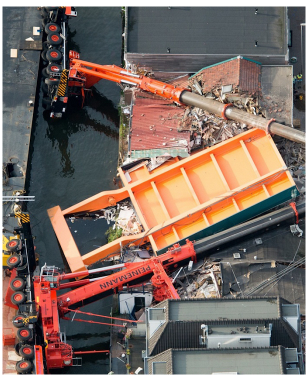
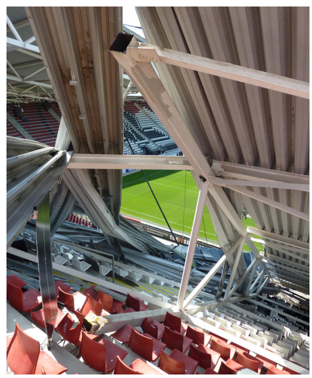
Conceptual Structural Design

Understanding Structural Performance



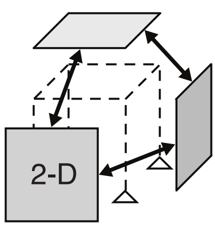


Systematic thinking

Scientific method:

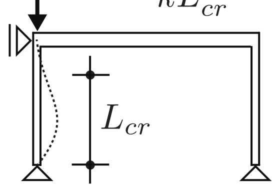
1. Problem definition
2. Research framework
3. Hypothesis
4. Validation
5. Conclusion

Systems thinking



Modelling

$$N_u = \frac{\pi^2 EI}{kL_{cr}^2}$$



Michiel Paul Horikx

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Executive summary

The subject of this textbook is a methodical approach on the complex problem-solving process of conceptual structural design.

Problem A lack of insight of the professional into structural engineering is considered to be a main obstacle for an overall performance/cost optimisation of the built environment.

For a better understanding and an effective solution approach this lack of insight can be divided into the two partial problems, “lack of insight into conceptual structural design” and the underlying “lack of insight into structural performance”.

Solution approach The methodical approach on conceptual structural design leads to a controlled build-up of insight into the behaviour of the structure and supports the actual successive design decisions during the conceptual design phase on the basis of the following coherent set of solution components:

Structural design cycle Independent of life cycle phase, complexity of design, and contractual commitments, the structural engineering practice can be outlined by a fundamental structural design cycle: creation, optimisation, and specification. Within each of the major design phases conceptual design, basic design, and detailed design, this fundamental structural design cycle is applicable.

Basic structural forms One of the main conceptual structural design activities is the determination of the structural form, based on "understanding" and "order of magnitude" of basic structural forms. Characterisation of structural forms, with regard to the capacity to bear and resist, and with regard to the interfaces with the built environment, turns out to be feasible on a two-dimensional subsystem level.

Structural design path The structural design process can fundamentally be characterised by the two simultaneous processes of specification and decomposition of the structural form from system to element. These two processes can be visualised together in a two-dimensional matrix in which the structural design can be explored. The design path follows the fundamental dimensioning routine from structural integrity, via load distribution, to failure mechanisms.

Structural design loops In order to have a successful solution to a complex design problem, a cyclic design process is inevitable. Every cycle goes through the phases of creation, optimisation, and specification. The cyclic convergent optimum strategy aims for an optimisation of both quality of design outcome and number of design cycles. The individual design loops are appointed and the corresponding basic principles clarified.

Shared knowledge-based conceptual design For an optimisation of the performance/cost ratio of the life cycle of a structure, an integral approach on conceptual design is a necessity. Control of the interfaces between all participating disciplines is largely dependent on experience and intuition. Definition and collection of the fundamental conceptual design parameters of the most influential participating disciplines serve as a concurrent-based breeding ground for integral design solutions.

Conceptual structural design parameters Both for understanding structural performance and for an effective knowledge exchange with the other participating disciplines, a balanced set of conceptual structural design parameters is conditional. On a two-dimensional subsystem level, approximation parameters for conceptual structural design are determined on the basis of timeless applied mechanics.

Case study This specific case study demonstrates how the methodical approach leads to a controlled build-up of insight into the behaviour of the structure and supports the actual successive design decisions during the conceptual design of the trusses of the Maeslant storm surge barrier.

The load paths, overall geometry, and principal detailing on the basis of performance, structural, and construction demands, are determined. Subsequently, the structural action in this outlined structure is optimised and the elements are dimensioned. Finally, a thorough risk analysis is conducted as a demarcation of the conceptual structural design phase.

Training The required knowledge, skills, and professional attitude have to be achieved by a mix of the learning methods lecture, training, and project work. Lectured theories and trained engineering practice can be applied during project work.

The training programme consists of a series of trainings, including a zero measurement training. All trainings have approximately the same entry professional on graduate master level, with a balanced complexity of geometry and modelling.

Epilogue It is important to study how the design is organised in practice, and especially the ways in which designers with different disciplinary expertise are able to work together, collaboratively in teams. For an integral conceptual structural design, the main contributing disciplines and corresponding interfaces have to be considered

This textbook discusses conceptual structural design on a high level of abstraction. However, a deepening research on conceptual structural design is valuable and feasible. Recommendations for research are given with respect to both understanding these complex interdisciplinary interfaces and structural performance.

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Chapter 1

Introduction

1.1 About this textbook

1.1.1 Structural design

Structural design consists of three major sequential design phases: conceptual, basic, and detailed design.

With detailed design - a phase of code checking, detailing, and specifying - all common material applications have been extensively researched and recorded in numerous textbooks and design codes of practice.

For basic design - a phase of deepening and optimisation - the main tools are applied mechanics-based and since Isaac Newton widespread available as textbook material.

Conceptual design - the creation phase with a complex and partly intuitive process and numerous complex interfaces between different fields of practice - is little touched by technological progress.

Complexity of design Design is to optimise the performance/cost ratio of the life cycle. For design, it is mostly the outcome that counts, rather than the followed path. The design path, however, does control both duration and flexibility of the complex design process.

The art of design, striving for the ultimate solution, is the process of getting oversight by abstracting complexity and crossing borders.

Abstracting complexity The standard engineering practice of handling complex reality is modelling this reality with an abstract and approximate representation. Besides structural modelling, which is the core of structural engineering, process modelling can attribute to a clarification of the complex cyclic design process.

Crossing borders between dissimilar things Scientific education in structural design and corresponding research is compartmented to such an extent that the interface between applied mechanics and material applications is underdeveloped. The interfaces between structural demand, performance demand, and construction demand are even less visible. Both professionals and higher education programmes will benefit from crossing borders and clarification of these interfaces.

Conceptual structural design Conceptual design is the first and decisive phase of design, providing the overall integrated system. Here, the proverbial "DNA" of the solution is constructed.

Present-day conceptual design of regularly complex structures with numerous boundary conditions, require a thorough understanding of design parameters and extensive experience in design.

1.1.2 Subject and knowledge level

The subject of this textbook is the intersection of professional structural engineering and the conceptual design of the built environment as shown in figure 1.1. In general, an effective and efficient conceptual structural design is based upon a profound practice of simplification and decomposition techniques.

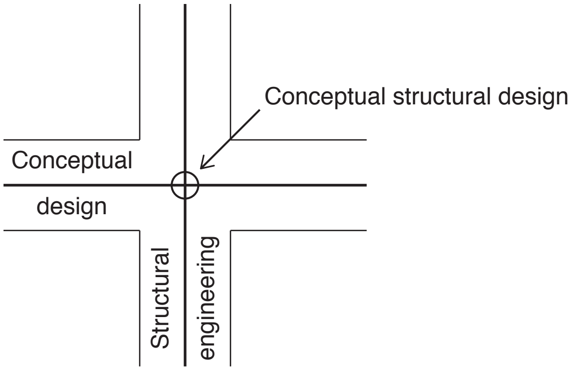


Figure 1.1: Conceptual structural design

However, the number and complexity of sophisticated high-end computer programs and interfaces with other disciplines, increasingly dominate daily practice of the present-day professional structural engineer. Due to this on-going expansion of depth and breadth, the simplification and decomposition techniques of the experienced structural engineer are steadily disappearing from practice training and higher educational programmes.

Knowledge level The simplification and decomposition techniques for conceptual structural design, as covered by the contents of this textbook, are on an entry professional on graduate master level. Within both professional and science graduate education, the focus can vary from a direct practical relevance to an in-depth academic research relevance.

Zero measurement training Out of educational considerations the arrangement of contents in this textbook is clustered around major subject matters, rather than following the overall chronological sequence of activities of the structural engineering practice. The required knowledge, skills, and professional attitude have to be achieved by a mix of the learning methods lecture, training, and project work. Lectured theories and trained engineering practice can be applied during project work. This textbook supports both lecture and training, in class as well as self-tuition.

The training programme consists of a series of trainings, including a zero measurement training, as listed in table 18.1 on page 216. The zero measurement training can be applied as a self-check at the beginning of the learning process.

1.1.3 Textbook outline

This textbook is the follow-up of a previous academic research on the complex problem-solving process of structural conceptual design [9]. The so explored methodical approach on conceptual structural design leads to a controlled build-up of insight into the behaviour of the structure and supports the actual successive design decisions during the conceptual design phase on the basis of the following coherent set of solution components:

Structural design cycle Capturing structural design by breaking down this complex process into the essential absolute minimum, resulting in a fundamental design cycle as an effective characterisation of both the overall design process, and the individual design phases.

Basic structural forms Basic set of two-dimensional subsystems, with each individual subsystem being an assembly of directly connected structural elements, designed to act together to resist loads.

Structural design path The structural design can be explored in a two-dimensional matrix, in which the design path follows the fundamental dimensioning routine from structural integrity, via load distribution, to failure mechanisms.

Structural design loops A combination of analysis, check, orientation, and correction loops to support a cyclic optimum design with regard to optimisation of both the quality of design outcome and the number of design cycles.

Shared knowledge-based conceptual design Definition and collection of the fundamental conceptual design parameters of the most influential participating disciplines can serve as a joint breeding ground for integral design solutions.

Load path design A first draft of the structure's integrity requires simple and clear three-dimensional modelling on the level of axial forces, directly or with a truss-analogy, including the three-dimensional effects.

Load distribution parameters Applied mechanics-based oversight of force- and deformation-driven load distribution and a matching balanced set of conceptual design approximations.

Dimensioning parameters Applied mechanics-based oversight of professional practice dimensioning and a matching balanced set of conceptual design approximations.

1.2 Reading guide

1.2.1 Partition of the textbook outline

The analysis and definition of the problem, the solution in the form of a combined physical and process decomposition, the quantification of the individual design parameters, and a case study and training programme are divided in four identifiable partitions as shown in figure 1.2.

Part I Problem definition For effective problem solving, the problem is divided into two partial problems "lack of insight into conceptual structural design" and the underlying "lack of insight into structural performance".

Part II Conceptual structural design The solution to the lack of insight into conceptual structural design consists of a combined physical and process decomposition.

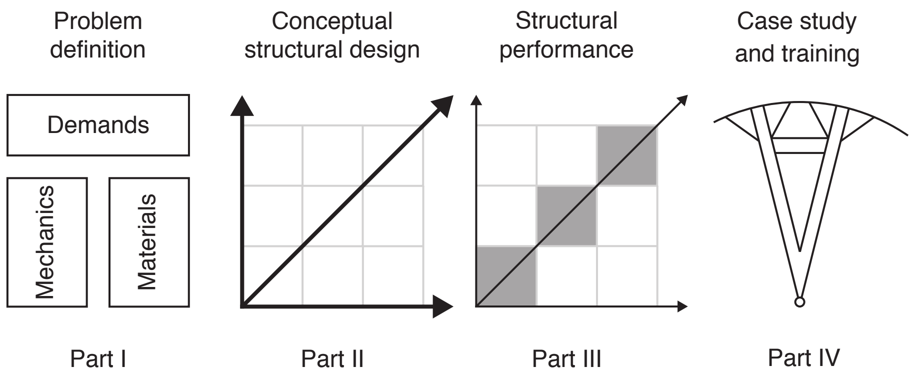


Figure 1.2: Partition of the textbook outline

Part III Structural performance The solution to the lack of insight into structural performance consists of qualification and quantification of the structural engineering fundamentals in the form of conceptual structural design parameters.

Part IV Case study and training Both qualified and quantified solutions are clarified by means of a case study and a series of trainings, including a zero measurement training.

1.2.2 Introductions

Each part starts with a chapter "Introduction" which describes the reason for its existence and a chapter guide of the specific part within this textbook.

Accompanying figures and descriptions clarify the coherence between chapters and the content of individual chapters.

Part I

Problem, analysis and solution approach

Chapter 2

Introduction to part I

A lack of insight of the professional into structural engineering is considered to be a main obstacle for an overall performance/cost optimisation of the built environment. A methodical approach with effective solutions to this lack of insight is the subject of this textbook. Present-day problems, orienting analyses, and a solution approach with corresponding solution components are described in part I with a chapter arrangement as shown in figure 2.1.

Present-day problems High-end computer programs, numerous interfaces with other disciplines, and a tendency towards more implicit performance-based provisions increasingly dominate daily practice of the present-day professional structural engineer. Due to this on-going expanding depth and breadth, the simplification and decomposition techniques of the experienced structural engineer are steadily disappearing from practice training and higher education programmes.

Because of this, the understanding and reliability of the young professional in structural engineering is on the decline. Computerised designing without sufficient insight, particularly in the conceptual design phase, is a dangerous operation both out of economical and safety points of view.

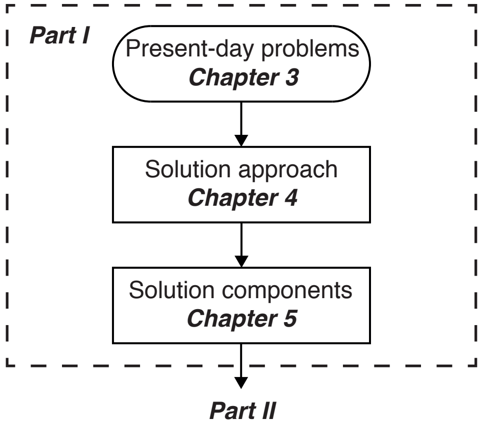


Figure 2.1: Reading guide for part I

Solution approach A thorough definition of the problem is a crucial step in the problem-solving process, because an effective problem definition is directing the solution approach.

For a better understanding, and effective problem solving, the lack of insight of the professional into structural engineering can be divided into the two partial problems, "lack of insight into conceptual structural design" and the underlying "lack of insight into structural performance".

The field of practice shift from calculating/checking to modelling/designing requires the following three pillars for a future proof structural design: universal systematic thinking, applied systems thinking, and applied mechanics-based modelling.

Solution components In general, complex problems can be effectively approached on a relatively high level of abstraction with an elaborate knowledge of the fundamentals and a simultaneous working "from the whole to the part" and "from coarse to fine".

The methodical approach is aimed to be a conceptual design tool for experienced structural engineers, rather than a black box operated by data typists. Therefore, the solution is carried out in the field of abstractions of a decomposed system instead of algorithm-based numerical power.

In order to secure durability of the design method, the solution is focused on being overall applicable to construction types and the use of timeless elements such as mechanics.

Chapter 3

Present-day problems in structural engineering

3.1 Structural (un)safety in the Netherlands

3.1.1 Structural failures

In the Netherlands, there has been a notable amount of structural failures over the last decades. For example, a lot of roofs and parking decks have collapsed while in use, as well as complete buildings, mostly during construction.

Some news-breaking structural failures include:

Theatre Het Park, Hoorn, 2001 Collapse of the theatre tower during construction due to a combination of multiple engineering and construction errors.

Hotel Van der Valk, Tiel, 2002 Parking deck collapse due to a lateral torsional instability and subsequent horizontal displacement of the supporting beams.

Bos en Lommerplein, Amsterdam, 2006 Near collapse of supporting parking garage beneath a residential complex due to missing concrete reinforcement.

Stadium De Grolsch Veste, Enschede, 2011 Roof collapse during construction due to a loading of the incomplete stabilised roof structure.

Queen Juliana bridge, Alphen aan den Rijn, 2015 Pontoon-based crane collapse during construction due to severe shortcomings in construction engineering and management.

Eindhoven Airport, 2017 Floor collapse due to an unusual orientation of wide-slab flooring in combination with insufficient overlapping of the reinforcement splices.

AFAS Stadium, Alkmaar, 2019 Roof collapse due to engineering errors with respect to wind loading and weld strength of the roof structure.

3.1.2 Structural safety codes of practice

In Eurocode [19] structural safety is defined as the capacity of a structure to resist all action(s), as well as specified accidental phenomena, it will have to withstand during construction work and anticipated use.

Eurocode EN 1990 [14] further defines reliability as the ability of a structure or a structural member to fulfil the specified requirements, including the design working life, for which it has been designed. Structural reliability covers in fact four aspects: safety, serviceability, durability, and robustness of a structure.

The semi probabilistic level I calculations in the material-related Eurocodes are based on the assumption that an element is sufficiently reliable if a certain margin is present between the representative values of the load effect and the resistance as shown in figure 3.1.

The representative value of the load effect S has a 5% probability of overshooting, whereas the representative value of the resistance R has a 5% probability of undershooting. The use of probability of exceedance

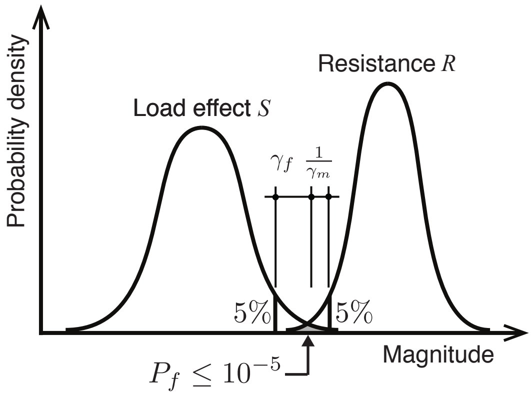


Figure 3.1: Structural safety Eurocode

instead of mean values incorporates the influence of probability distribution.

The resistance is the capacity of a structure to resist a load effect. The verification implies that the resistance R has to be greater than or equal to the load effect S. The risk of failure when $R < S$ should be sufficiently low.

When there is no margin between the representative values of S and R the probability of failure approximately amounts to $P_f \leq 10^{-1}$, which is sufficient for the Serviceability Limit State (SLS). However, for the Ultimate Limit State (ULS) this probability of failure $P_f \leq 10^{-1}$ is entirely insufficient and should be upgraded towards $P_f \leq 10^{-5}$.

The corresponding margin between the representative values of S and R can be obtained by global partial factors γ_f and γ_m for load effect and resistance, respectively. The verification of the safety is then based on an equation of the following type:

$$S \cdot \gamma_f \leq \frac{R}{\gamma_m} \tag{3.1}$$

With these partial factors stochastic variability is covered, which is related to uncertainties in loads, materials, geometry, and calculation models. However, stochastic variability does not include gross human errors.

3.1.3 Actual structural safety

The actual structural safety in the Netherlands with a probability of failure over the last decades $P \ll 10^{-5}$ satisfies easily the required structural safety level of the codes of practice with a probability of failure of about $P \leq 10^{-5}$. However, the actual safety ($P \ll 10^{-5}$) is not a subset of the regulated safety ($P \leq 10^{-5}$) as substantiated by Terwel [25]:

An extensive study of structural failures in the Netherlands has shown that the current number of fatalities among residents due to structural failures remains within assumed acceptable limits, although a high impact - low probability disaster did not occur in the observed time interval. This study showed also that about 90% of the failures are caused by human errors, although human behaviour is not included in the probabilistic calculation approach of the Eurocode. It seems a paradox that the individual risk remains within acceptable limits, although the main influencing factor, human error, is not included in the calculation approach. This can be explained because the actual strength of structures is often higher than the calculated strength due to redundancy.

3.2 Lack of insight of the professional

In recent years, in-depth investigations of a number of specific disasters have been undertaken by the Inspectorate for Housing, Spatial Planning and the Environment, research organisations, the Dutch Safety Board, university professors, expertise firms, and specially convened committees of enquiry.

These investigations reveal that it is almost never possible to identify one single cause for a disaster. Mostly, it is a combination of factors and circumstances - which are an inherent part of the participants of the building process - that can be identified. All these participants influence the structural safety of a building with their actions and interdependencies. Many failures, however, arise in the design phase.

Structural collapse in the Netherlands appears to be mainly a combination of the lack of supervision during all project phases and a lack of insight of the professional into structural engineering, as recorded in the problem statement "Castle or House of Cards" under the management of the Ministry of Housing, Spatial Planning and the Environment [23].

For the lack of insight of the professional into structural engineering this problem statement addresses some major perceptions of the Ministry of Housing, Spatial Planning and the Environment:

- Many in the construction industry realise that the level of professional skill among structural engineers, but also among other players, is on the decline.

- University professors in the field are noticing a general erosion of knowledge and command of applied mechanics, the mainstay of the structural engineering profession.

- The "black box" character of calculation software will further diminish people's understanding of the subject.

Eight organisations, including the Inspectorate for Housing, Spatial Planning and the Environment, the Concrete Association, and structural engineers and builders' organisations have published the joint Compendium for a Structural Safety Strategy [24].

This compendium contains a detailed description of how structural safety can be guaranteed in the various phases of the design and building process and what roles the various participants in the building process can play with regard to structural safety. Since 2018 this compendium has evolved into a national internet platform for structural safety "Kennis-Portaal Constructieve Veiligheid (KPCV)".

3.2.1 Problems, causes, and effects

The overall problem of structural unsafety in the Netherlands is caused by many shortcomings and developments. This textbook will discuss the problem of a lack of insight of the professional into structural engineering, with corresponding causes and effects, as recorded in the problem statement [23].

Two partial problems For a better understanding, the lack of insight of the professional into structural engineering can be divided into two partial problems:

1. Lack of insight into structural performance on micro level; human error and inadequacies of people working on building projects.

2. Lack of insight into conceptual structural design on macro level; problems relating to the structure and culture of the building sector.

Structural performance The lack of insight into structural performance is generally accepted to be caused by the following:

- General erosion of knowledge and command of applied mechanics, the mainstay of the structural engineering profession.

- Present-day extensive use of calculation software, essentially a "black box", further diminishing people's understanding of the subject.

Conceptual structural design The lack of insight into conceptual structural design is generally accepted to be caused by the following:

- Increasing number and complexity of interfaces with other disciplines and corresponding collaboration processes.

- Increasing complexity of contractual models such as the Design, Build, Finance, Operate, Transfer (DBFOT) model and the Value for Money (VfM) model.

Safety and costs Foregoing causes can have considerable effects on both safety and costs:

- The lack of insight and especially improper use of advanced computer programs can put structural safety at severe risk.
- An uncontrolled design process can bring about insufficient performance/cost optimisation and a lot of failure costs.
- Besides a tendency for excessive numerous functional requirements, process control and reliability of a tender build-up is obviously endangered by a lack of insight of participating professionals.

3.2.2 General requirements structural engineering

For competent structural engineering, the compendium [24] outlines the following general requirements:

- Experience with like projects.
- Adequate knowledge about the required type of structure with regard to the structural behaviour: actions, materials, structural action, fire resistance and, if applicable, dynamic effects and fatigue.
- Adequate knowledge about the required type of structure with regard to the integral aspects: construction, architecture, durability, maintainability, and sustainability.
- With attention to the geotechnical engineering, including interactions between structure, foundation, construction activities, and structural environment.
- The ability to judge the results of automated design tools.
- Insight into the interaction between detailed design and the behaviour of the structure.

3.2.3 Recommendations structural design

With regard to structural design, the compendium [24] outlines the following recommendations:

- Aim for an as-clear-as-possible structural design concept, with regard to the overall load distribution and the decisive failure mechanisms.
- Predicated on safety, e.g. structures must not collapse without due warning.
- Be aware of the potential consequences of a structural failure and design a second method of support to ensure that the forces are dispersed elsewhere when a vital structural component can give way.
- Apply comprehensive approximated design calculations as a check for black box automated complex design tools.

3.3 Historical perspective

3.3.1 Developments professional structural engineering

With increasing complexity of structures and corresponding design, the master builder of ancient times inevitably altered into a team of specialists, as shown in figure 3.2 and clarified in the following subsections:

Homo universalis Master builder with expertise on the fields of architectural, structural, and construction engineering. This master builder is figured as the “Vitruvian Man” by Leonardo Da Vinci [28], based on the correlations of ideal human proportions with geometry described by the ancient Roman architect Marcus Vitruvius Pollio [29].

Expanding depth and breath Structural engineering as a separated formalised discipline.

On-going expanding depth and breath Specialisations within the professional field of structural engineering.

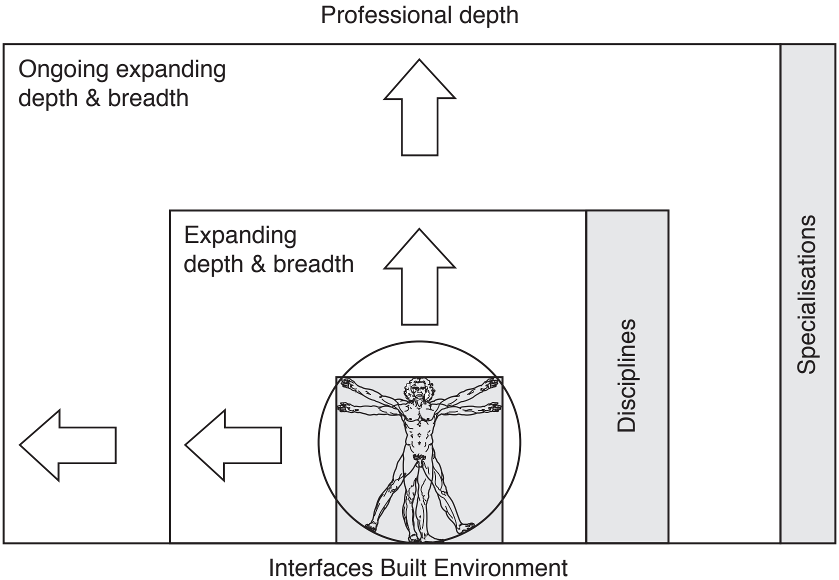


Figure 3.2: Expanding depth and breadth

3.3.2 Homo universalis

Structural engineering has existed since mankind started to construct its own structures. Throughout ancient and medieval history all architectural, structural and construction design was carried out by one person; often an artisan in the role of master builder. Structural comprehension was extremely limited and almost entirely empirically based.

The physical sciences underlying structural engineering began to be understood during the Renaissance in the late 15th century. It was then that architectural, structural, and construction design evolved into a more profound and controllable knowledge level but was still in the hands of one person, then called “Homo universalis”.

The Latin expression “Homo universalis” can be translated to “Universal person”, meaning a person with a broad knowledge of several fields and often with proficiency or accomplishments in at least some of these fields. Many notable universal persons lived during the Renaissance period such as Leonardo da Vinci and Michelangelo.

Until the 19th century, only one person was needed to integrally oversee the design, a generalist with profound expertise on the fields of architectural, structural, and construction engineering. The term “Generalist” is used to contrast this general approach to knowledge to that of the “Specialist”.

3.3.3 Expanding depth and breadth

With the development of specialised knowledge of structural theories, which emerged during the industrial revolution in the late 19th century, structural engineering came into existence as a more defined and formalised discipline.

The volume of knowledge of materials, technologies, and construction methods was increasing and structures became more complex. Due to the limited ability of comprehension of each individual professional, the field of building engineering was inevitably split into the separate disciplines of architecture, structural engineering, and construction engineering.

The modern structural engineer can rely on a long history of constant validation of theoretical approaches, building up extensive knowledge data bases such as applied mechanics-based structural analyses, previous designs, design rules, design codes of practice, and numerous researches.

To complete any project, it now takes a team of professionals that includes structural engineers working with other disciplines such as mechanical, geotechnical, electrical, and civil engineers, and urban planners and architects.

3.3.4 On-going expanding depth and breadth

The volume of knowledge of materials, technologies and building methods is still increasing enormously. Furthermore, there is a tendency away from the explicitly deemed to satisfy provisions towards more implicit performance-based contracting. This inevitably asks for corresponding expertise and brings with it ever more in-depth specialisation.

Within the field of structural engineering alone there is so much expertise that a structural engineer can never master it fully, resulting in specialisations such as geotechnical engineering, pre-stressed concrete engineering, finite elements engineering, and bridge engineering.

Numerous sophisticated high-end automated design tools increasingly support the daily practice of the present-day professional structural engineer. In spite of, or perhaps just because of these extensive design tools, the understanding and reliability of young professionals in structural engineering decreases dramatically.

They lack a fundamental understanding of structural behaviour, and they lack an overview and insight into the conceptual design process and related interfaces. In short, they lack the ability to abstract the basic design parameters of form, material, and dimension.

Chapter 4

Solution approach

4.1 Problem definition

4.1.1 The merit of a proper problem definition

Albert Einstein is quoted as having said that if he had one hour to save the world he would spend fifty-five minutes defining the problem and then five minutes solving it [4].

Furthermore, he is quoted as having said that the significant problems we face could not be solved at the same level of thinking we were at when we created them [6].

So finding a methodical approach for the complex conceptual structural design requires an elaborate problem definition on a high abstraction level. Subsequently, working out the solution is merely a derivative activity.

4.1.2 An interface control approach

For a better understanding, and effective problem solving, the lack of insight of the professional into structural engineering is divided into the two partial problems "lack of insight into structural performance" and "lack of insight into conceptual structural design", as appointed in subsection 3.2.1 and clarified in figure 4.1.

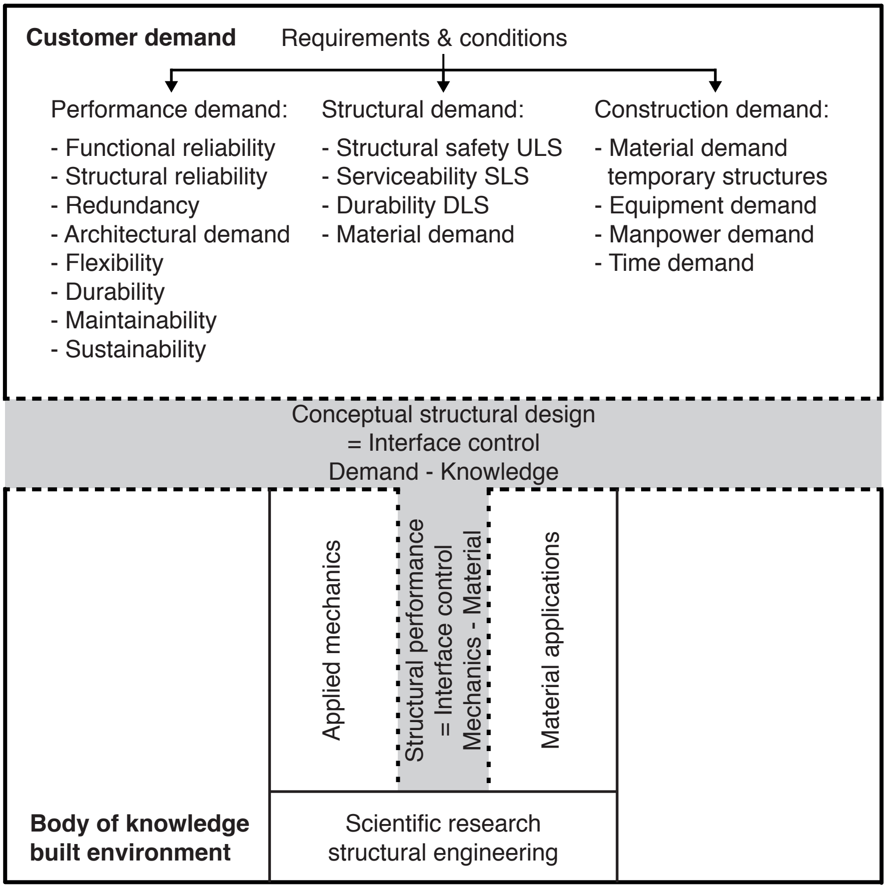


Figure 4.1: Structural design as an interface control approach

The relation of performance, structural, and construction demand with the performance/cost optimisation of the built environment will be clarified in figure 7.1 on page 60 and corresponding explanatory text.

The problem of structural performance Present-day understanding of structural performance is characterised by a constant expansion of complex analysis tools, without an adequate control of the fundamental interface between applied mechanics and the material applications.

The problem of conceptual structural design Present-day conceptual structural design is characterised by a constant expansion of requirements, related interfaces, and collaboration models, without an adequate organisation of the brought about possibilities by controlling the fundamental interface between the body of knowledge of the built environment and the demands of the customer.

4.1.3 Structural engineering activities and missing tools

Professional structural engineering encompasses both analysis and design; where analysis is related to structural performance and design is related to requirements, conditions, and interfaces with the built environment.

An effective structural design process will be characterised by convergence and optimisation, based on a progressive insight into the behaviour of the structure, and an integral control of all influential boundary conditions.

The corresponding structural engineering activities and tools as listed in table 4.1 reveal an obvious deficiency with regard to conceptual structural design tools.

The interface between applied mechanics and material applications is underdeveloped and the interface between structural demand and the built environment is not yet developed.

4.1.4 Definition of structural performance

Structural performance and its difference from structural action, is clarified by the following definitions.

Structural engineering activities and tools		
Activity	Subject	Tool
Code checking	Static scheme: - Load distribution - Displacement SLS Code check: - Sectional strength - Stability Optimisation: - Trial and error	Finite Element Method Design codes of practice Computerised frame analysis and code checking
Structural analysis	Fundamental insight into structural performance: - Equilibrium - Force-driven parameters - Deformation-driven parameters - Durability driven parameters	Applied mechanics Background documents design codes of practice ***Interface between applied mechanics and material applications is underdeveloped***
Conceptual structural design	Fundamental overview of basic interfaces with the built environment: - Performance demand - Structural demand - Construction demand	***Interfaces between structural demand and performance and construction demand are not developed***

Table 4.1: Structural engineering activities and tools

Structural performance Structural performance is a collective term for the following structural requirements:

Structural safety - ULS The safety of a structure or structural member is prescribed by its Ultimate Limit State (ULS). If structural behaviour is beyond this undesired state, one of the following failure mechanisms may occur: collapse due to loss of equilibrium of the structure; fracture due to excessive internal stresses or fatigue; or instability due to insufficiency or unbalance of the ground.

