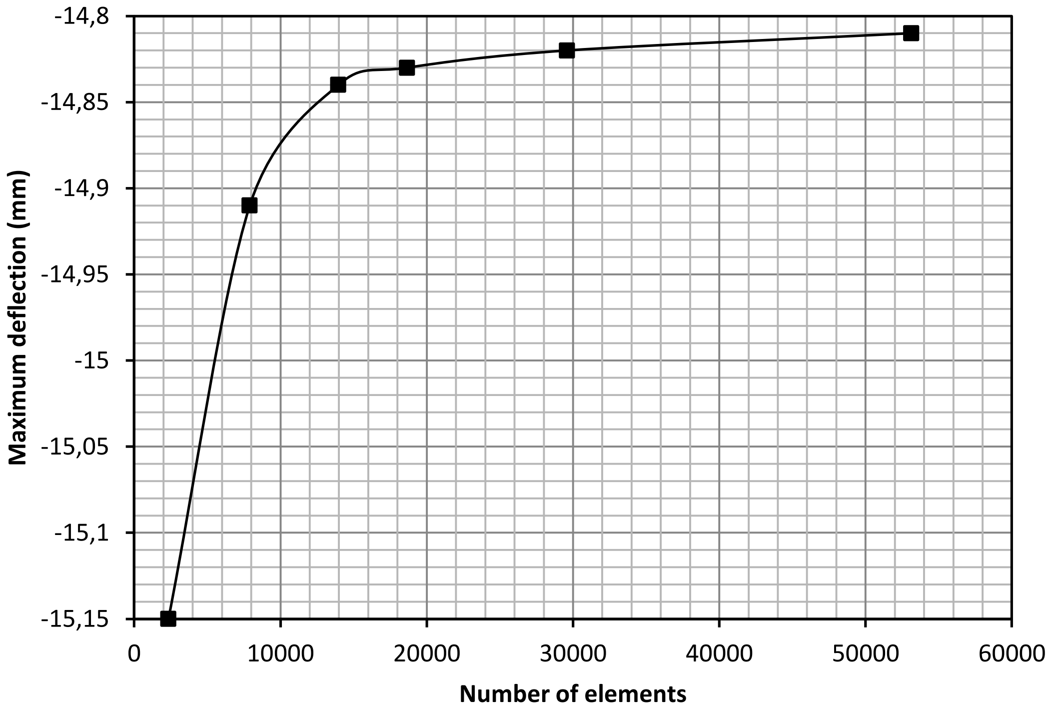

Figure 17 Convergence study of concrete bridge (Displacements)

As it can be seen in Figure 18, stresses presented a convergent behaviour as well, but not as stable as displacements did. Convergence of both fields was a key aspect of the project since performing large number of analyses requires much computational time and hence the maximum element size that provides good estimations is desired.

Once both stresses and displacements were checked regarding their convergence, it was time to decide how fine the mesh should be to keep both the smoothness of the process and its accuracy. Taking as "exact" values those obtained with the highest number of elements that the available resources allowed, the element size selected gave as a result stresses with 3.88% of error and displacement with 0.2%. This was considered acceptable for a predesign phase, and allowed to keep the process as smooth as necessary for an iterative approach.

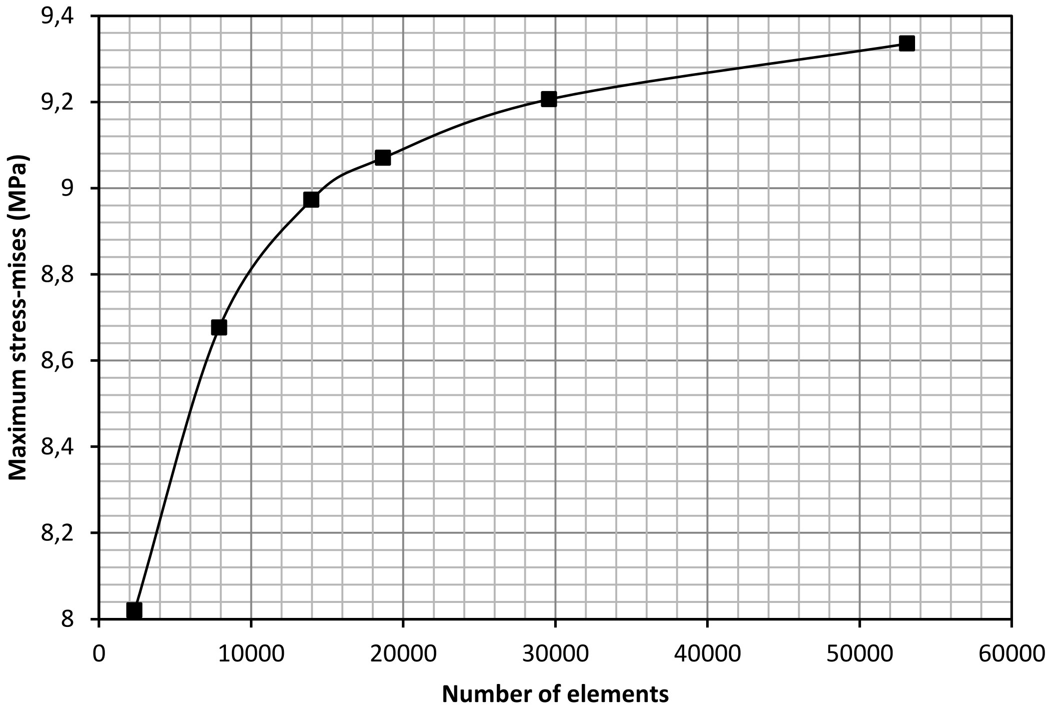

Figure 18 Convergence study of concrete bridge (Stresses)

Although the showed values correspond to the concrete model, the same procedure was followed for the composite bridge and similar results were obtained.

5.10.2 Parallelisation of the analysis

Parallelisation was an important aspect of the computational part of the project, since the tool was intended to submit a huge amount of calculations that could not be performed in a reasonable time span without any kind of parallelisation. Provided that the computational power comes from Chalmers cluster, it was needed to prepare the script for the use of its whole computational capacity.

Despite the fact that ABAQUS has some strategies implemented to parallelise routines and perform analyses faster, they are more oriented to large single analyses (usually very complex) that require huge amount of power. On the other hand, the tool made for this project was based on a sequential queue of not so large analyses, so it was not possible to take full advantage of the power available. Therefore, some modifications were made in order to exploit the computational power provided by the cluster. In *Figure 19* and *Figure 20* it is possible to see the difference between sequential way of running the script, and the parallel way of doing the same.

Figure 19 Sequential running of the job

Since ABAQUS has some multithreading features available for its jobs, every analysis included in the loop (rectangles in the figures) has some kind of parallelisation capabilities (more than one core is used for every single analysis). Anyway, it was necessary to finish the analysis in order to begin the next one. This was the reason that made the process so slow so it was therefore decided to address this issue in other different way.

Even if a whole computing cluster is available for use, it would be useless to spend its resources in parallelising every analysis one by one, since the performance gain would be marginal. It seemed more appropriate to find a way to submit parts of the original job as separate jobs, in order to be able to use the computational power as efficiently as possible.

By using the parallel running strategy, time consumption of the job was reduced proportionally to the number of jobs submitted, resulting in a huge save of calculation time.

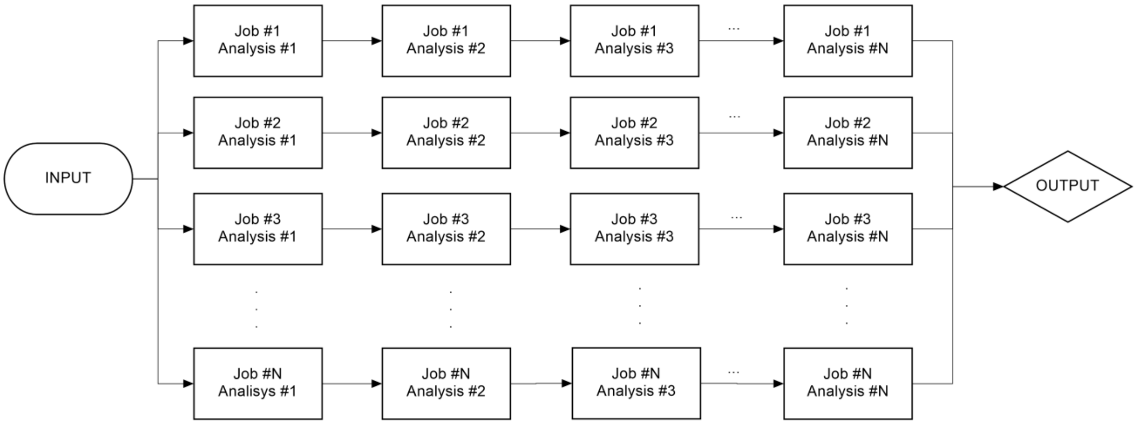

Figure 20 Parallel running of the job

In order to be able to take advantage of this, it was needed to modify the input and output flow of data. Instead of having a multiple nested loops that generated the geometry and analysed the model for each variable combination, all of them were considered, tagged and stored into a list. By doing this, it was possible to have a variable containing every value of the parameters from each bridge and its identification for further processes. This list could be divided afterwards in as many parts as desired by the user (number of jobs) and an analysis file was automatically created for each part.

By running every single analysis job, it was possible to get the same output as the sequential alternative only by performing some minor modifications on the data processing. This was achieved by putting all the outputs together so they were available to be processed in the same way as if they were obtained from a sequential job. It is easy to understand that merging a huge amount of data might be problematic if it is not done carefully, and some problems might appear in later phases i.e. optimisation phase. It is important to note that the output of the process is the same as if it had been performed with the sequential approach, but much faster.

It was essential that every combination of parameters submitted, analysed and stored was identified in order to point at it if necessary. This was done by means of the tag attribute included both in the input and output phase. By using it, it was possible to identify the exact characteristics of the bridge, a very important feature for the optimisation process.

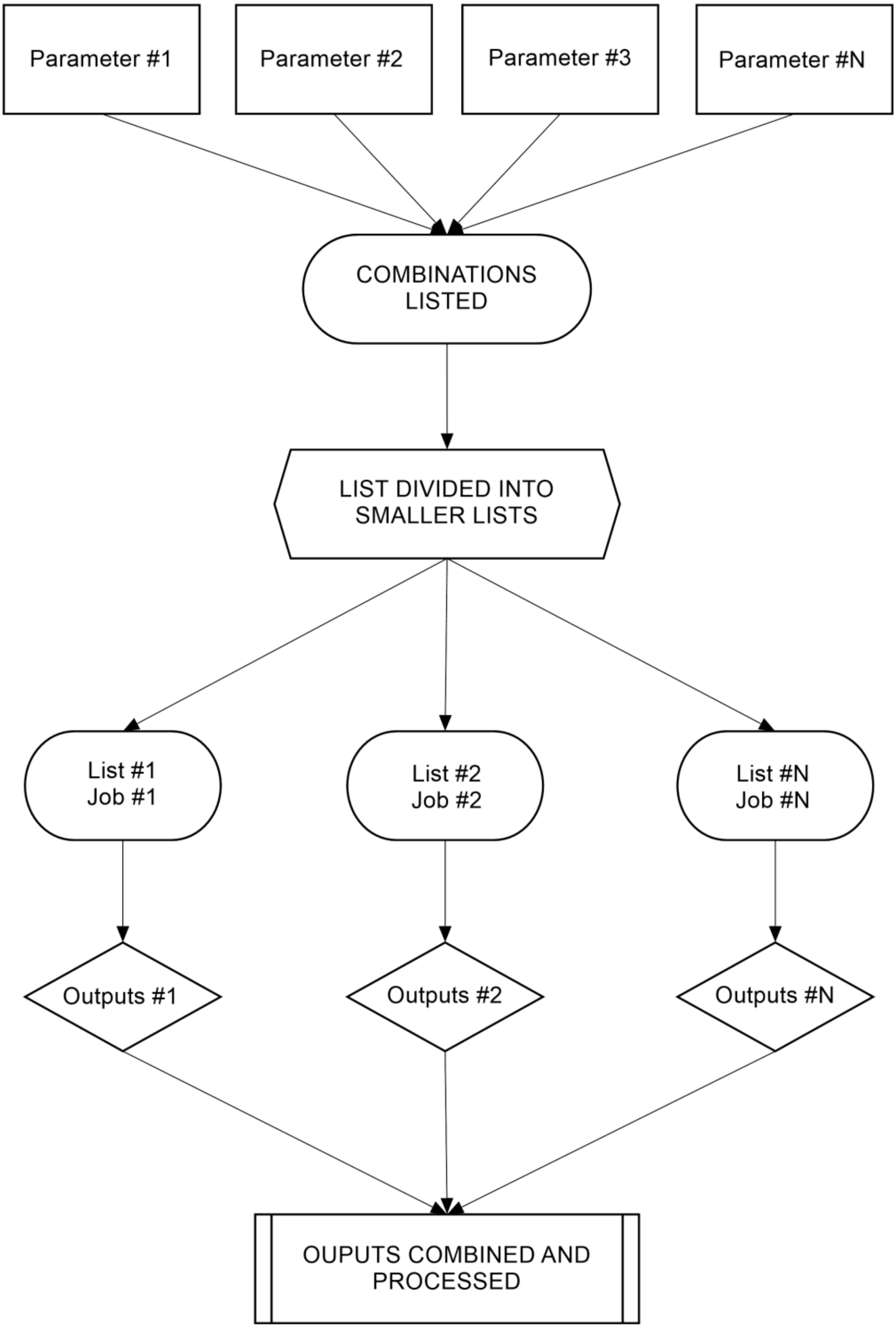

Figure 21 Overall process of the algorithm

6 CASE STUDIES

In this chapter, two different bridge concepts designed by the company NCC will be considered as case studies. They will be used for validating the aforementioned tool and for assessing the applicability of the methodology here presented.