Serviceability - SLS The functionality of a structure or structural member is prescribed by its Serviceability Limit State (SLS). Serviceability in structural design includes the following cases: deflection of a beam under loads; deformation like swing during vibration; or tilting of a column under external actions.

Durability - DLS The durability of a structure or structural member is prescribed by its Durability Limit State (DLS). The ability of structural members to perform adequately for normal use during the characteristic working life can among others be endangered by material deterioration that can within time decrease the strength capacity and serviceability of these members.

These specifically in the structural design codes of practice prescribed structural requirements ULS, SLS, and DLS, are part of a more overall sustainability objective to minimise the negative environmental impact by enhancing efficiency and moderation in the use of materials and energy.

Structural action Structural action can be characterised by the way in which a structure resists the loads acting on it - incorporating the load distribution within the structure; the corresponding deformation of the structure; and the strength of structural members to resist these loads.

The typifications "structural action" and "structural performance" show a high degree of similarity, except for the modern more specific contractual specifications with regard to functional behaviour and durability aspects. For this reason, both typifications are distinguished from each other, and the broader-ranging structural performance is therefore deliberately included in the problem definition.

When principally applied mechanics calculations are addressed, the typification "structural action" instead of "structural performance" will be used. Particularly, optimisation calculations with respect to load distribution, deformation, and strength will be addressed as an optimisation of the structural action.

4.1.5 Definition of conceptual structural design

Conceptual structural design and the identical integral design are clarified by the following definitions.

Conceptual structural design Conceptual design refers to the creation phase of the design of the built environment or objects within this built environment such as buildings and civil works. Particularly, this design phase is characterised by a multitude of co-operating disciplines and mutual interfaces.

Within this multi-disciplinary design phase, all disciplines in general are responsible for an overall optimisation of the performance/cost-ratio of the life cycle; the structural engineer in particular is responsible for a safe and efficient structural design within this overall optimisation.

With regard to a safe and efficient structural design, it should be emphasised that understanding structural performance is the starting point and an absolute prerequisite for conceptual structural design.

Integral design Integrality of design for the benefit of overall optimisation should ideally completely take place during conceptual design in general, and conceptual structural design in particular. In this textbook therefore, conceptual design is completely synonymous with integral design and "conceptual structural design" automatically implies full integrality of design.

4.2 Present-day solutions' field of practice

4.2.1 Copying reality

Most of the present-day solutions reducing failure and corresponding costs are aimed at a complete and an as-accurate-as-possible procedure capturing reality.

The most thorough procedure capturing reality is copying reality:

1. Qualify the complete set of structural engineering aspects, including all the related interfaces with the built environment.
2. Quantify all these aspects.
3. Describe all the interfaces between these quantified aspects with unambiguous processes.
4. Combine these processes in one converging flow diagram.

Most of the present-day solutions aim at such a copy of reality, as accurate as possible, often in one of the following forms:

Planning and control A data-driven approach concerned with planning and controlling all aspects of a process.

Numerical power Mostly in the form of advanced structural analysis applications or even a modest expert system, capturing an expert's knowledge by encoding it in a computer program.

4.2.2 Planning and control

The most usual present-day approach to reduce failure and corresponding costs is by establishing elaborate planning and corresponding control activities.

PDCA cycle The concept of planning and control is based on the scientific method of "hypothesis, experiment, and evaluation" or "plan, do, and check". Later developed into the present-day customary Plan, Do, Check, and Act (PDCA) cycle as an iterative four-step management method for the control and improvement of processes and products.

For an effective planning-control relationship, an accurate adjustment of the control activities to the planning is of importance:

- Establishing measurable standards together and corresponding with the objectives.
- An interaction between planning and control leading to changes occurs when taking corrective action with the final step of the control. This can take several forms, but two of the most effective are to change the objectives or alter the plan.
- A design plan must provide the framework for the design team control system. When objectives and plans change for whatever reason, control standards should change accordingly.

Systems engineering Systems engineering is an interdisciplinary approach and is used for enabling the realisation and deployment of successful systems. Systems engineering integrates other disciplines and specialty groups into a team effort, forming a structured development process that spans the whole system lifecycle.

Systems engineering became synonymous with the overarching responsibility for the development of the complete end product and enabling products. This role has increasingly expanded until the present; it now also being responsible for the interface between the complete device and the user, and even with the system's eventual disposal. Interface design and specification are concerned with assuring that the pieces of a system connect and interoperate with other parts of the system, and with external systems when necessary.

In general, systems engineering proceeds in the following steps:

1. Formalise the approach.
2. Control the overall system, regarding the entire life cycle, all interfaces included.

3. Apply explicit, clear, and provable selection processes.
4. Define interactions between requirements, objects, and organisations.

Building information modelling Building Information Modelling (BIM) is a digital representation of physical and functional characteristics of a facility. A building information model is a shared knowledge resource for information about a facility forming a reliable basis for decisions during its life cycle.

A basic premise of BIM is collaboration by different stakeholders at different phases of the life cycle of a facility to insert, extract, update, or modify information in the BIM to support and reflect the roles of that stakeholder.

For the professionals involved in a project, BIM enables a virtual information model to be handed from the design team to the main contractor and subcontractors and then on to the owner/operator; each professional adds discipline specific data to the single shared model. This reduces information losses, which traditionally occurred when a new team took "ownership" of the project, and provides more extensive information to owners of complex structures.

Traditional design was largely reliant upon two-dimensional (2-D) drawings. BIM extends this beyond the three-dimensional (3-D) physical geometry with time as the fourth dimension (4-D) and costs as the fifth (5-D), etc. BIM therefore covers more than just geometry. It also covers spatial relationships, light analysis, geographic information, and quantities and properties of building components.

4.2.3 Numerical power

Numerous sophisticated high-end automated design tools increasingly support daily practice of the present-day professional structural engineer.

Finite element method Present-day form-free complex architectural designs require an elaborate and profound structural analysis on a three-dimensional level. For a corresponding analysis on stress level, three-dimensional finite elements are applicable.

Advanced structural analysis may examine:

- Dynamic analysis; natural frequencies and frequency response.
- Geometric nonlinear analysis of second-order behaviour; linear stress-strain relationship and large displacements.
- Material nonlinear analysis of plastic load-carrying capacity; non-linear stress-strain relationship and small displacements.
- Induced deformation analysis of the structure, based upon geotechnical failure mechanisms and corresponding deformations.

Because of the present-day availability and self-evidence of these sophisticated high-end automated structural analysis tools, utilisation of these tools for even the simplest structural problem seems appealing.

Parametric design Parametric design is a type of rule-based modelling where geometric constraints and also scripting are used to ensure that the main objectives of the design intent within a project are preserved. It is about the use of variables and algorithms to generate a hierarchy of mathematical and geometric relations to explore the whole range of possible solutions that the variability of the initial parameters may allow. As a result, design teams are able to generate innovative forms.

Parametric design is not limited to only constraining geometry; it can also be used to define and constrict relationships such as thermal properties and material strength. With the coupling of finite elements, modelling material dimensions and even generative design with optimised organic structural load paths can be obtained.

Expert systems An expert system represents information and searches for patterns in that information. They are known as expert systems because they model how a human expert analyses a particular situation by applying rules to the facts, or compares the current case with similar cases, in order to reach a conclusion.

The underlying concept of an expert system is that it is possible via a series of carefully structured interviews, to capture an expert's knowledge and to encode it in a computer system in such a way that the system is able to mimic the decision-making behaviour of the expert.

Expert systems can include different types of reasoning such as case based, sequential based, rule based, and fuzzy logic.

Expert systems for conceptual structural design Computers do not possess common sense, so when an expert system is pushed outside the bounds of its knowledge, it has no way of judging whether or not something is sensible.

Much of the knowledge that an expert possesses is in a form that cannot be expressed clearly. Also the shear volume and depth of the information is such that it is just not possible to capture it all. Maintaining and updating knowledge bases is a demanding task that requires specialist staff.

In reality, matching a previous design to new design requirements is more complex than can be achieved by simple techniques. Also, automatically modifying the design proved to be a very difficult task.

4.2.4 Professional higher education

In the hands of experienced conceptual designers, present-day availability of sophisticated high-end automated structural analysis tools can contribute to conceptual structural design on a detailed scale such as an exploratory analysis of complex structural action.

In the hands of inexperienced young professionals however, the conceptual design capabilities of these sophisticated tools diminish rather than improve. Computerised designing with insufficient insight - particularly in the conceptual design phase - is a dangerous operation both from an economical and a safety point of view.

Due to the on-going expansion of high-end automated design tools, the simplification and decomposition techniques of the experienced structural engineer are steadily disappearing from practice training and higher education programmes.

University professors in the field are noticing a general erosion of knowledge and skills of applied mechanics, the mainstay of the structural engineering profession [23].

Furthermore, education about structural design in general, and conceptual structural design in particular, lacks an integral approach of the educational programme and corresponding emphasis on the interfaces between the disciplines.

4.3 Solution approach structural design

4.3.1 A need for control on system level

As with a lot of complex design problems, the standard planning and control mode is not enough to guaranty a satisfying solution as this formalistic, more bookkeeping-like approach only supports basic process control.

With regard to the quality of the design solution even planning and control with elaborate procedures and explicit supervision protocols - as proposed in the joint Compendium for a Structural Safety Strategy [24] - merely gives an illusion of control and reliability.

Furthermore, elaborate process control with an overkill of numerous regulations and control systems stifles creativity, progress and cooperation. Especially process control and reliability of a conceptual design - and in particular the preceding tender build-up - is endangered by a present-day tendency towards excessive numerous functional requirements and control procedures.

On the other hand, effective proven design tools such as systems engineering and BIM do offer control and clarity to open the way to creativity, progress and cooperation. Individual components of these applications, such as decomposition techniques and applied mechanics-based calculation routines can be very useful for a solution approach on conceptual structural design.

4.3.2 Shift from calculating to modelling

As a result of modern sophisticated automated design the field of practice shifts from calculating/checking to modelling/designing. This shift requires a fundamental understanding of modelling in combination with research skills:

1. **Modelling** of load distribution in complex structures and modelling of material behaviour of new structural materials, new applications of existing materials and new production techniques.
2. **Research skills** as an effective and efficient problem-solving tool for complex structural problems.

Partition of research skills For the professional field of practice, research skills can subsequently be divided into universal systematic thinking and applied systems thinking:

2a. **Systematic thinking** based on the scientific research method about problem definition, research framework, hypothesis, validation, and conclusion.

2b. **Systems thinking** as a holistic approach from the whole to the part and from coarse to fine, regarding complex interfaces, structural integrity, load distribution, and failure mechanisms.

4.3.3 Three pillars of future-proof structural design

In conclusion, the field of practice shift from calculating/checking to modelling/designing requires the following three pillars for a future proof structural design as listed in table 4.2: universal systematic thinking, applied systems thinking, and applied mechanics-based modelling.

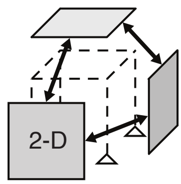
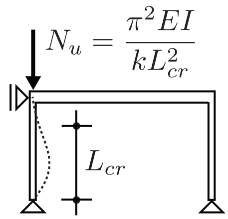
Systematic thinking	Systems thinking	Modelling
Scientific method: 1. Problem definition 2. Research framework 3. Hypothesis 4. Validation 5. Conclusion		

Table 4.2: Three pillars of future-proof structural design

Systematic thinking Scientific research has to comply with the basic principles of the scientific method. The essence of the scientific method is to test a hypothesis, and replication of this testing should get the same response; this response can be measured and recorded.

The following five steps can outline the scientific method:

1. State a problem and define a corresponding research question.
2. Investigate what is already known and structure the solution finding by means of a research framework.
3. Formulate a hypothesis as a solution to the problem.
4. Test the hypothesis and analyse the results on whether to accept, adjust, or reject the hypothesis.
5. Conclude, with recommendations for further research, and publish the results.

The underlying goal or purpose of science to society and individuals is to produce useful models of reality. To achieve this, one can form hypotheses based on observations of reality. By analysing a number of related hypotheses, scientists can form general theories. These theories benefit society and individuals who make use of them.

Systems thinking Systems thinking is about patterns and relationships to describe how things interact and gain insight into why systems behave the way they do. Systems thinking refers to various perspectives or interpretations of reality.

Present-day system theories development is aiming at tools and methods to better comprehend and manage the complexity of the total system life cycle. Modern developments include performance-based design, multidimensional modelling management, and quantitative risk management.

Because of the multitude of parameters and the complexity of interrelations, a workable and to optimisation leading numerical power-based overall design method seems far ahead. In the long run, however, the ultimate artificial intelligent expert system is surely a possibility.

The time gap between present-day needs and future successful applications of artificial intelligence will probably be large enough to excuse for a more simplified applied system thinking for conceptual structural design.

Applied system thinking for structural design can be based upon decomposition techniques for a holistic approach from the whole to the part and from coarse to fine, regarding complex interfaces, structural integrity, load distribution, and failure mechanisms.

Modelling Due to the complexity of material behaviour, structural analysis is completely dependent on abstract representations of the actual structure.

As an abstract representation, modelling has its limitations. For a reliable application of structural modelling, awareness of these limitations is of great importance; for example, when shear deformation is dominant.

To perform an accurate analysis, the structural engineer must determine information about structural loads, geometry, material properties and support conditions. The results of such an analysis typically include support reactions, member forces, and displacements. This information is then compared to criteria that indicate the conditions of failure.

Hand calculations of the structural action are based on analytical formulations that mostly apply to simple linear-elastic and ideal-plastic analysis models.

Computer calculations of the structural action are generally based on the finite element method, including the most commonly used displacement method. It is a numerical method generated by theories of mechanics and is applicable to structures of arbitrary size and complexity. The finite element method also helps in producing stiffness and strength visualisations.

Regardless of approach, the formulation is based on the same three fundamental relations of equilibrium, constitutive - stress-strain relationship, and compatibility - strength and stiffness transfer between elements. The solutions are approximate when any of these relations are only approximately satisfied, or an approximation of reality.

Chapter 5

Solution components

5.1 In search of a methodical approach

5.1.1 Solution approach

Insight into both structural performance and conceptual structural design is the primary requirement for a solution, based on universal fundamental understanding instead of specific applicability.

Present-day innovative accurate and integral design programmes such as expert systems are still small-scaled, insufficiently developed, and unreliable. The black-box character of such systems makes this lack of reliability worse.

So the methodical approach on conceptual structural design focusses on fundamental insight, by appointing and organising all factors that have been identified as important to the problem and to corresponding relationships.

5.1.2 Guiding principles

In response to the problem definition and subsequent analysis, guiding principles are searched for, in order to get hold of a coherent methodical approach. The following highly-correlated components substantiate the methodical approach:

T-shaped professional A balanced combination of in-depth understanding of structural performance and an in-breadth understanding of conceptual structural design, directly in line with the problem definition as expressed in figure 4.1 on page 26.

Applied mechanics Insight through simplification by way of an approximate mechanics-based modelling - the mainstay of the structural engineering profession - and corresponding conceptual structural design parameters.

Designing with progressive insight A need for understanding - from a safety and an economical point of view - based upon a controlled built-up of insight by working "from the whole to the part" and "from coarse to fine".

Decomposition Working "from the whole to the part" through a physical decomposition of the system and "from coarse to fine" through a process decomposition of the conceptual structural design process.

5.2 T-shaped professional

5.2.1 Modern demands

The problem of present-day collapses can be attributed to a lack of insight of the professional into structural performance and integral design as discussed in section 3.2.

The upcoming call for the return of the old-fashioned structural engineer can be part of the solution. After all, such an engineer has sufficient fundamental insight into structural performance to secure structural safety during conceptual design. Furthermore, this fundamental insight could prove to be an effective guide to acquire and utilise present-day complex software tools, codes, and research results.

However, the old-fashioned structural engineer would not be equipped for present-day and future performance-based integral design, with complex interfaces, contractual conditions, and collaboration processes. For an effective performance/cost optimisation with regard to service life and environment, one has to comply with modern demands.

5.2.2 T-shaped professional structural engineer

The problem definition of structural design as an interface control approach, as shown in figure 4.1 on page 26, represents these modern demands.

Therefore, within the integral Body Of Knowledge of the Built Environment (BOK BE), the present-day professional in structural engineering should possess a so-called “T-profile” as given in figure 5.1.

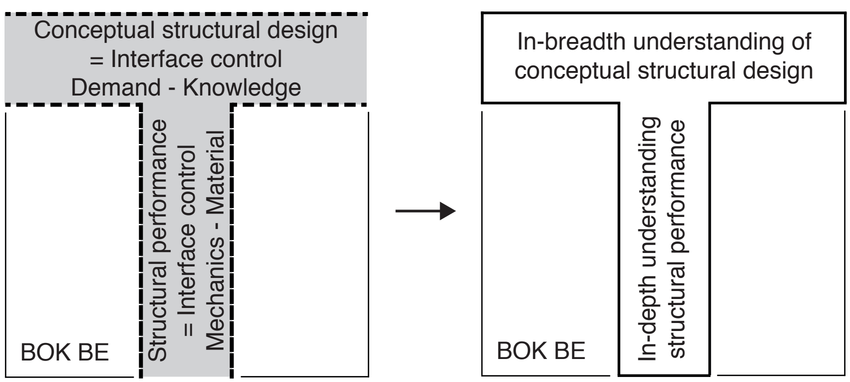


Figure 5.1: T-shaped professional structural engineer

This T-profile consists of sufficient in-depth understanding of structural performance to secure the structural safety, and sufficient in-breadth understanding of conceptual structural design to function effectively within an integral design team.

5.3 Applied mechanics

5.3.1 Necessity of insight into structural performance

Structural performance has to comply with structural safety, serviceability, durability, and sustainability over the life cycle of a structure as defined in subsection 4.1.4.

Non-conformities with respect to these structural requirements have to at the least be detected and corrected but preferably prevented by a preliminary insight into the structural performance.

Structural failure Every failure could lead to a catastrophic structural collapse. And therein lies the problem for structural engineers: engineering is the most unforgiving of professions.

Structural collapse is often a combination of causes and even if there is a single cause, it can lead to a progressive collapse. Usually, the causes can be attributed to human error: mistakes, misunderstandings, incompetence, ignorance, dishonesty; every facet of human failing is represented in construction failure.

Disaster is most likely when new designs are based purely on successful precedents and basic lessons of past failures are ignored, or not foreseen; for example the Tacoma Narrows Bridge. Other forces can be at work such as new materials, competitive pressure, and economical constraints.

Other non-conformities With structural safety being a self-evident prerequisite, serviceability is an upcoming contractual requirement to be met with regard to a controlled functional behaviour of the structure.

The further addition of durability as a contractual requirement is initiated through performance-based design as a logical step towards an integral performance/cost optimisation of the built environment of the entire life cycle. As a result, durability is extending to sustainability by minimising a negative environmental impact.

Necessity of insight Present-day sophisticated automated black-box design tools are an upcoming threat with respect to non-conformities. A black box permits no insight into the process; the quality of the process and the design outcome is completely dependent on the understanding and overall insight of the programmer and at that time current knowledge, practices, and perceptions.

Computerised designing with insufficient insight, particularly in the conceptual design phase, is a dangerous operation both from an economical and a safety point of view. Insight into the conceptual structural design process is a necessity for structural designers, who are directly responsible for the structural performance.

5.3.2 Timeless applied mechanics

Especially the lack of insight of the young professional during the design phase can be seen as a shortcoming of present-day professional education.

During the entire design process there must be a constant built-up of insight into the load distribution in combination with decisive failure mechanisms of the structure as a whole.

Insight into the load distribution and decisive failure mechanisms can primarily be based on a thorough knowledge and application of mechanics. The visibility of this application highly contributes to an insightful controlled structural design process during all design phases, but emphatically during the conceptual structural design.

Applied mechanics-based modelling is and will be the mainstay of the structural engineering profession.

In order to secure durability of the design method, this methodical approach focuses on an overall applicability for construction types, and the use of timeless elements and particularly applied mechanics.

5.4 Designing with progressive insight

5.4.1 General problem approach

Conceptual structural design is such a complex problem-solving process that it is in urgent need of increased accessibility to the field of practice.

In general, complex problems can be effectively approached on a relative high level of abstraction, with an elaborate knowledge of the fundamentals and a simultaneous working "from the whole to the part" and "from coarse to fine" as shown in figure 5.2.

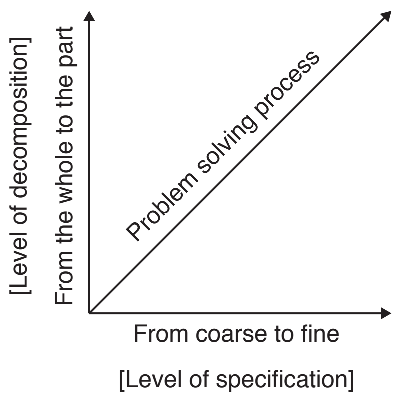


Figure 5.2: Problem approach

The vertical and horizontal axes can be formulated somewhat more abstract as "level of decomposition" respectively "level of specification".

5.4.2 Progressive insight from estimation to accuracy

An effective structural design process is characterised by convergence and optimisation, based on a progressive insight into the behaviour of the structure, and an integral control of all influential boundary conditions.

Working with an increasing number of related interfaces and using complex computational analysis and code checking asks for insight and oversight during all design phases.

The three phases of a global structural design process are designing the primary purpose of the structure, designing the basic structural concept, and the calculation process.

Progressive insight from estimation to accuracy on (sub)system level:

1. Determination of the structural form and the choice of material.
2. Approximate dimensioning by manual calculation of the load distribution and failure mechanisms with basic applied mechanics.
3. Computerised two-dimensional framework calculation of the load distribution and post-processed code checking.

For most buildings this will normally be sufficient, but high-rise buildings and civil structures may need a more thorough investigation:

4. Computerised three-dimensional framework calculation of the load distribution (three-dimensional torsion effects) and post-processed code checking.
5. Computerised Finite Elements calculation on stress level of the load distribution and post-processed code checking.
6. Computerised Finite Elements calculation on stress level for both load distribution and material strength, code checking is no longer applicable.

5.5 Decomposition

5.5.1 Abstractions of a decomposed system

Decomposition is a standard technique when dealing with complex systems and allows for certain abstractions. One way of decomposing may allow for natural and elegant abstraction in further system description, whereas other decompositions allow for less natural abstraction. So the decomposition has to be chosen with respect to the possible abstractions later.

Completeness When decomposing a system we need an argument that the composition of all decomposed components makes the whole system and its functionality complete again. For certain kinds of decompositions the completeness argument is easier than for others.

Loss of information When designing with abstractions of a decomposed system, the unavoidable loss of information to the system reality needs to be taken into account. Decomposing a system inevitably results in loss of information at the borders of the partitioned subsystems. Conscientiously chosen locations for the interfaces between subsystems - often a natural way of decomposing - will decrease insight and reduce loss, however.

Natural descriptions We use formal language to describe systems. This language can be based on different disciplines in the formal, mathematical world:

- There are logics, where we have basic properties and propositional and temporal operators to relate basic properties. Theorem provers are suitable tools that require a logic-based system description.
- There are process algebras, wherein processes and different ways of synchronisation between processes are the elementary bricks, and wherein model checkers are the tools corresponding to the process-algebraic way of system description.
- A physical system can also be described as a set of differential equations, and classical mathematics forms the basis for solving sets of differential equations and arguments about the functionality described.

Additionally, the composition mechanism that corresponds to the chosen decomposition is relevant. On the one hand, when this composition mechanism is reflected by a language primitive of the description language, the corresponding decomposition fits this language best. On the other hand, each decomposition corresponds to a most ideal way of system description.

5.5.2 Decomposition of complex systems

Particularly the complexity of the interdisciplinary interfaces makes both the design process and the overall behaviour of a structural system complex.

For an effective analysis of such a complex system, the following coherent decomposition approach can be applied as brought up by Kickert [13] and further substantiated by De Ridder [21]:

Physical decomposition Decomposing the physical parts of a system as the most natural way of decomposition.

Process decomposition Decomposing the process phases of a system such as design, construction, and operation.

Aspect decomposition Decomposing the aspect parts of a system such as strength, aesthetics, and durability.

5.5.3 Physical decomposition

This way of decomposition follows the physical parts of a system. Often, it is a very natural way of decomposition because we easily "see" all the physical parts.

The completeness criterion of physical parts is easy to check: when we have processed all physical parts, we have the whole system. Also, failure of physical parts can be located naturally and therefore described more straightforwardly.

The actual physical decomposition of a structural system into basic structural forms is further elaborated in chapter 9.

5.5.4 Process decomposition

When looking at chemical plants or production plants a recipe or a production plan forms the "essence" of their functionality. In this case, process decomposition is the most natural way and allows for the most effective abstractions.

The focus in the process decomposition lies in the form of causal chains that are relevant in the system to model.

The actual process decomposition of conceptual structural design into individual fundamental structural design phases is further elaborated in chapter 8.

5.5.5 Aspect decomposition

Particularly the interacting of the numerous aspect parts of the built environment such as functionality, costs, aesthetics, strength, redundancy, constructability, flexibility, durability, maintainability, and sustainability becomes complex when crossing disciplinary boundaries.

Although an integral conceptual design is a highly cyclic process, the complexity lies in the interdisciplinary aspects rather than the process itself. As a result of this complexity of the interdisciplinary interfaces, the loss of information at the borders of the partitioned disciplines will be unacceptably large and decomposition cannot be applied effectively.

Then concurrent engineering of all system theories in general, and decomposition-based methods in particular, will be an appropriate solution. Utilisation and corresponding preconditions is further elaborated in section 7.3.

Concurrent engineering Concurrent engineering is a systematic approach to integrated product development that emphasises the response to customer expectations. It embodies team values of co-operation, trust and sharing in such a manner that decision-making is by consensus, involving all perspectives in parallel, from the beginning of the product life cycle.

Essentially, concurrent engineering provides a collaborative, co-operative, collective and simultaneous engineering working environment. The concurrent engineering approach is based on the five key elements of process, multidisciplinary team, integrated design model, facility, and software infrastructure.

Concurrent engineering is recognised as a strategic weapon that businesses must use for effective and efficient product development. It is not a trivial task, but a complex strategic plan that demands full corporate commitment, and therefore strong leadership and teamwork go hand in hand with successful concurrent engineering programmes.

Part II

The art of conceptual structural design

Chapter 6

Introduction to part II

Both principal and design team are in search of an overall performance/cost optimisation. A lack of insight of the professional into structural engineering is considered to be a main obstacle for such an optimisation.

For this relatively unexplored problem, an exploratory research is conducted by a systematically zooming in from the whole to the part and from coarse to fine. The so explored methodical approach on conceptual structural design consists of a process control component and an underlying structural performance component. The process control component is discussed in this Part II, “The art of conceptual structural design”. The underlying structural performance component is discussed in the following Part III, “Understanding structural performance”.

Part II starts with present-day complex and ambiguous conceptual structural design.

Then the conceptual structural design process is subdivided into the individual components process decomposition, physical decomposition, and a cyclic control of the process, as shown on the axes in figure 6.1.

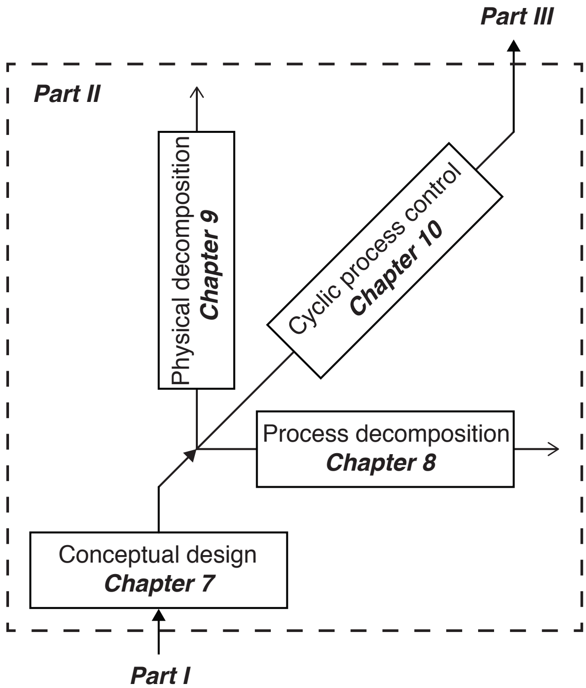


Figure 6.1: Reading guide for part II

Conceptual design Conceptual structural design is a creative and dimensioning process in which the structural form, the materials and the basic dimensions are determined, taking into account all influential aspects such as aesthetics, constructability, sustainability, and costs.

Until the present day, it appears challenging to capture performance-based solutions out of the multitude of aspects and their complex interrelations. Modelling these interrelations is capturing intuition; the most ambiguous, and therefore the most intangible aspect of conceptual design.

A definition and collection of the fundamental conceptual design parameters of the most influential participating disciplines can serve as a joint breeding ground for a concurrent "shared knowledge-based conceptual design".

For a controlled conceptual design process, both process and physical decomposition techniques are inevitable.

Process decomposition Capturing structural design by breaking down this complex process into the essential absolute minimum, resulting in a "structural design cycle" as an effective characterisation of both the overall design process and the individual design phases.

Physical decomposition A structure as a three-dimensional physical system can be decomposed in clear two-dimensional "basic structural forms", with each individual structural form being an assembly of directly connected one-dimensional structural elements, designed to act together to resist loads.

Cyclic process control A complete control of the present-day complex and highly cyclic conceptual design process is still out of reach. Success in this design phase is dependent on the level of experience and intuition of the conceptual structural designer.

Structural design can be explored on a "structural design path", following the fundamental dimensioning routine from structural integrity, via load distribution, to failure mechanisms.

A cyclic optimum design with regard to optimisation of both the quality of design outcome, and the number of design cycles can be supported by "structural design loops" such as analysis, check, orientation, and correction loops.

Solution components The so-obtained coherent set of solution components for conceptual structural design, and corresponding chapter arrangement, is listed in table 6.1.

Chapter arrangement and solution components		
No.	Chapter	Solution component
7	Conceptual design	- Shared knowledge-based conceptual design
8	Process decomposition	- Structural design cycle
9	Physical decomposition	- Basic structural forms
10	Cyclic process control	- Structural design path - Structural design loops

Table 6.1: Solution components for conceptual structural design

Chapter 7

Conceptual design

7.1 Determination of the structural form

7.1.1 Conceptual structural design

A conceptual design team consists of a representative of the principal and representatives of each discipline with a major design involvement with regard to feasibility and the performance/cost ratio of the specific required system.

After a reduction of the mostly numerous requirements and contractual conditions into a controllable set of presumably dominant requirements and conditions, the first conceptual draft on system level can be executed. Only on this three-dimensional level an integral design is feasible. The process of creating system outlines by the conceptual design team is shown in figure 7.1.

Customary, this creation is a composition of basic structural forms on object level. More innovative structural design, however, requires a system creation on aspect level. On such an aspect level, a set of functional requirements is directing the design of possible load paths instead of just combining basic forms.

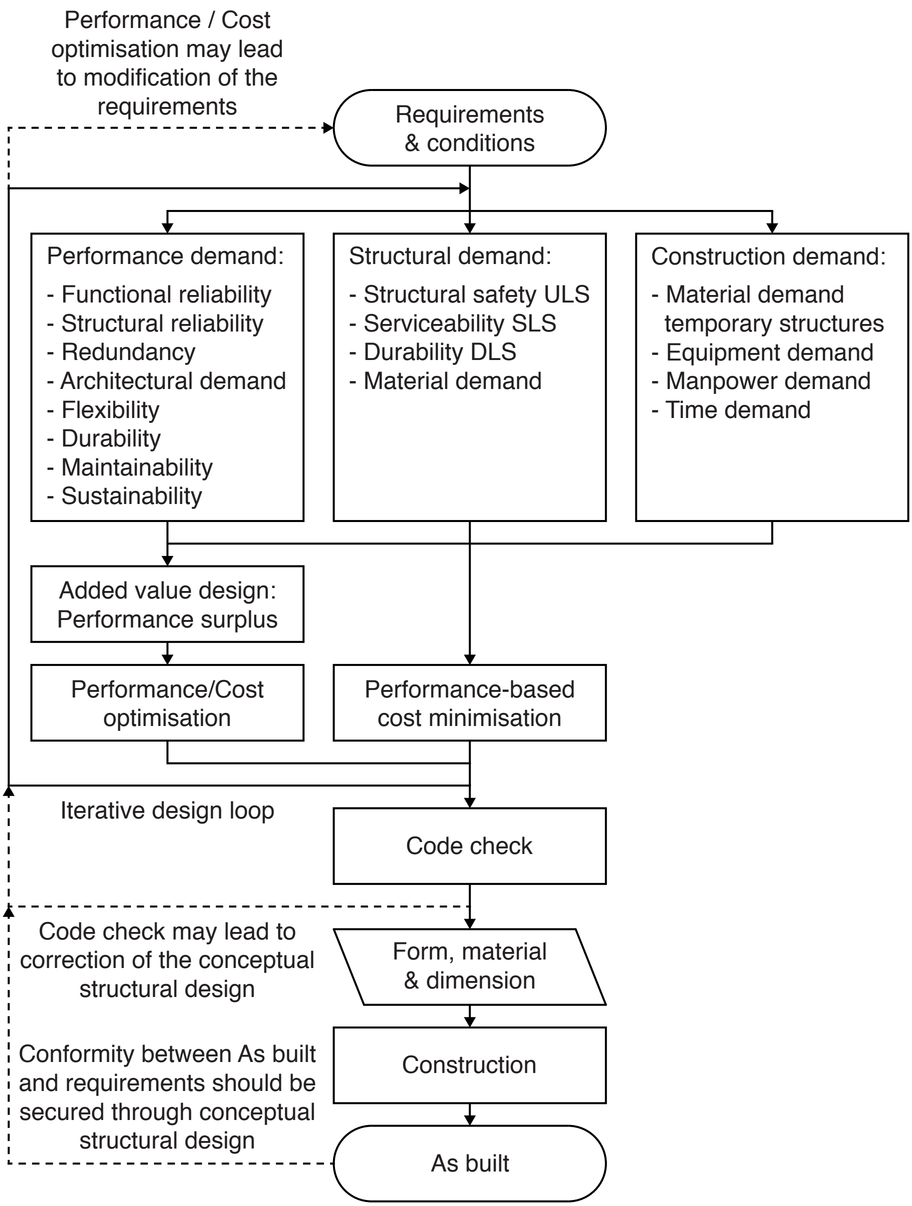


Figure 7.1: Conceptual structural design process

For a coherent and complete integral design all fundamental demands, namely performance demand, structural demand, and construction demand, have to be complied with. Moreover, these three demands are, one by one, indispensable:

Performance demand A system derives its reason for existence from its function. Principally, performance demand has to secure this function by means of explicit reliability demands. Furthermore, architectural expression, future customisation, service life, and environmental sustainability can be demanded.

Structural demand The safety and serviceability of the structure are inevitably conditional for a safe and functional use of the system. Failing structural safety has proved to cause serious civil disturbances.

Construction demand Design engineers optimise cost drivers of the scheme - such as simplicity, uniformity, repetition, and phasing - to be executed with respect to manpower, equipment, and material demand. The cost optimisation of the execution is implicitly part of the life cycle costs and is, among other things, dependent on location and the individual contractor. Constructability in itself, and corresponding construction time are inevitably conditional for coming into existence of the system.

The quality of the system can best be valued by its performance/cost ratio; with a performance in the broadest sense, including structural and construction demands. Due to the large amount of design freedom, the performance/cost ratio can best be established and optimised in the creation phase. In the process of further specification, an optimisation of the performance/cost ratio will turn out to be increasingly limited.

The system outlines that only comply with the requirements are so-called "performance-based" solutions. On the basis of these requirements, striving for full optimisation of the performance/cost ratio will lead to a performance-based cost minimisation.

However, such a minimisation is not automatically the ultimate goal of a principal and the design team; sometimes, supplementary added-value solutions offer a considerable added-value for a relatively small cost increase. Then, a real performance/cost optimisation, despite the earlier established requirements, is preferable.

7.1.2 Design process

Conceptual structural design is a creative and dimensioning process, in which the structural form, the materials, and the basic dimensions are determined. The structural engineer should be involved in the project from the start of the conceptual design phase because of the influence of complex aspects such as functionality, costs, aesthetics, constructability, and sustainability.

The structural form One of the main conceptual structural design activities is the determination of the structural form, based on "understanding" and "order of magnitude" of standard structural forms, fundamental insight into structural behaviour, and an overview of the basic interfaces with the built environment.

Materialisation The choice of material is dependent on a combination of structural, construction, and architectural demand parameters; structural demand with respect to reliability, redundancy, and durability; construction demand parameters for both permanent and temporary structures such as construction time, mass, simplicity, and uniformity; and finally, architectural demand, which can span a wide spectrum of expressions from power and massiveness to minimalism and transparency.

Dimensioning Dimensioning of a materialised structural form is a quantification process. The load distribution on the system, through the subsystems into the elements, is one of the flow parameters. The second parameter in a reversed flow is the determination of the approximated capacity of the materialised elements, the subsystems, and the system.

The starting point in this process is usually the adoption of dimensions derived from similar projects. Final dimensioning is obtained through a "trial and error" procedure involving repeated analysis of the structure. Only in some uncomplicated cases this process can be rationalised using an algorithm flow diagram.

7.1.3 Flow diagram

The framework of procedures in conceptual structural design can be modelled by flow diagrams, giving an unambiguous solution.

Flow diagrams can support structured analysis and design, by showing the flow of data from external entities into the system, and how the data move from one process to another, as well as its logical storage. Within processes, optimisation loops can be implemented. There are common modelling rules creating flow diagrams:

- All processes must have at least one data flow in and one data flow out.
- All processes should modify the incoming data, producing new forms of outgoing data.
- Each data store must be involved with at least one data flow.
- Each external entity must be involved with at least one data flow.
- A data flow must be attached to at least one process.

All influential aspects are known in conceptual structural design, the basic interfaces with the built environment included. The interrelation and interaction between all these aspects, however, is so complicated that besides experience, intuition has to be used frequently.

Using a flow diagram is a way of making effective choices out of all we already know. In doing so, throwing away aspects that cannot be captured in this way - such as the ambiguous but vital intuition - is unavoidable.

7.1.4 Experience and intuition

In many cases the interacting of the numerous aspects of the built environment such as functionality, costs, aesthetics, strength, redundancy, constructability, flexibility, durability, maintainability, and sustainability becomes complex when crossing disciplinary boundaries.

Then a lot of experience is advisable, and furthermore and more intangibly, intuition becomes a necessity as stated by famous engineers:

Eduardo Torroja In his book "Philosophy of Structures" [27]: "The achievement of the final solution is largely a matter of habit, intuition, imagination, common sense, and personal attitude. Only the accumulation of experience can shorten the necessary labour or trial and error involved in the selection of one among the different possible alternatives." "The calculation of stresses can only serve to check and to correct the sizes of structural members as conceived and proposed by the intuition of the designer. The work itself is never born from calculation."

Pier Luigi Nervi In his book "Structures" [20]: "It is highly regrettable that some of the highest qualities of the human mind, such as intuition and direct apprehension, have been banned from our schools and have been overwhelmed by abstract and impersonal mathematical formulas... The essential part of the design of a building consists in conceiving and proportioning its structural system... then and only then we can and we should apply the formulas of mathematical theory of elasticity to specify with greater accuracy its resisting elements."

Modern structural engineers can rely on a long history of constant validation of theoretical approaches, building up an immense database of knowledge.

Until the present day, it appears challenging to capture performance-based solutions out of the multitude of aspects and their complex interrelations. The unsolved topic in modelling these interrelations is capturing intuition, the most ambiguous, and therefore the most intangible aspect of conceptual design.

Furthermore, some unambiguous structural aspects such as serial effects, induced deformation, and the stability of the equilibrium are difficult to insert in a linear optimisation process. At a certain level, decomposition and a linear approach to modelling the design process is no longer feasible.

Consequently, the determination of the structural form is partly a qualification process. The field of application, and the characteristics of the structural forms with respect to the individual relations with the built environment, are the variables in this partly unambiguous and partly ambiguous process.

The ambiguous part is often specified as experience and intuition. The most tangible of the two - experience - consists of conscious and subconscious knowledge.

It is a reasonable assumption that intuition is nothing more than the subconscious part of experience. After all, there is a noticeable difference between the intuitive capacities of a young professional and an experienced engineer. This difference is contradictory to the sometimes-suggested idea of a cosmic consciousness.

It is worthwhile revealing the experienced-based subconscious knowledge, keeping in mind never to take something for granted.

7.2 How to capture the intangible

7.2.1 Abstracting conceptual design

In general, communication can be divided in three basic levels of abstraction: object, experience, and concept. Going up the levels of abstraction, ideas increase while reality decreases:

At the object level Communication is about tangible material or an unambiguous representation of it.

At the experience level Communication is about (common) experience. Although reality is the point of departure, it has the abstraction of interpretation.

At the concept level Communication is about ideas and thoughts. Concepts can be accepted or rejected.

Considering the levels of abstraction in conceptual structural design, the basic levels of communication are applicable. The experience level in engineering practice, however, is mainly restricted to physical behaviour, described with physical laws.

These laws are scientific generalisations that have become accepted universally within the scientific community. Describing the observable laws of nature is based on observations and repeated scientific experiments.

Some extremely important physical laws are simply definitions of the observable laws of nature, such as the mathematical definition of force by Newton's second law of mechanics. Some laws are only approximations of other more general laws, with a restricted domain of applicability.

Modelling Due to the complexity of load distribution and above all material behaviour, structural analysis is completely dependent on abstract representations of the actual structure.

As an abstract representation, modelling has its limitations. For a reliable application of structural modelling, awareness of these limitations is of great importance, for example when shear deformation is dominant.

7.2.2 Directing parameters of conceptual design

During the conceptual design phase, the individual participants of the integral design process such as the structural engineer, the architect, and the contractor, measure, so to speak, with different scales:

Form The structural engineer designs in terms of materialised structural forms, static schemes, and dimensioning processes; in brief "form".

Concept The architect designs in terms of architectural concepts, design philosophy, space, and expression; in brief "concept".

Process The contractor designs in terms of building techniques, planning, phasing, and repetition; in brief "process".

Dependent on the discipline, the parameters "form", "concept", and "process" give direction to the design solution as shown in figure 7.2.

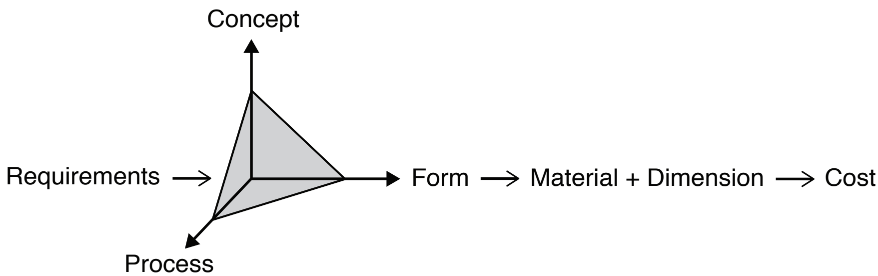


Figure 7.2: Directing parameters of conceptual design

Present-day misunderstanding between professional disciplines within an integral design process can partly be ascribed to these almost perpendicular oriented areas of attention.

Participation in this process requires a certain level of abstraction. Particularly the architect is used to abstraction, considering the nature of architectural designing. On the contrary, both the structural engineer and the contractor have to deal with a lot of down-to-earth activities in the field of code checking and execution. For an effective cooperation, it is of the utmost importance that every discipline contributes on the required conceptual design level of abstraction.

7.2.3 Sharing the knowledge of the built environment

For an optimisation of the performance/cost ratio over the life cycle of a structure, an integral approach and control of all participating influential disciplines is an absolute necessity. In many cases, the interacting of the numerous aspects of the built environment is so complex that an unambiguous flow diagram cannot be applied. Then, a lot of experience and intuition becomes advisable.

In consequence of gathering the required information for both the consciousness and sub-consciousness of participants of conceptual design, an accessible knowledge base is to be researched. For this purpose, the immense complete body of knowledge of the built environment is totally unsuitable and consequently, resulted in the present-day numerous disciplines and specialisations.

Nevertheless, the ability to have insight in multiple fields of discipline is the most effective way of crossing borders between disciplines and thus more effectively handles the design interfaces between these disciplines. It is of importance to determine and bring together the required fundamental knowledge of these disciplines as an effective basis for conceptual design.

A mutual knowledge base is possibly feasible, determining the absolute required minimum:

1. Sharing the knowledge of the built environment on an approximation level suitable for conceptual design;
2. and restricted to only the knowledge of the disciplines involved with a major interface during conceptual design.

7.2.4 Back to the fundamentals of conceptual design

The required T-profile of the professional structural engineer as shown in figure 5.1 on page 43, can be refined by a visualisation of the overlap between the in-depth and the in-breadth understanding. This overlap, being the conceptual design parameters, is visualised in figure 7.3.

Conceptual design parameters The conceptual design parameters are the specific part of the in-depth understanding that contributes to the conceptual design. These conceptual design parameters have to encompass all fundamental aspects of structural engineering. After all, the

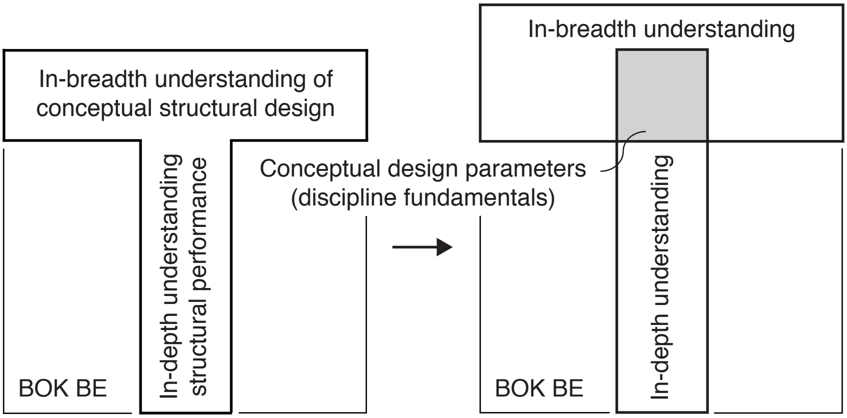


Figure 7.3: T-shaped professional for conceptual design

inherent quality of a design is established during conceptual design, taking into account all influential interfaces with the other disciplines.

Although subsequently a lot of optimisation, detailing, and verification has to be carried out, conceptual design and corresponding approximate parameters establish, so to say, the "DNA" of the final design.

Discipline fundamentals Therefore, the conceptual design parameters represent the fundamentals of the discipline, in this specific case the professional field of structural engineering. In view of the nature of conceptual design, this generalisation is applicable to all disciplines within the built environment.

7.3 Shared knowledge-based conceptual design

7.3.1 Splitting process and technical breadth

For a better understanding of the typical conceptual design activities, the in-breath understanding of conceptual design can furthermore be divided into a technical breadth of the built environment and the integral process control of conceptual design as shown in figure 7.4.

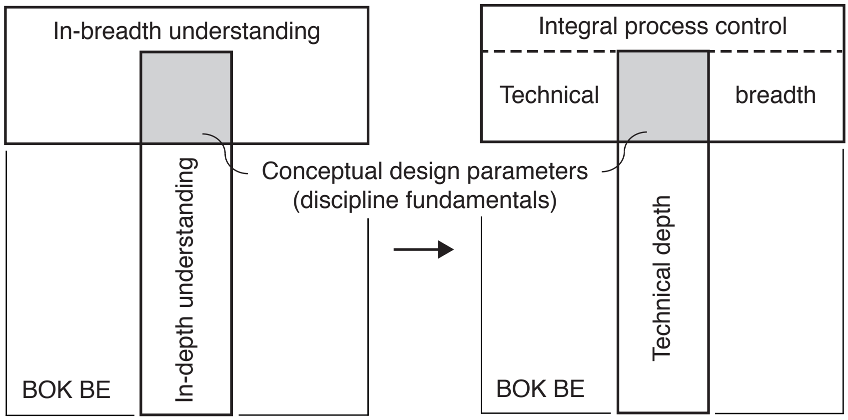


Figure 7.4: T-shaped professional technical breadth

Integral process control The ability to work in a multi-disciplinary manner with the profile to be "integrator" - besides the standard required process control abilities with respect to problem solving, project management, and self-management - includes the following:

- Functional requirements as starting point for conceptual design.
- Decomposition techniques as used in systems engineering.
- Collaboration models such as concurrent engineering and building information modelling.

Technical breadth The combined technical breadth and depth of the Built Environment (BE) encompasses the complete Body Of Knowledge (BOK) as indicated in figure 7.4 with the abbreviation "BOK BE", applicable to all design phases of conceptual, basic, and detailed design.

A part of this body of knowledge, indicated in figure 7.4 as "Technical breadth", refers only to the conceptual design phase. On the level of conceptual design, the technical breadth of the built environment specifically encompasses a fundamental understanding and application of the conceptual design parameters of all disciplines.

This understanding and application includes an understanding and control of the relevant interfaces between the individual disciplines with attention to constructability, safety, and durability.

However, due to the complexity of present-day design and the limited extrapolation capacities of previous designs, the consequences of design decisions in the early phase of conceptual design are difficult to oversee.

This underlines both the necessity to have insight in multiple fields of discipline, and to develop numerous concepts in order to bring them to a lower degree of complexity as further elaborated in section 10.4.

Technical depth structural engineering The technical depth equals the in-depth understanding of structural performance. This depth encompasses the complete body of knowledge of professional structural engineering as practiced during all design phases of conceptual, basic, and detailed design.

7.3.2 Principal disciplines of the built environment

For an integral conceptual design, the main contributing, and thus principal disciplines and corresponding interfaces, have to be considered. With respect to the quantity and character of the interfaces between the participating disciplines within a conceptual design team, a distinction between environment and object level as shown in figure 7.5 is appropriate.

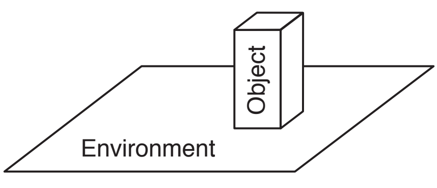


Figure 7.5: Environment and object level

Besides this distinction between environment and object level, an effective classification of principal disciplines can be based upon obvious requirements such as value, functionality, safety, aesthetics, and realisability. The so-classified principal disciplines for the built environment are listed in table 7.1.

Principal disciplines of the built environment		
Requirements	Environment level	Object level
Value	Property Development	Property Management
Functionality	Transportation Engineering	Installation Engineering
Safety	Hydraulic Engineering	Structural Engineering
Aesthetics	Urban Planning	Architectural Engineering
Realisability	Environmental Engineering	Construction Engineering

Table 7.1: Principal disciplines of the built environment

7.3.3 Concurrent-based shared knowledge

On object level, the interfaces of the object-related structural engineering with the other object-related disciplines are within the system functionality of the object and are further substantiated as follows:

Property management The performance/cost ratio is the main driver for the overall optimisation of conceptual design. Regarding life cycle costs, maintenance, and management are gaining importance, and are therefore substantial input for conceptual design.

Installation engineering Particularly, the main ducts of air conditioning systems have the same scale as girders and are preferably designed parallelly. Smaller scaled installations normally have minor to no influence at all on structural dimensioning.

Architectural engineering Architecture is a conscious creation of utilitarian space and construction of materials in such a way that the whole is both technically and aesthetically satisfying. Creation of utilitarian space with materialised forms is a main influential design interface with the structural form.

Construction engineering The practical feasibility of the execution focuses on avoiding unnecessary complexity, on influences on dimensions and tolerances, and on possible choices between alternatives.

On environmental level, the interfaces with the object-related structural engineering mainly consists of geometrical and loading constraints such as free space profiles, road cross sections, traffic loads, and hydraulic loads.

So for the determination of the fundamental shared knowledge with respect to conceptual structural design, the emphasis is on the object-related disciplines.

Because of the complexity of the interdisciplinary interfaces between these object-related disciplines, concurrent engineering will be an appropriate solution as discussed in subsection 5.5.5 of the solution components.

After all, the concurrent engineering approach provides a collaborative, co-operative, collective and simultaneous engineering working environment, based on the five key elements of process, multidisciplinary team, integrated design model, facility and software infrastructure.

7.3.4 Conceptual structural design parameters

Both for understanding structural performance and sharing our fundamental knowledge with the other directly related interface disciplines, a set of conceptual structural design parameters has to be established.

For professional Structural Engineering (SE), the applied mechanics-based conceptual structural design parameters can be split into the structural integrity on system level via the load distribution on subsystem level to the failure mechanisms on element level, as shown in figure 7.6.

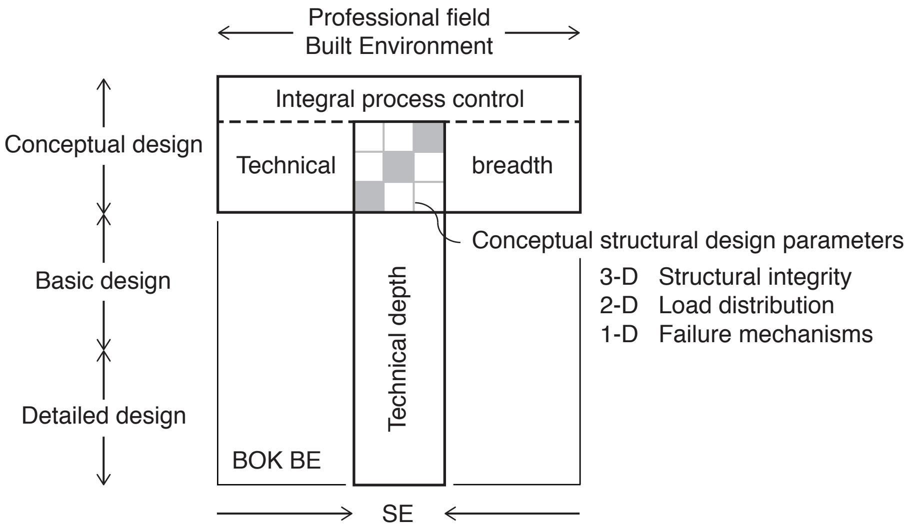


Figure 7.6: T-shaped conceptual structural design

Structural integrity The system as a whole can only be captured on the corresponding three-dimensional system level. A typical three-dimensional system effect such as overall torsion has to be incorporated here. Insight into and control of the load distribution within the three-dimensional system is merely feasible with axial forces on the level of load paths.

For a more accurate insight into the structural action and corresponding analysis of load distribution and dimensioning, the three-dimensional system is too complex and has to be decomposed into more accessible two-dimensional subsystems.

Load distribution On the two-dimensional subsystem level, a more accurate load distribution can be executed, including insight into optimisation, redistribution and possibly induced deformation of statically indeterminate structures. Consequently, a distribution of the prime actions can be executed to determine the individual element forces and corresponding required capacity.

Failure mechanisms The failure mechanisms on a one-dimensional element level represent the ability of materialised elements or cross sections to resist the distributed loads per element. These failure mechanisms are dependent on resistance of materials, resistance of cross sections, and stability of elements.

The deformation of the individual, materialised elements culminates in a deformation and corresponding displacements of the subsystems and subsequently, the whole system.

Chapter 8

Process decomposition

8.1 Fundamental design cycle

8.1.1 Structural life cycle phases

In the life cycle of a structure, the following phases and corresponding structural engineering activities can be chronologically distinguished as listed in table 8.1: initiation, design and specification, construction, operation and maintenance, and demolition.

8.1.2 Structural design phases

Out of the functional requirements and boundary conditions, a system is created and the performance/cost ratio is optimised, taking into account the interfaces with the built environment. Specifications for construction are prepared. This process can be divided into three major design phases with increasing accuracy, as listed in table 8.2: conceptual design, basic design, and detailed design.

Conceptual structural design The conceptual design phase is of special interest with respect to the problem definition as identified in section 4.1. The starting point in this phase is usually the adoption

Life cycle phases of a structure	
Life cycle phases	Structural engineering activities
Initiation	User needs are secured by a set of functional requirements, which define what the system is ultimately supposed to do. At this level of abstraction, structural requirements are not directly referred to.
Design and specification	Out of the functional requirements, a system is created and the performance/cost ratio is optimised, taking into account the interfaces with the built environment. Specifications for construction are prepared.
Construction	Feasibility and cost optimisation of construction is secured by design, including temporary structures. Structural site engineers operate on the level of postponed design activities, including corrective actions in case of non-conformities.
Operation and maintenance	After a maintenance or flexibility demand, structural engineers design the reconditioning or rebuilding of the structure or parts of it, and again operate on the level of design activities.
Demolition	A controlled structural collapse is based upon a structural action analysis and structural engineers operate on the level of partial design activities.

Table 8.1: Life cycle phases

of conceptual structural design solutions derived from similar projects. Any new type of structure, however, requires an extended lead-time to obtain a thorough understanding of the structural action.

It is not expected that corresponding conceptual design calculations and estimated costs be precise, but rather within accepted tolerance. The level of acceptance has at least to be qualified, and preferably be quan-

Design phases of a structure	
Design phases	Structural engineering activities
Conceptual design	The purposes of conceptual design of any structure are to obtain a clear picture of the structural action, approximated dimensions of the structure, principal details, quantities of materials for making estimates of costs, and a reliable prediction of the basic and detailed design.
Basic design	The purposes of basic design are to obtain exact dimensions of the structure and approximated dimensions of the details.
Detailed design	The purposes of detailed design are to obtain exact dimensions of the details and specifications for construction.

Table 8.2: Design phases

tified, with a risk analysis of the costs of the conceptual design.

8.1.3 Cyclic design process

The immense number of individual parameters and mutual interactions cannot be unambiguously controlled and will inevitably lead to a highly cyclic process as shown in figure 8.1.

In the process of materialisation from requirements to construction, ongoing design and check activities can influence foregoing activities with regard to choice of geometry, material, and matching dimensions. In an effective converging process, however, the number of optimisation loops will diminish during the process of further specification.

Both principal and design team are in search of an overall performance/cost optimisation. In some cases, this performance/cost optimisation may even lead to a modification of the requirements.

The conformity between the as-built situation after completion of the

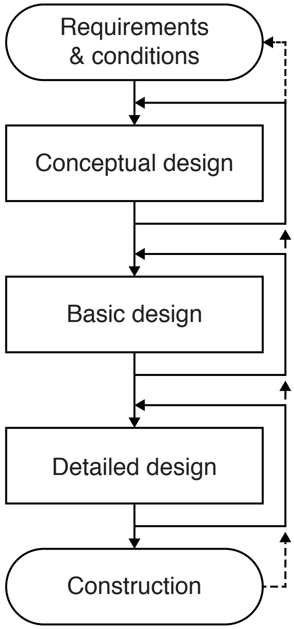


Figure 8.1: Structural design process

construction and the requirements should be secured through conceptual design.

8.1.4 Fundamental structural design cycle

The objective is capturing structural design by breaking down this complex and unambiguous process to the essential absolute minimum, resulting in a fundamental design cycle as an effective characterisation of both the design process as a whole, and the individual design phases.

The major contribution of professional structural engineering to the overall life cycle of a structure is on the level of design activities as clarified in table 8.1 on page 78.

Each individual design activity starts with requirements as to what the (part of the) structure is supposed to do and ends with a specification for construction. Independent of the life cycle phase, complexity of design, and contractual commitments, structural engineering practice can be outlined by the fundamental structural design cycle as shown in figure 8.2.

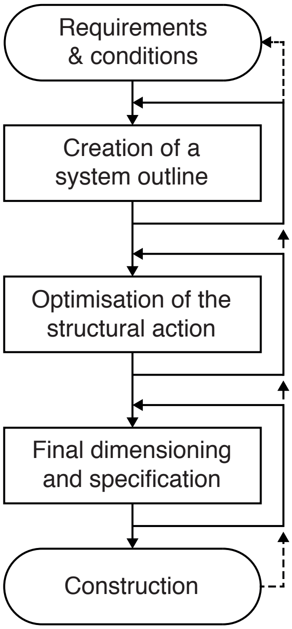


Figure 8.2: Fundamental structural design cycle

Creation of a system outline The most directing functional requirements and contractual conditions are interpreted into an overall geometry, choice of materials, and approximate material dimensions. All influential interfaces with other disciplines are taken into account and overall performance/cost is optimised.

Optimisation of the structural action Within the boundaries of the overall integral design, a more thorough analysis of the structural action leads to optimisation of this structural action and a further accuracy of the material dimensions. An overall structural system check and compliance with all functional requirements and contractual conditions is carried out.

Final dimensioning and specification Final material and detail dimensioning is determined on the basis of code checks with respect to structural safety, serviceability, and durability. Specification for construction is prepared.

8.2 Level of accuracy

8.2.1 Level of specification

To meet a client's demand, functional requirements will specify the starting point for the design process. Functional requirements, with function as a higher order, can lead to form and material. Functional requirements are decisive with respect to the quality of a durable structure, boundary conditions included, and therefore fixed.

The process from functional requirements to as-built can be divided into identifiable phases as shown in figure 8.3: conceptual design, basic design, detailed design, and construction.

Time schedule The time schedule on the horizontal axis consists of the three major design phases and the subsequent construction phase. These major design phases - conceptual, basic, and detailed design - can be substituted by the fundamental structural design cycle-based phases "exploration & creation", "selection & optimisation", and "verification & specification".

Level of reliable specification The corresponding achieved level of specification per individual design phase is strongly dependent on the time schedule. Particularly, in the conceptual design phase during exploration and creation most of the design is specified. This phase is decisive for both quality of design and a prosperous completion of subsequent phases.

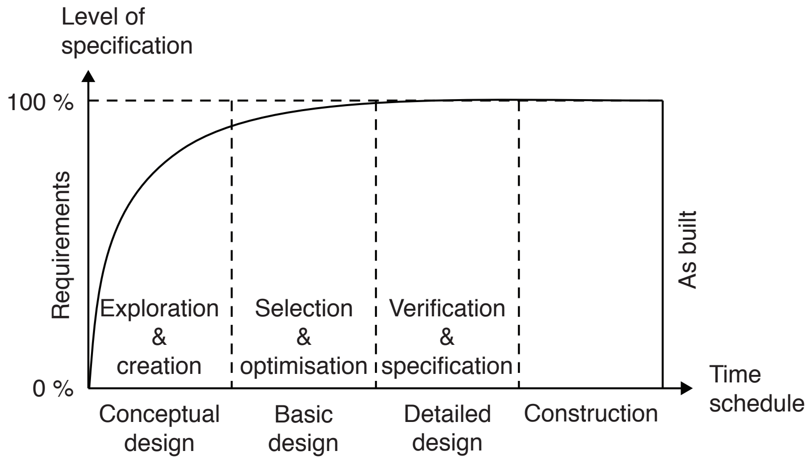


Figure 8.3: Level of specification

For an effective decision supporting, the reliability of each design phase outcome requires a corresponding accuracy. A feasibility study of this accuracy per design phase is provided by the International Federation for Structural Concrete.

8.2.2 Fib Model Code 2010

The International Federation for Structural Concrete fib, "fédération internationale du béton", is a pre-normative organisation. "Pre-normative" implies pioneering work in codification. This work has been realised in 2012 with the fib Model Code 2010 [8].

The fib Model Code 2010 includes the whole life cycle of a concrete structure, from design and construction to conservation and dismantlement, in one code for buildings, bridges, and other civil engineering structures.

In this code, subsection 3.5.3, "Quality Management in Design", a serious study is reported on the feasible accuracy of individual design phases. These individual phases, compatible with the decision process employed by the owner, are listed in table 8.3.

Fib Model Code 2010 design phases of a structure	
Design phases	Structural engineering activities
Scouting	Initial feasibility scan of the scheme, global functional requirements are specified. Target accuracy project costs estimation ±30%.
Basis of design	Functional requirements, data, and design criteria will be developed, service criteria agreed. Target accuracy project costs estimation ±20%.
Project specification	Further development into a preliminary design, evaluation alternative structural concepts. Target accuracy project costs estimation ±10%.
Final design	All primary structural members will be specified and typical details will be designed. Target accuracy project costs estimation ±5%.
Detailed design	The output of this phase will allow construction of the project.

Table 8.3: Fib design phases

Scouting phase The "Scouting Phase" is an initial feasibility scan of the scheme. From an abstract level of perception, global functional requirements are specified. In common practice, the design effort expenses will then be limited, because the feasibility of the project will usually still be uncertain. At this stage, the target accuracy for the estimate of overall project costs might typically be ±30%.

Basis of Design phase During this phase, the functional requirements, basic data, and design criteria will be developed and the service criteria agreed upon. A conceptual design will also be developed to support a more accurate budget estimate. Quite some effort is required as the "Basis of Design" should be agreed upon, fixed and frozen upon completion. At this stage, the target accuracy for the estimate of overall project costs might typically be ±20%.

Project Specification phase With the "Basis of Design" as starting point, the design will be developed into a preliminary design. Alternative structural concepts will generally be developed and evaluated against each other. Specifications for workmanship, materials, and detailed design will then be developed. Significant effort is generally required. At this stage, the target accuracy for the estimate of overall project costs might typically be ±10%.

Final Design phase During this phase, all primary structural members will be specified and typical details will be designed. The structural analysis should consider the behaviour of the structure in relation to the envisaged dimensioning situations, taking into account the relevant factors that significantly influence the potential performance of the structure concerned. At this stage, the target accuracy for the estimate of overall project costs might typically be ±5%.

Detailed Design phase The output of this phase shall allow construction of the project. The level of detail of drawings and specifications and site instructions shall allow unambiguous understanding by the contractor of what is required and how the scheme must be executed, as well as how compliance with the documents must be demonstrated.

Target accuracies costs The target accuracies for the estimate of overall project costs, however, could also be applied to other factors such as environmental impact and the evaluation sustainability parameters.

8.2.3 Accuracy fundamental structural design phases

The fib Model Code 2010 study on the feasible accuracy of corresponding individual design phases can easily be transformed into the required accuracy of the individual major design phases and structural phases of the fundamental structural design cycle, as listed in table 8.4.

Structural design phases and corresponding accuracy			
Structural phases	Major phases	Fib phases	Accuracy
Creation of a system outline	Conceptual design	Scouting	±30%
		Basis of design	±20%
Optimisation of the structural action	Basic design	Project specification	±10%
		Final design	±5%
Final dimensioning and specification	Detailed design	Detailed design	For construction

Table 8.4: Level of accuracy

8.3 Fundamental design process

8.3.1 Structural design characteristics

Independent of life cycle phase, complexity of design, and contractual commitments, the structural engineering practice can be most effectively outlined by the structural design characteristics as listed in table 8.5, directly based upon the fundamental design cycle as shown in figure 8.2 on page 81.

These structural design characteristics can be an effective guide for structuring structural engineering activities and a corresponding professional profile of structural engineering.

8.3.2 Fundamental structural design process

Within each of the major design phases of conceptual design, basic design, and detailed design, the fundamental structural design cycle as

Professional structural engineering
Structural design characteristics
Conceptual design: Creation of a system outline
- Overall geometry - Choice of materials - Approximate section characteristics: ±20% - Conceptual variant studies of performance-based solutions - Main optimisation of the performance/cost ratio - Open mind for ideas and innovative design
Basic design: Optimisation of the structural action
- Thorough analysis of the structural action - Optimisation of the structural action - Further dimensioning with an accuracy of ±5% - Overall structural system check with the complete set of functional requirements and contractual conditions
Detailed design: Final dimensioning and specification
- Final section and detail dimensioning - Check calculations structural safety ULS - Check calculations serviceability SLS - Check calculations durability DLS - Specification for construction - Monitoring decisive parameters

Table 8.5: Structural engineering characteristics

shown in figure 8.2 on page 81 is applicable. Consequently, the structural design process as shown in figure 8.1 on page 80, can be filled in as shown in figure 8.4.