6.1 NCC Montagebro

NCC Montagebro (Figure 22) is a reinforced concrete bridge concept that can be used for crossing roads, railways or water streams. Developed by NCC, it can be implemented quickly and with minimal disruption of traffic thanks to a fully-industrialised process. There are bridges belonging to this concept built all around Sweden, with different geometries or details but the same common characteristics.

6.1.1 Geometry of the model

Since the project was oriented to the optimisation of the concept, the geometry was introduced according to the its real configuration. It was based on a concrete deck resting on concrete beams (variable number) that were simply supported at the abutments. As it was not important for the project to analyse the behaviour of the abutments itself, they were not introduced into the FE model, placing the boundary conditions on the beams as it will be discussed later.

Figure 22 NCC Montagebro bridge

The modelling of the deck was decided to be carried out with shell elements since that part of the structure is a slab, which can be very accurately represented by shell elements just by establishing the thickness of the plate and the offset direction in which the thickness of the plate is generated from the base element. The latter aspect was very important, since the implementation of the regular offset from the middle section created some distortion on the results due to material overlapping from beams and deck.

For the modelling of the beams, shell elements were considered as well. Although theoretically, it could have been possible to use beam elements due to obvious similarities with the structural element, a good definition of a shell could also be used to represent beam behaviour. Additionally, since full interaction between beams and deck was considered due to the nature of the joint (casted concrete joint), it was more suitable to model the whole bridge out of shell elements, for avoiding problems in assembling different types of elements. The number of integration points along the thickness of the shell elements was set to five according to the recommended value for the integration method chosen, i.e. Simpson.

It might be argued whether solid elements should have been considered, but since the purpose of the project was the assessment of the bridge as a structure, and not a detailed study of some structural components, they were discarded. Using solid elements would have dramatically increased the calculation time, without reporting any significant advantage within the scope of this project. Since this model is implemented in a loop that will repeat similar runs sequentially, the analysis time was considered one important aspect for the modelling of the bridge, as increasing the time for one single run would have a large impact in the total computational time.

6.1.2 Parameterisation

The parameterisation of the bridge was based on considering the possibility of simulating a huge amount of different bridges with different dimensional parameters.

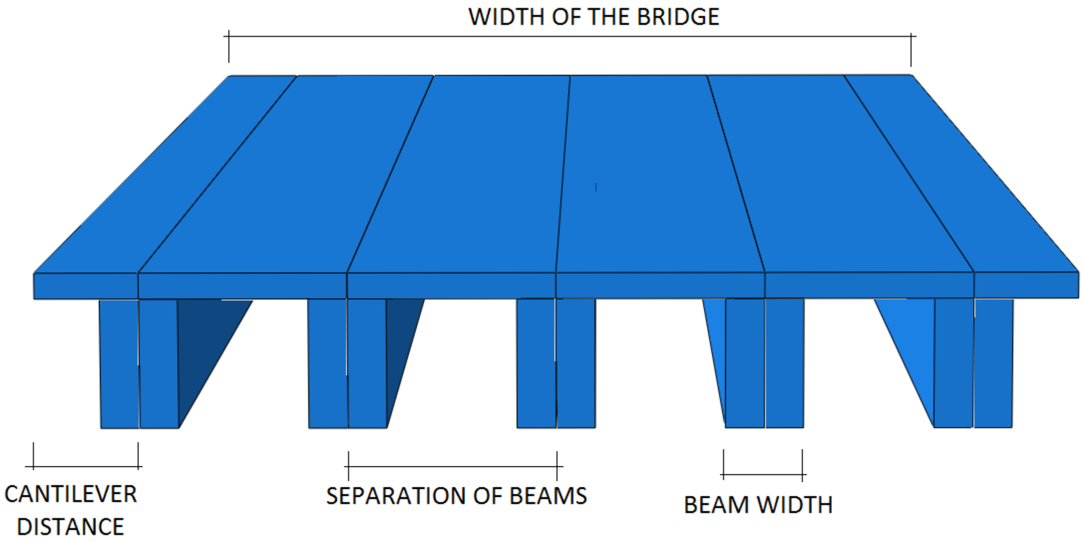

Figure 23 Definition of parameters affecting Separations of beams

While some of the parameters were common to the whole set of bridges, such as the length or the width of the bridge, another group of them were iteratively changed to generate new bridge alternatives to be assessed. This was the case of the number of beams, cross-sectional properties of the beams (height and width), thickness of the slab and cantilever distance. The latter was introduced to give more possibilities to the design but the original concept does not include this possibility, having edge beams at the end of the slab at both sides.

Some other important parameters were derived from the aforementioned ones. This was the case of the separation of the beams, which was depending on the width of the bridge, the width of the beams, the cantilevering distance and obviously the number of beams. It is defined as:

$$Sep.Beams = \frac{Bridge\ width - 2.cantilever\ dist - beam\ width}{number\ of\ beams - 1}$$

Equation 7 Separation of beams derivation

6.1.3 Mesh properties

Meshing of the FE model resulted to be very important both for the accuracy and validity of the results as well as for the proper functioning of the script itself.

As described in *Subchapter 5.10.1*, the mesh density was important to have not too big elements that might compromise the integrity or accuracy of the obtained results. At the same time it was not acceptable to use too small elements since the computational time would be increased dramatically.

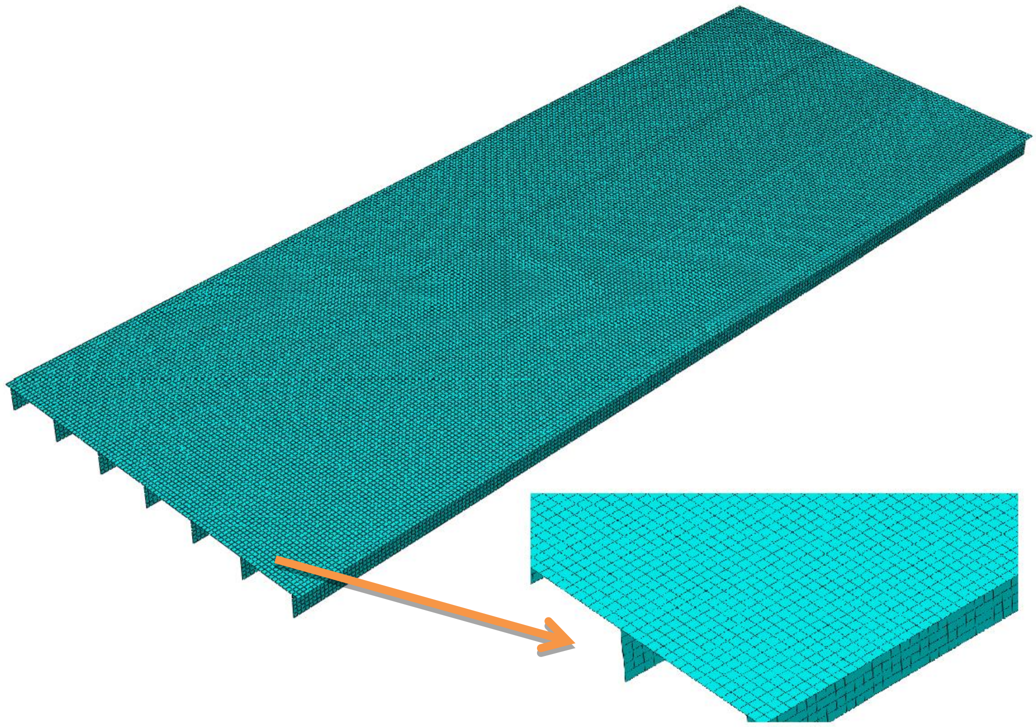

Figure 24 Mesh of the model with quadrilateral elements (S4R)

Regarding the element type, it was decided to use quadrilateral elements since it was a suitable shape according to the bridge geometry. As it can be seen in Figure 24, the different parts configuring the bridge have right angles that make quadrilateral elements to fit perfectly along the model. Additionally, although it might lead to slightly stiffer behaviour than reality, reduced integration was chosen to save computational resources. As a result, S4R elements were used in the ABAQUS model, which are known to be suitable for general purpose analyses.

6.1.4 Boundary conditions

One important aspect of every Finite Element simulation is the definition of proper boundary conditions for the model. This will be essential to determine how close the model is to reality for relying on its results.

Boundary conditions were carefully analysed and established as fixed points on the beam supports. This was decided by paying attention to the configuration of the concept bridge, where the beams are supported by the abutment on both sides. At the beginning it was considered to introduce a beam element to represent the abutment, but it was discarded in favour of a simpler way of restraining the geometry since the structural behaviour resulted to provide no relevant additional information. Therefore, the introduction of beam elements for this purpose would not improve the results while it would increase slightly the computational load of the model, making this modelling choice not optimal.

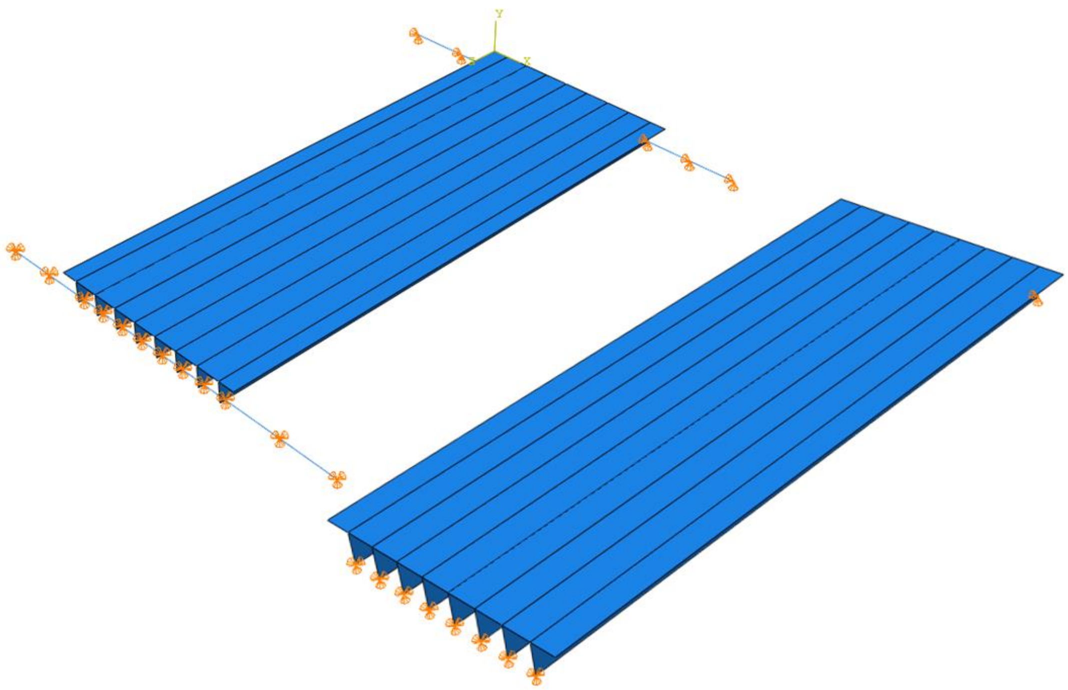

Figure 25 Comparison between the model with beam element and the simpler model

These boundary conditions simulate a simply supported condition of the bridge, which is coherent while the number of spans is restricted to one. If more spans were considered, continuous beam conditions might be introduced and boundary conditions would change. However, this was outside the scope of the project.

6.2 NCC Nynäshamn

The case of study of the composite bridge was based on a bridge already built in Nynäshamn, Sweden. Although it was the only one built according to this design (or similar) by the date of writing, it might be the intention of the company to use the design to develop a concept that could fit in other locations as happened with NCC Montagebro.

6.2.1 Geometry of the model

The geometry of the second bridge assessed in this project was fairly different from the previous one. Instead of having a high density of beams supporting a concrete deck, in this case the bridge had a limited number of beams (two or three) supporting a concrete slab. Another main difference was regarding materials. While NCC Montagebro was completely built of concrete, this concept included stainless steel girders, introducing an innovative construction material that allows having virtually no maintenance. Both the geometrical and material differences gave as a consequence a different structural response of the bridge that led to different design approaches and assumptions as it can be read in *Chapter* 4. One example of this was the different approach used for the design of the slab, as commented previously.

Figure 26 Nynäshamn Bridge

The modelling of the bridge was decided to be carried out with shell elements for similar reasons discussed for the previous concept. Moreover, shell elements fitted even better for the design of the I-profiled girders. Similar strategies as commented in the previous concept bridge were used for the overlapping issues between the deck and the top flange of the beams, i.e. offset of shell thickness.

In order to be able to solve the merging between the deck and the beams, it was necessary this time to use a *Tie function* from ABAQUS because of the different properties (material) of the girders and the slab. Anyway, full interaction was assumed between concrete and steel for simplicity and building aspects of the joint.

It is important to point out that this model was significantly more complex (geometrically speaking) that the previous one, mainly because of the girders geometry. Its varying section over the length (provided by an arch shape of the lower flange) required extra work both during the implementation of the geometry and the data extraction of the curved surfaces of the beam.