In the process of increasing insight into the system's functionality, each design phase is characterised by a change of both accuracy and character of the activities. The corresponding increasing level of specification is reflected by the required accuracy of the material dimensioning from ±20%, via ±5%, to final specification for construction.

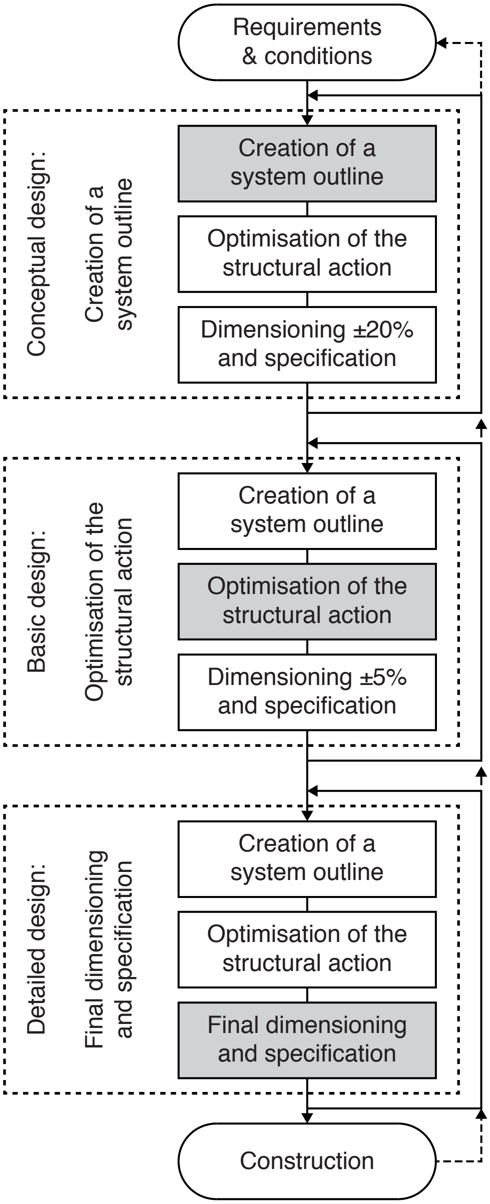


Figure 8.4: Fundamental structural design process

Chapter 9

Physical decomposition

9.1 Qualification of the structural form

9.1.1 Classification

A physical decomposition of a structure requires a methodical classification as a guidance for the partition of the system into subsystems.

There are numerous possibilities of classifying structural systems. The two most notable classification techniques are the following:

- Classification by physical appearance; in this case structural form.
- Classification by behaviour; in this case structural action.

The majority of applied classifications are directly based on the structural form with clear identifiable structural elements as beams, columns, floors, walls, arches, and trusses.

Oskar Büttner und Erhard Hampe performed an extensive study and reported it in the corresponding publication “Bauwerk, Tragwerk, Tragstruktur, Band 2: Klassifizierung, Tragqualität, Bauwerkbeispiele” [3] of a structural action-based classification.

In this classification, specifically the bending flexibility of linear and plane structural elements is distinguished. These elements are then assembled into one-dimensional linear elements, and two-dimensional and three-dimensional structural systems. De facto it is a distinction between tensile structures and all other structures dominated by bending.

9.1.2 Form follows function

The starting point of the design process is the function expressed by a set of functional requirements. In order to successfully conceive and plan a structure it is necessary to investigate and well know its reason for existence, and its major and minor loads to resist and to bear. The function, however, is not a static specification but can vary during the life cycle of a structure.

Eduardo Torroja Stated in his book "Philosophy of Structures" [27]: "The primary functions of all structure can be summarised as follows: To enclose a certain space and to protect it from the natural elements of wind, rain, and snow, from changes in temperature, and from noise. This function is achieved by the use of walls and roofs. To provide passageways for the movement of persons and vehicles; floors, staircases, and ramps of buildings, and bridges and viaducts are used for these functions. To resist the lateral thrust of earth, water, or other fluids; included in this category are dams, dikes, reservoirs, storage tanks, silos, and retaining walls."

Eladio Dieste Stated in his book "La Estructura Cerámica" [7]: "The resistant virtues of the structures that we seek depend on their form; it is through their form that they are stable, not because of awkward accumulation of material. There is nothing more noble and elegant from an intellectual viewpoint than this: to resist through form."

9.1.3 Interfaces with the built environment

The main interfaces with the built environment, which are decisive with respect to influencing the feasibility in general and the overall performance/cost ratio in particular, are: reliability, architectural demand, air conditioning system, constructability (practical feasibility), and relative life cycle costs.

To secure a clear recognisable interface between conceptual structural design and the mainly form-oriented other conceptual design disciplines of the built environment, a structural form-based, rather than structural action-based classification is preferable.

Furthermore, a structural action-based classification can vary completely on different levels. For example, the global load distribution of a truss consists of bending moments and shear forces. The corresponding local load distribution within the truss consists completely of axial compression and tension forces in the individual members of the truss.

9.2 Decomposition of the structural form

9.2.1 System decomposition

Physical decomposition follows the physical parts of a system. Often, it is a very natural way of decomposition, because we easily "see" all the physical parts. Also, the completeness criterion above is easy to check; when we have all physical parts, we have the whole system.

The whole structure is a three-dimensional system such as a frame building, multi-storey building, barrier, bridge, and tunnel.

The three-dimensional system can be disassembled in two-dimensional (plane) subsystems such as a frame, floor slab, cable-stayed beam, truss, arch, and shear wall.

The two-dimensional subsystems can subsequently be assembled in one-dimensional (linear) elements such as a compression and tension bar, a bending beam, and a (shear) corbel.

The system "Arch bridge" as shown in figure 9.1 for example, can be seen as an assembly of the subsystems frame, truss, arch, and orthotropic deck.

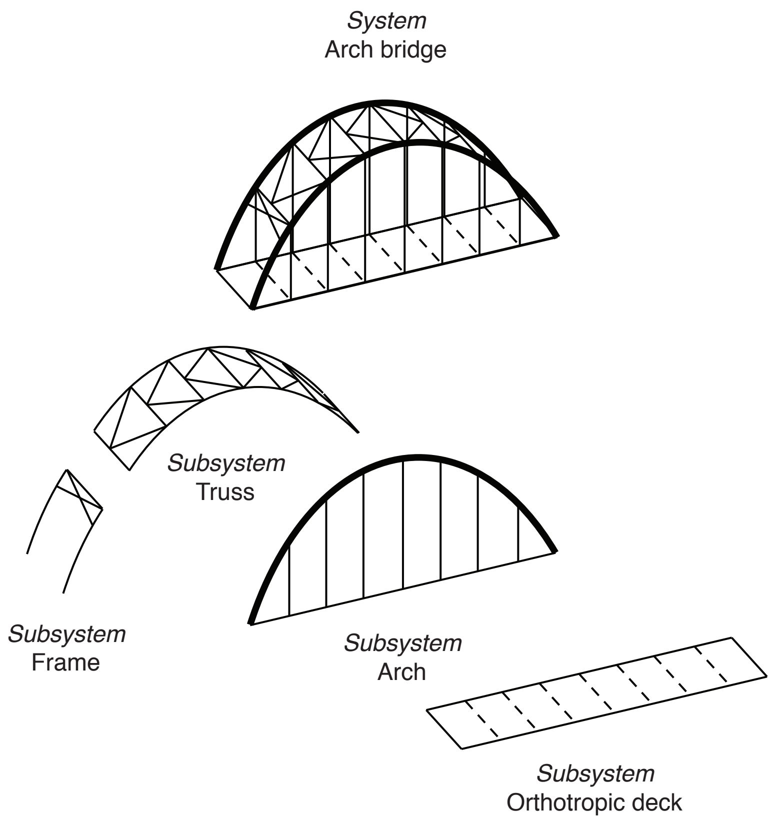


Figure 9.1: Assembly of an arch bridge

A two-dimensional subsystem is an assembly of directly connected structural elements, designed to act together to resist loads.

9.2.2 Structural form on subsystem level

Characterisation of a problem is part of the solution. But characterisation of a bridge, barrier or building as a whole is nearly impossible due to the variety and complexity of possible structural forms. Characterisation of individual forms with regard to the capacity to bear and resist, and with regard to the interfaces with the built environment, appears feasible. It is plausible that on this subsystem level, as shown in figure 9.2, enough insight and oversight can be gathered to produce performance-based solutions.

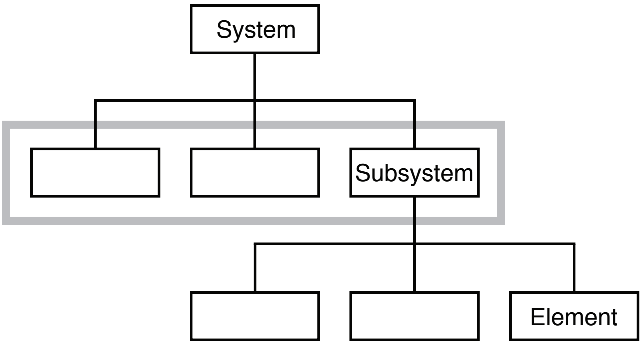


Figure 9.2: Subsystem level

9.2.3 Basic structural forms

On subsystem level the following basic structural forms can be distinguished as shown in figure 9.3: frame, floor slab, cable-stayed beam, truss, arch, and shear wall.

Global and local load distribution On the basis of uniformly distributed loads the global load distribution of all basic structural forms consists of bending moments and shear forces. Dependent on the individual basic form this global load distribution is internally transferred differently into a local load distribution.

Each individual basic structural form is characterised by its corresponding local load distribution. This characteristic local distribution of the loads is described in the following paragraphs, whereby members subjected to axial compression or tension, are relatively stronger and generally more economical than those designed for pure bending.

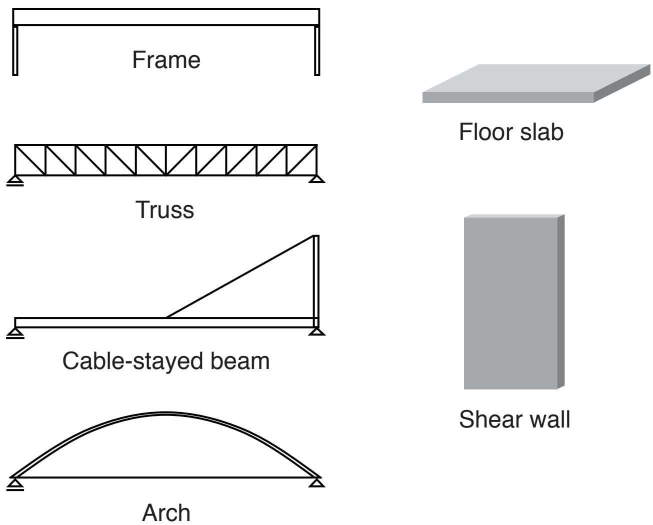


Figure 9.3: Basic structural forms

Frame Frames consist of bending beams and supporting columns. The local load distribution of a beam consisting of one element equals the global load distribution and is therefore subjected to bending and shear. The supporting columns are subjected to compression. The load distribution in a frame is worked out in subsection 14.2.1.

Floor slab The local load distribution of a floor slab equals the global load distribution and is therefore subjected to bending and shear. A two-way spanning floor slab, whether concrete slab or orthotropic steel deck, is subjected to bending and shear in both spanning directions. The load distribution in a floor slab is worked out in subsection 14.2.2.

Cable-stayed beam A supporting cable diminishes the supported length and corresponding local load distribution of beams; by each half, strength - the effect of section modulus W - is quadrupled and stiffness - the effect of moment of inertia I - is eight-fold.

The effectiveness of the local load distribution in a cable-stayed, statically indeterminate continuous beam is dependent on the stiffness of the cable. The cross-sectional area of the cable is therefore governed by both tension strength and supporting stiffness. The load distribution in a cable-stayed beam is worked out in subsection 14.2.3.

Truss Every member of the truss is in pure compression or pure tension. Shear, bending moments, and other more complex stresses are all practically zero. This makes trusses physically stronger and stiffer than other ways of arranging material. Some or all of the joints may be fixed rather than pinned but the main contribution to the strength and stiffness is provided by the triangulation. The load distribution in a truss is worked out in subsection 14.2.4.

Arch The form is more or less parabolic and usually, the horizontal reaction forces are resisted by a tension rod. Uniformly loaded, the entire arch is in compression and has little bending moments and shear. This makes arches relatively stronger than other ways of arranging material, resulting in smaller cross-sectional dimensions. However, an arch requires a more than average amount of structural space.

So-called “false” arches, when bending is dominant over compression, are unfavourable: they are circular instead of parabolic arches and/or have highly-concentrated loads, instead of more or less uniform loads. The load distribution in an arch is worked out in subsection 14.2.5.

Shear wall The most common application of shear walls is securing the global stability of a system. For non-sway buildings, these can be in the form of concrete shear walls or steel wind bracing. The load distribution of a stocky shear wall consists of dominant shear in combination with minor bending. The load distribution in a shear wall is worked out in subsection 14.2.6.

Chapter 10

Cyclic process control

10.1 Exploration of the solubility space

10.1.1 Creation-process requirements

During the creation process, the structural engineer is creating a materialised form out of the functional requirements and taking into account the fundamental related interfaces with the built environment.

Key words in the search for a structural form are "insight", "overview", and "breeding ground".

Insight Insight into each individual aspect of the design process such as reliability, structural form, load distribution, choice of material, failure mechanisms, dimensioning, architectural demand, constructability, and costs.

Overview Overview of the interrelations of the aspects, such as:

- Reliability versus form, load distribution, material, failure mechanisms, and dimensions.
- Architectural demand versus form, material, and dimensions.

- Constructability versus form, material, and dimensions.
- Costs versus reliability, architectural demand, and constructability.

Breeding ground The way of presenting and visualising the individual aspects and their interrelations can serve as a fertile breeding ground for design solutions. All data do not necessarily have to be quantified to fulfil their function. Therefore, hardly quantifiable phenomena such as intuition, imagination, common sense, and personal attitude can be implemented in the creation process.

10.1.2 Design strategies

Four typical design strategies with increasing complexity can be distinguished as shown in figure 10.1:

Linearity The simplest design strategy describes designing as an unambiguous one-way movement among the design activities. This strategy is only working for the design of extremely simple structures.

Cyclic For most of the design problems, a cyclic design process is inevitable. Every time the designed structure turns out to be insufficient a new cycle is appropriate.

Cyclic Convergent In a converging process, the number of loops will diminish during the process of further specification by making goal-oriented design choices. Convergence, however, does not automatically mean optimisation.

Cyclic Optimum Optimisation of both quality of design outcome and number of design cycles is dependent on how strong the initial idea is and the improvements one chooses to make. For the most successful possible outcome, corresponding fundamental design choices have to be searched for.

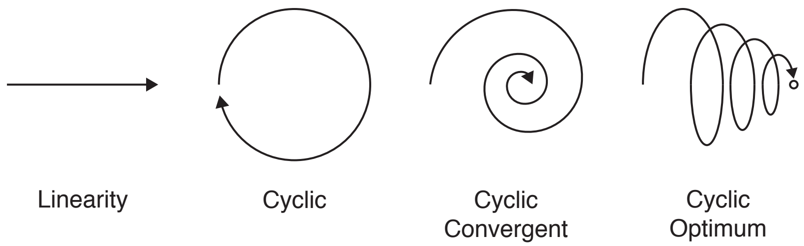


Figure 10.1: Design strategies

10.2 Linear design

10.2.1 Linear process

Each individual design activity starts with requirements as to what the structure or part of the structure is supposed to do and ends with a specification for construction. Independent of life cycle phase, complexity of design, and contractual commitments, structural engineering practice can be outlined by the fundamental structural design cycle as shown in figure 8.2 on page 81.

Linear design describes designing as an unambiguous one-way movement among these design phases as shown in figure 10.2. This strategy is only working for the design of extremely simple structures.

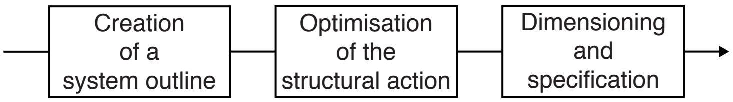


Figure 10.2: Linear structural design

10.2.2 Specification of the structural form

Essentially, the design process of a structure is a process of composing the structural form. For an effective control of this composition process both the level of specification and the level of composition have to be considered.

The integral composition process of a conceptual draft of the system outline will inevitably lead to a process of decomposing the structure in controllable basic structural forms, mostly on a two-dimensional subsystem level.

The immense number of individual parameters and mutual interactions cannot be unambiguously controlled and will inevitably lead to a cyclic process.

10.3 Cyclic design

10.3.1 Cyclic process

In order to have a successful solution to a complex structural design problem a cyclic design process is inevitable. Every cycle goes through the phases of creation, optimisation, and specification. Then, the concept is reviewed with respect to the functional requirements in particular, and a performance/cost optimisation over the life cycle in general.

When the reviewed concept turns out to be insufficient, a new cycle is appropriate. In the process of materialisation from requirements to construction, on-going design and check activities can influence foregoing activities with regard to choice of geometry, material, and matching dimensions.

The number of cycles depends on how strong the initial idea is, and on the improvements one chooses to make; intelligent improvements will reduce the number of times that one has to go through the cyclic design process.

The cyclic design process evolves over time in order to produce the most successful possible outcome but at least a performance-based solution. In an effective converging process, the number of optimisation loops will diminish during the process of further specification.

An example of this cyclic design process is when the famous Thomas Edison, who designed more than ten thousand prototypes of the light bulb until he was satisfied and knew it worked, said; “I have not failed

10,000 times, I have successfully found 10,000 ways that will not work" [30].

10.3.2 Fundamental structural design path

The structural design process can fundamentally be characterised by two simultaneous processes:

Specification The process of specification of the structural form from approximate to accurate.

Composition The process of composition or rather decomposition of the structural form from system to element.

These two processes can be visualised together in a two-dimensional matrix as shown in figure 10.3. Within this matrix, the cyclic structural design can be explored.

Level of specification On the horizontal axis, the phases of specification are arranged from creation of the system outline, via optimisation of the structural action, to dimensioning and specification.

Level of decomposition On the vertical axis, the phases of decomposition of the structural form are arranged from three-dimensional system, via two-dimensional subsystem, to one-dimensional element.

Design path Complex structures can effectively be analysed "from system to element" and "from approximate to accurate". The corresponding design path is directed by the effectiveness of this combination of breadth and depth. For an effective converging process, this combination has to be well balanced.

The design path follows the fundamental dimensioning routine from structural integrity, via load distribution, to failure mechanisms:

- Structural integrity; three-dimensional system design and decomposition in subsystems.

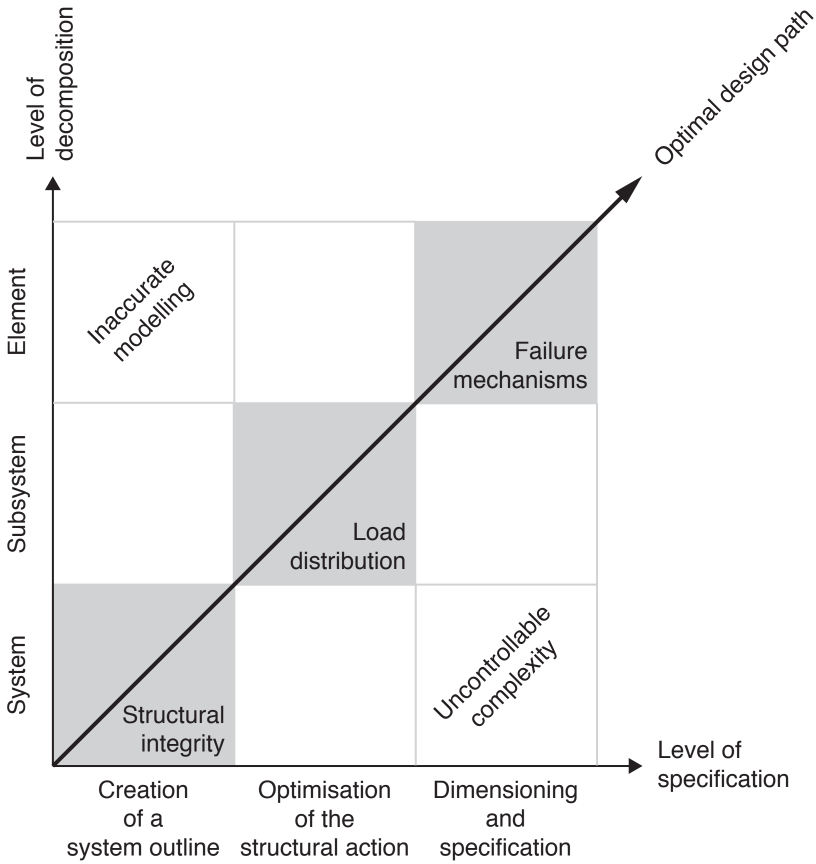


Figure 10.3: Fundamental structural design path

- Load distribution; distribution of the actions within the subsystem on element level.
- Failure mechanisms; distributed actions can be resisted by materialising the elements.

Outside the boundaries of this design path, ineffectiveness such as inaccurate modelling or even uncontrollable complexity can be found.

Inaccurate modelling The integral creation of a system outline requires a three-dimensional modelling. Due to the huge degree of freedom and the complexity of interfaces with other disciplines, this three-dimensional modelling and corresponding dimensioning is only approximating and without great detail.

For a more refined dimensioning, load distribution and capacity calculations on element level are appropriate. The required data for an accurate calculation on element level are not available in such an early conceptual design stage.

Therefore, a creation of a system outline on element level inevitably leads to inaccurate and thus unusable modelling.

Uncontrollable complexity The specification of the design is determined on element level in order to get the required detailed depth and accuracy. Solely on system level, it is not achievable to control the multitude of information and interrelations from creation, via optimisation, to specification.

So inevitably, designing the structure on system level only results in uncontrollable complexity of the dimensioning and specification.

10.3.3 Structural design path

Out of the functional requirements, a system is created and the performance/cost ratio is optimised, taking the interfaces with the built environment into account. Then, specifications for construction are prepared. This process can be divided into three major design phases with increasing accuracy: conceptual design, basic design, and detailed design, as shown in figure 8.4 on page 88.

The fundamental structural design path as shown in figure 10.3 on page 102 is applicable for all these three design phases. Furthermore, this fundamental path is also applicable for the structural design process as a whole - from creation to specification - as shown in figure 10.4.

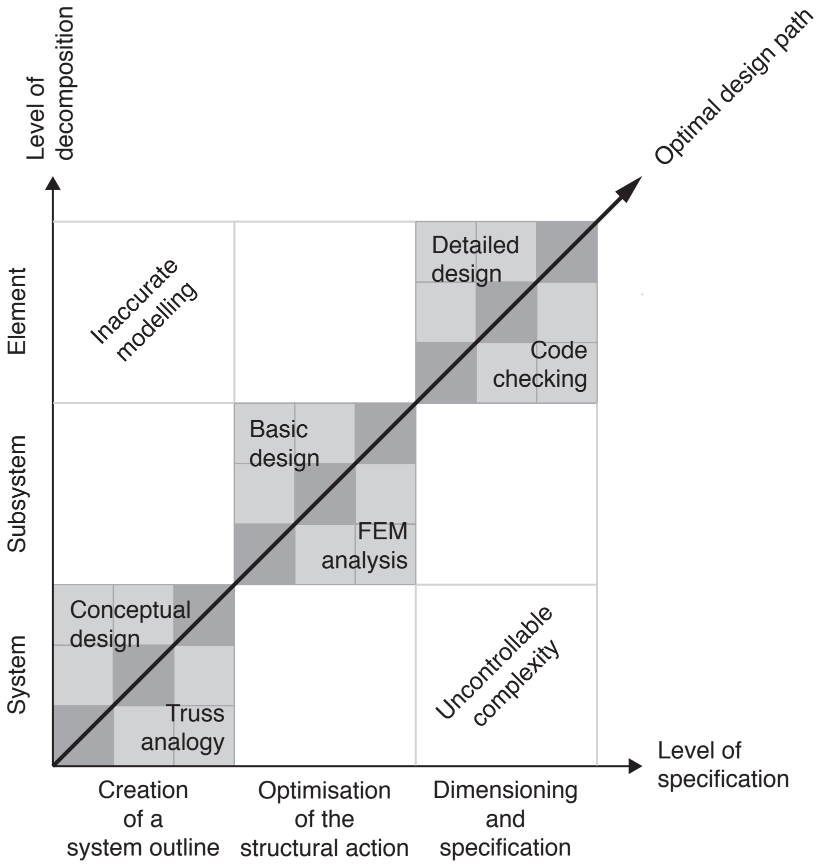


Figure 10.4: Structural design path

In this case, the standard level of decomposition is combined with a level of specification over the major structural design phases of conceptual, basic, and detailed design:

Conceptual design Creation of a system outline with the help of truss analogy.

Basic design Optimisation of the structural action with the help of a

Finite Element Method (FEM) analysis.

Detailed design Dimensioning with the help of code checking, followed by a specification for construction.

These include the ineffectiveness that can be found outside the boundaries of the design path, as mentioned before, in the form of inaccurate modelling or uncontrollable complexity.

10.4 Cyclic convergent design

10.4.1 Volume of complexity

The number of interdependent parameters that can be simultaneously controlled is limited and subject to the complexity of the individual parameters, the complexity of the interdependency between the parameters, and the level of abstraction of both parameters and interdependency.

The limitation of the complexity of either the individual parameters, or the interdependency between the parameters, has a similarity with communicating vessels. On the one hand, the control of processes with complex interdependencies will unavoidably lead to a limitation of the quantity and/or complexity of the individual parameters. On the other hand, control of processes with large quantities and/or complex parameters, will unavoidably lead to a limitation of the complexity of the interdependency between the parameters.

Due to the limited volume of complexity per design phase, the integral design process with complex interdependencies inevitably has to be separated from the individual architectural, structural, and construction design processes with complex parameters as shown in figure 10.5.

10.4.2 Three steps in conceptual design

In order to control the conceptual design phase, we can consider three steps:

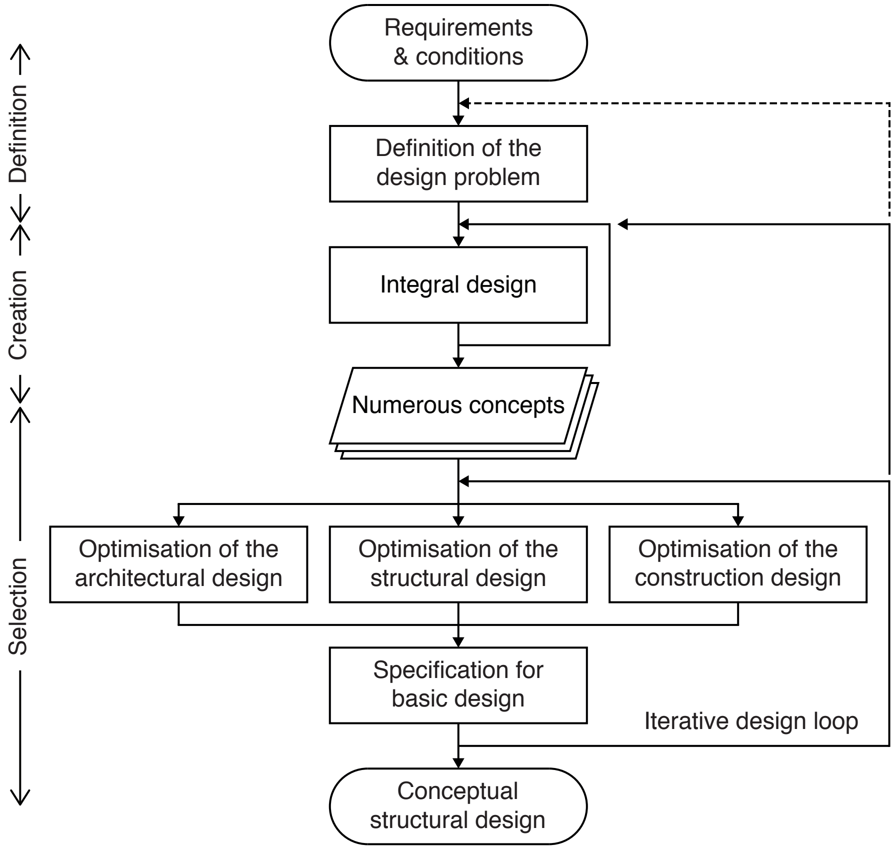


Figure 10.5: Limited volume of complexity per design phase

Definition step Defining the design problem.

Creation step Structural engineering-based creation; diverging into a rough overview of possible solutions.

Selection step Value engineering-based selection; a more refined checking of the boundary conditions, converging into a solution.

10.4.3 Definition step

Common practice during conceptual design is formulating a quick definition of the problem in order to save time for a heated search for performance-based solutions.

Albert Einstein is quoted as having said that if he had one hour to save the world he would spend fifty-five minutes defining the problem and then five minutes solving it [4].

Einstein's wisdom with regard to the problem-solving process is directly applicable to the conceptual structural design process. Thoroughly formulating the definition of the design problem during the problem definition phase is a time-consuming but crucial effort before starting the search for performance-based solutions.

Besides an effective partition of available time, it is a misconception that extending the duration of the search would lead to a better solution. It is far more effective to optimise available time, than stretching it.

The conceptual design team has to evaluate the complete set of functional requirements and contractual conditions, in order to determine a workable reduced set of fundamental requirements and conditions: presumably dominant, required performances and aspects - including interfaces with the adjacent built environment and prerequisite constraints.

The so-determined set of fundamental functional requirements will then be the starting point of the creation of concepts by the conceptual integral design team.

Effective functional requirements should be defined on an as-high-as-possible level of abstraction in order to serve as a starting point for creative freedom, rather than stifling restriction.

Furthermore, on such a high abstraction level, market tendencies and other undesirable influences are neutralised.

10.4.4 Creation step

First of all, it is important to create concepts that are likely to meet the performance requirements. It is hard enough to create a materialised form out of the functional requirements, and to take the fundamental related interfaces as reliability, architectural demand, constructability, and costs, into account.

To be secured of at least one performance-based solution at the end of the conceptual design process, it is advisable to create numerous concepts and develop them to a lower degree of complexity for a considered selection. Dependent on the complexity one has to cope with during the selection phase, the number of initial concepts can be reduced.

10.4.5 Selection step

The goal is convergence by optimising the cost-drivers such as the complexity of execution - shape, repetition and planning, maintenance, and possibilities regarding future extension capability.

Striving for an optimisation of the performance/cost ratio will lead to a performance-based cost minimisation. Sometimes, supplementary added-value solutions offer a considerable added value for a relatively small cost increase. Then, a real performance/cost optimisation - despite the earlier established requirements - will be executed.

10.5 Cyclic optimum design

Design cycles encompass complete updates of design. Within a design cycle, individual design loops can be distinguished representing particular design actions. Independent of life cycle phase, complexity of design, and contractual commitments, fundamental design loops can be identified as shown in figure 10.6.

In principal, design loops can occur at any level of specification, at any level of decomposition, in any direction, and with any range. Usually, in

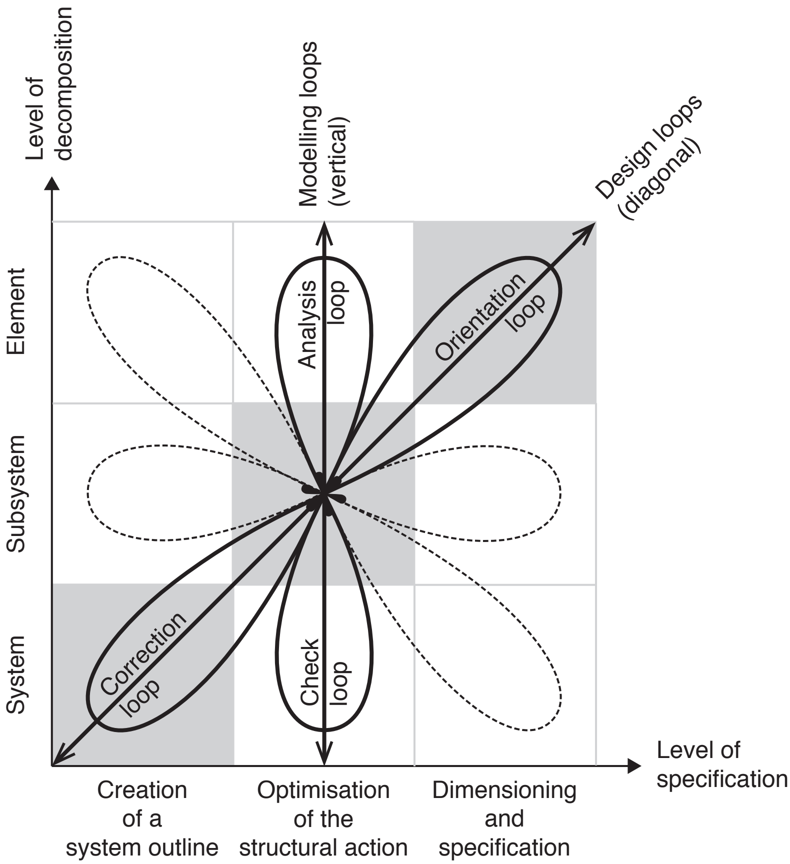


Figure 10.6: Fundamental structural design loops

design practice however, most frequently occurring design loops can be identified:

Modelling loops Vertical calculation-based loops with regard to structural safety and serviceability.

Typical design loops Diagonal creation-based loops with regard to a controlled build-up of the structural system.

10.5.1 Modelling loops

Modelling loops are the backbone of structural design. Within each specification phase from creation to specification, the complexity of reality has to be modelled with the help of calculation models with regard to structural safety and serviceability of the structure, or a specific part of the structure.

Although the point of attention differs per specification phase, for research on the structural integrity, the whole system or a specific part of the system is involved, covering all the levels of decomposition from system to element.

Such modelling loops are vertically orientated in the fundamental structural design loops as shown in figure 10.6 on page 109.

Refinement requires an analysis loop, whereas keeping the integrity of the whole system in mind requires check loops.

Analysis loop Going to a higher level of decomposition, complexity increases and further analysis is necessary to get a grip on the more detailed structural action.

Check loop With increasing level of decomposition and corresponding further analysis, checking the calculation model on a lower level of decomposition - preferably the whole system - is necessary in order to secure the integrity of the structure.

10.5.2 Typical design loops

Typical design loops are directing the design process. Within each specification phase from creation to specification, requirements-based design choices are made. Although the point of attention differs per specification phase, each design choice has to be valued and made ready for further refinement.

These design loops are diagonally orientated in the fundamental structural design loops as shown in figure 10.6 on page 109. On-going, more detailed specification requires an orientation loop, whereas diagnosed inaccuracies require correction loops.

Orientation loop Going to a higher level of specification in combination with a higher level of decomposition, complexity increases and research is necessary in order to orientate on possible further design refinements.

Correction loop With on-going detailing, insight into the structural action increases. With this increasing insight, foregoing structural design approximations and decisions can turn out to be insufficient or even incorrect. Such diagnosed inaccuracies require correction.

10.5.3 Optimal design cycle

The cyclic character of the conceptual structural design process is a continuous process of zooming in and zooming out with (diagonal) design loops, thus constantly changing both the level of decomposition and the level of specification.

In between, the dimensioning with the help of calculation models is a process of zooming in and zooming out with (vertical) modelling loops, changing only the level of decomposition. This process from creation to dimensioning is shown in figure 10.7.

The clearly directed fundamental design loops result in small-scaled and thus shorter, more clarifying individual design cycles. Furthermore, these small-scaled clarifying cycles potentially lead to a reduction of the total amount of cycles.

It should be noted that an effective optimal design cycle is a process of reduction rather than exclusion and limitation. This process of reduction, however, is completely dependent on known and explicitly unknown parameters. In the case of implicitly unknown parameters, there is automatically an unwanted exclusion and consequently uncontrollable risk with corresponding uncertainties in cost estimation.

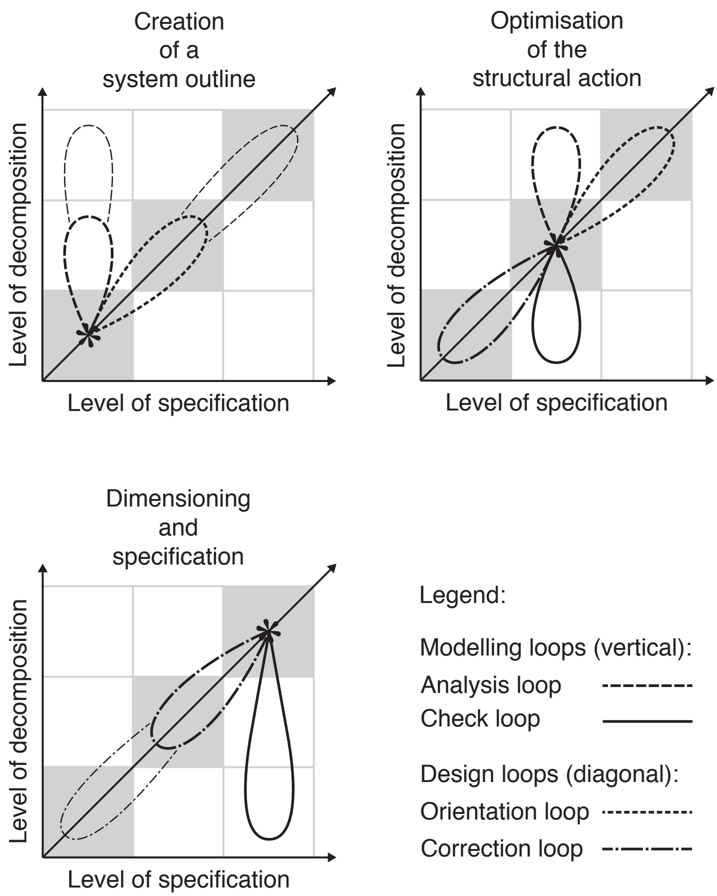


Figure 10.7: Structural design loops

10.5.4 Risk analysis

The specification of a conceptual design demarcates the concerning project phase. Dependent on the contractual arrangement, this specification can be used as internal transfer documentation or for tenders. In both cases, a risk analysis is essential.

A risk analysis of the material demand gives an accuracy estimate of the material quantities and corresponding unit cost indications. The risk analysis has to be executed on the level of individual components of a decomposed system, principal details included.

For an effective risk analysis the following, preferably quantified data have to be determined:

Performed structural analysis Depth and breath of the structural analysis are dependent on the complexity of the structural action and the available time schedule, financing, and resources of the conceptual structural design phase.

Dimensioning The result of the performed structural analysis is an approximate section-dimensioning of the materialised overall geometry. This dimensioning of the conceptual design is specified by a list of material quantities.

Cost weighting The cost weighting equals the material quantities times the unit cost indications, and is a measure for cost optimisation opportunities and corresponding risk.

Uncertainties The difference between the performed approximate structural analysis for conceptual design and the required depth and breath to meet the in-use requirements for structural safety and serviceability, can be defined as uncertainties of the conceptual design. The difference in depth and breath generally concerns load combinations, load distribution, and failure mechanisms.

Coverage by conceptual dimensioning The uncertainties of the performed approximate structural analysis with respect to the required depth and breath can be partially, or even not at all, covered. The corresponding status of the coverage gives an indication of the risk influence of uncertainties on conceptual design.

Reserves Reserves can be intentionally incorporated or are the result of rounding-up to the nearest standardised product dimensions. Occasionally, an optimisation of requirements during the conceptual design phase can result in a reserve.

Optimisations Foreseen but time-consuming cost optimisations can be postponed to basic design but registered as a potential reserve with regard to the completed conceptual design.

Part III

Understanding structural performance

Chapter 11

Introduction to part III

The methodical approach on conceptual structural design consists of a process control component and an underlying, structural performance component. The process control component has been discussed in the previous Part II, “The art of conceptual structural design”. The underlying, structural performance component is discussed in this Part III, “Understanding structural performance”.

Although the individual, conceptual structural design parameters are fully applied mechanics-based, it is choice, combination, and balanced depth of these parameters that define their professional applicability. It is thereto that this part derives its reason for existence within this textbook.

Part III starts with a qualification of structural performance.

The fundamental structural design path, as shown in figure 10.3 on page 102, gives the basic arrangement of the required conceptual structural design parameters “3-D structural integrity”, “2-D load distribution”, and “1-D failure mechanisms”. The quantification of these design parameters as described in part III is subdivided into the individual parameters as shown on the diagonal in figure 11.1.

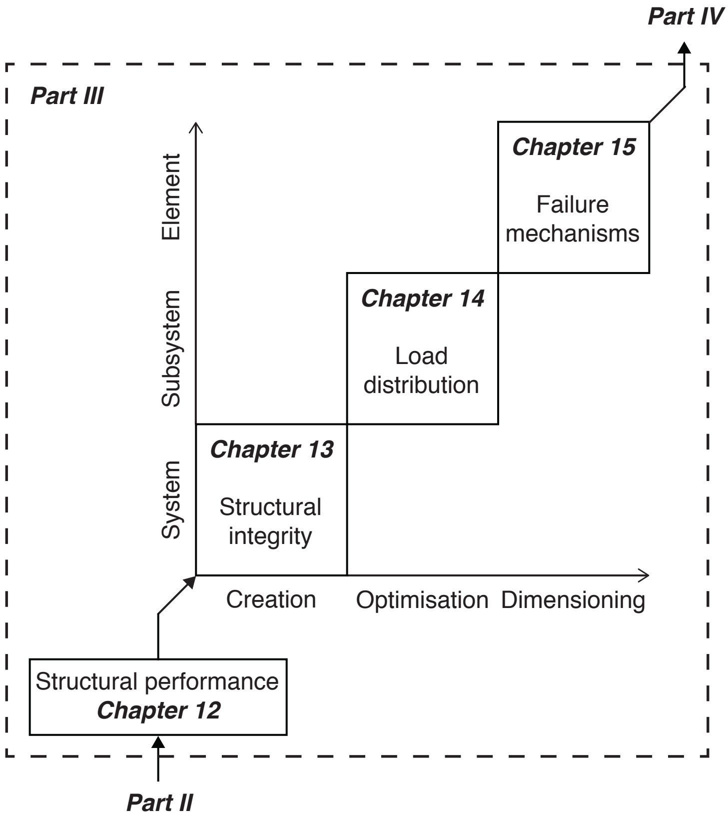


Figure 11.1: Reading guide for part III

Structural performance Structural performance is a collective term for structural safety, serviceability, and durability. As a major part of structural performance, structural action with regard to load distribution, corresponding deformation, and strength is researched.

For an effective and efficient conceptual structural design, approximations and especially deformation-driven parameters have to be determined. The choice and combination of these parameters, however, define their professional applicability.

Structural integrity In the course of the conceptual design, and particularly in the first design loops, the insight into the structural action has to be built up step by step. Due to the huge degree of freedom and the complexity of interfaces with other disciplines, conceptual structural designing requires simple and clear, three-dimensional modelling.

This is possible on the level of axial forces, directly or with a truss analogy for modelling bending action and more complex forms.

Load distribution The load distribution on the system through the subsystems into the elements, can best be determined on subsystem level with design approximations of the load distribution in two-dimensional, basic structural forms.

For the basic structural forms, the design parameters concerning load distribution and deformation are determined and guides are given for the design of parallel load distribution with regard to optimisation, redundancy, and induced deformation.

Failure mechanisms With the determined forces in the individual one-dimensional elements, the required dimensions of these elements can be determined by means of design approximations of the load-carrying capacity with regard to the ultimate limit state.

With the determined deformations, the required dimensions of these elements can be determined by means of design approximations of the deformation with regard to the serviceability limit state.

For the elements of basic structural forms, the design parameters concerning material properties, sectional strength, stability, and deformation are determined.

Solution components The so obtained coherent set of solution components for structural performance, and corresponding chapter arrangement, is listed in table 11.1.

Chapter arrangement and solution components		
No.	Chapter	Solution component
15	Structural integrity	- Load path design
16	Load distribution	- Load distribution parameters
17	Failure mechanisms	- Dimensioning parameters

Table 11.1: Solution components for structural performance

Chapter 12

Structural performance

12.1 Present-day structural performance

12.1.1 Structural requirements

Structural performance is a collective term for the following structural requirements as defined in subsection 4.1.4:

Structural safety The safety of a structure or structural member is prescribed with its Ultimate Limit State (ULS).

Serviceability The functionality of a structure or structural member is prescribed with its Serviceability Limit State (SLS).

Durability The durability of a structure or structural member is prescribed with its Durability Limit State (DLS).

These specifically in the structural design codes of practice prescribed structural requirements ULS, SLS, and DLS, are part of a more overall sustainability objective to minimise the negative environmental impact.

12.1.2 Modern developments

The three structural requirements, structural safety, serviceability, and durability, combined with corresponding directions of development are shown in figure 12.1.

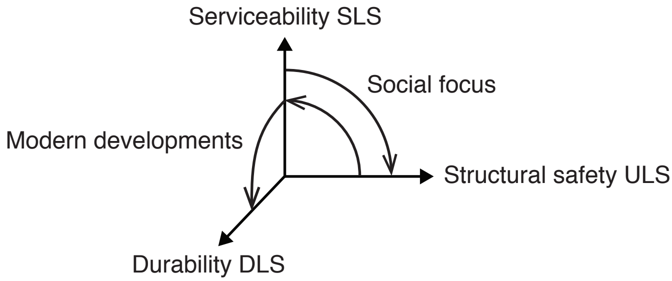


Figure 12.1: Developments of structural performance

With structural safety being a self-evident prerequisite, serviceability is an upcoming contractual requirement with regard to controlled functional behaviour of the structure. Due to the absence of an additional safety factor for serviceability, the outcome of measurement results typically is sensitive to downward crossing.

Particularly, the recent addition of a durability limit state to the ultimate and serviceability limit states, is initiated through performance-based building design as a logical step towards an integral performance/cost optimisation of the built environment of the entire life cycle. As a result, durability is extending to sustainability by minimising negative environmental impact, and further to the Triple Bottom Line (TBL) accounting framework with a social, environmental, and financial partition.

Due to the amount of structural failures, however, society is shifting its focus back towards structural safety.

12.2 Force and deformation-driven parameters

12.2.1 Equilibrium and strength

For an effective and efficient determination of the capacity to bear and resist, a fundamental insight into structural behaviour is necessary. The four subjects to be considered profoundly are equilibrium, strength, deformation, and parallel load distribution.

Further qualification and coherence of these four subjects as shown in figure 12.2 will be clarified subsequently.

Equilibrium Equilibrium is the first, most obvious aspect and an absolute necessity for each building and all civil work; otherwise the structure or parts of the structure will translate, rotate, or just tumble down.

Strength Strength is the second most obvious aspect and also an absolute necessity; without enough strength the functional requirements with respect to the ultimate limit state, and particularly the structural safety, can never be met.

The two failure mechanisms concerning the strength are sectional strength and stability. Sectional strength is force-driven, whereas the stability of the equilibrium - from element up to system level - is deformation-driven.

This deformation-driven stability is subject to many parameters such as length, cross-sectional bending stiffness, boundary conditions, deformation due to fabrication, residual stresses, and material faults. Therefore, the stability analysis is often merely based upon iterative blind code checking, without availing the opportunity of essence-based deformation-driven designing.

The necessity of both equilibrium and strength is described thoroughly in built environment related legislation by structural safety regulations.

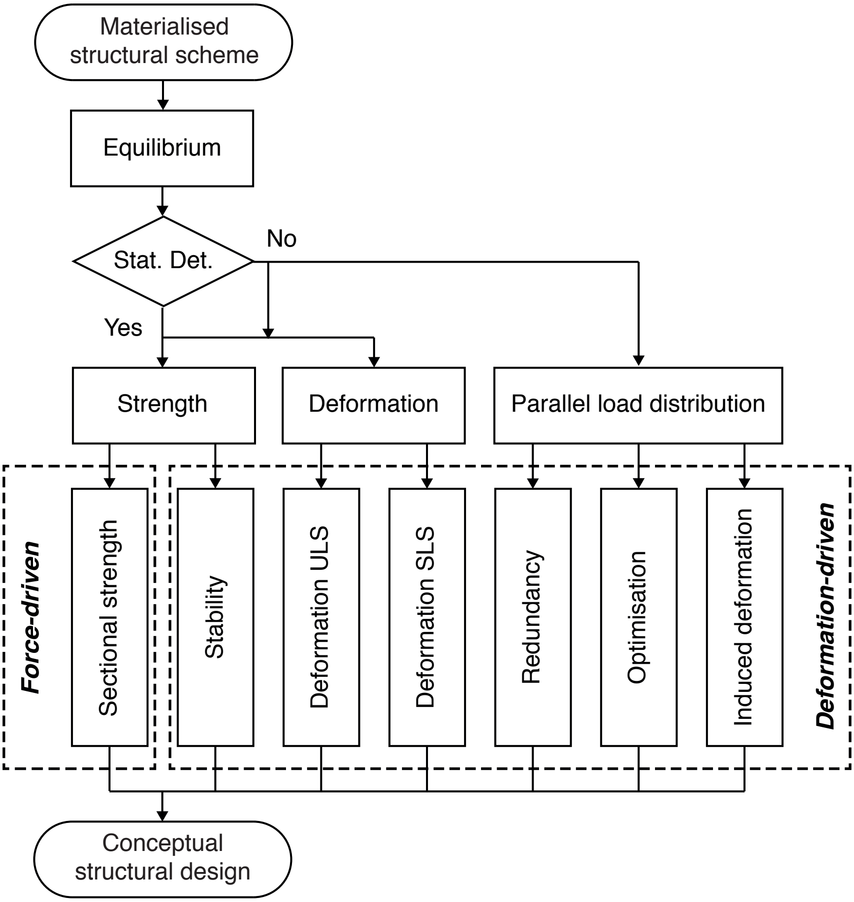


Figure 12.2: Force and deformation-driven parameters

12.2.2 Deformation-driven parameters

One of the main pitfalls in conceptual designing, however, is focusing mainly on the combined equilibrium and strength, just because it is an obvious necessity.

As for stability, deformation-driven parameters in structural designing are somewhat underexposed in higher education as well as in common, modern structural engineering practices.

Especially the conceptual designer needs to comprehend the deformation-driven parameters as shown in figure 12.2 on page 124, in addition to the standard engineering basics such as applied mechanics, design codes, and related background documents.

Deformation In case of functional requirements with regard to the serviceability limit state, displacements of parts of the structure and the structure as a whole have to be met.

Parallel load distribution Foregoing strength and deformation parameters have to meet the requirements concerning structural safety in the ultimate limit state, and displacements in the serviceability limit state. The extent of compliance with these requirements determines the effectiveness of the design.

Nowadays, a lot of structures are statically indeterminate out of economical efficiency and redundancy considerations. Corresponding parallel load distribution in the ultimate limit state is largely determined by deformation parameters.

Statical indeterminacy The structural system is statically indeterminate when the static equilibrium equations are not sufficient for determining the internal forces and reactions on the structure. A difference in deformation will influence the load distribution. Consequently, the behaviour can be deformation-driven, besides force-driven.

The distribution of the load in detailed design will be on the level of stress and strain. Due to a high degree of parallel behaviour - statical indeterminacy - the stress distribution is proportional to the stiffness distribution, which is inversely proportional to the strains.

Optimisation of the performance/cost ratio The optimisation of the stiffness distribution is a main factor in the optimisation of the performance/cost ratio. Focusing mainly on the strength will produce a safe design but not automatically an economical efficient one. Therefore, it is of great importance that the structural designer, besides performance and construction demand, focuses on deformation in addition to strength.

12.2.3 Determination of deformation-driven parameters

The relative small group of conceptual structural design practitioners does not have the time to analyse its mainly intuition-based designing in order to make its knowledge accessible for both professionals and higher education programmes.

Scientific education is compartmented to such an extent that the interface between applied mechanics and material applications is underdeveloped. The interfaces between the structural engineering and architectural demand, constructability and life cycle engineering are even less visible.

The practice of conceptual structural design with regard to deformation-driven parameters, is not covered by most of the higher education programmes, as listed in table 12.1.

Misfit of structural design parameters in education		
Structural design parameters:	Force-driven	Deformation-driven
Field of practice	Minor part	Major part
Higher education programmes	Major part	Minor part

Table 12.1: Misfit of design parameters in education

Therefore, a missing set of deformation-driven design parameters has to be determined as a basis for conceptual structural design in higher education programmes.

12.3 Approximate dimensioning

12.3.1 Conceptual structural design approximations

It is of the utmost importance that design assumptions during conceptual structural design are based on insight into the behaviour of the structure to assure structural safety and to facilitate design optimisation.

Effective conceptual structural design approximations should address, and give insight into all influential structural parameters namely material properties, geometry, support conditions, and loading.

Ineffective approximations So-called "rules of thumb" when solely based on geometric properties and therefore load indifferent, do not address all influential structural parameters, have a low accuracy and are almost never provided with a clear and comprehensive insight into the application boundaries.

High-end black box application programs, on the other hand, address all structural parameters and have a high accuracy but do not give a direct insight into the behaviour of the structure.

Modified applied mechanics-based approximations Applied mechanics-based approximations give a calculated insight into load distribution and likely decisive failure mechanisms. Furthermore, it facilitates an overview of the optimisation possibilities of the structural design.

For realistic approximations of complex material behaviour, applied mechanics approximations can be modified with identifiable empirical data.

On the level of integral conceptual design, geometrical parameters should be related to outer dimensions.

12.3.2 Conceptual structural design parameters

Although applied mechanics are textbook material-based, defining an adequate set of conceptual structural design parameters is appropriate within this methodical approach.

This set of conceptual structural design parameters has to fulfil the following requirements:

- Be applied mechanics-based to assure structural insight.
- Have clear and unambiguous applicability.
- Have performance-based approximation depth for conceptual design.
- Be balanced with respect to deformation-driven parameters.
- Have manageable limited overall size.

The fundamental structural design path, as shown in figure 10.3 on page 102, gives the basic arrangement of the required conceptual structural design parameters:

- Load path design on a three-dimensional system level.
- Load distribution on a two-dimensional subsystem level.
- Failure mechanisms on a one-dimensional element level.

This basic arrangement can be extended into an effective set of applied mechanics-based conceptual structural design parameters as listed in table 12.2.

12.3.3 Dimensioning routine

In conformity with the fundamental structural design cycle as shown in figure 8.2 on page 81, the dimensioning routine of the conceptual structural design can be constructed as shown in figure 12.3.

Conceptual structural design parameters		
Structural integrity	Load distribution	Failure mechanisms
3-D system level	2-D subsystem level	1-D element level
System design on load path level and decomposition in subsystems	Distribution of the prime actions within the subsystem on element level	Distributed actions can be resisted by materialising and dimensioning
Basic applied mechanics based		
System design: - Rigidity - System effects - Principal details	Distribution: - Basic structural forms - Parallel load distribution - Induced deformation	Capacity: - Material strength and stiffness - Sectional strength and stiffness - Stability

Table 12.2: Conceptual structural design parameters

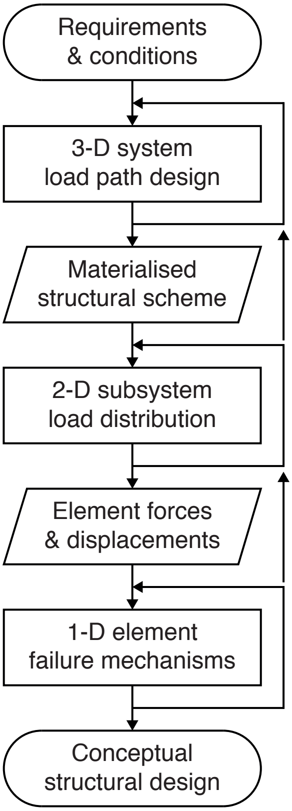


Figure 12.3: Dimensioning routine of the conceptual structural design

Chapter 13

Structural integrity

13.1 Conceptual design on system level

13.1.1 Creation phase of the conceptual design

The creation phase of a system outline starts with requirements and conditions of what the system is supposed to do and ends with one or more materialised structural schemes as shown in figure 12.3 on page 130.

13.1.2 Structural integrity

Structural integrity is the term used for the performance characteristics of a structure with regard to the ability to support designed loads and the redundancy capacity of the whole system.

In an integral conceptual design process, the performance/cost-ratio of the life cycle has to be optimised. Every structure will have its own process of creation, influenced by its bearing capacity and resistance, its economy, its construction site, and last but not least a more or less pronounced aesthetic concept and appearance. Its structural integrity, however, will always be an undoubted demand and has to be unambiguously secured by a clear, three-dimensional basic structural concept.

The vertical imposed loads, the self-weight of the structure and natural impacts such as snow loads have to be borne. In addition to these vertical loads, other equally essential horizontal loadings must be taken into account due to wind actions and global initial sway imperfections.

On system level, the static resistance and equilibrium during the life cycle of the structure have to be secured. Particularly, the design of additional stabilising elements to resist the horizontal loads requires attention and insight on a three-dimensional system level.

Furthermore, the redundancy of the structural system with respect to accidental loads has to be secured, based on a risk analysis and dependent on specific structural performance demands.

13.2 Load path design

13.2.1 Load path design on system level

Due to the huge degree of freedom and the complexity of interfaces with other disciplines, conceptual structural designing requires simple and clear three-dimensional modelling. This is merely possible on the level of axial forces, directly, with a truss-analogy or an arch depending on the structural form.

Axial forces-based, a rough three-dimensional outline of the load paths can be constructed as shown in figure 13.1.

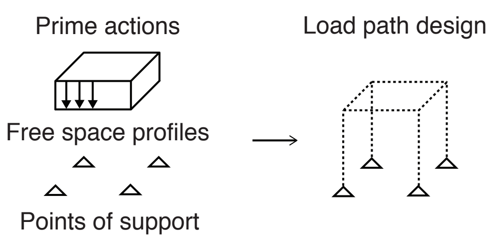


Figure 13.1: Load path design

These load paths are directly caused by the prime actions, by-passing the free space profiles, and borne by the available points of support.

Load path design during the creation phase of conceptual design is the process of determination and optimisation of the load paths, in close collaboration with all the influential participating disciplines.

13.2.2 Truss-analogy in load path design

Modelling bending action in more complex forms can be axial forces-based with the help of a truss-analogy, as shown in figure 13.2.

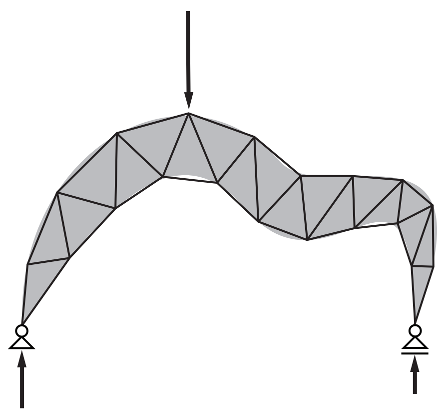


Figure 13.2: Truss-analogy in load path design

The complex form can be filled out by a statically determinate truss configuration, two-dimensional or when appropriate three-dimensional. The load distribution in the truss can then easily be calculated using the method of sections. The strength-based dimensioning results in sectional properties and corresponding construction heights. The rigidity of the truss can further be optimised with a minimal potential energy-based analysis.

13.2.3 Modelling the system decomposition

Getting a grip on possible three-dimensional load distribution effects in an early stage of the conceptual design process requires retention of three-dimensional effects during decomposition of the three-dimensional system in two-dimensional subsystems. This system decomposition can be principally done either by defining the compatibility functions between the subsystems; or by separating the three-dimensional effect as shown in figure 13.3.

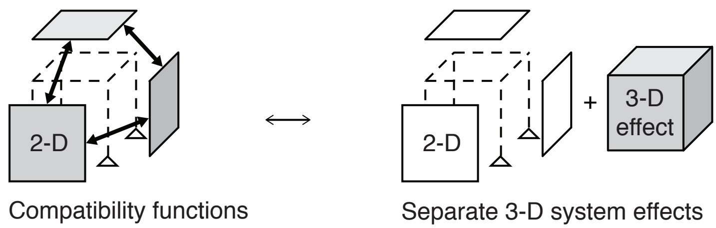


Figure 13.3: Modelling the system effects

Defining the compatibility functions to such an extent that they can be used for a neat quantification of the three-dimensional load distribution is too complex and time-consuming for a conceptual dimensioning. Therefore, qualification and approximate quantification of a separated three-dimensional effect is the remaining feasible option.

For example, the three-dimensional system effect of a double track truss bridge can be separated by defining the overall torsion due to one track loading, with the help of basic applied mechanics.

13.2.4 Dimensioning routine of the load path

The load distribution on the system, through the subsystems into the elements can best be determined on subsystem level with design approximations of the load distribution in basic structural forms. Possible overall three-dimensional system effects then have to be separately evaluated, estimated and added to the load distribution of the subsystem. The result is an approximate determination of the forces in each individual element.

With these forces, the dimensions of the individual elements can be determined by means of design approximations of the load-carrying capacity of elements with regard to sectional strength and element stability. The dimensioning routine, in conformity with figure 12.3 on page 130, is shown in figure 13.4.

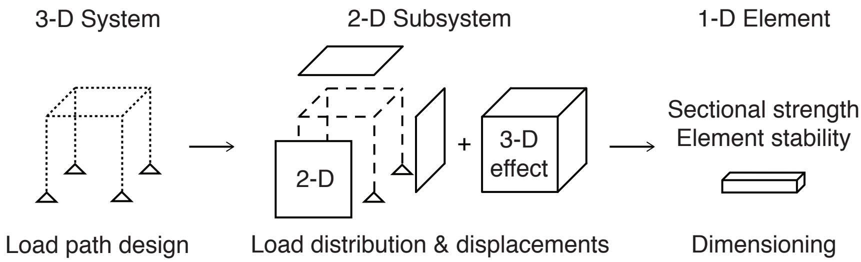


Figure 13.4: Dimensioning routine

This determination of the approximated capacity of the elements, the subsystems and the system is a reversed flow resulting in a dimensioning of the whole structural system.

13.3 Conceptual structural design

13.3.1 Principal details

The materialisation of the structural form is not only dependent on the choice of material and corresponding dimensioning, but also on the design of influential architectural and cost dictating details of the structure, the so-called "principal details" as shown in figure 13.5.

The design approach of these often three-dimensional details is identical to that of the structural system. The choice of material and the matching dimensions of the principal detail can influence the foregoing qualification process with regard to the field of application and characteristics of the chosen structural form; then, an optimisation loop is applicable.

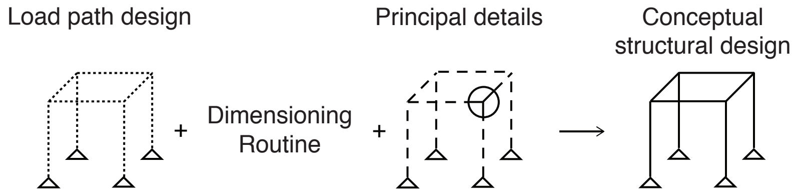


Figure 13.5: Conceptual structural design

13.3.2 System configuration

Independent of requirements and performance/cost optimisation of the conceptual design, structural design should keep the following more generic rules in mind with regard to a simple and clear structural system configuration:

Structural economy Members subjected to axial compression or tension are relatively stronger than other ways of arranging material and generally more economical than those designed for pure bending.

Structural safety In a simple and clear structure, which is rigorously functional, the transmission of forces in the whole structure and in each of the elements will be simple, clear, and without twist.

Instinctive security A structure with few and strong elements always gives an impression of ease and security.

13.3.3 Material properties of conceptual design

In conceptual design, clear and concise material properties are strived for. The strength of structural elements loaded by axial forces or bending moments, is based on the material-dependent normal stress strength f_d. For concrete and its reinforcement, the design material strength f_c respectively f_s, incorporates a material safety. For structural steel, the design material strength equals the yielding stress f_y.

Other material properties such as shear strength f_v, normal stiffness E

and shear stiffness G, can be expressed in this basic strength property f_d or are constant.

The material properties are based upon the applicable Eurocodes for concrete [15] and structural steel [17].

Elementary material properties for the conceptual structural design of reinforced concrete and steel structures are listed in table 13.1.

Material properties for conceptual design		
Property:	Reinforced concrete	Structural steel
Normal stress strength f_d $\left[\frac{\text{N}}{\text{mm}^2}\right]$	$f_c = \dfrac{\text{Concrete grade}}{1.5}$ $f_s = 435$	$f_y =$ Steel grade
Shear stress strength f_v $\left[\frac{\text{N}}{\text{mm}^2}\right]$	$f_v = 0.2 f_c$	$f_v = \dfrac{f_y}{\sqrt{3}}$
Normal stiffness modulus E $\left[\frac{\text{N}}{\text{mm}^2}\right]$	$E_c = \dfrac{f_c}{\frac{1.75}{1000}}$	$E = 2.1 \cdot 10^5$
Shear stiffness modulus G $\left[\frac{\text{N}}{\text{mm}^2}\right]$	$G_c = \dfrac{f_c}{\frac{1.75}{1000} \cdot 2.4}$	$G = 8.1 \cdot 10^4$

Table 13.1: Material properties for conceptual design

Normal stress strength for reinforced concrete Common material grades from conventional to high strength, and their normal stress strengths f_c, are as follows:

Concrete grade	f_c $\left[\frac{\mathrm{N}}{\mathrm{mm}^2}\right]$
C30	20
C45	30
C60	40
C90	60

Higher concrete grades offer increasing strength and corresponding increasing stiffness in order to reduce structural dimensions and increase available space. Furthermore, a shorter cure time allows for quick removal of formwork, and subsequently, putting it into use.

Normal stress strength for structural steel Common material grades from conventional to high strength, and their normal stress strengths f_y, are as follows:

Structural steel grade	f_y $\left[\frac{\mathrm{N}}{\mathrm{mm}^2}\right]$
S235	235
S355	355
S460	460
S690	690

Higher steel grades offer increasing strength in order to reduce strength-driven structural dimensions.

Shear stress strength for reinforced concrete The shear stress strength f_v for most materials is linearly proportional to the normal stress strength f_d and considerably lower. For reinforced concrete, the shear stress strength is approximately 20% of the normal stress strength: $f_v = 0.2 f_c$.

Shear stress strength for structural steel The shear stress strength for structural steel can be derived from the Huber Hencky and Von Mises yield criterion:

$$\sqrt{\sigma^2 + 3\tau^2} \leq f_y \Rightarrow \tau \leq \frac{f_y}{\sqrt{3}} = f_v \tag{13.1}$$

Normal stiffness modulus for reinforced concrete For concrete, the normal stiffness modulus or Young's modulus E_c is dependent on the material grade and its corresponding normal stress strength f_c. This modulus E_c can be derived from the simplified bi-linear modelled stress strain relationship of concrete with an elastic strain limit of 1.75‰ [15], combined with the practical usage for the modulus E_c, including the influence of time related effects [1]:

$$E_c = \frac{f_c}{\frac{1.75}{1000}} \tag{13.2}$$

Normal stiffness modulus for structural steel For steel, the normal stiffness modulus or Young's modulus E has a constant value of $2.1 \cdot 10^5 \, \frac{\mathrm{N}}{\mathrm{mm}^2}$.