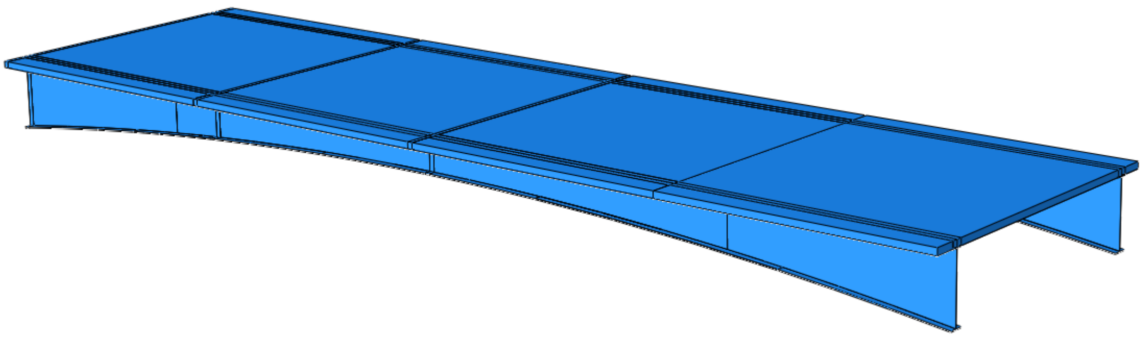

Figure 27 FE model geometry of the Nynäshamn bridge

6.2.2 Parameterisation

Due to the complex design of the girders included in the concept, parameterisation of this bridge was a major issue of this project. As it can be seen in Figure 28, the beams have three different regions in the top flange, three different regions in the web and five different regions in the lower flange. By different regions it is meant that there are different parts of the girder with different thicknesses or widths, i.e. flanges and web. The combination of all of these characteristics, together with thickness of the slab, the height of the girders, their curvature, position and the number of girders provided up to 18 parameters that were possible to be varied to get different alternatives within the concept presented.

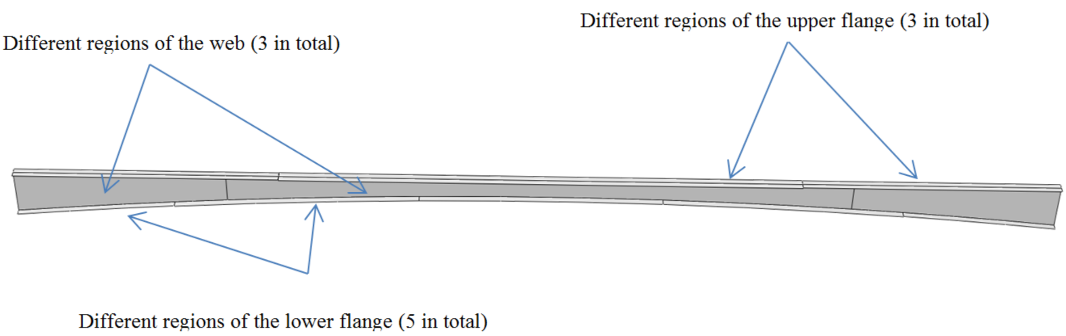

Figure 28 Different regions of the parameterisation of the girders

In order to be able to easily iterate over all the parameters described before, and reach every combination possible within the concept, it was necessary to define very carefully the different parameters that would describe the beam configuration.

By simple observation of the documentation provided by NCC, it was observed that the distribution of the different zones was arranged symmetrically over the length of the bridge, as it would be expected. This meant that, for example, first and third upper flange regions had exactly the same length, width and thickness, reducing the number of necessary variables for the parameterization. Moreover, the lower flange was designed with its three central regions having the same characteristics. As a consequence, a parameterization based on central entities and side entities was performed.

Central entities were the regions of the upper flange, the lower flange and the web that were not in contact with any end of the girder. Therefore, there was one central region in the upper flange, three central regions in the lower flange and one central web region. Side regions were defined as the opposite, completing all the regions constituting the beam.

Once this parameterisation was done, it was simple to effectively assign width and thickness to all the entities in the beam giving the possibility of design fairly complex girder geometries. On the other hand, a big limitation arose at this point: length of the different regions described in Figure 28.

The main issue about the length was that, while it was something that should be possible to be changed when creating new bridge alternatives, it had to be changed according to the limitations explained before (same size of some regions). Additionally, the length of the regions had to be derived in some way from the length of the bridge, keeping its symmetric definition. Therefore, the parameter controlling that issue was decided to be a factor between 0 and 1 that would establish the portion of the length defining the side region of the girder (one different factor for each flange and the web). The definition of these parameters *l* can be checked in Figure 29. It is important to note that by using the symmetric design of the girder, it is possible to derive the length of every region only with the *l* factor of the entity and the total length of the beam, as shown in Figure 29.

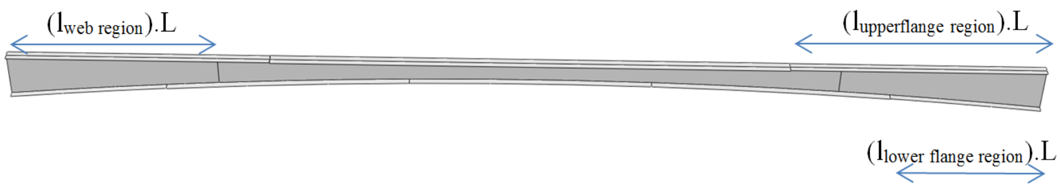

Figure 29 Definition of the length of the entity regions

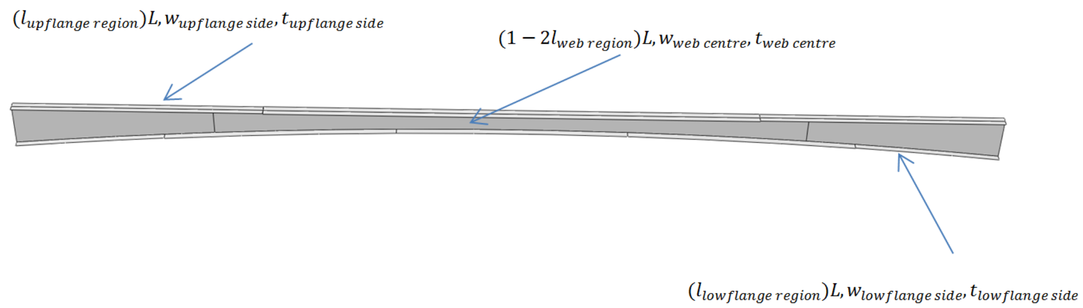

Figure 30 Examples of the definition of three different entities

Parameterisation of the girder was therefore performed in a way that only by using three different parameters (excluding the total length) it was possible to define the properties of each entity included in the beam. These properties were the thickness of the entity (t), the width (w) and the length of the region (defined by a factor *l*). Some examples are shown in Figure 30, where the properties of the side region of the upper flange, the side region of the lower flange and the central region of the web are derived from the available parameters.

Once the distributions of the different regions of the girders and their properties were established, the only remaining parameters to define the whole beam were the height of the girder and the curvature. The latter was defined by the circumference built from three different points, being two of them the end points of the girder and the central one the lower part of the girder in the central section (where the beam cross section is smaller, as seen in Figure 31). The purpose of adding the variable h_{curv}, was giving the tool the possibility of varying the curvature of the beam only by changing the value of that distance. It is easy to understand that changing h_{curv} as defined in Figure 31, one of three points defining the arch will change (as the other two are fixed) and therefore the curvature will change proportionally.

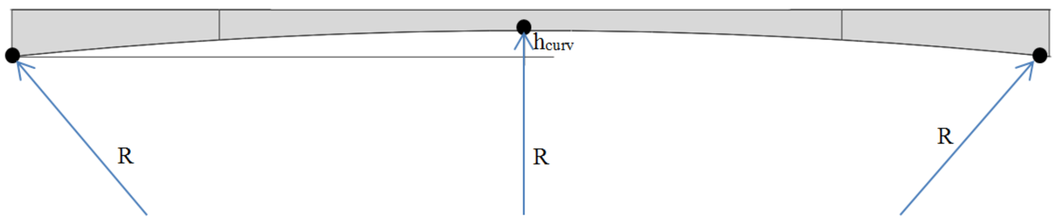

Figure 31 Beam curvature definition

To finish with the parameterisation, at this point the full parameterisation of the beam was performed. The only remaining parameters that characterised the whole bridge were the number of girders, the width of the slab and the position of the girders, defined by the cantilevering distance (as seen in Figure 32).

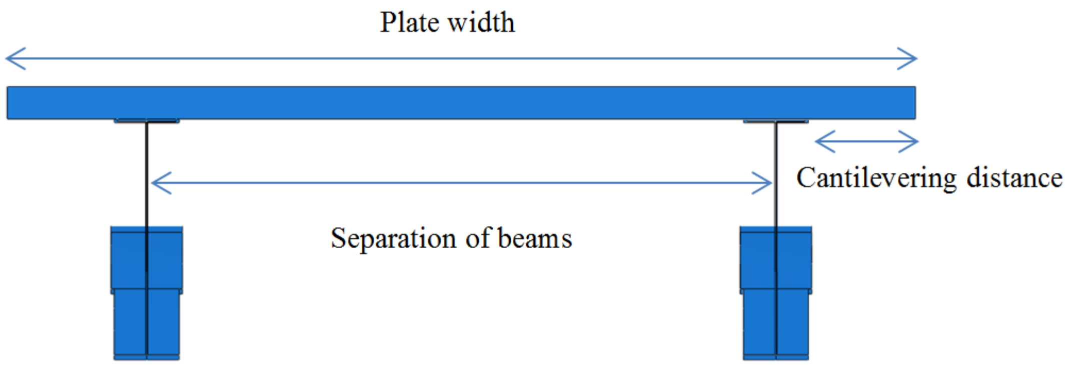

Figure 32 - Definition of cantilevering distance and separation beams

It is important to note that cantilevering distance is measured from the end of the slab to the widest top flange, while separation of beams will be defined as the distance from web to web. Anyway, separation of beams is not necessary to be introduced since it can be derived from other parameters as seen in Equation 8, taking advantage of the symmetry conditions that were previously used in the longitudinal definition of the beam parameters.

$$Separation\ beams = \frac{platewidth - 2.\,cant\ distance - \max(flangewidth)}{num\ beams - 1}$$

Equation 8 Separation of beams derivation for composite bridge

It is important to note in Equation 8 that the term named $\max(flangewidth)$ is defined as the largest flange width of the beam, considering that width as the width of the whole beam.

6.2.3 Mesh properties

As described in the previous concept, meshing of the model was a key aspect to ensure both valid results and a good ratio between accuracy and resources consumption. A similar convergence study to the one shown in *Subchapter 5.10.1* was carried out to confirm that convergence of results was happening and determining the optimal mesh density for the process.

As in the concrete concept bridge, the element type was decided to be quadrilateral elements for the same reasons. Moreover, reduced integration was also chosen to save computational resources, as happened previously for the concrete model. Same general purpose elements were used for the analysis, as said in the previous subchapter, S4R elements.

6.2.4 Boundary conditions

As stated before, boundary conditions were a really important aspect of the model to be able to reproduce in the FEM the real behaviour of the bridge.

In this case, the real bridge had its girders anchored with a connection as shown in Figure 33, and concrete was afterwards casted around the connection to ensure no movement of the fixed end. Therefore, the FEM boundary conditions that were imposed were fixed conditions in both sides.

Figure 33 Anchorage of the steel beams (NCC Construction, Brodag 2011)

In order to establish the fixed conditions in the model, the whole web was fixed in both ends. It was investigated the effect of fixing the flanges as well, but similar results were obtained. The detail of the connection in the FE model is shown in Figure 34.

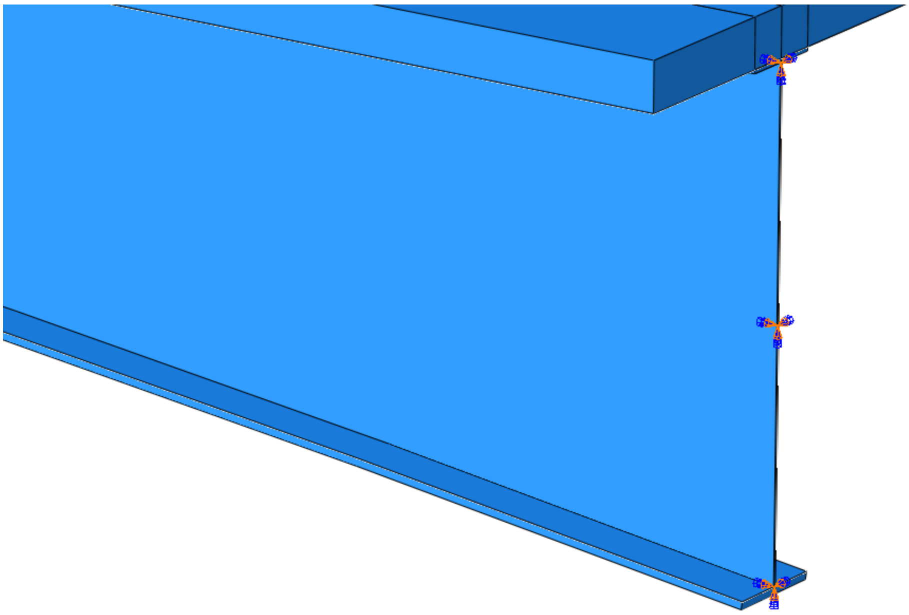

Figure 34 Fixed boundary conditions in FEM composite bridge model

7 OPTIMISATION

7.1 General Information

As presented in *Subchapter 5.9*, Output data, as a result of the iterative analysis process, large amounts of data are extracted, regarding among others the parameters defining the dimensions of every bridge alternative and its reinforcement design. These two categories are indispensable for estimating the material amount required for building an alternative, and hence, most of the main characteristics defining the performance of a bridge, i.e. material cost, environmental impact, construction cost and expected maintenance.

The methodology described in this thesis could take advantage of advanced optimisation techniques for developing its full potential. Nevertheless, it is outside the scope of this project the assessment and use of these advanced techniques and a simple multi-objective optimisation was carried out in order to find optimal solutions within the set of possible alternatives.

Two main reasons motivated the selection of a set of optimal alternatives rather than a single alternative in this specific methodology, besides its relation with the Set-Based Design approach. First, as the tool was developed for pre-design, it might happen that the optimal alternative does not fulfil one of the requirements checked during the performance of the final full analysis, and therefore the alternative chosen in pre-design would not be optimal and rework would be needed. On the other hand, optimising regarding multiple criteria needs the opinion of multiple stakeholders and therefore, letting the persons in charge of taking the final decision choosing between a set of optimal alternatives makes the process more transparent.