Shear stiffness modulus for reinforced concrete For uncracked concrete the shear modulus G_c is dependent on the material grade and its corresponding normal stress strength f_c:

$$G_c = \frac{E_c}{2\,(1+\nu)} = \frac{\frac{f_c}{\frac{1.75}{1000}}}{2\,(1+0.2)} = \frac{f_c}{\frac{1.75}{1000} \cdot 2.4} \tag{13.3}$$

Shear stiffness modulus for structural steel For steel the shear modulus G has a constant value of approximately $8.1 \cdot 10^4 \, \frac{\mathrm{N}}{\mathrm{mm}^2}$:

$$G = \frac{E}{2\,(1+\nu)} = \frac{2.1 \cdot 10^5}{2\,(1+0.3)} \approx 8.1 \cdot 10^4 \, \frac{\mathrm{N}}{\mathrm{mm}^2} \tag{13.4}$$

Chapter 14

Fundamental parameters of load distribution

14.1 Conceptual design on subsystem level

14.1.1 Load distribution phase of the conceptual design

The load distribution phase starts with materialised structural schemes and ends with element forces and displacements as shown in figure 12.3 on page 130.

14.1.2 Load distribution

The distribution of the loads within a two-dimensional subsystem is dependent on the geometrical characteristics of the subsystem. For the two-dimensional basic structural forms as defined in figure 9.3 on page 94, the approximated load distribution for conceptual design has to be determined.

In search of a basic set of knowledge and approximation parameters for conceptual structural design in general, and load distribution in particular, the following principles are employed:

Applied mechanics-based to secure insight and durability. Basic applied mechanics textbook material, being established knowledge, is not included in the bibliography.

Professional practice factors for improvement of approximation accuracy, including reference in the bibliography or verification in this textbook.

Emphasis on deformation-driven aspects in accordance with the force and deformation-driven parameters as shown in figure 12.2 on page 124.

Performance-based and well-balanced with respect to depth and breadth of underlying knowledge and applicability for conceptual design.

14.1.3 Parallel load distribution

Statically determinate structures An assembly of directly connected elements is defined "statically determinate" when the static equilibrium equations are sufficient for determining the internal forces and reactions: $\Sigma H = 0$, $\Sigma V = 0$, and $\Sigma M = 0$.

A common statically determinate structure is a simply supported beam as shown in figure 14.1.

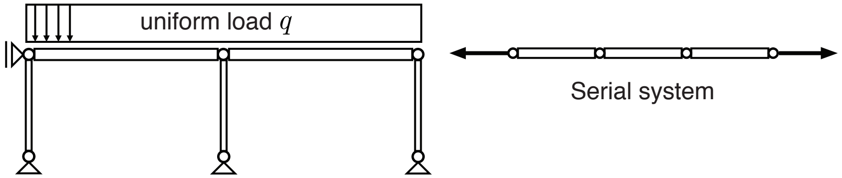


Figure 14.1: Statically determinate structure

Practically, a statically determinate structure has just enough (internal) members and (external) supports to secure structural form and fixation, in analogy to a serial system.

As a consequence, the behaviour is completely force-driven and failure of only one random element immediately leads to failure of the structure.

Statically indeterminate structures An assembly of directly connected elements is defined "statically indeterminate" when the static equilibrium equations are not sufficient for determining the internal forces and reactions on the structure.

A common statically indeterminate structure is a continuous beam as shown in figure 14.2.

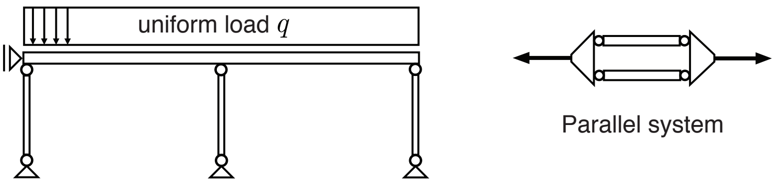


Figure 14.2: Statically indeterminate structure

Practically, a statically indeterminate structure has more (internal) members and/or (external) supports than necessary for structural integrity, in analogy to a parallel system.

As a consequence, the behaviour can be deformation-driven, besides force-driven. A difference in stiffness will influence the load distribution within the parallel system.

Parallel load distribution can be effectively utilised with regard to the following:

Redundancy Design a second method of support to ensure that the forces are dispersed elsewhere when a vital structural component can give way.

Optimisation Optimisation through redistribution by altering the stiffness of the individual elements.

Induced deformation Deformation of a primarily statically indeterminate structure can cause unwanted loading of secondary structures.

14.2 Load distribution in basic structural forms

14.2.1 Load distribution in a frame

Frames consist of bending beams and supporting columns. The local load distribution of a beam consisting of one element equals the global load distribution and is therefore subjected to bending and shear. The supporting columns are subjected to compression.

The effects of bending moments versus axial forces can be analysed with a statically indeterminate frame structure, consisting of a combined cantilever beam and supporting column as shown in figure 14.3.

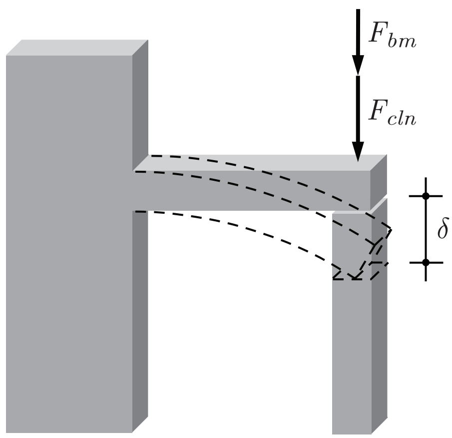


Figure 14.3: Parallel bending and axial force

Axial versus bending strength The strength of the individual cantilever beam, the individual column and the ratio between both, can be determined:

$$\sigma_{bm} = \frac{M}{W_{bm}} = \frac{F_{bm} \cdot l_{bm}}{\frac{1}{6} \cdot b_{bm} \cdot h_{bm}^2} \Rightarrow F_{bm} = \frac{\sigma_{bm} \cdot b_{bm} \cdot h_{bm}^2}{6 \cdot l_{bm}} \tag{14.1}$$

$$\sigma_{cln} = \frac{N}{A_{cln}} = \frac{F_{cln}}{b_{cln} \cdot h_{cln}} \Rightarrow F_{cln} = \sigma_{cln} \cdot b_{cln} \cdot h_{cln} \tag{14.2}$$

$$\frac{F_{cln}}{F_{bm}} = \frac{\sigma_{cln} \cdot b_{cln} \cdot h_{cln} \cdot 6 \cdot l_{bm}}{\sigma_{bm} \cdot b_{bm} \cdot h_{bm}^2} \tag{14.3}$$

When both beam and column are equally homogeneous materialised and dimensioned $\square$ $300 \cdot 300$ mm^2 with lengths of 3 m:

$$\frac{F_{cln}}{F_{bm}} = \frac{\sigma \cdot b \cdot h \cdot 6 \cdot l}{\sigma \cdot b \cdot h^2} = \frac{6 \cdot l}{h} = \frac{6 \cdot 3}{0.3} = 60 \tag{14.4}$$

The axial strength exceeds the bending strength by far. In general, members subjected to axial compression, or tension, are relatively stronger than other ways of arranging material.

Axial versus bending stiffness The distribution of a load in a statically indeterminate structure, will be on the basis of the ratio between the axial stiffness and the bending stiffness:

$$F = F_{cln} + F_{bm} \tag{14.5}$$

$$\delta = \frac{F_{cln} \cdot l_{cln}}{EA_{cln}} = \frac{F_{bm} \cdot l_{bm}^3}{3 \cdot EI_{bm}} \Rightarrow \frac{F_{cln}}{F_{bm}} = \frac{EA_{cln} \cdot l_{bm}^3}{3 \cdot EI_{bm} \cdot l_{cln}} \tag{14.6}$$

When both beam and column are equally homogeneous materialised and dimensioned $\square$ $300 \cdot 300$ mm^2 with lengths of 3 m:

$$\frac{F_{cln}}{F_{bm}} = \frac{E \cdot b \cdot h \cdot l^3}{3 \cdot E \cdot \frac{1}{12} \cdot b \cdot h^3 \cdot l} = \frac{4 \cdot l^2}{h^2} = \frac{4 \cdot 3^2}{0.3^2} = 400 \tag{14.7}$$

Because axial stiffness mostly exceeds the bending stiffness by far, a combined load distribution will result in a domination of the axial forces.

14.2.2 Load distribution in a floor slab

The local load distribution of a floor slab equals the global load distribution and is therefore subjected to bending and shear. A two-way spanning floor slab, whether concrete slab or orthotropic steel deck, is subjected to bending and shear in both spanning directions.

The effect of parallel bending can be analysed with a homogeneous two-way spanning floor slab with a simplified load distribution as shown in figure 14.4.

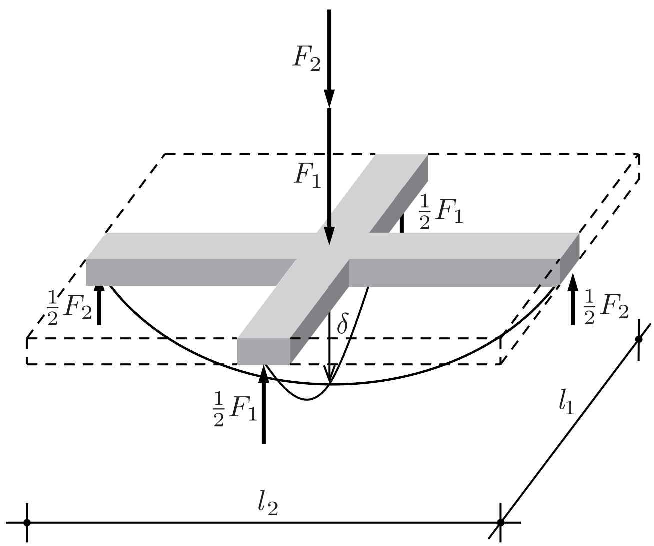


Figure 14.4: Load distribution in a two-way spanning floor slab

The load distribution in statically indeterminate structural systems with parallel bending is dependent on the length of the load paths; the effectiveness of the load paths is linearly proportional to the difference in stiffness.

An approximation of the distribution of the loads in the short span (F_1) respectively long span (F_2) of the floor slab:

$$F = F_1 + F_2 \tag{14.8}$$

$$\delta = \frac{F_1 l_1^3}{48EI} = \frac{F_2 l_2^3}{48EI} \Rightarrow \frac{F_1}{F_2} = \left(\frac{l_2}{l_1}\right)^3 \tag{14.9}$$

For example, a rectangular two-way spanning slab with spans of l respectively $2l$:

$$\frac{F_1}{F_2} = \left(\frac{2l}{l}\right)^3 = 8 \tag{14.10}$$

The short load path is eight times as effective as the long load path, due to the proportional difference in stiffness.

The outcome is directly appropriate for a slab of reinforced concrete. The usually low reinforcement percentage has after all a negligible influence on the stiffness $E_c I$ of a reinforced concrete slab.

14.2.3 Load distribution in a cable-stayed beam

A supporting cable diminishes the supported length and corresponding local load distribution of beams; by each half, strength - the effect of section modulus W - is quadrupled and stiffness - the effect of moment of inertia I - is eight-fold. The required section modulus W, and moment of inertia I, are worked out in subsection 15.4.3.

The effectiveness of the local load distribution in a cable-stayed statically indeterminate continuous beam is dependent on the stiffness of the cable. The cross-sectional area of the cable is therefore governed by both tension strength and supporting stiffness.

In such a statically indeterminate structure, optimisation by redistribution within the deformation-driven parallel load distribution can be utilised. An example of such an optimisation by a controlled redistribution is the cable-stayed continuous beam as shown in figure 14.5.

The effectiveness of the beam is governed by the stiffness of the cable; the strength of the cable has no influence at all with regard to the redistribution.

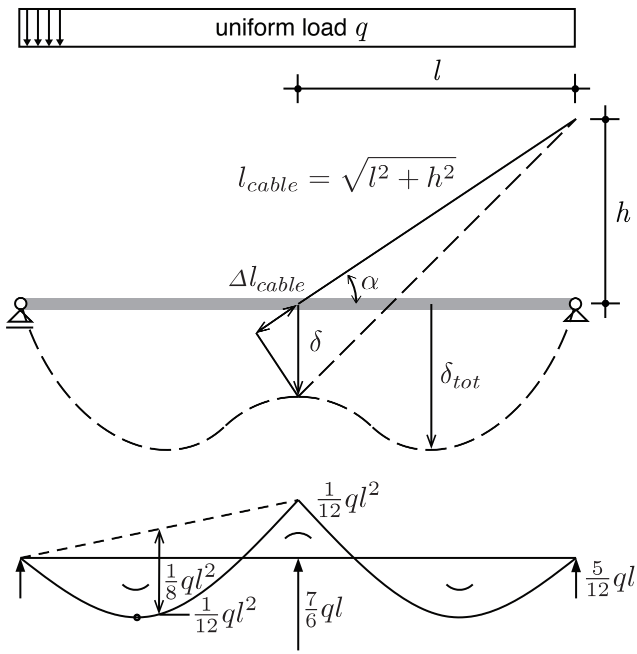


Figure 14.5: Redistribution in a cable-stayed beam

Regardless of the required cross-sectional area of the cable with respect to the strived-after redistribution, the structural strength and serviceability always remain prerequisite requirements.

Optimisation An optimisation of the moment distribution in the continuous beam can be accomplished by sagging the middle support:

$$\delta = \frac{\frac{5}{12}ql \cdot l^3}{3EI} - \frac{ql^4}{8EI} = \frac{ql^4}{72EI_{bm}} \tag{14.11}$$

The corresponding recommended cross-sectional area of the cable with respect to the stiffness (ULS) amounts to:

$$\delta = \frac{ql^4}{72EI_{bm}} = \frac{\Delta l_{cable}}{sin\alpha} = \frac{N_{cable} \cdot l_{cable}}{EA_{cable} \cdot sin\alpha} = \frac{\frac{\frac{7}{6}ql}{\sin\alpha} \cdot \sqrt{l^2+h^2}}{EA_{cable} \cdot \sin\alpha}$$
$$\Rightarrow A_{cable,\,stiffness\,ULS} = \frac{84 \cdot I_{bm} \cdot \sqrt{l^2+h^2}}{l^3 \cdot \sin^2\alpha} \cdot \frac{E_{bm}}{E_{cable}} \quad (14.12)$$

Structural strength The required cross-sectional area of the cable with respect to the strength (ULS) amounts to:

$$A_{cable,\,strength\,ULS} \geq \frac{\frac{7}{6}q_d \cdot l}{\sin\alpha \cdot f_y} \quad (14.13)$$

No redistribution and both spans under extreme loading amounts to:

$$R_{mid} = \tfrac{5}{4}q_d l \neq \tfrac{7}{6}q_d l \quad (14.14)$$

Using high-strength steel cables, the required stiffness usually will be decisive due to the same Young's modulus for all steel grades.

Serviceability The required cross-sectional area of the cable with respect to the displacements (SLS) amounts to:

$$\delta_{tot} \approx \frac{1}{150} \cdot \frac{q_k l^4}{EI_{bm}} + \frac{1}{2} \cdot \frac{\frac{\frac{7}{6}q_k l}{\sin\alpha} \cdot \sqrt{l^2+h^2}}{EA_{cable} \cdot \sin\alpha} \leq \frac{l}{250}$$
$$\Rightarrow A_{cable,\,stiffness\,SLS} \geq \frac{146 \cdot q_k \cdot \sqrt{l^2+h^2}}{E_{cable} \cdot \sin^2\alpha \cdot \left(1 - \frac{5q_k l^3}{3EI_{bm}}\right)} \quad (14.15)$$

Concerning the displacements of the beam, the stiffness of the beam I_{bm} influences the required stiffness of the cable A_{cable}.

14.2.4 Load distribution in a truss

Every member of a truss is in pure compression or pure tension. The main contribution to the strength and stiffness is provided by the triangulation. Shear and bending moments, and other more complex stresses are all practically zero. This makes trusses relatively strong and stiff.

The load distribution in a simply supported simplified parallel chord truss is shown in figure 14.6.

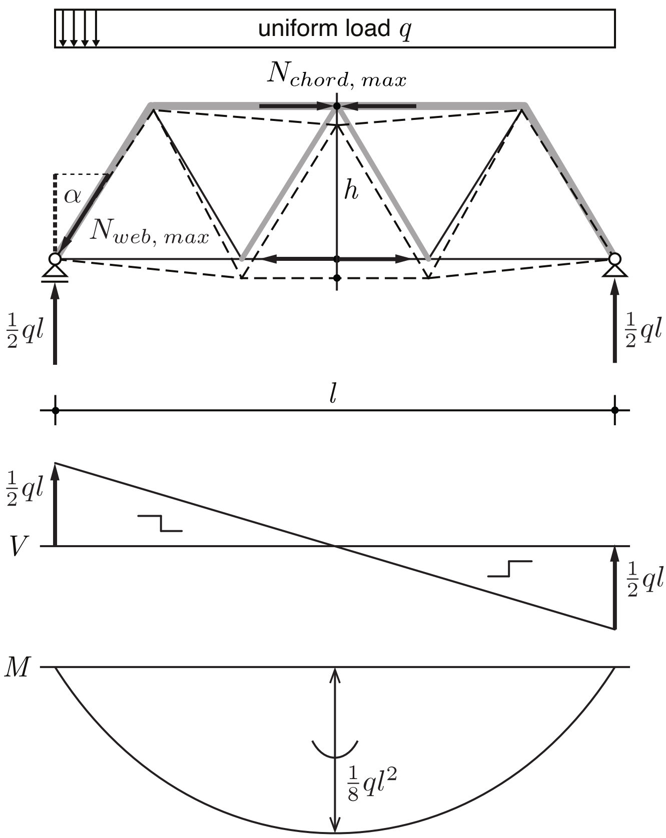


Figure 14.6: Load distribution in a truss

The global load distribution consists of bending moments and shear forces. The local load distribution within the truss consists completely of axial compression and tension forces in the individual truss members.

A simply supported truss with a uniform load results in the following maximum axial forces in the chord and the web members:

$$N_{chord,\,max} = \frac{M}{h} = \frac{\frac{1}{8}q_d l^2}{h} \tag{14.16}$$

$$N_{web,\,max} = \frac{V}{\cos\alpha} = \frac{\frac{1}{2}q_d l}{\cos\alpha} \tag{14.17}$$

A non-uniform load still results in local axial forces only, with the possibility of a change of compression and tension in the web members near the shear transition point.

The global stiffness of the truss can be approximated as follows:

$$I_{truss} \approx 0.8 \cdot 2 \cdot A_{chord} \cdot \left(\tfrac{1}{2}h\right)^2 \tag{14.18}$$

The moment of inertia I_{truss} has an approximated loss of 0.2 because of global shear deformation of the truss due to elastic deformation of the individual web members.

During conceptual design this stiffness parameter I_{truss} can be applied to determine the global bending deflection (SLS):

$$\delta = \frac{5}{384} \cdot \frac{q_k l^4}{EI_{truss}} \qquad \text{and } I_{truss} \approx 0.8 \cdot 2 \cdot A_{chord} \cdot \left(\tfrac{1}{2}h\right)^2 \tag{14.19}$$

Furthermore, this stiffness parameter I_{truss} can be applied to determine the global buckling strength (ULS) as discussed in subsection 15.3.3.

14.2.5 Load distribution in an arch

The form of an arch is more or less parabolic and usually, the horizontal reaction forces are resisted by a tension rod. Uniformly loaded, the entire arch is in compression and has little bending moments and shear; this makes arches relatively strong.

The load distribution in a simply supported arch is shown in figure 14.7.

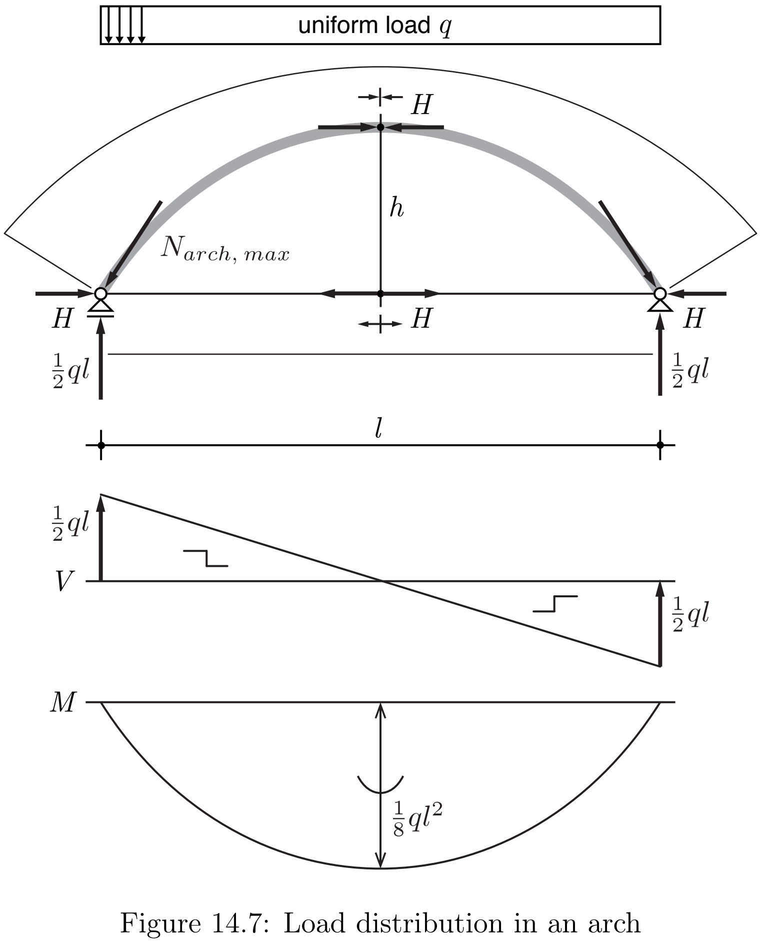


Figure 14.7: Load distribution in an arch

The subsystem as a whole is simply supported and therefore globally statically determinate. The corresponding global load distribution consists of bending moments and shear forces.

The tied arch, consisting of an arch and a tie-rod, is locally statically determinate. When loaded with a uniform load per m^1 span length, the corresponding local load distribution within a tied parabolic arch consists solely of axial compression in the arch and axial tension in the tie-rod.

A uniform load results in the following maximum axial forces in the tie-rod and the arch:

$$H = \frac{M}{h} = \frac{\frac{1}{8} q_d l^2}{h} \tag{14.20}$$

$$N_{arch,\,max} = \sqrt{\left(\tfrac{1}{2} q_d l\right)^2 + H^2} \tag{14.21}$$

When the uniform load is dominant, the arch more or less follows the line of compression. Loaded with a uniform load per m^1 span length it equals a parabola. Loaded with a uniform load per m^1 arch length it equals a hyperbola (hyperbolic cosine function), the so-called “catenary”.

A non-uniform load results in additional bending of the arch, besides the axial compression. So-called “false” arches, where local bending is dominant over compression, are unfavourable:

- They have circular instead of parabolic arches.
- And/or they have highly concentrated loads, instead of more or less uniform loads.

A practical design value for the maximum bending moment within the arch can be obtained by half the variable uniform loading as shown in figure 14.8.

The so-obtained practical design value for the maximum bending moment within the arch amounts to:

$$M_{arch,\,max} \approx \tfrac{1}{64} q_d l^2 \tag{14.22}$$

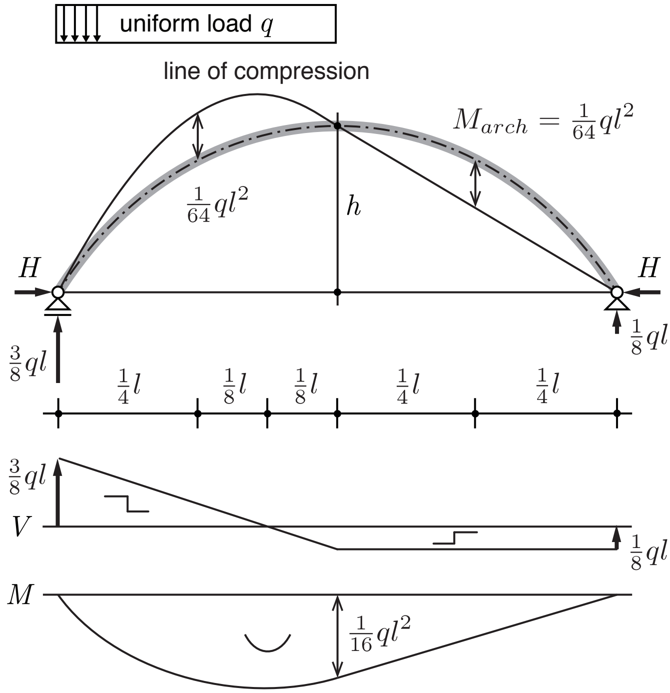


Figure 14.8: Bending moments in an arch

The corresponding variable axial forces in tie-rod and arch amount to:

$$H = \frac{M}{h} = \frac{\frac{1}{16}q_d l^2}{h} \tag{14.23}$$

$$N_{arch} = \sqrt{\left(\tfrac{3}{8}q_d l\right)^2 + H^2} \tag{14.24}$$

14.2.6 Load distribution in a shear wall

A most common application of shear walls is securing the global stability of a system against wind actions and global initial sway imperfections. For non-sway buildings these can be in the form of concrete shear walls or steel wind bracing.

The load distribution of a stocky shear wall consists of dominant shear in combination with minor bending.

Concrete shear wall The load distribution within the shear wall takes place on element level. For conceptual design, the approximate calculation of shear strength and deformation in subsection 15.2.1 respectively 15.2.2 is applicable.

Steel wind bracing The load distribution within the steel wind bracing takes place on subsystem level as discussed in subsection 14.2.4 on the load distribution in trusses.

Because of the stocky dimensions, shear is governing. For conceptual design, the approximate calculation of shear strength and deformation in subsection 15.2.3 on the shear deformation in cantilevered trusses is applicable.

14.3 Parallel load distribution on detailed level

The distribution of the load in detailed design will be on the level of stress and strain. Due to the high degree of complexity of common three-dimensional details, these details reveal a corresponding high degree of statically indeterminacy. The stress distribution within the details is proportional to the stiffness distribution, which is inversely proportional to the strains.

Conceptual design of principal details can be interpreted as the design of a small-scaled system. Thus, an identical methodical approach such as for system design, with a decomposition in subsystems and elements, is appropriate.

Because both axial and shear stiffness mostly exceed the bending stiffness by far, a combined load distribution will result in a domination of the axial and shear forces.

A study on the optimum design of a stiffening plate in a beam-to-column joint shows the difference between bending and combined axial/shear deformation. In this specific case, it is by far preferable to connect the plate only to the loaded flange, thus avoiding high fitting costs of each individual stiffener. Both options are given in the same figure 14.9.

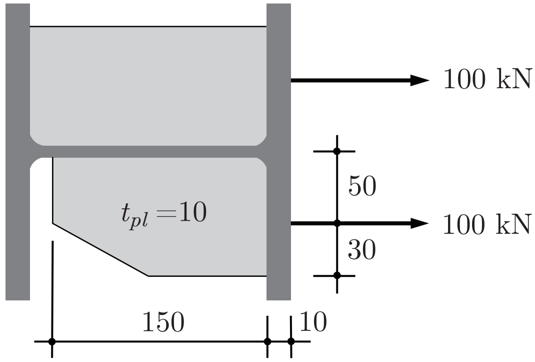


Figure 14.9: Stiffener plate in a beam-to-column joint

When the contribution of the flange turns out to be a negligible factor, the stiffener plate only needs to be connected to the loaded flange for force transition.

By calculating the deformation of each individual (parallel) failure mechanism, the decisive mechanism can be determined under the condition that there is an order of magnitude difference in deformation. When the deformations lie in the same range, an accurate finite elements calculation has to be executed.

In this specific case the following failure mechanisms are applicable:

- Bending of the flange.
- Axial force on the stiffener.
- Shear force on the stiffener.

Bending of the flange The bending deformation of the flange is shown in figure 14.10.

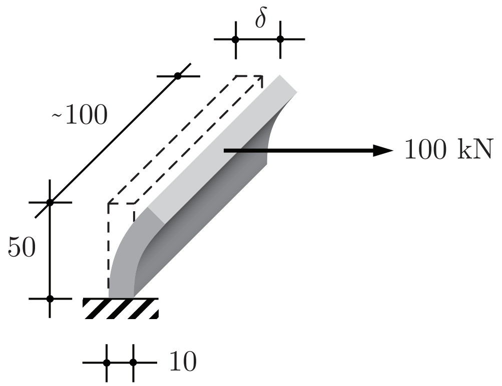


Figure 14.10: Bending deformation of the flange

The displacement, due to bending deformation of the flange, amounts to:

$$\delta = \frac{Fl^3}{3EI} = \frac{100 \cdot 10^3 \cdot 50^3}{3 \cdot 2.1 \cdot 10^5 \cdot \frac{1}{12} \cdot 100 \cdot 10^3} = 2.38 \text{ mm} \tag{14.25}$$

Axial force on the stiffener The axial deformation of the stiffener is shown in figure 14.11.

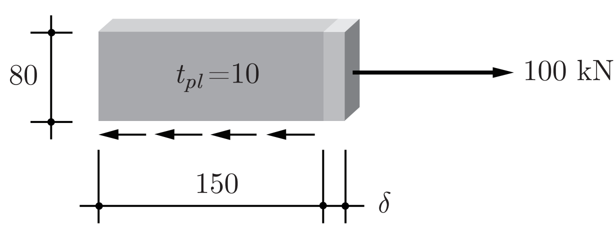


Figure 14.11: Axial deformation of the stiffener

The displacement, due to axial deformation of the stiffener, amounts to:

$$\sigma = \varepsilon \cdot E \Rightarrow \frac{N}{A} = \frac{\delta}{l} \cdot E$$
$$\Rightarrow \delta = \frac{N \cdot l}{E \cdot A} = \frac{\frac{1}{2} \cdot 100 \cdot 10^3 \cdot 150}{2.1 \cdot 10^5 \cdot 80 \cdot 10} = 0.04 \text{ mm} \quad (14.26)$$

Shear force on the stiffener The shear deformation of the stiffener is shown in figure 14.12.

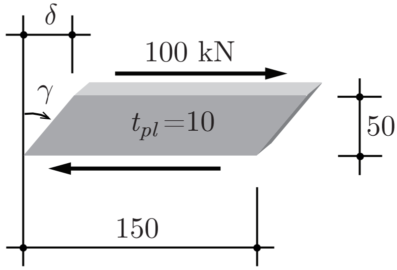


Figure 14.12: Shear deformation of the stiffener

The displacement, due to shear deformation of the stiffener, amounts to:

$$\tau = \gamma \cdot G \Rightarrow \frac{V}{A_v} = \frac{\delta}{h} \cdot G$$
$$\Rightarrow \delta = \frac{V \cdot h}{G \cdot A_v} = \frac{100 \cdot 10^3 \cdot 50}{8.1 \cdot 10^4 \cdot 150 \cdot 10} = 0.04 \text{ mm} \quad (14.27)$$

The deformation due to bending of the flange is an order of magnitude higher; consequently, the contribution of the flange is negligible.

14.4 Induced deformation

14.4.1 Principle of induced deformation

An induced deformation may only occur in a statically indeterminate and thus deformation-driven structure.

Problem of induced deformation Within a statically indeterminate subsystem, a stiff element can cause an induced deformation of an adjacent relatively flexible element. The same can occur on a higher system level with an induced deformation of a relative flexible subsystem within a statically indeterminate system.

As a result this induced deformation can cause a structural failure of the concerning relatively flexible element, respectively subsystem.

Solution to the problem of induced deformation Failure is commonly opposed by an increase of the cross-sectional strength and accompanying stiffness. When the stiffness and corresponding structural action increases more than the strength of the structure, the solution is to reduce the stiffness.

14.4.2 Induced deformation on subsystem level

An example of induced deformation on subsystem level is a concrete floor slab on a torsional clamped steel beam as shown in figure 14.13.

When the problem of possible torsional failure is force-driven, the only solution is the necessary increase of the torsional strength to oppose structural failure.

However, when the problem of possible torsional failure is deformation-driven, the solution is to reduce the torsional stiffness to prevent structural failure.

14.4.3 Induced deformation on system level

Within a statically indeterminate system a stiff subsystem can cause an induced deformation of a relatively flexible subsystem.

A common application is, for example, a bridge with a top lateral bracing as shown in figure 14.14. The effect of the induced deformation, however, also appears in other types of bending systems.

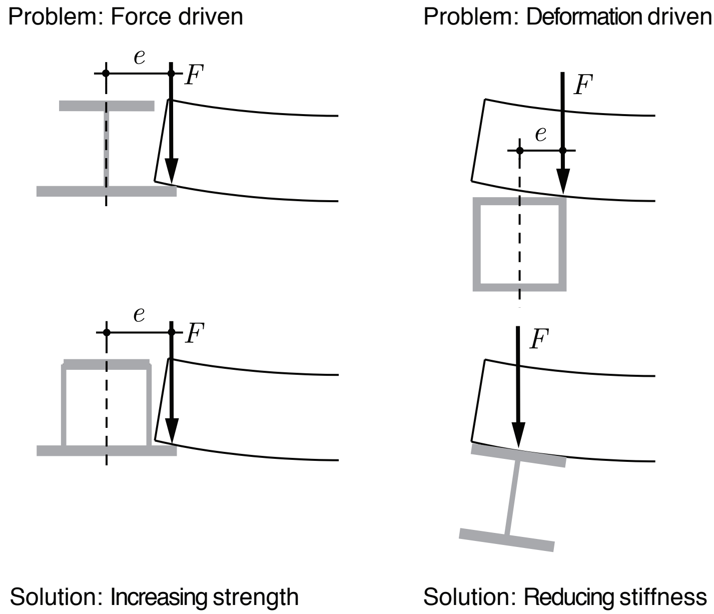


Figure 14.13: Induced deformation torsional clamped beam

The position of the neutral axis is dependent on the difference in stiffness between the top lateral bracing and the bottom deck.

The shortening of the neutral axis $\Delta l_{neutral\,axis}$ due to the pure mathematical deflection is a negligible factor with regard to the mechanical deformations $\Delta l_{bottom\,edge}$ and $\Delta l_{top\,edge}$ related to the cross-sectional and material stiffness parameters.

The lengthening of the bottom edge $\Delta l_{bottom\,edge}$ gives an induced displacement of the bearings. The shortening of the top edge $\Delta l_{top\,edge}$ gives an induced deformation of the top lateral bracing.

Triangular bracing When the system as a whole is statically determinate, induced deformations cannot occur. The matching deformation of a triangular hinged top lateral bracing is shown in figure 14.14.

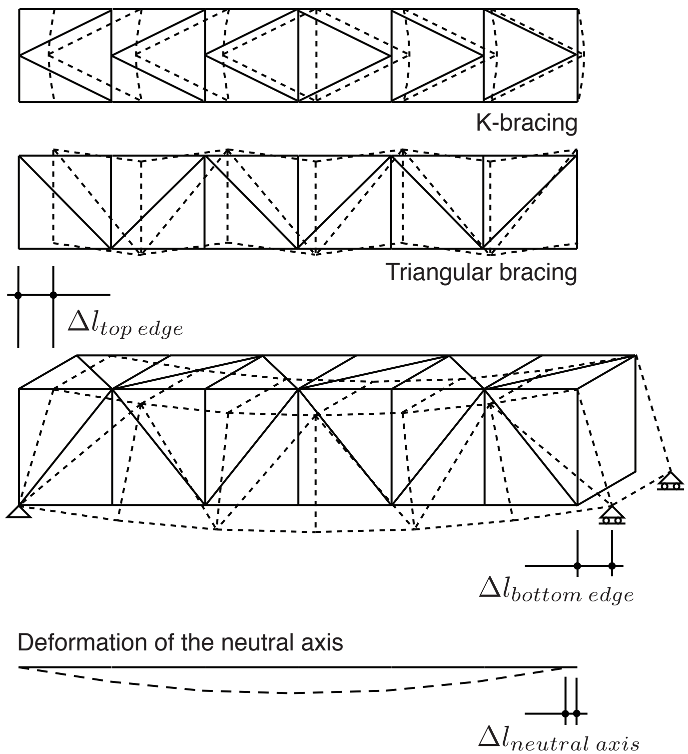


Figure 14.14: Induced deformation in a truss bridge

Normally however, the top chord is continuous over the truss span length, making the structure statically indeterminate and inducing a deformation of the top lateral bracing. When designing a stiff truss bracing, the compressed members will probably fail due to this induced deformation.

K-bracing Therefore, a more flexible K-bracing is an effective design to reduce the overall bracing stiffness by introducing flexible bending elements among the stiff axial elements, thus reducing the load on all bracing elements. The deformation of the top lateral K-bracing is shown in figure 14.14.

Chapter 15

Fundamental parameters of failure mechanisms

15.1 Conceptual design on element level

15.1.1 Dimensioning phase of the conceptual design

The dimensioning phase starts with element forces and displacements, and ends with the required section properties for conceptual design, as shown in figure 12.3 on page 130.

15.1.2 Failure mechanisms

Failure mechanisms typically include structural failures in the Ultimate Limit State (ULS) and serviceability failures in the Serviceability Limit State (SLS). Corresponding minimum required section properties have to be defined to comply with functional requirements and contractual conditions.

In search of a basic set of knowledge and approximation parameters for conceptual structural design in general, and failure mechanisms in particular, the following principles are employed:

Applied mechanics-based to secure insight and durability. Basic applied mechanics textbook material is considered established knowledge and is therefore not included in the bibliography.

Professional practice factors for improvement of approximation accuracy, including reference in the bibliography or verification in this textbook.

Emphasis on deformation-driven aspects in accordance with the force and deformation-driven parameters as shown in figure 12.2 on page 124.

Performance-based and well-balanced with respect to depth and breadth of underlying knowledge and applicability for conceptual design.

The following appropriate applied mechanics-based knowledge and approximation parameters are discussed as listed in table 15.1.

Conceptual design parameters of failure mechanisms		
Shear	Stability	Bending & axial
Shear strength	Principle	Concrete elements
Shear deformation	Euler based	Steel elements
Stocky beam	Buckling	Foundations

Table 15.1: Conceptual design parameters of failure mechanisms

The material properties are based upon the applicable Eurocodes for concrete [15] and structural steel [17].

15.2 Shear

15.2.1 Shear strength

For a reliable determination of the shear strength it is essential to comprehend the two major parameters f_v and A_v as shown in figure 15.1.

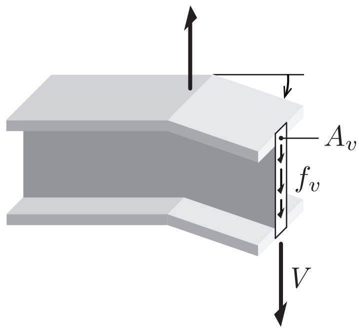


Figure 15.1: Shear strength

Material shear strength f_v The shear strength f_v for most materials is linearly proportional to the axial strength f and considerably lower. For reinforced concrete the shear strength is approximately 20% of the axial strength. Decisive for this global shear strength is the local compressive strength of the concrete diagonals in the truss-like load distribution of the global shear force, provided that the shear reinforcement is sufficient. For structural steel the shear strength is approximately 60% of the axial strength.

Effective shear area A_v The stiffness contribution of the perpendicular to shear force orientated material such as flanges, is negligible with regard to the material parallel to the shear force. Therefore, the shear force will be borne solely by the shear area A_v parallel to this force. Failure will result in rupture of the shear area before the flange material will contribute, resulting in a propagating shear rupture of the whole cross section.

The required effective shear area A_v can be calculated as follows:

$$\frac{V_d}{V_u} = \frac{V_d}{A_v \cdot f_v} \leq 1 \Rightarrow A_v \geq \frac{V_d}{f_v}$$
$$\text{Reinforced concrete } A_v \geq \frac{V_d}{0.2 f_c}, \text{ steel } A_v \geq \frac{V_d}{0.6 f_y} \quad (15.1)$$

For standard slender beams, however, bending strength is mostly decisive over shear strength.

15.2.2 Shear deformation

A beam as shown in figure 15.2 is subjected to a combined bending and shear deformation.

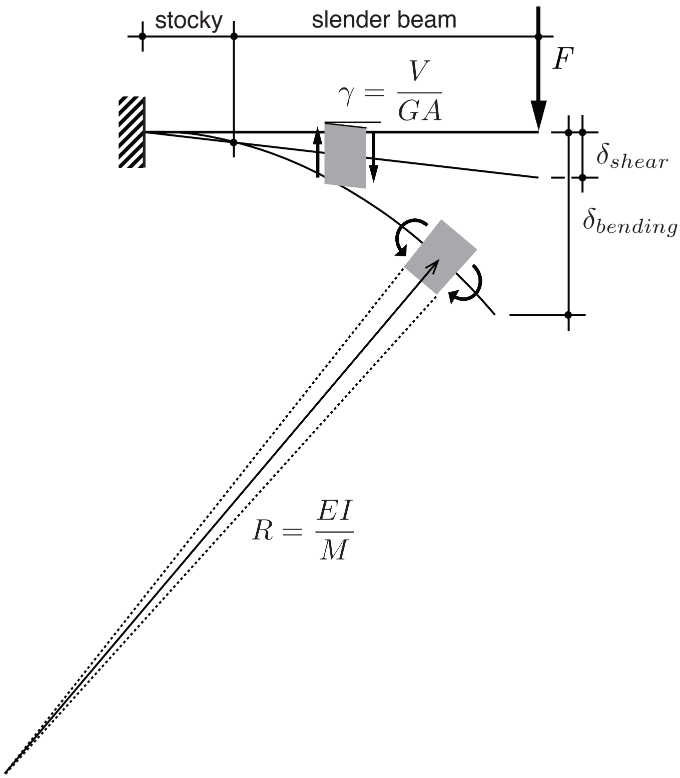


Figure 15.2: Bending and shear deformation

For slender beams the bending deformation is decisive and the shear deformation may be neglected, for stocky beams the shear deformation is decisive and the bending deformation may be neglected.

For a slender beam, the bending deformation is in general:

$$\delta_{bending} = \iint_{l} \frac{M}{EI} \tag{15.2}$$

And in this specific case:

$$\delta_{bending} = \frac{Fl^3}{3EI} \tag{15.3}$$

For a stocky beam, the shear deformation is in general:

$$\delta_{shear} = \int_{l} \frac{V}{GA} \tag{15.4}$$

And in this specific case:

$$\delta_{shear} = \frac{Fl}{GA} \tag{15.5}$$

The transition from “stocky” to “slender” depends on the bending versus shear stiffness of the material. The transition slenderness cannot be determined by an equilibrium of foregoing shear and bending formulae, because the abstraction of bending is based upon slender beam theory and the transition slenderness is beyond the validity range of this theory.

In mechanics-based analyses of the transition slenderness from “stocky” to “slender” of a simply supported beam, the span-over-height ratio $\frac{l}{h}$ varies between 3 and 4. The Dutch National Annex to the Eurocode 2 [16] defines this transition slenderness for simply supported concrete beams explicitly at a span-over-height ratio of $\frac{l}{h} = 3$.

Considering the relatively small difference in $\frac{E}{G} = 2\,(1+\nu)$ between concrete and steel, $2\,(1+0.2) = 2.4$ respectively $2\,(1+0.3) = 2.6$, the corresponding transition slenderness of the two will be approximately similar, resulting in span-over-height ratios $\frac{l}{h}$ as shown in figure 15.3.

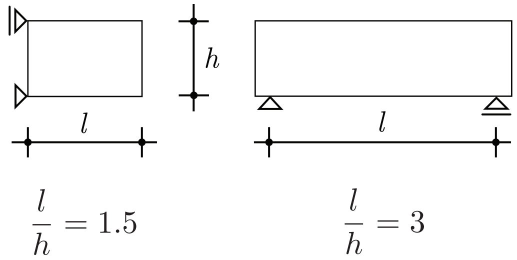


Figure 15.3: Transition slenderness for stocky to slender beams

15.2.3 Shear deformation on subsystem level

Shear deformation on subsystem level occurs in cantilevered truss structures. For a comprehensible conceptual design, the shear contribution on element level has to be appointed. An example of a usual, present-day cantilevered parallel chord building design is shown in figure 15.4.

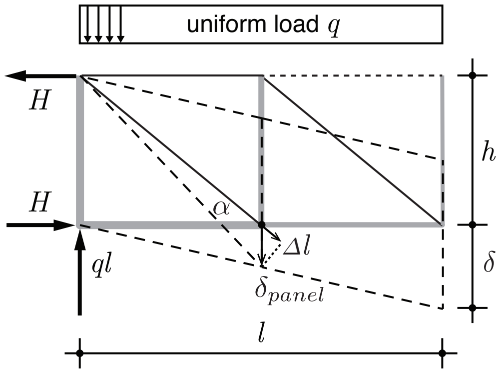


Figure 15.4: Shear deformation in a cantilevered truss

Because of the usually heavy dimensioned floor-carrying chord members in combination with the nearby stocky dimensions of this cantilevered truss, the displacement at the cantilevered end of the truss will be dominated by shear deformation, rather than bending deformation. To realise this, the design in the serviceability limit state should focus on shear deformation and the corresponding required section properties of the web members. The linearity of deformation is based on a strength-optimised design of the web members.

Approximated loads (ULS):

$$N_{chord,\,max} = H = \frac{\frac{1}{2} q_d l^2}{h} \tag{15.6}$$

$$N_{web,\,max} = \frac{q_d \cdot \left(n_{panel} - \frac{1}{2}\right) \cdot l}{\sin\alpha} \tag{15.7}$$

Displacement due to shear deformation (SLS):

$$\begin{aligned} \delta = n_{panel} \cdot \frac{\Delta l}{sin\alpha} &= n_{panel} \cdot \frac{N_{web,\,k} \cdot l_{web}}{EA_{web} \cdot \sin\alpha} \\ &= n_{panel} \cdot \frac{\frac{q_k \cdot \left(n_{panel} - \frac{1}{2}\right) \cdot l}{\sin\alpha} \cdot \sqrt{\left(\frac{l}{n_{panel}}\right)^2 + h^2}}{EA_{web} \cdot \sin\alpha} \leq \delta_{required} \end{aligned} \tag{15.8}$$

Required cross-sectional area with regard to shear deformation (SLS):

$$A_{web,\,max} \geq \frac{\frac{q_k \cdot \left(n_{panel} - \frac{1}{2}\right) \cdot l}{\sin\alpha} \cdot \sqrt{\left(\frac{l}{n_{panel}}\right)^2 + h^2}}{\frac{\delta_{required}}{n_{panel}} \cdot E \cdot \sin\alpha} \tag{15.9}$$

15.3 Stability

15.3.1 Stability of the equilibrium

The basic principles of the stability of the equilibrium can be demonstrated by the uncomplicated structural element as shown in figure 15.5.

A body may be in one of three states of equilibrium: stable, unstable, and neutral. Analysis on the state of stable equilibrium can be effectively executed by a displacement over a slight distance δ. The structural element is in stable equilibrium if it returns to its equilibrium position.

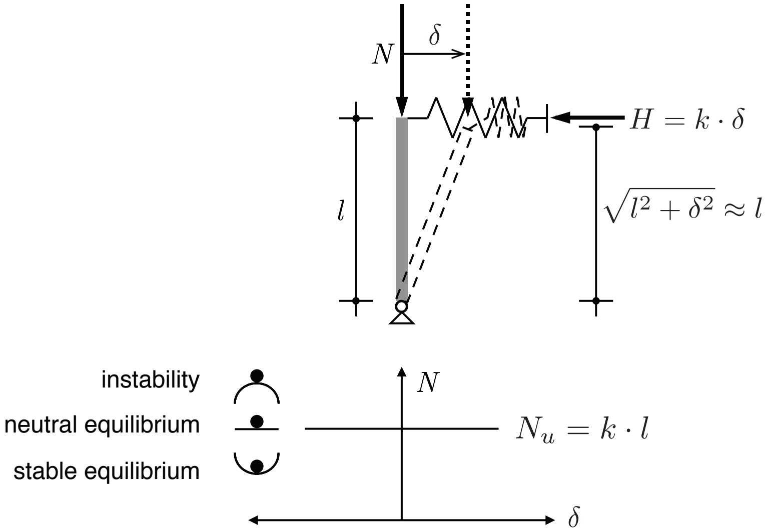


Figure 15.5: Stability of the equilibrium

Driving moment leading to an instability failure mode:

$$N_u \cdot \delta \tag{15.10}$$

Resisting moment providing a possible equilibrium:

$$H \cdot l = k \cdot \delta \cdot l \tag{15.11}$$

Equilibrium and corresponding ultimate failure load:

$$N_u \cdot \delta = (k \cdot \delta) \cdot l \Rightarrow N_u = k \cdot l \tag{15.12}$$

What we can learn from the equilibrium of this uncomplicated structural element is applicable for stability problems in general:

Stability is deformation-driven The ultimate failure load N_u with respect to stability has no relation whatsoever with strength parameters, but is solely determined by flexural stiffness parameters such as length, cross-sectional bending stiffness and boundary conditions.

Cross-sectional strength is naturally determined by the cross-sectional area in combination with the material strength: $N_u = A \cdot f_d$.

In addition, this analysis shows that every rule has its "field of application" to be considered; in this case, the assumption $\sqrt{l^2 - \delta^2} \approx l$ is valid under the condition of small displacements with respect to the overall geometry.

15.3.2 Second-order effects

There are second-order effects when the first-order deformation due to loading affect the distribution of internal forces.

The basic principles of second-order effects can be demonstrated by a column with an initial deformation due to fabrication e_0 and subjected to an axial load N as shown in figure 15.6.

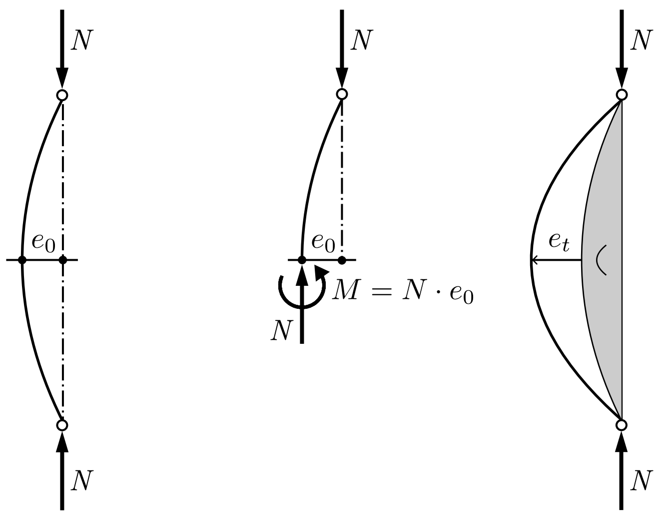


Figure 15.6: Second-order effects

The initial deformation e_0, combined with the axial load N, causes a first-order bending moment distribution with $M = N \cdot e_0$ in the middle of the column. The corresponding bending deformation is additional to the initial deformation and consequently leads to an amplification of the bending moments.

In a stable structure, the second-order effects are self-limiting; the second-order deformations are small enough that they do not continue to amplify. Unstable structures exhibit a different behaviour; second-order deformations are large enough to induce even larger forces, leading to greater and greater deformations and ultimately to collapse.

For such a combined compression and bending, the quantification of the second-order effects is substantiated with a magnification factor $\frac{n}{n-1}$ in subsection 15.3.4.

15.3.3 Euler-based design approximations

The ultimate failure load with respect to stability, is defined by Euler as follows:

$$N_{Euler} = \frac{\pi^2 EI}{L_{cr}^2} \tag{15.13}$$

Where L_{cr} is the buckling length with design approximations as shown in figure 15.7.

For braced frames, a buckling length of l is a safe approximation. For unbraced frames, there is no absolutely safe approximation; for common frames a buckling length of $2.5\,l$ is appropriate [12].

For in-plane buckling of arches, a buckling length of $0.5\,l_{arch}$ is appropriate [26]. For out-of-plane buckling the overall arch length is appropriate, which is normally reduced by adding lateral supports, such as bracings between the arches.

The mathematical formula of Euler is valid for a slender, perfectly straight member of homogeneous material that is free from initial stress. Because

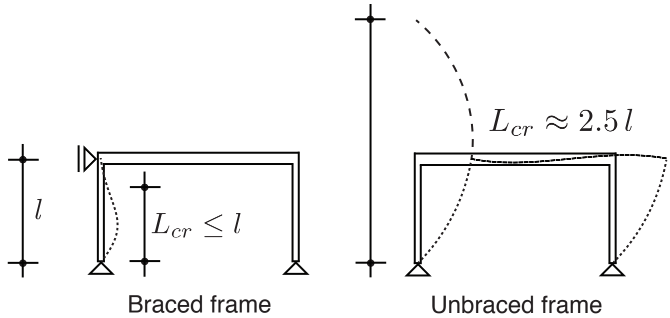


Figure 15.7: Buckling length

of deformation, material inhomogeneity, and residual stress due to fabrication, the Euler critical load has to be reduced by an adjustment factor k:

$$N_u = \frac{\pi^2 EI}{k L_{cr}^2} \tag{15.14}$$

Whereby the lowest N_u of both axes is decisive for the failure load.

In the professional practice, the value for the adjustment factor k can be set at 1.7, based on experimental results of buckling failure as included in Eurocode 3 [17] and listed in table 15.2.

For both an adjustment factor $k = 1.7$ and a buckling curve b, the critical buckling stress σ_{cr} is determined for structural steel grade S355. The accuracy of the adjustment factor k in relation to the Eurocode-based experimental results amounts to $\pm 30\%$.

The sectional strength-related, basic applied mechanics-based, design approximations have an accuracy far within the required $\pm 20\%$ as listed in table 8.5 on page 87.

The stability-related design approximations, however, are more difficult to capture. Probably the most sensitive is the buckling design approximation with an accuracy of $\pm 30\%$.

Critical buckling stress steel grade S355			
EC3 buckling curve b		Adjustment factor $k = 1.7$	
λ	σ_{cr}	σ_{cr}	$\Delta\%$
0	355	355	0
20	347	355	+2
40	310	355	+15
60	260	339	+30
80	201	190	-6
100	150	122	-19
150	77	54	-30

Table 15.2: Critical buckling stress comparison

Global buckling strength Whether it is local buckling on element level or global buckling on subsystem level, the same conceptual design formulae for buckling strength N_u and buckling length L_{cr} are applicable. Both the moment of inertia I and buckling length L_{cr} have to be related to the subsystem as a whole.

Lateral torsional buckling strength Lateral torsional buckling of a beam in bending is dependent on both buckling failure of the compressed flange and opposing effects such as torsional resistance and stabilisation of the tensioned flange.

As a safe approximation for conceptual design, opposing effects can be neglected. Lateral torsional buckling can then be determined on the basis of the buckling of the compressed flange. The same conceptual design formulae for buckling strength N_u and buckling length L_{cr} can then be applied.

Lateral buckling restraints All lateral restraints of buckling-sensitive structures or parts of structures have the function of a stiffener since stability in general, and buckling in particular, is deformation-driven.

However, the required stiffness can be obtained with a strength-driven design approximation F_d for the lateral restraint: $F_d = 1\% \cdot N_d$ [18].

It should be noted that a lateral restraint is not just a detailing matter but a noticeable structural element, which has to be connected to a fixed support.

15.3.4 Combined compression and bending

The combination of compression and bending with respect to stability is too complex to handle on the level of conceptual design approximations.

However, when possibly decisive, this combination can be designed on stress level, including the estimation of an initial deformation e_0. Considering the relative complex calculation, such an analysis functions more as a check than as a design.

On stress level, the combined compression and bending can be described with the following equation:

$$\sigma = \frac{N_d}{A} + \frac{n}{n-1} \cdot \frac{N_d \cdot e_0}{W} + \frac{n}{n-1} \cdot \frac{M_d}{W} \leq f$$
$$\text{and } n = \frac{N_{Euler}}{N_d} = \frac{\frac{\pi^2 EI}{L_{cr}^2}}{N_d} \quad (15.15)$$

The following corresponding unity check is equivalent:

$$\frac{N_d}{A \cdot f} + \frac{n}{n-1} \cdot \frac{N_d \cdot e_0}{W \cdot f} + \frac{n}{n-1} \cdot \frac{M_d}{W \cdot f} \leq \frac{f}{f}$$
$$\Rightarrow \frac{N_d}{N_u} + \frac{n}{n-1} \cdot \frac{N_d \cdot e_0}{M_u} + \frac{n}{n-1} \cdot \frac{M_d}{M_u} \leq 1 \quad (15.16)$$

15.4 Bending and compression

15.4.1 Conceptual design of concrete in bending

Bending strength of concrete members Because of the insignificant tensional strength of concrete, global bending moments and shear forces in a concrete member cannot be resisted through homogeneous material behaviour.

The local load distribution within a concrete beam consists of axial compression and tension forces in the concrete and the reinforcement, respectively, in complete analogy with the load distribution in an arch for stocky beams and a truss for slender beams, as shown in figure 15.8.

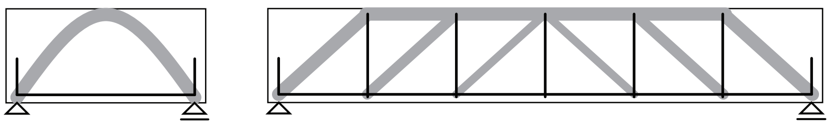


Figure 15.8: Arch and truss analogy in concrete beams

A design approximation of the ultimate strength of the usually governing bending rather than shear, can be effectively modelled by a lever arm with an approximated value of $0.8h$ between the centres of gravity of the compressed concrete zone and the reinforcement [2] as shown in figure 15.9.

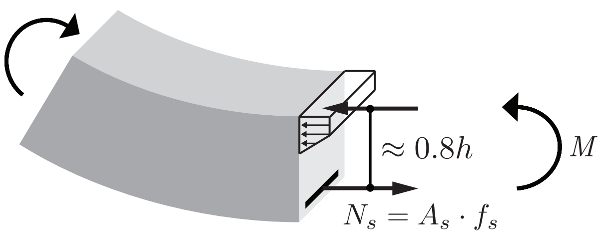


Figure 15.9: Bending strength of a concrete member

Within the cost optimal limits of reinforcement percentage, the strength of the reinforcement will be decisive for the bending strength of concrete members. As a professional practice-based average for cost optimal concrete design for an individual reinforcement direction and layer, a reinforcement percentage of 0.5% of the gross concrete cross section $b \cdot h$ will be applied.

Consequently, the amount of reinforcement A_s to provide the required bending strength M_d can be based on the reinforcement strength f_s and the lever arm of $0.8h$:

$$\frac{M_d}{M_u} = \frac{M_d}{(A_s \cdot f_s) \cdot 0.8h} \leq 1 \Rightarrow A_s \geq \frac{M_d}{f_s \cdot 0.8h} \tag{15.17}$$

The required section height h, relative to the member length l, for a simply supported beam under a uniform load can be derived as follows:

$$A_s \geq \frac{\frac{1}{8}q_d l^2}{f_s \cdot 0.8h} = \tfrac{0.5}{100}bh \Rightarrow \frac{h}{l} \geq \sqrt{\frac{\frac{1}{8}q_d}{b \cdot 435 \cdot 0.8 \cdot \frac{0.5}{100}}} \tag{15.18}$$

The required section height can be adjusted to resist load distributions due to non-uniform loading or beam end restraints, for example for the end field of a continuous beam:

$$A_s \geq \frac{\frac{1}{10}q_d l^2}{f_s \cdot 0.8h} = \tfrac{0.5}{100}bh \Rightarrow \frac{h}{l} \geq \sqrt{\frac{\frac{1}{10}q_d}{b \cdot 435 \cdot 0.8 \cdot \frac{0.5}{100}}} \tag{15.19}$$

The width b of a poured-in-place concrete beam will mostly be adapted to the supporting concrete structure for reasons of formwork economics. In any case, lateral torsional failure must be prevented. A common value for the $\frac{b}{h}$-ratio is between $\frac{1}{2}$ and $\frac{3}{4}$ [1].

For a floor slab, the load q_d is directly proportional to the width b, so usually, the reinforcement design will be executed per m^1 unit width.

Displacement of concrete members For the displacement of a concrete member, the material and cross-sectional related stiffness EI_c has to be approximately determined. The material stiffness E_c is derived from the simplified bi-linear modelled stress-strain relationship of concrete with an elastic strain limit of 1.75‰, including the influence of time-related effects, as discussed in subsection 13.3.3 about material properties for conceptual design. The cross-sectional stiffness I_c is based on an uncracked cross-section in the serviceability limit state.

$$\delta = \frac{5}{384} \cdot \frac{q_k l^4}{EI_c} \qquad \text{and } EI_c = \frac{f_c}{\frac{1.75}{1000}} \cdot \tfrac{1}{12}bh^3 \tag{15.20}$$

The required section height h, relative to the member length l, for a simply supported beam can be derived as follows:

$$\delta = \frac{5}{384} \cdot \frac{q_k l^4}{\frac{f_c}{\frac{1,75}{1000}} \cdot \frac{1}{12} b h^3} \leq \frac{l}{250}$$
$$\Rightarrow \frac{h}{l} \geq \sqrt[3]{\frac{5}{384} \cdot 250 \cdot 0.021 \frac{q_k}{b \cdot f_c}} \quad (15.21)$$

The required section height can be adjusted to resist load distributions due to non-uniform loading or beam end restraints, as for the end field of a continuous beam:

$$\delta = \frac{1}{150} \cdot \frac{q_k l^4}{\frac{f_c}{\frac{1,75}{1000}} \cdot \frac{1}{12} b h^3} \leq \frac{l}{250}$$
$$\Rightarrow \frac{h}{l} \geq \sqrt[3]{\frac{1}{150} \cdot 250 \cdot 0.021 \frac{q_k}{b \cdot f_c}} \quad (15.22)$$

15.4.2 Conceptual design of concrete in compression

Axial strength The axial strength capacity of a more or less homogeneous material is dependent on the strength f_d of the material in combination with the area A of the cross section.

The required area of concrete can simply be calculated as follows:

$$\frac{N_d}{N_u} = \frac{N_d}{A \cdot f_c} \leq 1 \Rightarrow A \geq \frac{N_d}{f_c} \quad (15.23)$$

This simple model can also be applied as a design approximation for reinforced concrete whereby the contribution of the reinforcement may be neglected.

Buckling strength of concrete members For reinforced concrete the Euler critical load has to be reduced by an adjustment factor of $k = 1.7$.

The required sectional height can be derived as follows:

$$\frac{N_d}{N_u} = \frac{N_d}{\frac{1}{k} \cdot \frac{\pi^2 E_c \frac{1}{12} bh^3}{L_{cr}^2}} \leq 1 \Rightarrow h_{weak\,axis} \geq \sqrt[3]{N_d \cdot \frac{1.7 \cdot L_{cr}^2}{\pi^2 E_c \cdot \frac{b}{12}}} \quad (15.24)$$

15.4.3 Conceptual design of steel in bending

Bending strength of steel members Global bending moments and shear forces in a steel member can be resisted through nearly homogeneous material behaviour with a plastic stress distribution for standard rolled sections with the lowest steel grade S235 and a mostly elastic stress distribution for higher grades [17] as shown in figure 15.10.

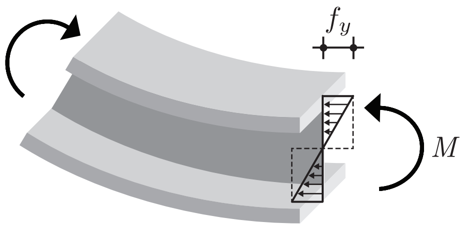


Figure 15.10: Bending strength of a steel member

Due to material costs and weight of structural steel, the design of cross sectional areas has to be fully optimised. In order to support this optimisation process, design approximations have to be based upon the section modulus W:

$$\frac{M_d}{M_u} = \frac{M_d}{W \cdot f_y} \leq 1 \Rightarrow W \geq \frac{M_d}{f_y} \quad (15.25)$$

The required section modulus for a simply supported beam under a uniform load is as follows:

$$W \geq \frac{\frac{1}{8} q_d l^2}{f_y} \quad (15.26)$$

The required section modulus can be adjusted to resist load distributions due to non-uniform loading or beam end restraints, for example for the end field of a continuous beam:

$$W \geq \frac{\frac{1}{10} q_d l^2}{f_y} \tag{15.27}$$

Displacement of steel members For the displacement of a steel member, the material and cross-sectional related stiffness EI is governing. For steel, the material stiffness E has a constant value of $2.1 \cdot 10^5 \, \frac{\mathrm{N}}{\mathrm{mm}^2}$ and is completely independent of the steel grade.

The required cross-sectional stiffness or moment of inertia I for a simply supported beam can be derived as follows:

$$\delta = \frac{5}{384} \cdot \frac{q_k l^4}{EI} \leq \frac{l}{250} \Rightarrow I \geq 250 \cdot \frac{5}{384} \cdot \frac{q_k l^3}{E} \tag{15.28}$$

The required moment of inertia can be adjusted to resist load distributions due to non-uniform loading or beam end restraints, for example for the end field of a continuous beam:

$$\delta = \frac{1}{150} \cdot \frac{q_k l^4}{EI} \leq \frac{l}{250} \Rightarrow I \geq 250 \cdot \frac{1}{150} \cdot \frac{q_k l^3}{E} \tag{15.29}$$

15.4.4 Conceptual design of steel in compression

Axial strength The axial strength capacity of a more or less homogeneous material is dependent on the strength f_d of the material in combination with the area A of the cross section.

The required area of steel can simply be calculated as follows:

$$\frac{N_d}{N_u} = \frac{N_d}{A \cdot f_y} \leq 1 \Rightarrow A \geq \frac{N_d}{f_y} \tag{15.30}$$

Buckling strength of steel members For structural steel, the Euler critical load has to be reduced by an adjustment factor of $k = 1.7$.

The required moment of inertia can be derived as follows:

$$\frac{N_d}{N_u} = \frac{N_d}{\frac{1}{1.7} \cdot \frac{\pi^2 EI}{L_{cr}^2}} \leq 1 \Rightarrow I_{weak\,axis} \geq N_d \cdot 1.7 \cdot \frac{L_{cr}^2}{\pi^2 E} \tag{15.31}$$

15.4.5 Conceptual design of foundations

Spread foundation strength The structural action of a spread foundation consists of a load spread, a stabilisation by a possibly present top loading, and a complex failure mechanism of slipping soil particles as shown in figure 15.11.

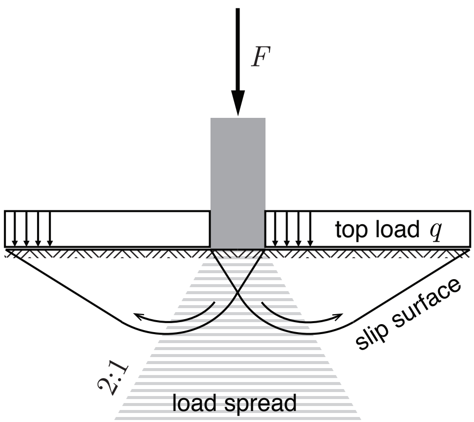


Figure 15.11: Failure mechanism of a spread foundation

Soil failure occurs with the formation of a load and soil dependent slip surface, characterised by the slope of the slip surface and the shear strength along this slip surface.

A safe and considerably simplified, ultimate limit state design approximation is a maximum of uniform assumed soil stresses at the foundation contact area of $0.2\ \frac{\mathrm{N}}{\mathrm{mm}^2}$.

This professional practice-based conceptual design approximation is based on a common foundation type on sand ground, a base width of about 1 metre and a minimal embedment depth of 0.3 metre.

In combination with horizontal loading, intermediate layers or adjacent (future) excavations, a more deepening analysis is required.

Pile foundation strength The structural action of a pile foundation consists of a pile head bearing capacity, a pile shaft friction-bearing capacity and a complex failure mechanism of slipping soil particles as shown in figure 15.12.