7.2 Optimisation Criteria

Due to the lack of more information regarding bridge performance estimation, only two criteria were used in order to select the set of optimal alternatives, i.e. material cost and CO_2 equivalent of the required material amount. These two parameters are only based on material amount and follow similar proportions regarding different materials, and hence, their combination gives no much more information than only considering one of them. However, they were used in order to show a hint of the potential of combining multi-criteria optimisation with the methodology here presented.

7.2.1 Material Cost

Due to manufacturing issues related with the benefits of design homogeneity, in the composite bridge all girders were equal. The material cost was defined as the sum of the individual cost of the different elements, i.e. stainless steel girders, concrete deck and reinforcing steel.

On the other hand, manufacturing prefabricated concrete beams is more adaptable for considering different reinforcement configuration within the beam itself but also between different beams. The former leads to the possibility of designing different regions of a beam to adapt to the bending moment and shear force distributions, which was implemented within the script by selecting these regions boundaries, and the latter was considered by adding no restriction of the number of different beams that could be used in a bridge in order to adjust the capacity of the individual beams to their needs. In the present work, neither limitation about the maximum number of different beams nor more than three regions were considered, although it could be modified by the designer. Another aspect to consider when designing reinforcement is the rebars diameter, which in this occasion was forced to not be changed within the bridge as it was considered an important factor for reducing secondary costs. Due to the reinforcing of the different regions, the estimation of the material cost of a concrete bridge alternative can be better explained as follow:

- The different regions of the different beams for different diameters are analysed and tagged as feasible or unfeasible regions.
- If all the regions of a bridge are considered feasible for a certain rebar diameter, the bridge is considered as feasible, and its material cost estimated as the sum of the individual costs of concrete, reinforcement of the beams and reinforcement of the deck.
- Otherwise, if there is at least one region missing for completing the whole bridge by using the same diameter in all the beams, the bridge is discarded.

7.2.2 CO_2 Equivalent

One way of estimating the environmental impact of a certain structure is through its CO_2 equivalent. This concept is difficulty assessed due to the large amount of different parameters involved in the environmental impact of a construction, e.g. energy consumption for extracting the raw material, energy consumption for producing the material, energy consumption for transporting the goods and so on. In this project, a simplified CO_2 equivalent considering only extraction, production and manufacturing issues of the different materials was used, and the total CO_2 equivalent of a bridge was simply estimated as the sum of the CO_2 equivalent of the different material amounts.

7.3 Optimisation Implementation

Different ways of implementing multi-criteria analysis of results are available, although for the sake of simplicity, the Weighted Sum Model (WSM)(Fishburn 1967) was used as first approach. This method consists on assigning different degrees of importance to the different criteria by applying weights, and defining the equivalent importance of an alternative as the sum of the criteria multiplied by these weights. The main drawback of this method is the selection of the weights, which is recommended to be made by consensus of the different stakeholders and by means of ranking techniques such as direct weighting method, swing weighting method or the hierarchy method.

In order to consider cases where the stakeholders have very different priorities regarding bridge performance, three different scenarios were presented an assessed according to the direct weighting method:

- *90% Material Cost – 10% $C0_2$ equivalent.* Even if extreme, it can be seen as the classical approach from industry.
- *10% Material Cost – 90% $C0_2$ equivalent.* Since the concept environmentally-friendly is getting continuously more importance in practice, this could show the tendency.
- *50% Material Cost – 50% $C0_2$ equivalent.* This situation corresponds to a balanced criteria importance.

8 OTHER APPLICATIONS

Besides the main application of this tool discussed in this project, i.e. bridge selection and suitability assessment, other possible applications for this tool, both in the field of Civil Engineering and others, can be identified.

8.1 Parametric study

One possible application of the tool would be the easy and automatic performance of parametric studies. As the development of a concept bridge in the tool includes the complete parameterisation of the concept, all of these parameters could be easily adjusted to create a loop that iterates throughout a whole set of values for a parameter or group of parameters. For example, it is easy to set up an automatic simulation that could analyse and assess the same bridge for different number of beams, getting as a result the structural response of all the alternatives included. By doing this type of study, it is easier for the designer to make decisions about a specific parameters or characteristics of the structure even before having the preliminary design.

This possibility could be also used for the refurbishment or reparation of old structures, trying different parameter combinations at the same time and being able to make a decision of how to repair the structure. This would give more possibilities to the engineer, having other possibilities different from just restoring the structure to its initial characteristics.

8.2 Applications in other fields

As it was commented at the beginning of this report, Set-Based Design methodology was initially developed completely out of the Civil Engineering industry, and it was first commercially applied by Toyota, working in the automotive industry. Since the purpose of this project was finding the way to introduce principles of Set-Based Design into Civil and Structural Engineering field, it is possible to be applied in some other industries as well. In this project, bridges were the study cases, but while keeping the principles of the methodology, including the FEA part and major modules of the tool, it should be possible to be easily ported to any kind of mechanical design. As long as there is something to optimise, some criteria to do it (Price, time, etc.) and a way to analyse the behaviour of the study case (FEM, CFD, etc.), Set-Based Design would be possible to be introduced.

It would important to remember that this methodology is useful mainly for the predesign phase, so it would be more beneficial to be applied in cases or fields where this phase is not necessary to be extremely detailed. Otherwise, performing an iterative analysis of alternatives would take too long and would make the approach less suitable.

9 RESULTS AND DISCUSSION

In this chapter the results are presented together with their discussion for a better understanding of the methodology outputs.

9.1 Comparison with real cases

Generally speaking, this methodology should produce valid results because it is not in any case trying to replace or substitute the detailed analysis performed to get final results. The aim is narrowing down the solution space by applying different criteria and doing a detailed evaluation of the final set. It is therefore not probable to get worse results than the ones obtained from traditional design, since the last step of the process i.e. detailed analysis, is exactly the same. Moreover, same requirements and quality controls are applied there. It is then only a better way to reach the same point, and very suitable for concepts that are built several times. Additionally, since this methodology introduces the preference and requirements of all the stakeholders, it is very probable that the final solution would be more satisfying for them than the one obtained from the classic approach.

9.1.1 NCC Montagebro

In order to assess how accurate were the results produced by the tool, dimensions of a real bridge already built were introduced and results were compared with the bridge documentation.

The comparison of the model with a real case was done in two different steps. First, it was expected that the tool would consider as valid the introduced alternative, since it was known to be feasible because it was already built. Nevertheless, although this was considered a promising result, it was considered not to be enough for a successful comparison.

It was then decided to use the reinforcement design implemented in the tool to try to check the validity of the results produced by the script, according to the regions where different reinforcements were designed.

Two real bridges were used for the comparison between the results from the model and real cases. Both bridges were initially designed according to NCC Montagebro bridge concept, so their predesign was supposed to be successfully compared with the designed tool. In order to get more realistic results, only the design of one internal beam and one edge beam was allowed, to match the real construction requirement. This is done because while conceptual optimisation would lead to one specific design for each beam of the bridge, it is afterwards more expensive to build that design in reality than designing only one or two types of beams.

After checking the documentation of the bridges, and introducing their geometrical values in the tool, the reinforcement designs that were obtained were the ones shown in Table 4 and Table 5.

Table 4 Comparison of the model with Örebro bridge

ÖREBRO

Support Region	Internal beams	
	Number of bars	Stirrups separation
Real case	5	100
Estimation	4	100
	Edge beams	
	Number of bars	Stirrups separation
Real case	5	100
Estimation	4	100

Field Region	Internal beams	
	Number of bars	Stirrups separation
Real case	9	150
Estimation	10	100
	Edge beams	
	Number of bars	Stirrups separation
Real case	9	150
Estimation	9	100

Mid-span Region	Internal beams	
	Number of bars	Stirrups separation
Real case	13	200
Estimation	15	NO STIRRUPS
	Edge beams	
	Number of bars	Stirrups separation
Real case	13	200
Estimation	12	200

Table 5 Comparison of the model with Väg 50 bridge

VÄG 50 MJÖLBY - MOTALA

Support Region	Internal beams	
	Number of bars	**Stirrups separation**
Real case	8	100
Estimation	7	100
	Edge beams	
	Number of bars	**Stirrups separation**
Real case	8	100
Estimation	6	100

Mid-span Region	Internal beams	
	Number of bars	**Stirrups separation**
Real case	12	200
Estimation	11	200
	Edge beams	
	Number of bars	**Stirrups separation**
Real case	12	200
Estimation	11	200

As it can be seen in Table 4 and Table 5, the comparison is good enough to consider the model suitable to be used for preliminary design. Bearing in mind that the results estimated by the tool, preliminary design values, were compared directly with the real bridge final design, figures can be considered close enough. This is because the final design was expected to have some important influences from issues considered in the final (and detailed) analysis but not in pre-design. Anyway, there are no remarkable disagreements that could suggest that the pre-design results are unreliable. The biggest deviation from the predesign and the final model was two rebars, while stirrups separation was usually the same or very close.

It is important to explain here that the script was allowed only to use spacing of the shear reinforcement in multiples of 100 millimetres. Therefore, 150 millimetres separation was not an expected value from the predesign algorithm. Moreover, the tool was prepared to avoid shear reinforcement if concrete capacity was estimated to be enough, displaying "NO STIRRUPS" as a result in mid-span region in Table 4, where shear force is prone to be low due to its distance to the supports.

9.1.2 NCC Nynäshamn

As described in *Subchapter 9.1.1*, comparison of the concrete bridge model was based on two different sources. Firstly, it was checked that the bridge was considered as feasible according to the criteria considered. Secondly, the reinforcement design produced by the tool was compared with the real reinforcement design of the bridge, extracted from the actual bridge documentation.

In the case of this concept, the first comparison was possible to be performed, since a similar strategy to discard possible bridges is followed. On the other hand, the second verification was impossible to be done, since there is no beam reinforcement design in this bridge concept.

While seeking for other ways to show some comparisons of the models with reality, some interesting results were discovered in the parametric study of the bridge. Some of the parameters showed stabilization after a critical value was reached. That means that some parameters were increasing the bridge capacity up to a certain value, and oversizing the parameter further gave almost no benefit for the structural capacity. This behaviour was found in some parts of the bridge, such as the flanges (both upper and lower).

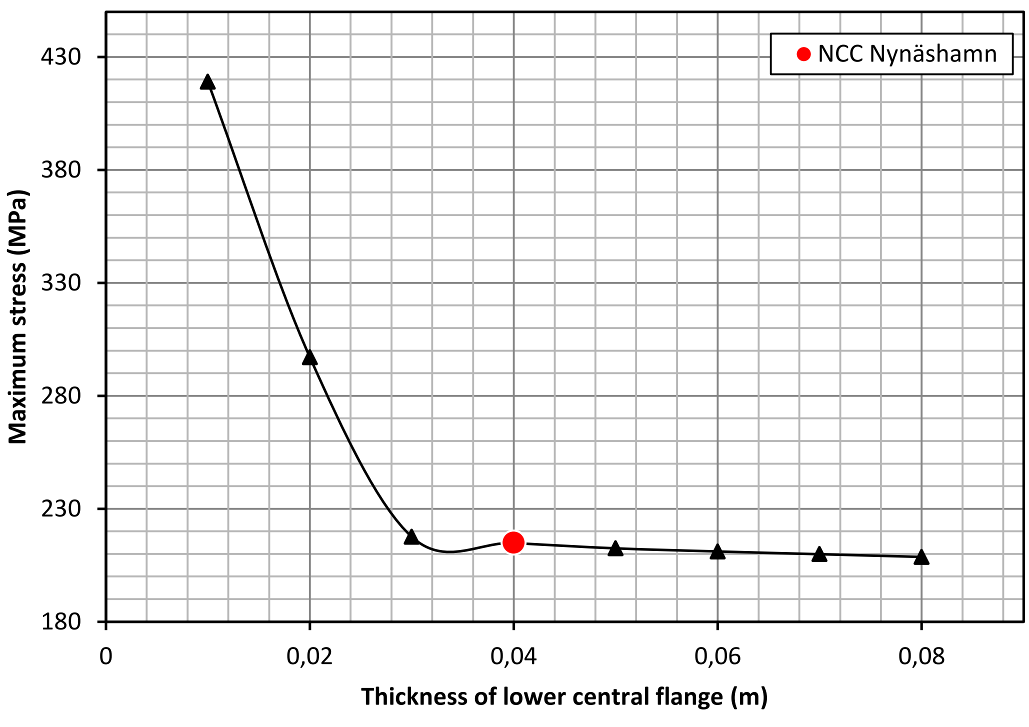

Figure 35 Parametric study of the thickness of the lower central flange (Maximum stress)

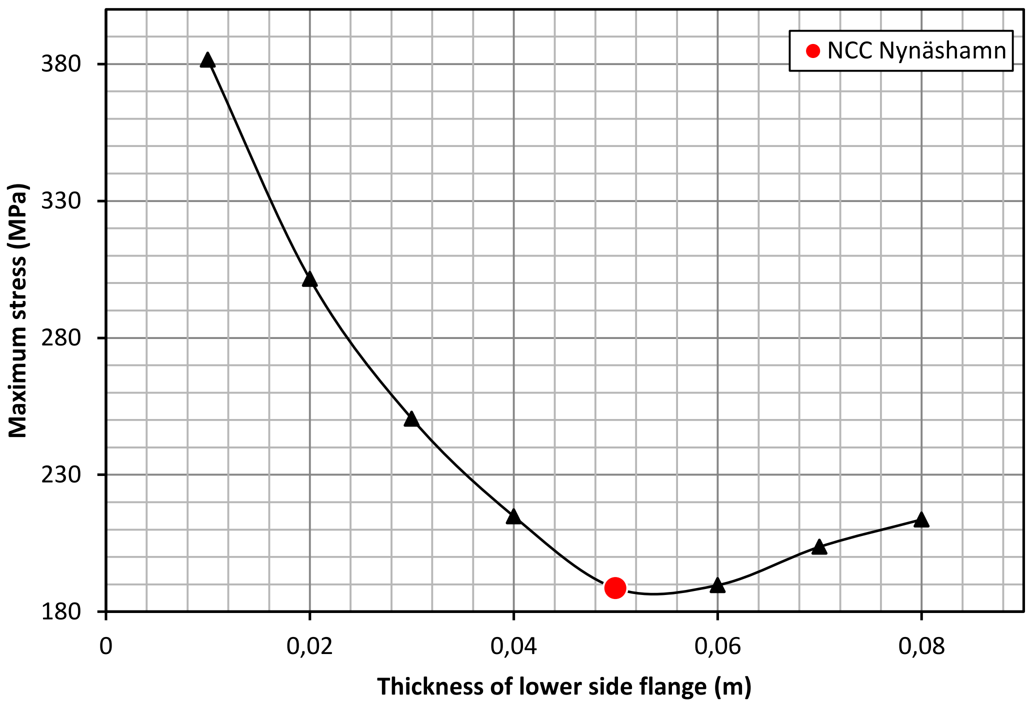

Figure 36 Parametric study of the thickness of the lower side flange (Maximum stress)

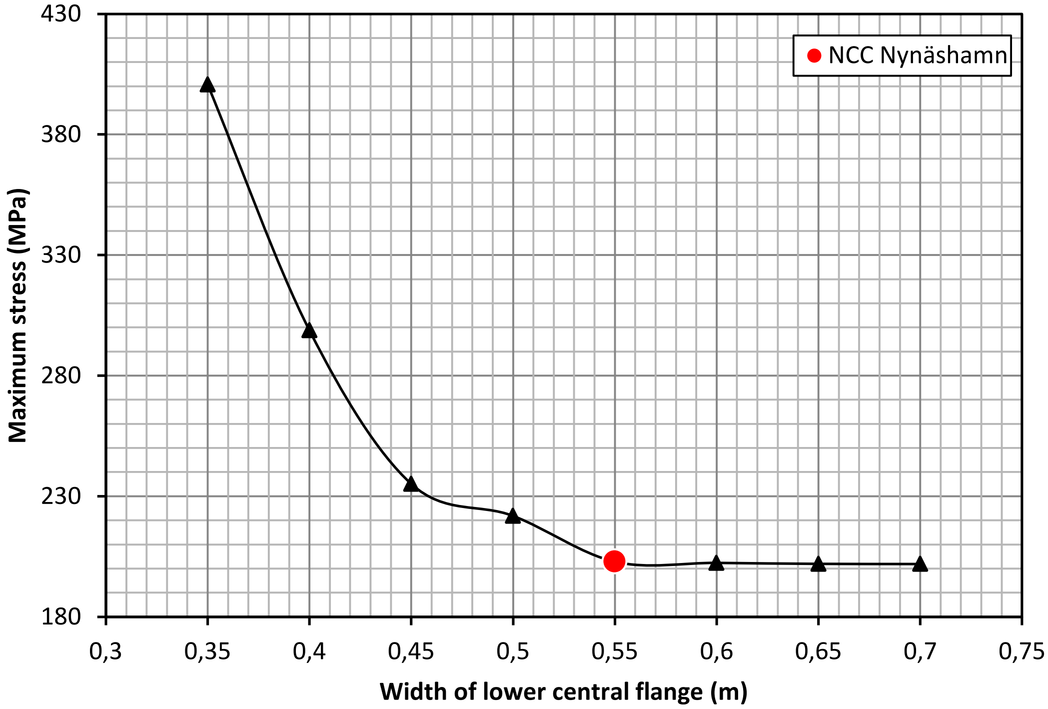

Figure 37 Parametric study of the width of the lower central flange (Maximum stress)

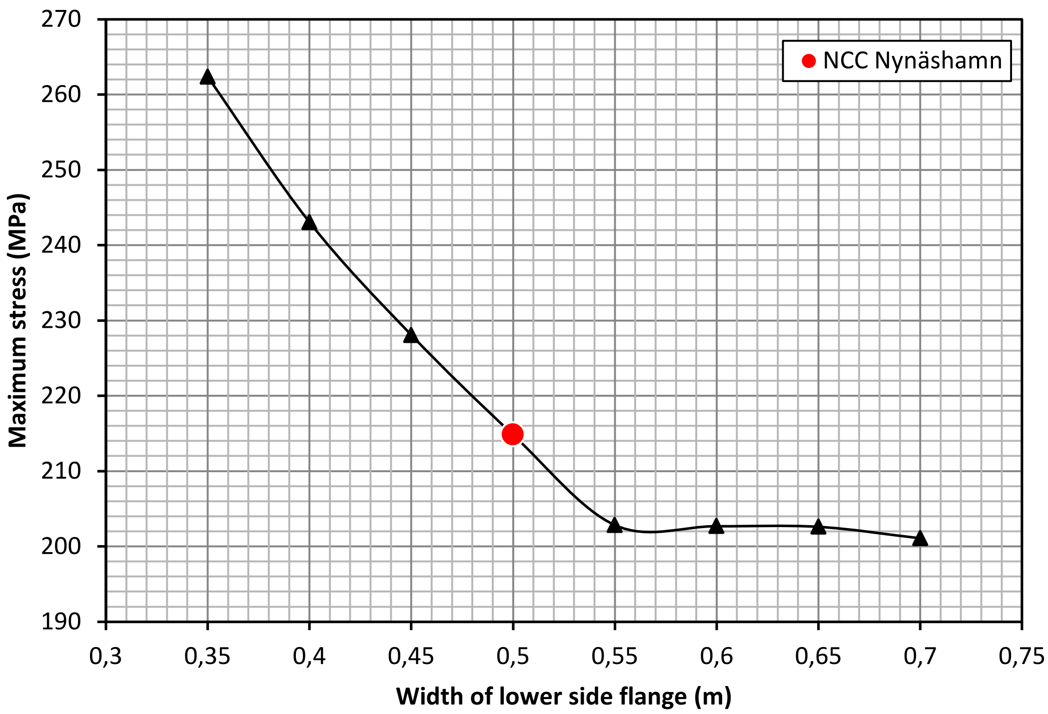

Figure 38- Parametric study of the width of the lower sideflange (Maximum stress)

As the composite bridge didn't have any beam reinforcement design to compare with, as they were made of stainless steel, other approaches were considered to perform the comparison with reality.

After an in-depth parametric study of the concept, it was discovered that some of the parameters used in the real bridge were very near to the optimal values showed in the study. As it can be seen in Figure 35 to Figure 38, the real bridge was designed on the verge of the limiting values, showing clear intentions of performing a design as optimal as possible, avoiding waste of material. This fact was considered as a good symptom for the comparison of the model in the predesign phase, since the structural response of the whole FE model seemed to be very similar to the design targets of the original designers that presumably used the traditional approach.

Although the results obtained were considered fair enough for a successful comparison of the composite bridge with reality, one of the checks included could not be considered accurate enough. Lateral torsional buckling under traffic load conditions could not be correctly assessed with the tool. Eurocode models suitable for the geometry of the bridge were limited (beams are not uniform and their section varies along the length), so the capacity regarding lateral torsional buckling was heavily underestimated. It is worth to comment that the tool even considered that lateral torsional buckling would happen in the already built bridge, which has no stiffeners in reality. This was one of the reasons that suggested the consideration of buckling as a *want criterion*, since further FE buckling analysis should be performed to assess buckling capacity of the alternatives.

9.2 Comparison of bridge performance

In order to get some interesting results from the developed tool, comparison with real cases was used for assessing the its performance in selecting better alternatives than the ones built by traditional design approaches.

Figure 39 and Figure 40 show the normalised material cost and CO_2 equivalent of the alternatives selected by the tool as structurally-feasible, and those from the real cases. The normalisation of the results has been performed by dividing the material costs and CO_2 equivalents by the minimum values. As first glance, it can be seen that, if still being considered as feasible after the required full analysis, there are alternatives suggested by the tool which are more optimal in terms of material cost and CO_2 equivalent than those built in reality. However, a full analysis of the most optimal alternatives needs to be done in order to verify whether the methodology really provides better designs than the ones made traditionally.

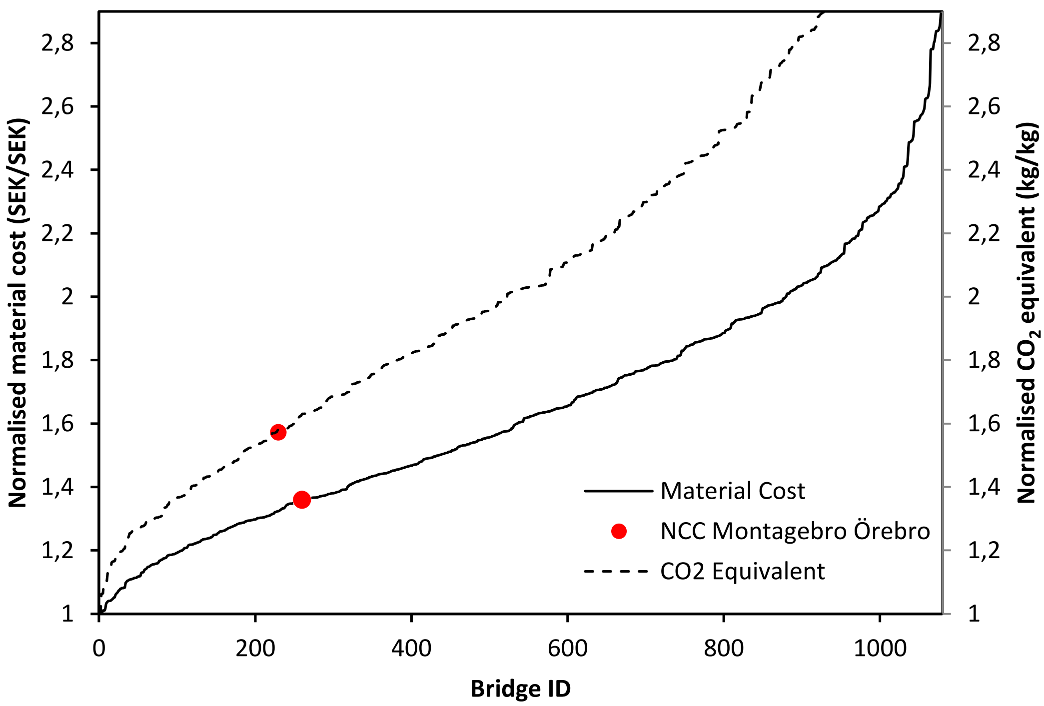

Figure 39 Distribution of analysed alternatives regarding normalised material cost and CO2 equivalent for the concrete bridge concept Montagebro

It needs to be mentioned that the parameters for the concrete bridge were chosen in broader ranges due to there were less parameters defining the bridge concept, and therefore this can explain the higher performance of the optimal alternative compared to the real case. On the other hand, the parameter ranges of the composite bridge were taken by using as the reference the built bridge parameters, and hence they are a less representative sample. In both cases more alternatives can be considered by incrementing the number of parameters in the ranges, which can lead to more computational time, although for these tests, not even a day was required. However, all the parameters of the composite bridge cannot be varied at the time, because so many combinations lead to unmanageable time consumptions and the ones with more influence have to be pre-selected.

Based on the aforementioned, if a bridge wants to be placed within a completely new scenario and there are no reference parameters for defining the ranges, the composite bridge might become unsuitable since it is defined by too many parameters. Referred to this issue, a more compact parameterisation of the composite bridge based on less tailor-made girders could solve the problem.

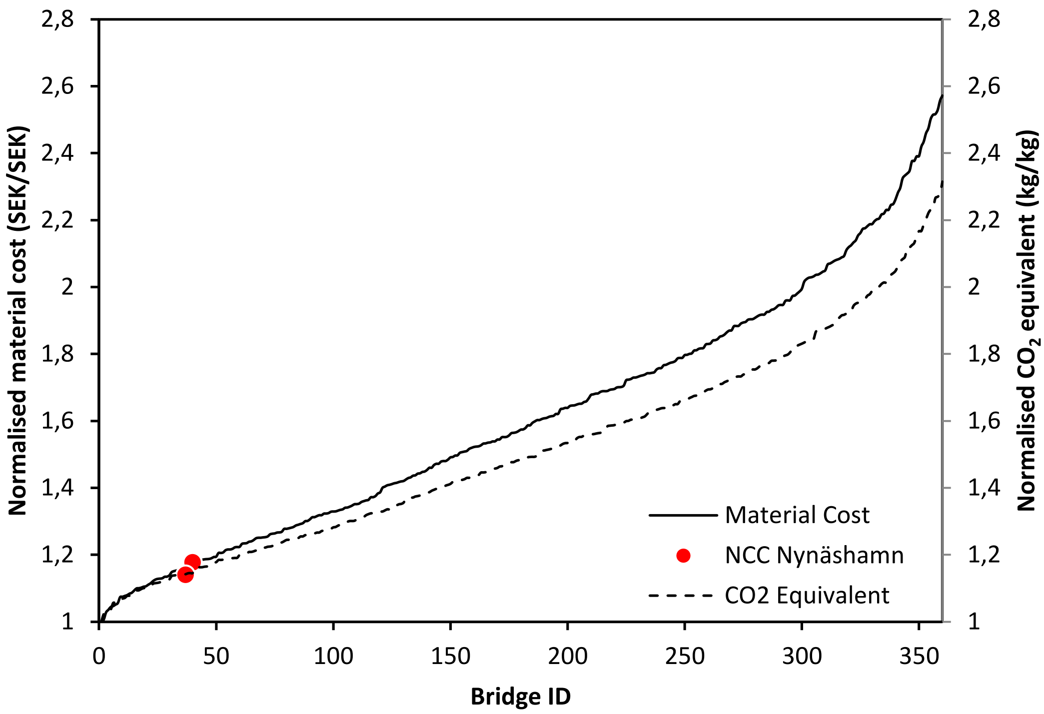

Figure 40 Distribution of analysed alternatives regarding normalised material cost and CO_2 equivalent for the composite bridge concept

The shape of the figures is similar in both cases which first presents a slightly higher performance reduction ratio, then a linear zone and ends with a very high performance reduction ratio. The former can be explained by considering that only few parameters provide the best performance when combined to each other, then the combination of parameters stabilise the rate because the parameters compensate each other and at the end the combination of the larger parameters in term of dimensions give too big bridges which are much more expensive in relation.

Table 6 Cost reductions between the real case and the optimal alternative given by the tool for different optimisation scenarios for the concrete bridge concept Montagebro

90% Material Cost – 10% $C0_2$ equivalent	***10% Material Cost – 90% $C0_2$ equivalent***	***50% Material Cost – 50% $C0_2$ equivalent***
35%	35%	35%

Table 7 Cost reductions between the real case and the optimal alternative given by the tool for different optimisation scenarios for the composite bridge concept

90% Material Cost – 10% C0₂ equivalent	***10% Material Cost – 90% C0₂ equivalent***	***50% Material Cost – 50% C0₂ equivalent***
18%	17%	17%

Table 6 and Table 7 show the cost reduction of the most optimal alternative considered in the three different optimisation scenarios presented in *Subchapter 7.3*. As it will be discussed in next chapter, and as it happened in the figures previously mentioned, the differences between material cost and CO_2 equivalent are very small, and almost the same optimal alternative is selected in all scenarios.

9.3 Correlation between optimisation criteria

Figure 41 and Figure 42 show the relation of material cost and CO_2 equivalent for all assessed alternatives, normalised according to the same procedure as before. As it was commented in *Subchapter 7.2*, material cost and CO_2 equivalent regarding only extraction and production are highly related and therefore little advantage is gotten when performing a multi-criteria optimisation with only these two criteria rather than considering only one. First, they are both estimated through the material amount and secondly, high extraction and production work-time consuming materials are commonly more expensive in terms of material cost, and therefore both are relatively similar in order of magnitude for different materials, i.e. costly materials are expected to have high CO_2 equivalent.

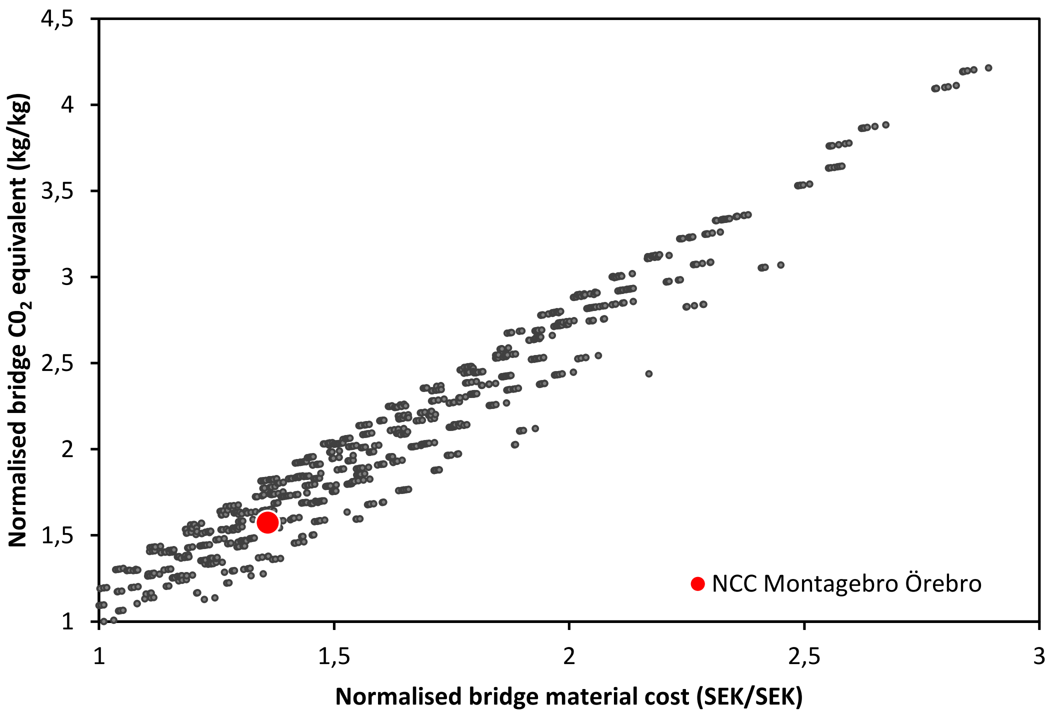

Figure 41 Correlation between normalised material cost and CO_2 equivalent for the concrete bridge concept Montagebro

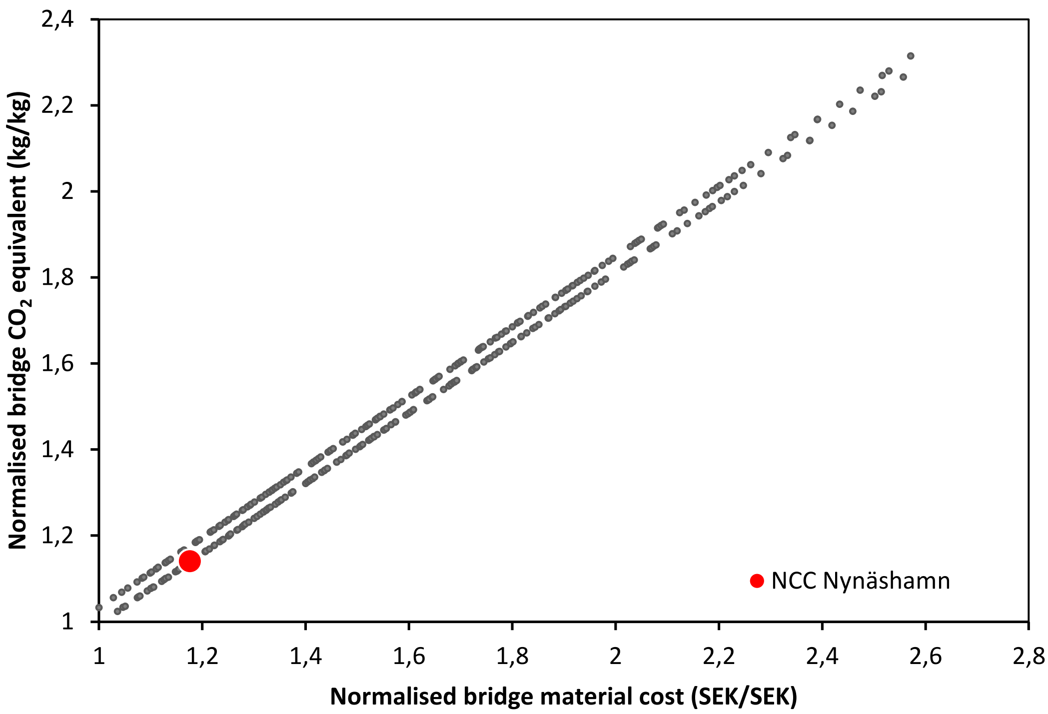

Figure 42 Correlation between normalised material cost and CO_2 equivalent for the composite bridge concept

It can be seen that the concrete bridge chart is more scattered due to the compensation between materials, e.g. less concrete beams implies more reinforcement and vice versa, and because of the higher variation of the number of beams considered in the analysis. On the other hand, the composite bridge chart shows that the girders have the highest influence in both parameters, and hence they follow two parallel lines corresponding to bridges with two and three girders respectively.

9.4 Suitability of bridge concepts regarding span length

Figure 43 and 44 show the most optimal alternative regarding material cost and CO_2 equivalent for both concepts over the span. It is important to note that the cost has been normalised so that the most optimal alternative for the shortest span is defined as value 1, and all other alternatives are weighted according to that value.

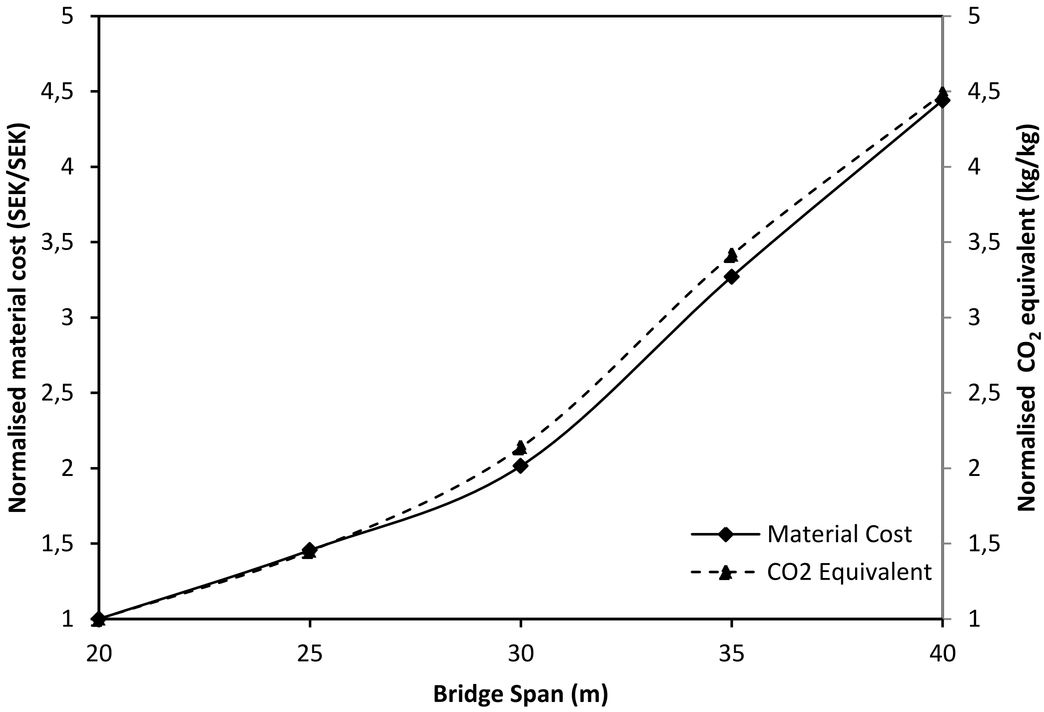

Figure 43 Cheapest alternatives regarding normalised material cost and CO_2 equivalent for different spans of the concrete bridge concept Montagebro

Regarding the shape of the curves, in Figure 43, i.e. Concrete Bridge, the increasing slope is due to the nature of the bridge. The longer the bridge is built, the more concrete is needed to extend it, but more reinforcement is needed per unit length as a consequence of the increasing stresses. This means than the increasing demand of material is not linear when increasing the span, but it is growing faster at longer spans. As an example, the price of increasing a concrete bridge from 30m to 35m long is higher than the investment that would be needed to increase it from 20m to 25m, although the increment of length is 5m in both cases. This is due to the higher amount of reinforcement per unit length that would be required in the first case.

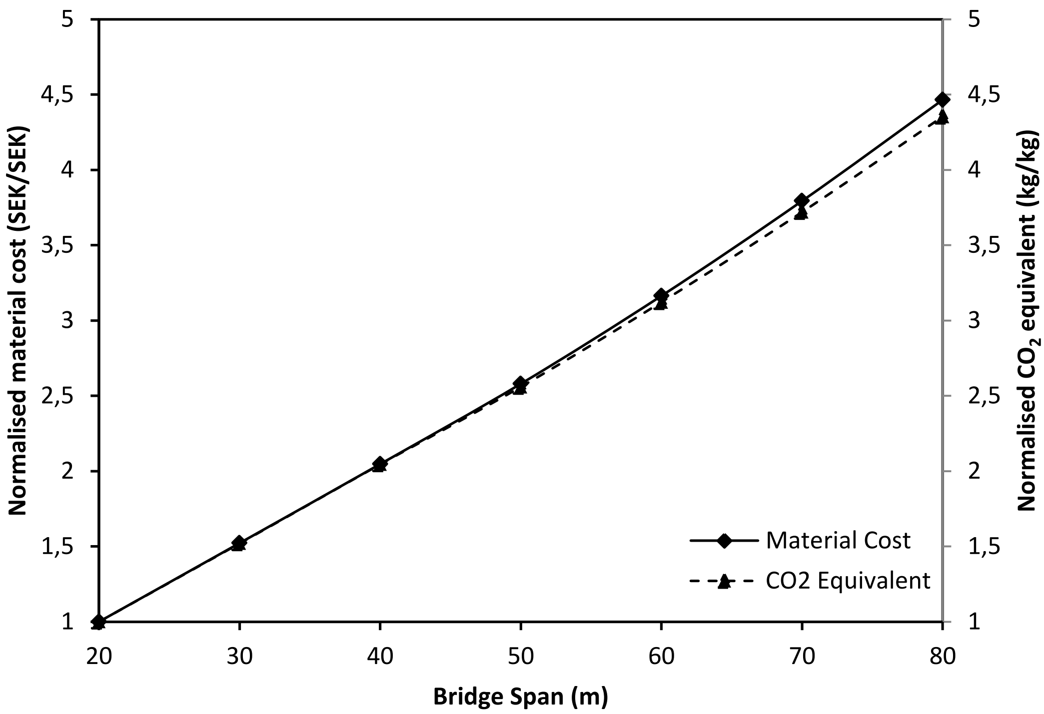

Figure 44 Cheapest alternatives regarding normalised material cost and CO_2 equivalent for different spans of the composite bridge concept

In case of the composite bridge in Figure 44, this tendency is almost linear because the longer the bridge is the bigger amount of material is needed, and in this case beams are not made out of reinforced concrete. If it is considered the high cost of stainless steel in comparison with concrete or regular reinforcing steel, together with the low increase of reinforcement rate that would happen in the slab, the tendency is almost linear. In this case increasing 5m of the bridge would be almost the same for every span because of the low increase of the slab reinforcement ratio as well as its low impact on then cost compared with the extra length of the stainless steel beams.

It would have been interesting to be able to assess in some way the best option between two concepts for a given situation i.e. span, but the different materials used in the concepts made it impossible to do it. At any span, if both options were compared stainless steel bridge would be much more expensive because life cycle cost is not performed in the methodology. If it had included maintenance in any way, life cycle cost comparison could be done. Therefore, prices are normalised according to the cheapest bridge of each concept and are not comparable from one concept to the other.

Figure 45 to 47 are histograms showing the number of alternatives considered as feasible for each span after the analysis phase. These histograms show the increasing mechanical demand of the bridges that the designer has to face when increasing the span while keeping the initial ranges of parameters.

As it can be seen in Figure 45, for the initial sets introduced as input data, concrete bridges had a feasible solution up to 40m span. Any longer bridge alternative for those values was not considered feasible by the tool. The reasoning for the different failures of the bridges will be discussed in the next section.

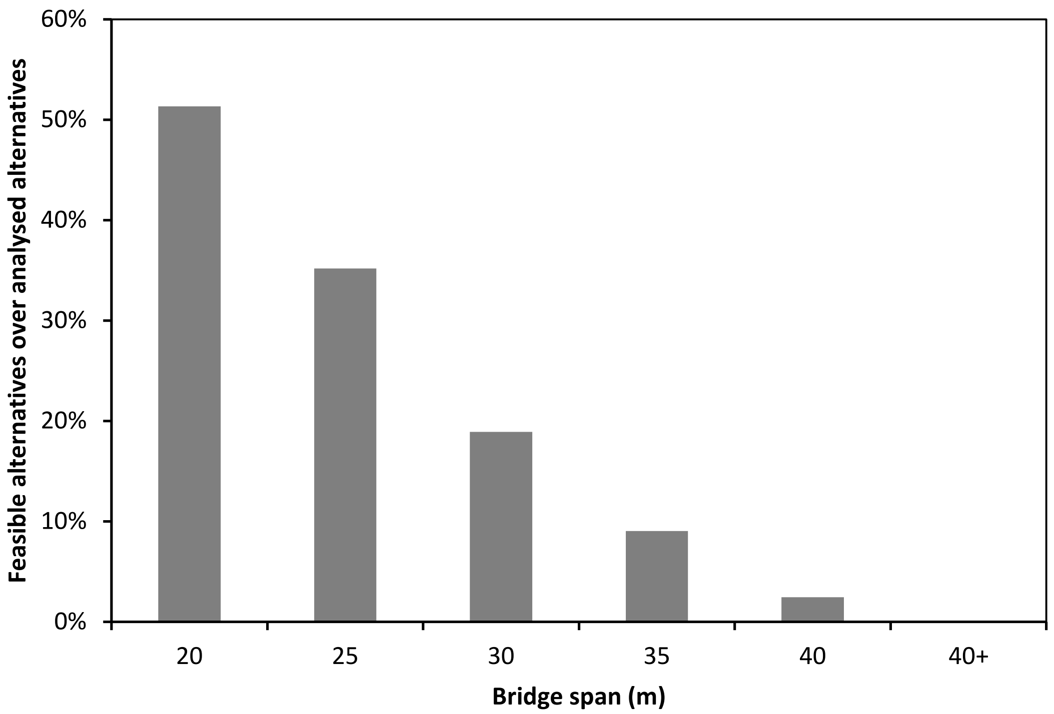

Figure 45 Number of possible alternatives after the analysis depending on the bridge span for an initial set of 2100 bridges per span. Concrete bridge concept Montagebro

Regarding the composite bridge, it is necessary to remember the two different selection criteria introduced in section 5.8, *must criterion* and *want criterion*. Hence, two different figures are showed according to the span length comparison of the composite bridge, one considering *want* and *must criteria* Figure 46 and another one only considering *must criteria* Figure 47. These two alternative histograms were included as well due to the underestimated Lateral Torsional Buckling discussed in *Subchapter 5.8.2.4*. If both figures are compared, it is easily noticeable that buckling is limiting the maximum span to 50m and reducing dramatically the number of feasible possibilities. On the other hand, if it is not considered as discarding criteria, it was possible to design bridges up to 80m, reaching the span limitations that the company claims for the concept.

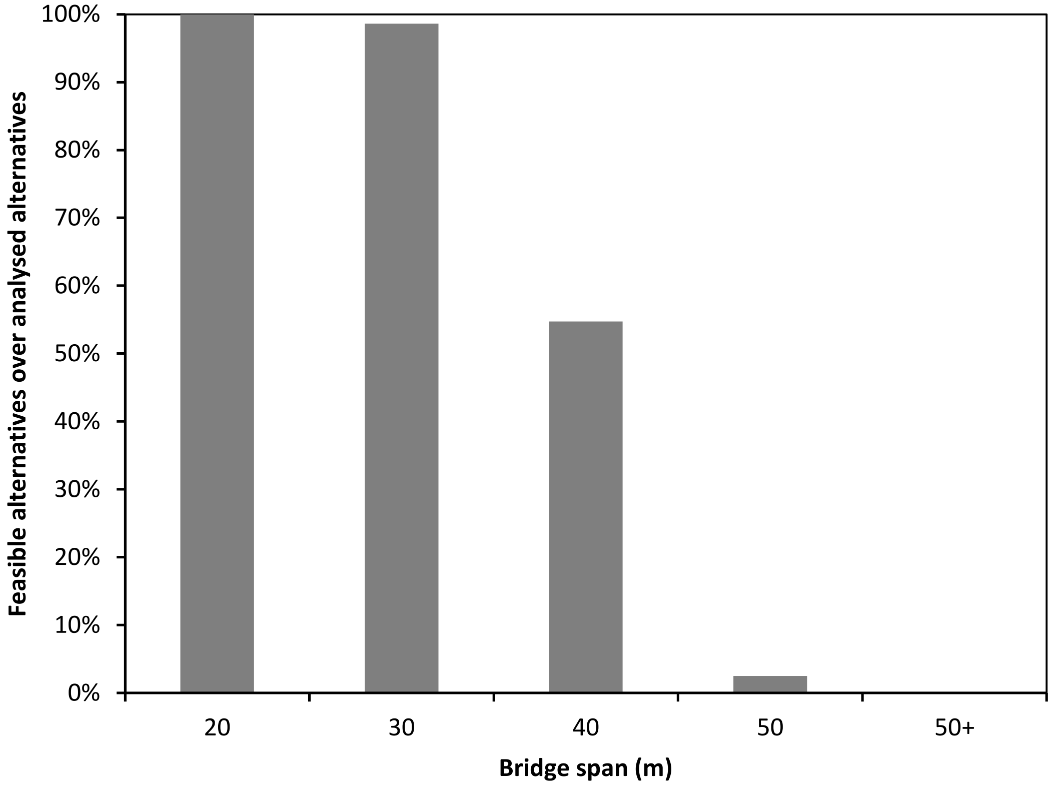

Figure 46 Number of possible alternatives after the analysis depending on the bridge span for an initial set of 360 bridges per span. Want and must criteria considered for discarding alternatives. Composite bridge concept

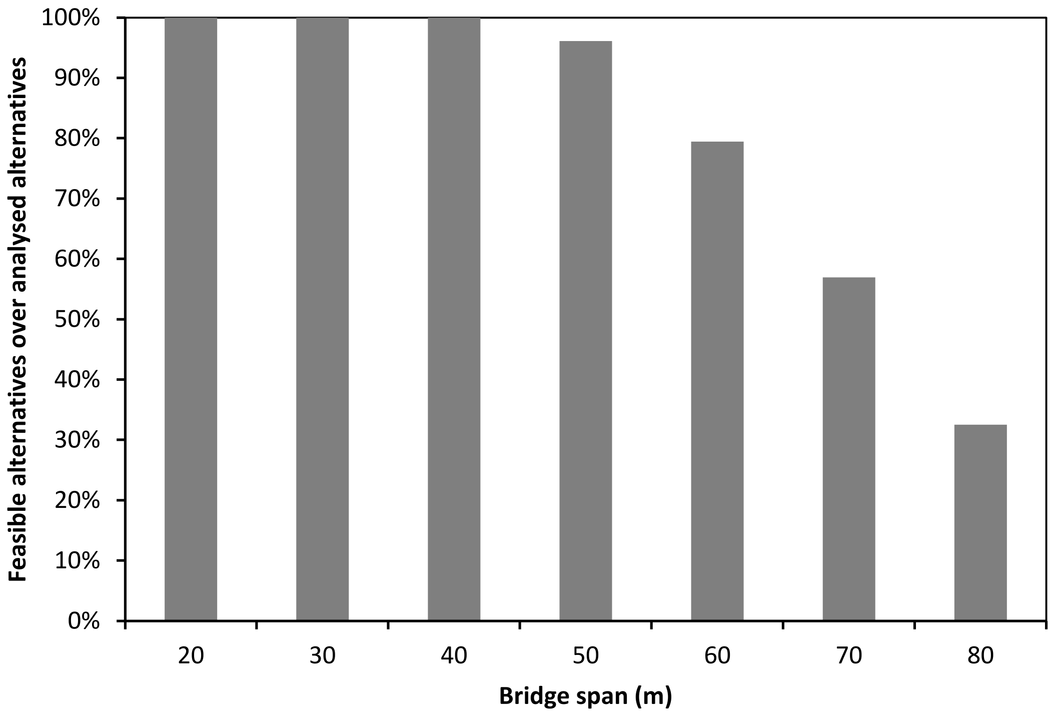

Figure 47 Number of possible alternatives after the analysis depending on the bridge span for an initial set of 360 bridges per span. Only must criteria considered for discarding alternatives. Composite bridge concept

It is worth to comment that if the figures discussed in this section are examined thoroughly, the number of different alternatives is much higher for the concrete concept than the composite concept. This is due to the different possibilities of design in both concepts. While the concrete bridge considers different reinforcement diameters for the beams, reinforcement design is calculated once for each diameter value, and up to 7 alternatives are obtained from the same geometry, due to 7 different rebar diameters. Reinforcement design is computationally fast because it is an analytical method, and it does not require any FE calculation. However, input data for the design are extracted from the FE analysis of the geometry of the bridge.

On the other hand, the generation of a single alternative in the composite bridge requires a FE analysis, because there is no possible alternative generated with analytical design within the same geometry. Therefore, it is much more computationally expensive to generate a similar number of alternatives with this concept if it is compared with the concrete bridge. Besides, it can be seen from the total acceptance of alternatives up to 40 metres, than the selected parameters were not representative enough.

9.5 Failure types distribution

The following pie charts show the distribution of the different failures identified when considering the alternatives as unfeasible.

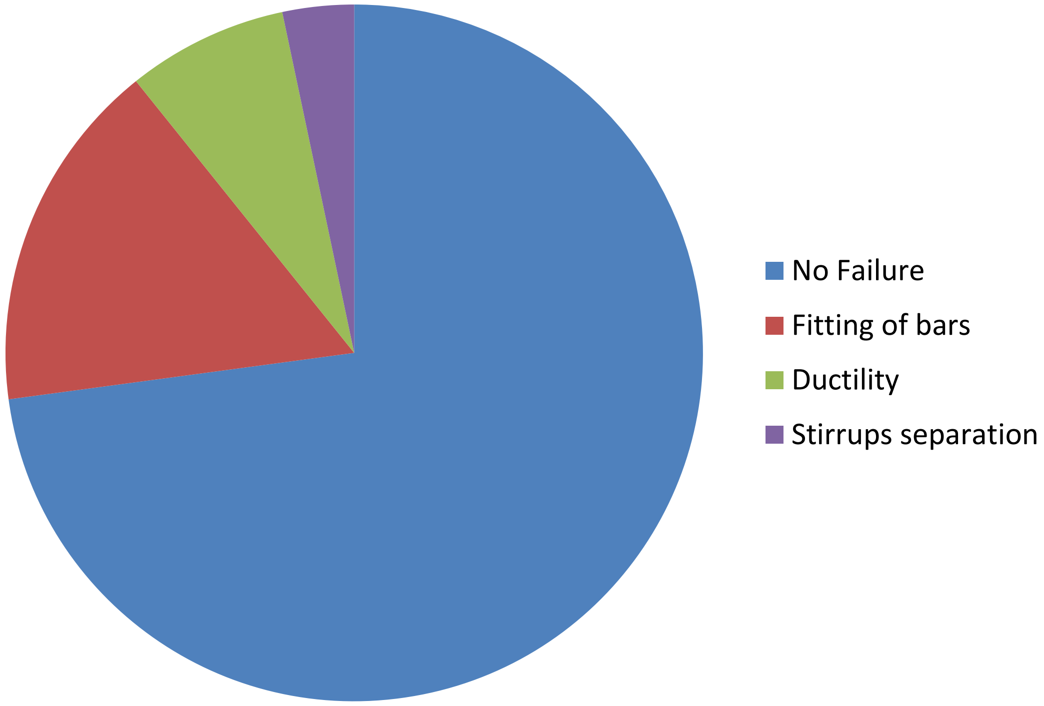

Figure 48 Failure types distribution for the concrete bridge concept Montagebro.

As it can be seen in Figure 48, three main failure types influence the discard of regions for the concrete concept, which corresponds to around 25% of the total amount of analysed regions. This figure does not show the number of bridges discarded when checking if all regions of a bridge are feasible which might be interesting to analyse in further studies. The figure reveals that the most restrictive criterion is the fact of limiting the possible number of reinforcement layers up to three followed by not fulfilling the ductility requirement and then by too low separation of stirrups needed. All these aspects correspond to the higher bending moment and shear force that needs to be carried when the length of the bridge is increasing, specially excess of bending moment due to the configuration of the bridge and characteristics of the loads.

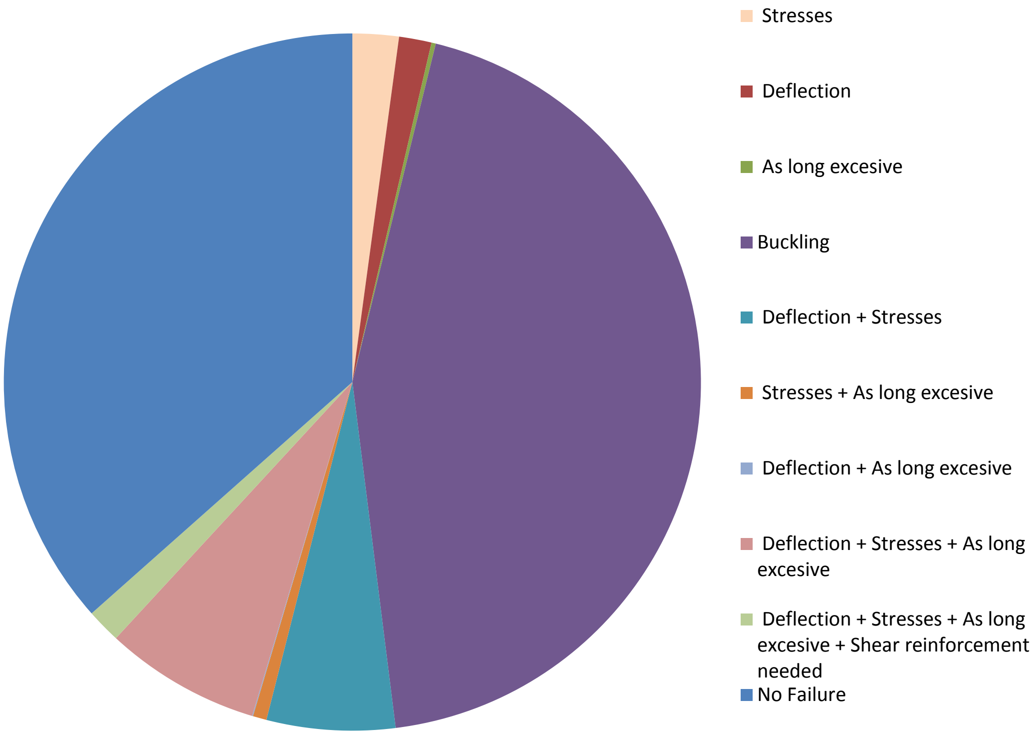

Figure 49 Failure types distribution for the composite bridge concept. Want and must criteria considered as failure types.

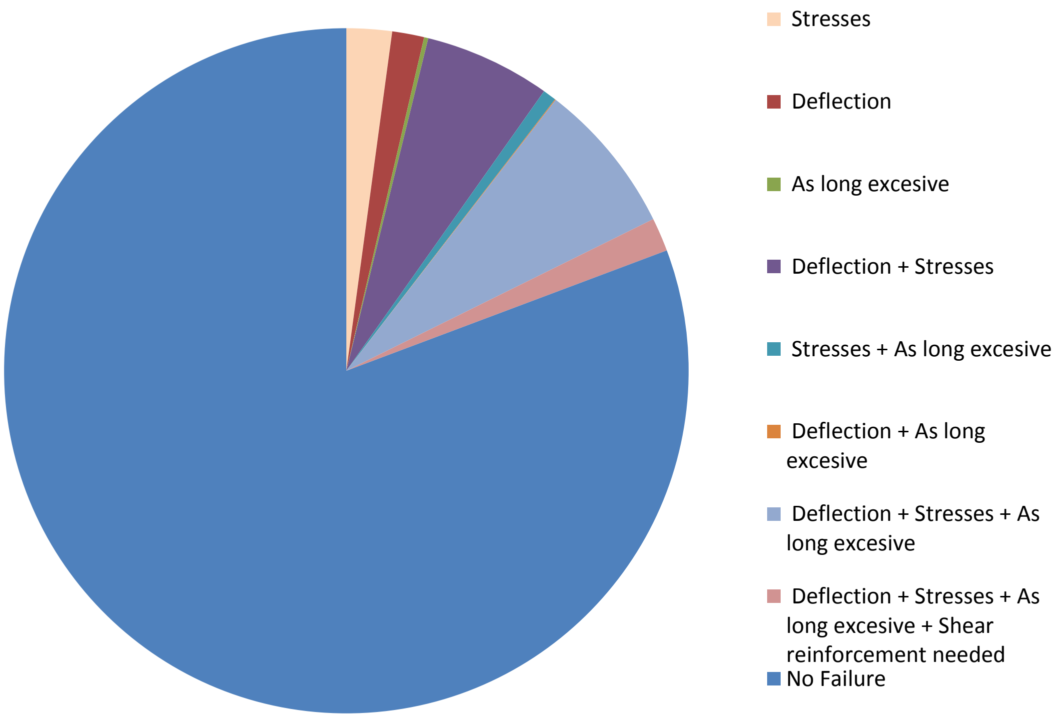

Figure 50 Failure types distribution for the composite bridge concept. Only must criteria considered as failure types.

Figure 49 and Figure 50 show the failure types according to the discarding process for the composite bridge. Both charts showing considering *want* and *must criteria* and only *must criteria* have been shown in order to verify that the buckling analysis as performed in the tool is too restrictive and a better approximation needs to be done. If buckling is not considered as failure, the most common unfulfilled criteria are the combination of too high deflection and stresses, and these two combined also with too high longitudinal reinforcement amount over the supports. This allows verifying that all three criteria are strongly related and become all a restriction when the span is increased too far. This is due to the increase of the span length leads to a higher amount of material and hence higher loads due to self-weight. It can be seen that shear reinforcement is not required often for this kind of bridges.

9.6 Applicability of set-based design on structural engineering

As presented in *Chapter 2,* the Set-Based Design approach seems to present many advantages over the traditional Point-Based Design approach, e.g. rework reduction, deeper knowledge of the design or better cooperation between stakeholders. However it had not been widely applied specifically into structural design where it could improve the design process and lead to more optimal designs. The following discussion intends to justify the potential of this approach within bridge design, as an example of structural design.

Since the bridge pre-design stage usually implies simple checks from the standards, they are suitable for being implemented into a computer script when the required data from the bridge behaviour is available. All these data related with the structural behaviour of the bridge, can be obtained by means of a Finite Element Analysis, and hence, the combination of both leads to the possibility of setting an automatic process. This automatic process facilitates the application of these pre-design checks to many bridge alternatives, and therefore the principles of Set-Based Design for finding a set of optimal alternatives can be implemented into the design of bridges.

First, for selecting the initial set of alternatives which will be analysed, the different alternatives are built as random combination of parameters from ranges defined by the designer, which requires a previous parameterisation of the analysed bridge concept. Secondly, the stakeholders can determine those criteria that need or are preferred to be fulfilled by these alternatives and hence the initial set of alternatives can be narrowed down. These criteria will mainly correspond to two different phases of the narrowing process; the determination of the structurally-feasible alternatives, and the optimisation of those possible alternatives regarding cost or environmental issues.

This methodology, supported by the development of a computer script which controls and manages the outcomes from a Finite Element Analysis, presents several direct and indirect advantages in the design process of structures, although some limitations are also presented.

This tool is not intended for replacing the bridge designer but for helping him in the pre-design phase. As discussed in *Chapter 9*, this tool has shown good fitting with some concepts that were designed by traditional means, which proves that the checks commonly used in practice have been correctly introduced in the script, and that the script is just doing what a designer normally does, but automatically. This possibility facilitates the analysis of huge amounts of alternatives unlike the traditional methodology of assessing only few alternatives by hand calculations, and in consequence, to find more optimal alternatives.

The tool also allows considering the opinion of the different stakeholders from the very beginning of the design process, by including the checks required by the administration or the criteria preferred by the client to optimise the bridge against.

Even if a computer script seems not very clear in terms of verification of the implementation of the standards, the methodology incorporates easy-to-read calculations sheets that can be easily verified of producing the same results as the script. These sheets can also be provided by the administration as specific required standards to be implemented into the script.

As mentioned in *Chapter 7*, is it essential to allow the person in charge of the project the possibility of making the last decision, and the tool can provide a set of optimal alternatives regarding different optimisation criteria, for this person to be able to select the most suitable alternative according to his criteria. This set also gives the possibility to have alternative designs in case the final analysis considers the chosen design as unfeasible, regarding one of the analysis that are not performed in pre-design.

Besides direct design applications, the tool can also be used for performing parametric studies or assessing suitability of alternatives regarding different situations, e.g. whether a concrete bridge or a composite bridge is more suitable for a specific purpose or location.

On the other hand, some limitations need to be considered. First, this methodology requires a bridge concept which can be parameterised by a reasonable amount of parameters, or at least, that can be proved that some of the parameters have little influence in the structural behaviour and can be estimated beforehand. This tool does not give the best bridge for a certain purpose or location but selects an optimal design according to a bridge concept, and therefore is intended for industrialised bridges and not for unique designs.

Furthermore, the methodology fits well with early stages of the design process, where the required checks and analyses are not very complex or time consuming. If the time consumption for designing an alternative is too high, the possibility of considering large amounts of alternatives becomes unfeasible. To find the equilibrium between accuracy degree and computational time of the analysis is fundamental for this methodology. The use of a computer cluster as did in this project is highly recommended for performing high number of analyses, which can limit the applicability of this tool.

Regarding the aspect aforementioned, in this project, the assessment of the lateral torsional buckling capacity of the steel girders in the composite bridge concept has been a difficult task to overcome due to the curved profile of those elements. After considering suitable methods for an iterative process, it was found that none of them gave a good estimation of the real capacity of the girder, and that the general method needs to be used. This general method implies a specific buckling Finite Element Analysis of each alternative, and it has not been included. Nevertheless, as mentioned in *Chapter 5*, this criterion is described as a *want criterion* because it can be solved by relatively low cost increment through stiffeners.

10 CONCLUSIONS

In this project the applicability of Set-Based Design in structural engineering has been studied and it can be concluded that this design approach suits well the needs of this field.

- The methodology here presented follows properly the principles of Set-Based Design and allows an iterative preliminary design of large amounts of design alternatives.

    - A literature review of the principles of Set-Based Design has been performed, concluding that even though it shows high potential in structural engineering, it has not been widely applied in this field.

    - In order to be able to carry out an iterative design process, a parameterisation of the structure, i.e. a bridge; is necessary for defining a set of different alternatives, which has been successfully done.

    - The methodology requires a computer script for controlling and evaluating the outcomes from a Finite Element Analysis, which has been developed showing multiple applications and great potential.

- The methodology has been successfully compared with two different study cases and applied for estimating better alternatives than the ones based on the traditional design practice, which has led to more optimal designs. Furthermore, the possibility of pre-design a large amount of bridges suitable for different situations increases the competitiveness.

    - Since traditional design practice begins with a pre-design stage where simple checks are performed by hand calculations over a limited number of design alternatives, the possibility of simulating this process automatically and being able to analyse large amount of alternatives in order to find a more optimal solutions is very advantageous. Furthermore this methodology leads to a better understanding of the design. However, it is mainly suitable for bridge concepts that are intended to be built several times.

    - The methodology that has been developed, not only permits direct design application but other sort of studies such as suitability assessment of different bridge concepts or parametric studies.

    - Different techniques for the comparison of the analysis performed by the computer script have been used. In the case of a concrete bridge, the script suggests a very similar reinforcement design than the one of the study cases. On the other hand, in the composite bridge, through an assessment of different parameters, it has been proved that the analysis of the script considers a similar behaviour than the one considered by the designers in the case study.

- The methodology has been developed and described in a structured way, which shows the different modules, explains how it is done and how it works, and it allows the designer to apply it for different purposes. Its limitations have been also presented and some techniques for overcoming these constraints have been presented, e.g. a parametric study to reduce the number of parameters for a given design.

    - By using computational tools, the management of the raw data from the analyses has been successfully performed. Even though the amount of data is huge, by using a script the size of this data is not an issue, and an automatic management carried out by the script itself allows presenting only the very necessary data to the designer in a user friendly way. A basic multi-criteria optimisation has been performed in order to show the high potential of this tool together with this methodology.

    - A computer cluster has been used for reducing the time consumption required by the methodology, when big amounts of alternatives are analysed. The methodology has been developed for being performed either in this type of clusters or in a network of desktop computers, which provides more flexibility.

11 FURTHER INVESTIGATIONS

After the initial development of the methodology carried out in this project, some further development and investigations might be performed in order to improve both its applicability and results.

Considering some improvements that might be investigated regarding the enhancement of the applicability of the methodology, some new features could be implemented. One of the limitations of the current tool is the only compatibility with single span bridges, reducing the situations where it can be applied. A very good boost for applicability would be overcoming this limitation, by adapting the existing tool for continuous beam conditions and other characteristics included in multiple span bridges.

Regarding parametric issues, it would be beneficial for the methodology to improve the parameter management, providing the possibility of wider ranges i.e. bigger design space, as well as develop some relations between them that could allow a more effective parameterisation. Although it is now possible to enlarge the design space as far as desired, time consumption becomes huge, so this could be optimised and improved.

Another applicability enhancement that could be introduced is the improvement of the optimisation process, by including more advanced methods and new criteria e.g. construction time, labour cost, traffic issues… It is important to note that the more criteria included there, the better alternatives will be obtained since the tool would take into account everything it has been provided.

As a final thought regarding applicability, lifecycle cost assessment has arisen as a very important feature that should be investigated in order to be able to compare different concepts of very different nature in a global framework, assisting the decision of the designer in the concept bridge selection.

Regarding the improvement of the results, more checks could be implemented in the tool, providing more Eurocode limitations to the assessment of the structures. It is important to highlight here the necessity of further investigation regarding to buckling, since the approach used in this project was not reliable enough to be trusted.

Additionally, some other important phenomena might be studied and included in the tool, such as long term effects of concrete or fatigue assessment of steel. The more of this features included, the closer the predesigned bridge will be to the final design reducing the time needed for the whole design process.

Finally, it would be interesting to investigate the feasibility of including the automatic final analysis of the best alternatives aiming to create a global tool going from blank paper to the final design. Although it might look like a greedy objective, at least some research on this topic would be much of interest.

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