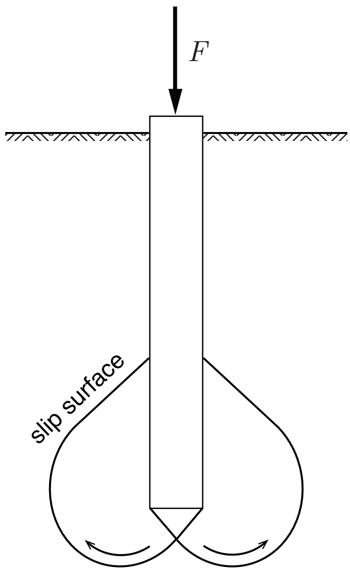


Figure 15.12: Failure mechanism of a pile foundation

Soil failure occurs with the formation of a load and soil dependent slip surface, characterised by the slope of the slip surface and the shear strength along this slip surface.

A safe and considerably simplified ultimate limit state design approximation is a maximum of uniform assumed soil stresses at the pile head area of 5 $\frac{\mathrm{N}}{\mathrm{mm}^2}$.

This professional practice-based conceptual design approximation is based on a common typical Dutch foundation type on a deeper sand layer with substantial base resistance, partial shaft resistance, and partial negative skin friction.

This design approximation can even be applied to the conceptual design of tension piles, despite the difference in failure mechanism.

Part IV

Case study and training

Chapter 16

Introduction to part IV

Both qualified and quantified solutions are clarified by means of a case study and a series of trainings, including a zero measurement training. The resulting chapter arrangement is visualised in figure 16.1.

Case study This specific case study demonstrates how the methodical approach leads to a controlled build-up of insight into the behaviour of the structure and supports the actual successive design decisions during the conceptual design of the trusses of the Maeslant storm surge barrier.

The load paths, overall geometry, and principal detailing on the basis of performance, structural, and construction demands, are determined. Subsequently, the structural action in this outlined structure is optimised and the elements are dimensioned. Finally, a thorough risk analysis is conducted as a demarcation of the conceptual structural design phase.

Training The required knowledge, skills, and professional attitude have to be achieved by a mix of the learning methods lecture, training, and project work. Lectured theories and trained engineering practice can be applied during project work.

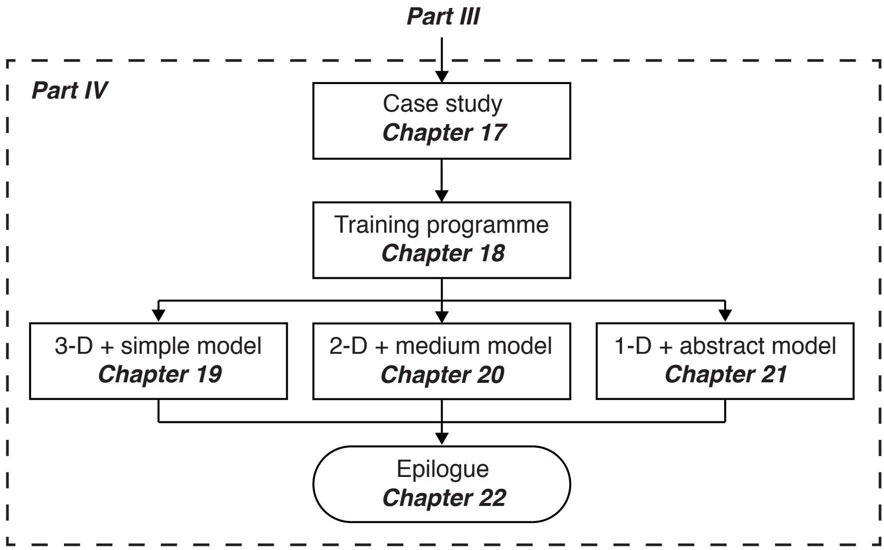


Figure 16.1: Reading guide for part IV

The training programme consists of a series of trainings, including a zero measurement training. All trainings have approximately the same entry professional on graduate master level, with a balanced complexity of geometry and modelling. The zero measurement training can be applied as a self-check at the beginning of the learning process.

Epilogue It is important to study how the design is organised in practice, and especially the ways in which designers with different disciplinary expertise are able to work together, collaboratively in teams. For an integral conceptual structural design, the main contributing disciplines and corresponding interfaces have to be considered

This textbook discusses conceptual structural design on a high level of abstraction. However, a deepening research on conceptual structural design is valuable and feasible. Recommendations for research are given with respect to both understanding these complex interdisciplinary interfaces and structural performance.

Chapter 17

Case study trusses Maeslant storm surge barrier

17.1 Demonstration of the methodical approach

17.1.1 A case study of conceptual structural design

The purpose of this specific case study is to demonstrate how the methodical approach can lead to a controlled build-up of insight into the behaviour of the structure and supports the actual successive design decisions during conceptual design.

For a clear comparison between standard practice and the methodical approach, diffuse conceptual design variables such as experience, and especially intuition, have to be eliminated. Therefore, (a part of) an actual personally executed conceptual structural design such as the trusses of the Maeslant storm surge barrier is preferred rather than a new project.

The load paths, overall geometry, and principal detailing on the basis of performance, structural, and construction demands are determined. Subsequently, the structural action in this outlined structure is optimised and the elements are dimensioned. Finally, a thorough risk analysis is conducted as a demarcation of the conceptual structural design phase.

17.1.2 Actual design method

The actual design method at the time of the conceptual structural design of the Maeslant barrier doors can be characterised by the following:

3-D Modelling From the beginning, full three-dimensional modelling is applied for all conceptual design activities as geometry design, load distribution analysis, and code checking.

Force-driven The initial geometry design and optimisation is force-driven based. Subsequently, code-based unity checks on sectional strength, stability, and the hollow section joints are conducted.

Reactive The reactive unity check-based conceptual design indicated a severe failure problem of the web members. Analysis revealed an induced deformation problem and corresponding solution.

Risk analysis As one of the first Design and Construct contracts, a thorough risk analysis of the material demand was conducted for tendering, and subsequently, as internal transfer documentation for the basic design team.

17.1.3 Methodical approach

The methodical approach as practiced on the conceptual structural design of the Maeslant barrier door trusses, can be characterised by the following:

2-D Modelling Decomposition-based conceptual design ensures a controlled build-up of insight into the behaviour of the structure, with a progressive insight from two-dimensional estimations to three-dimensional accuracy.

Deformation-driven Deformation-driven conceptual design with continuous consideration of aspects such as local stability, global stability, and induced deformation.

Pro-active A pro-active methodical approach including principal details and foreseen induced deformation, with the help of orientation, analysis, and check loops.

Risk analysis For complex design and construct-based contracts - being today's standard in the field of civil engineering - a thorough risk analysis of the material demand is inevitable as transfer documentation for basic design.

17.1.4 Outcome of the case study

The methodical approach leads to a controlled build-up of insight into the behaviour of the structure and supports the actual successive design decisions during conceptual design, on the basis of the following coherent set of solution components:

Structural design path The structural design is effectively explored from structural integrity, via load distribution, to failure mechanisms.

Structural design loops Conceptual structural design is based on a progressive insight with orientation, analysis, check and, if necessary, correction loops.

Load path design Out of the functional requirements, an initial system outline is created based on major load paths, an order of magnitude, and principal details.

Load distribution parameters After a decomposition in two-dimensional subsystems, further geometrical optimisations are clarified, and the load distribution is established.

Dimensioning parameters On the basis of so-obtained member loads in combination with deformation-driven conditions, conceptual dimensioning can eventually be specified.

Structural design cycle The fundamental design cycle with the decomposed design phases creation, optimisation, and dimensioning proved to be viable and effective.

Basic structural forms Decomposition of the three-dimensional structural system into two-dimensional basic structural forms - in this case trusses - proved an effective basis for structural analysis.

Shared knowledge-based conceptual design The interface of structural demand with architectural, and in this case particular construction demand, relies mainly on professional experience. A successful material demand and risk analysis is conducted.

17.2 Maeslant storm surge barrier

17.2.1 Final piece of the Delta works

After the North Sea flood of 1953, a commission was installed which had to come up with a plan to research the causes and seek measures to prevent such a disaster in future; they came up with a plan for the so-called "Delta works".

The plan consisted of blocking the estuary mouths of the Oosterschelde, the Haringvliet, and the Grevelingen. This reduced the length of the dikes exposed to the sea by 700 kilometres. The mouths of the Nieuwe Waterweg and the Westerschelde were to remain open because of the shipping routes to the ports of Rotterdam and Antwerp. The dikes along these waterways were to be heightened and strengthened.

The construction of the Maeslantkering was the final stage of these Delta Works. The main objective was improving the safety against flooding of the Rotterdam harbour and the surrounding towns and agricultural areas. In the original plan, this had to be carried out by the reinforcement of existing dikes as far as 50 kilometres inland. During the 1980s, it became clear that dike reinforcement would take at least 30 years and would only have disadvantages compared to a barrier as listed in table 17.1.

Therefore, the initial plan was put aside and the Ministry of Waterways and Public Works organised a competition in which construction companies could make plans for the construction of a reliable, yet relatively inexpensive, storm surge barrier.

Dike reinforcement versus barrier		
	Dike	Barrier
Costs (in billion euros)	0.82	0.45
Uncertainty costs	±20%	±10%
Delta safety schedule	2020	2000
Uncertainty schedule	10 years	2 years
Storm surges exposure	300 km	35 km
Environmental impact	Large scale	Limited

Table 17.1: Dike reinforcement versus barrier

17.2.2 Requirements

The storm surge barrier had to be located in the waterway that connects Rotterdam with the North Sea. As this waterway is the main route to the port of Rotterdam, a wide opening, unlimited headway, and a minimum disturbance of ship movements, among others, were required as listed in table 17.2.

Functional requirements for the Maeslant storm surge barrier
Unlimited headway
360 metres' wide opening
Barrier failure 1/1,000,000 in any one year
Minimum disturbance of ship movements
100 years' lifetime

Table 17.2: Requirements for the Maeslant storm surge barrier

17.2.3 Conceptual design Maeslant storm surge barrier

The winning barrier design with two huge hollow floating barrier doors was put forward by the Bouwcombinatie Maeslant Kering (BMK) consortium and became one of the first large Design and Construct projects in the Netherlands.

Each barrier door consists of a 210 metres' long retaining wall with floodable buoyancy chambers, supported by 250 metres' long trusses and ending in a ball joint with a diameter of 10 metres, and embedded in a concrete caisson as shown in figure 17.1.

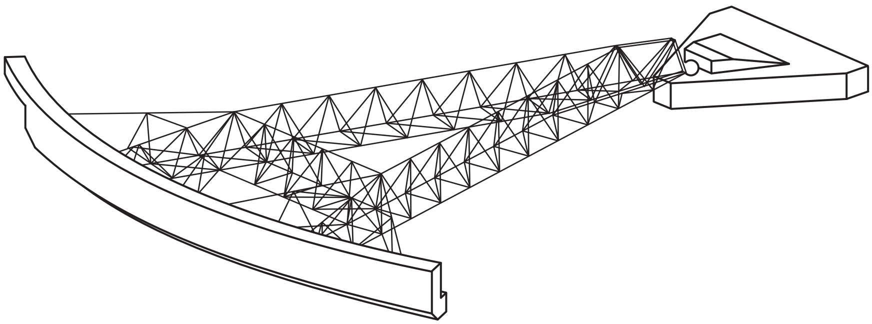


Figure 17.1: Maeslant storm surge barrier

Under normal conditions, the barrier doors are fully opened to allow ships to sail to and from Rotterdam. However, if the water level rises by three metres above the designated norm, the barrier doors are closed and flooded with water. This causes them to sink slowly onto the sill blocks at the bottom of the waterway. The entire process takes about 90 minutes.

During a storm surge, the water level on the North Sea side rises relative to the water level on the Rotterdam side. The corresponding hydraulic design load against one door equals approximately 350 meganewton (MN).

A major advantage of this design was that construction of the storm surge barrier could take place under dry conditions, in dry docks. Other advantages were that no vital parts of the barrier had to be placed under water, and maintenance of the barrier would be easy because of the dry docks. Finally, there would be almost no inconvenience for passing ships.

17.2.4 Performance/cost optimisation

For a coherent and complete integral design, all fundamental demands - namely performance demand, structural demand, and construction demand - have to be complied with. Within a Design and Construct contract, the performance/cost optimisation within the functional requirements is up to the integral constituted conceptual design team.

With respect to performance and construction demand, the following major performance/cost optimisations are essential for the conceptual design of the trusses:

Architectural demand The functionalism-based concept of architect Wim Quist for the design of the trusses, consists of a combination of tubular offshore-like appearance and a powerful clear transmission of the enormous hydraulic forces.

Maintainability Maintenance costs are a major issue for such a functional storm surge barrier. Therefore, the conceptual design of the steel trusses demands special attention to the coating by minimising surface area and avoiding sharp edges.

Construction demand Welding is, for the construction of steel trusses, a major laborious and thus cost-dominant design parameter. The design objective is thus an optimisation of weldability by minimising weld volume and maximising welder accessibility.

17.3 Creation of a system outline

17.3.1 Load-path design on system level

The primary design load consists of a combination of approximately 6 metres of hydraulic head and approximately 3 metres of transversing wave load, and can initially be simplifiedly modelled by a quasi-static hydraulic head design load of 9 metres, and a corresponding uniform load of $q_d = 90\,\frac{\mathrm{kN}}{\mathrm{m}}$ in accordance with figure 17.2.

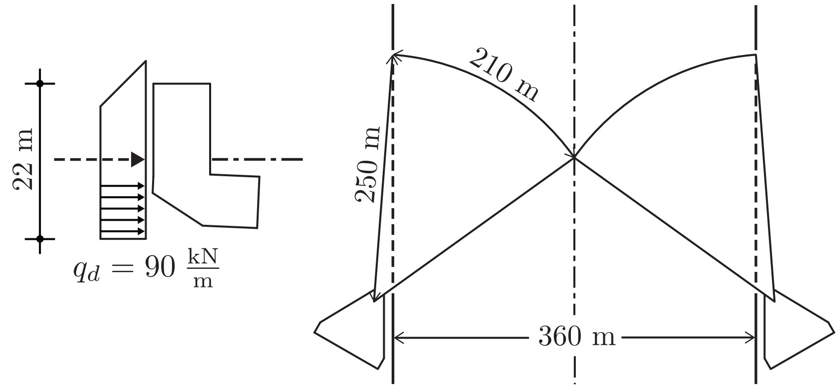


Figure 17.2: Hydraulic load

The resulting force of the ball joint amounts to 320 MN as a result of the projected hydraulic head in combination with the influence of the top and bottom of the wave.

Resisting this primary load and bridging 250 metres to the balancing concrete caisson requires a large amount of material. The corresponding self-weight of the structure is an inevitable secondary load of importance.

17.3.2 Principal details

Circular Hollow Section (CHS) members and joints are by far preferable, in analogy with offshore structures and based on the following performance/cost optimisation considerations:

Coated surface Surface area and sharp edges highly affect the influential costs of coating and corresponding maintenance. Minimisation of the coated surface, and avoiding sharp edges evidently, leads to a circular section.

Water pressure To avoid coating the inner surface, and the corresponding need for corrosion inspection and maintenance on the inside of such a complex structure, the trusses are completely water-tight. Consequently, a circular section is the best way to resist the water pressure.

Drag coefficient The drag coefficient quantifies the drag or resistance of an object in a fluid environment such as air or water. A circular section has a very low drag coefficient and is therefore less prone to both wind loading and dynamic hydraulic loading when submerged.

Element buckling Circular hollow sections have an excellent element buckling strength due to their all-directional high moment of inertia, low geometrical tolerances and low residual stresses.

Plate buckling A higher moment of inertia can be obtained by making sections thin-walled. Due to their circular shape, circular hollow sections effectively combine a high moment of inertia and excellent plate buckling resistance. Plate buckling, however, imposes a limit to the extent to which sections can be made thin-walled; $\frac{d}{t} \leqq 50$.

Uniformity To obtain cost-saving identical tubular gap joints and web member lengths, the trusses are designed with parallel bottom and top chords.

Joint geometry Aiming for a full capacity connection, the multi-planar gap joints are designed with canned sections and an average $\frac{d_{web}}{d_{chord}}$ ratio β of 0.6.

In this way, a principal joint as shown in figure 17.3 leads to a life cycle design that is both economically and architecturally satisfying.

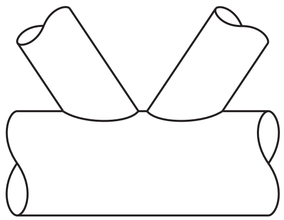
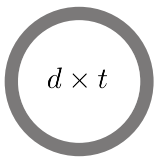


Figure 17.3: Principal joint

17.3.3 Circular hollow section elements

For an effective design of trusses with CHS members, section properties, steel grade, and the transition between sectional strength and element buckling have to be determined.

Section properties CHS The section properties of a CHS, with approximations based on a relatively small wall-thickness, are given in figure 17.4.



$$A \approx \pi d t$$

$$I \approx \tfrac{1}{8}\pi d^3 t \approx \tfrac{1}{8} A d^2$$

$$W \approx \tfrac{1}{4}\pi d^2 t \approx \tfrac{1}{4} A d$$

Figure 17.4: Section properties of a CHS

Steel grade At the time of the design of this storm surge barrier, standardised grades for structural steel varied from S235 up to S355. Given the functional requirements with respect to failure, this specific design is primarily force-driven instead of deformation-driven. Thus, the highest steel grade S355 is appropriate, thereby reducing the enormous self-weight of the structure.

Transition slenderness CHS The slenderness $\lambda_{el} = \frac{l_{el}}{d}$, where the axial strength equals the buckling strength, defines the transition between both failure mechanisms.

Axial strength:

$$N_u = A \cdot f_d \tag{17.1}$$

Buckling strength of an element in a braced structure:

$$N_u \approx \frac{1}{1.7} \cdot \frac{\pi^2 EI}{L_{cr}^2} \approx \frac{1}{1.7} \cdot \frac{\pi^2 EI}{l_{el}^2} \tag{17.2}$$

Transition slenderness $\lambda_{el,trans}$ CHS:

$$A \cdot f_d \approx \frac{1}{1.7} \cdot \frac{\pi^2 EI}{l_{el}^2} \approx \frac{1}{1.7} \cdot \frac{\pi^2 E \cdot \frac{1}{8} A d^2}{l_{el}^2} = \frac{1}{1.7} \cdot \frac{\pi^2 E \cdot \frac{1}{8} A}{\lambda_{el}^2}$$

$$\Rightarrow \lambda_{el,trans} = \frac{l_{el}}{d} = 0.85 \sqrt{\frac{E}{f_d}} = 0.85 \sqrt{\frac{210000}{355}} = 20.7 \quad (17.3)$$

17.3.4 Decomposition in subsystems

For a clear and effective design of the primary and secondary load paths, the three-dimensional system is decomposed into two-dimensional horizontal and vertical subsystem planes, as shown in figure 17.5.

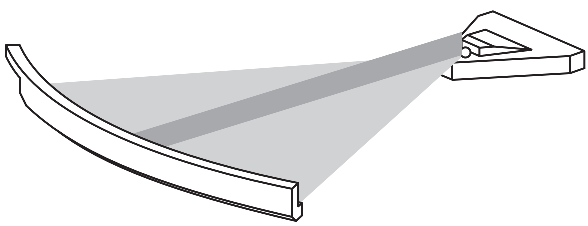


Figure 17.5: Decomposition in subsystem planes

Horizontal plane The horizontal plane directs the load paths of the primary hydraulic load. To provide a short transition of this primary hydraulic load to the ball joint, the level of this horizontal plane coincides with the resulting hydraulic design load.

The total required cross-sectional area of the bottom chords per barrier door amounts to:

$$A_{bottom} = \frac{l \cdot h \cdot q_d}{f_d} = \frac{210 \cdot 22 \cdot 90}{355} \cdot 10^3 = 117 \cdot 10^4 \, \text{mm}^2 \quad (17.4)$$

Vertical plane The vertical plane directs the load paths of the secondary structural self-weight. Furthermore, this plane directs the load paths caused by an eccentricity of the resulting hydraulic load due to the transversing wave.

The secondary top chords are less loaded than the primary bottom chords. Nevertheless, out of strength and fabrication considerations the preferred principal CHS joints are appropriate for both bottom and top chord joints. In combination with stability considerations, a three-dimensional truss configuration as shown in figure 17.6 is the logical consequence.

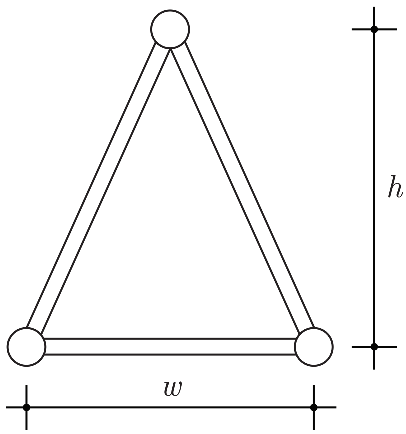


Figure 17.6: Triangular cross-sectional truss configuration

Intersection of horizontal and vertical plane The intersecting line between the horizontal and vertical plane gives an interaction between the hydraulic load and the structural self-weight.

The maximum hydraulic load in the horizontal plane is relieved by the structural self-weight in the vertical plane. However, when submerged, the buoyancy of the bottom chords will partly neutralise this relief since the buoyancy of the submerged members approximately equals their own weight.

The before mentioned eccentricity of the resulting hydraulic load on the spot of the bottom of the transversing wave, and the corresponding reduced water level also generates a relieving load with respect to the structural self-weight. Because of its temporal character, this relief will not be taken into account.

3-D system effects Possible three-dimensional load distribution effects require retention of these effects during decomposition of the three-dimensional system into two-dimensional subsystems.

In case of such a relatively flat structural system, three-dimensional system effects, and especially overall torsion, cannot be expected to be influential.

17.4 Optimisation of the structural action

17.4.1 Optimisation on horizontal subsystem level

In search of an optimal configuration, the absolute minimum performance-based configuration will be determined first; and then the possibility of redundancy as an added value, will be explored.

Performance-based cost minimisation The absolute minimum configuration consists of a statically determinate arrangement of two supports of the retaining wall. To secure global stability over a length of 250 metres, in plane horizontal stabilisation elements have to be designed in addition to the vertical weight bearing structures. However, the great distance between both supports requires heavy bracings.

Therefore each support is designed as a three-dimensional truss configuration. An efficient design of the retaining wall, however, requires more supports. Considering the long load path between the retaining wall and the ball joint, additional bracings instead of complete trusses are the most economical solution. Additional supporting bracings are designed as shown in figure 17.7.

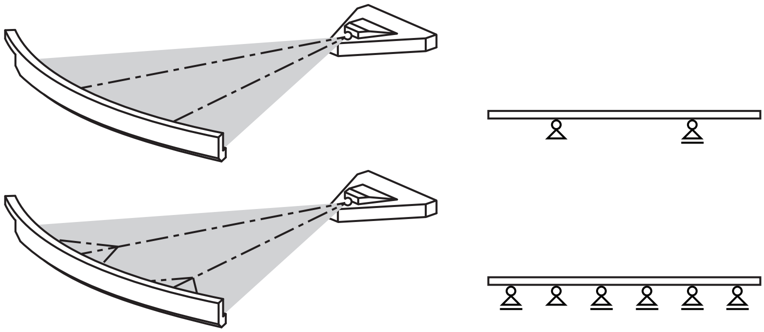


Figure 17.7: Optimisation of the supports of the retaining wall

Added-value design Nevertheless, one or more extra supporting trusses can be included to incorporate redundancy in this statically determinate design. However, even with four instead of two three-dimensional supporting trusses, failure of only one truss will inevitably lead to failure of the whole system due to an insurmountable loss of retaining wall capacity. So extra supporting trusses are still useless and costly.

17.4.2 Optimisation on vertical subsystem level

The vertical height h and the horizontal width w of the three-dimensional truss configuration as shown in figure 17.6 on page 200, have to be determined.

Vertical truss height The optimal height/span ratio of trusses varies between $\frac{1}{10}$ and $\frac{1}{15}$. Because of the secondary load character of the truss design, a height/span ratio near $\frac{1}{15}$ is appropriate with a height h of 18 metres.

The resulting corresponding equilateral triangle has a web member, and bottom chord and top chord lengths of approximately 20 metres.

Horizontal truss width For a sufficient connection angle of both web members to the top chord, with regard to the cross-section of the truss as shown in figure 17.6 on page 200, the width w requires 15 metres.

17.4.3 Induced deformation

A timely determination of deformation-driven design parameters is of importance for an effective optimisation of the structural action during design. This statically indeterminate truss structure with primary loaded chord members and secondary loaded web members can be potentially sensitive to induced deformation of the web members.

Because the primary loaded chord members can induce an impermissible deformation and corresponding failure of the web members, a sensitivity study, as input for final dimensioning of these web members, has to be conducted.

17.5 Dimensioning

17.5.1 Cross-sectional area of the bottom chord

In the statically determinate arrangement, the hydraulic load will be divided over $2 \cdot 2 = 4$ cross sections. Each cross section requires the following area:

$$A_{bottom} = \frac{1}{4} \cdot 117 \cdot 10^4 = 29.3 \cdot 10^4 \text{ mm}^2 \tag{17.5}$$

17.5.2 Global stability

The load-bearing capacity of the truss with respect to global buckling can be approximated as follows:

$$N_u \approx \frac{1}{1.7} \cdot \frac{\pi^2 EI}{l_{sys}^2} \qquad \text{and } I_{truss} \approx 0.8 \cdot 2 \cdot A_{chord} \cdot \left(\tfrac{1}{2}w\right)^2 \tag{17.6}$$

With known cross-sectional area A_{bottom} of the bottom chord members, the minimum required truss width w with respect to global buckling amounts to:

$$w \geq \sqrt{\frac{4.25 \cdot N_d \cdot l_{sys}^2}{\pi^2 \cdot E \cdot A_{bottom}}} = \sqrt{\frac{4.25 \cdot 207.9 \cdot 10^6 \cdot 250^2 \cdot 10^6}{\pi^2 \cdot 2.1 \cdot 10^5 \cdot 29.3 \cdot 10^4}} \cdot 10^{-3}$$
$$= 9.5 \leqq \text{actual } 15 \text{ m} \quad (17.7)$$

17.5.3 Cross-sectional area of web and top chord

The uniform structural self-weight load of the bottom chords per barrier door amounts to:

$$q_{bottom} = A_{bottom} \cdot \rho_s = 1.17 \cdot 78.5 = 91.8 \; \frac{\text{kN}}{\text{m}} \quad (17.8)$$

An additional top chord and a three-dimensional web member configuration will approximately double this weight. On the basis of post calculation, the design value of the uniform structural self-weight load amounts to approximately 200 $\frac{\text{kN}}{\text{m}}$ per barrier door.

The uniform structural self-weight load of approximately 100 $\frac{\text{kN}}{\text{m}}$ per three-dimensional truss results in the following maximum cross-sectional areas in the top chord and the web members:

$$A_{top} = \frac{\frac{1}{8} q_d l^2}{h \cdot f_d} = \frac{\frac{1}{8} \cdot 100 \cdot 250^2 \cdot 10^6}{18 \cdot 10^3 \cdot 355} = 12.2 \cdot 10^4 \text{ mm}^2 \quad (17.9)$$

$$A_{web} = \frac{\frac{1}{2} \cdot \frac{1}{2} q_d l}{\cos \alpha \cdot \cos \beta \cdot f_d} = \frac{\frac{1}{2} \cdot \frac{1}{2} \cdot 100 \cdot 250 \cdot 10^3}{\cos 22.6^\circ \cdot \cos 30^\circ \cdot 355}$$
$$= 2.2 \cdot 10^4 \text{ mm}^2 \quad (17.10)$$

17.5.4 Induced deformation of web members

Because the primary loaded chord members can induce an impermissible deformation and a corresponding failure of the web members, a sensitivity study is conducted.

Phenomenon An induced deformation on subsystem level is applicable for the web members of the trusses of the Maeslant storm surge barrier as shown in figure 17.8.

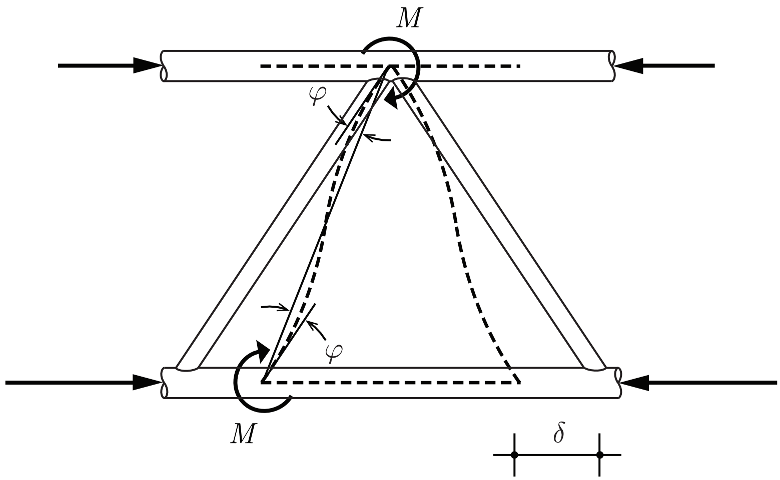


Figure 17.8: Induced deformation of the Maeslant storm surge barrier

The angle $\varphi = \frac{Ml}{6EI}$ is induced because of the elastic deformation δ of the bottom chords due to the immense horizontal water load, in combination with the stiffness of the welded tubular hollow section joints.

As a consequence of the induced angle φ, fixed web member length l, and fixed normal stiffness modulus E, the fraction $\frac{M}{I}$, is a constant.

So reducing the bending moment M, as caused by the induced angle, is only possible by reducing the cross-sectional moment of inertia I of the web member.

Modelling During conceptual design the barrier door is modelled with an overall finite elements model including retaining wall, trusses, and ball joint. The load cases consist of hydraulic head, transversing wave, and the structural self-weight.

The results are imported into a computerised code check including the failure mechanisms of the individual truss members and their connection to the canned joints.

Analysis To be on the safe side for the web members, the computerised calculation input is initially based on CHS sizes of $\varnothing$ 900 $\times$ 30 mm. The corresponding load distribution within the trusses results in a cross-sectional failure of these web members, primarily due to bending.

Subsequently, the wall thickness is incremented until the bending strength of the web members is sufficient. Even with a massive section of $\varnothing$ 900 mm, cross-sectional failure still occurs. This huge sectional area is completely out of proportion with the initial required axial strength.

The problem of induced deformation where the bending stiffness and the corresponding bending moments increase more than the strength of the structure, can be effectively solved by reducing the bending stiffness of the concerning member.

Reducing the bending stiffness of the web members by reducing the wall thickness to CHS sizes of $\varnothing$ 900 $\times$ 20 mm, the corresponding strength proves more than sufficiently that it resists all forces including the induced deformation-driven bending moments.

Design solution The potential induced deformation-driven problem of cross-sectional failure of CHS web members with high plate thicknesses, can be effectively prevented by applying the relatively low plate thickness of a $\varnothing$ 900 $\times$ 20 mm web member.

To prevent this potential induced deformation-driven problem during further design optimisation, the cross-sectional bending stiffness of the web members may not exceed the moment of inertia of a CHS $\varnothing$ 900×20 mm.

17.5.5 Section dimensions of conceptual design

The induced deformation-based dimensions of the web members, in combination with the average $\frac{d_{web}}{d_{chord}}$ ratio β of 0.6, results in the dimensions as shown in figure 17.9.

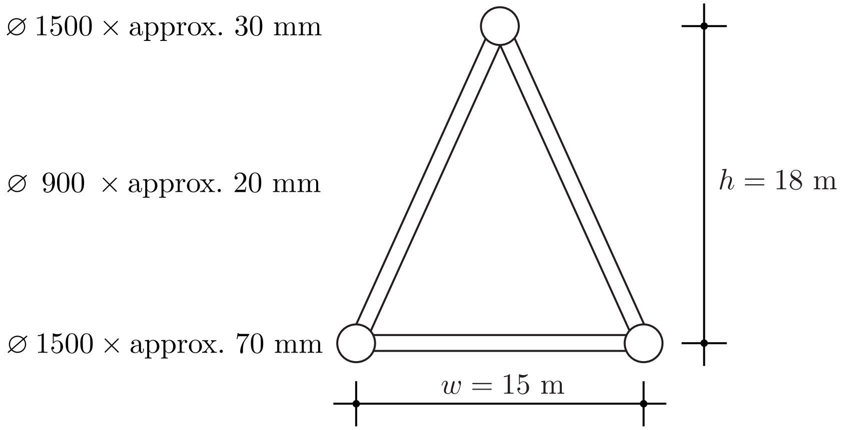


Figure 17.9: Truss dimensions

17.6 Specification and risk analysis

17.6.1 Material demand

The specification of conceptual design demarcates this project phase. As a Design and Construct project with a separated competitive tendering and assignment phase, this specification is used as a risk analysis for tendering, and subsequently, as internal transfer documentation for the basic design team.

A risk analysis of the material demand gives an accuracy estimation of the material quantities and corresponding unit cost indications. The risk analysis has to be conducted on the level of individual components of a decomposed system, principal details included.

For an effective risk analysis, as discussed in subsection 10.5.4, quantified data have to be determined.

17.6.2 Dimensioning and cost weighting

The result of the performed structural analysis is an approximate section dimensioning of the materialised overall geometry. This dimensioning of the conceptual design is specified by a list of material quantities.

An executed example of the dimensioning and cost weighting during the conceptual design of the trusses of the Maeslant storm surge barrier is listed in table 17.3.

The cost weighting equals the material quantities times the unit cost indications, and is a measure for cost optimisation opportunities and corresponding risk.

17.6.3 Uncertainties and coverage by dimensioning

The difference between the performed approximate structural analysis for conceptual design and the required depth and breath to meet the in-use requirements for structural safety and serviceability can be defined as uncertainties of the conceptual design. The difference in depth and breath generally concerns load combinations, load distribution, and failure mechanisms.

The uncertainties of the performed approximate structural analysis with respect to the required depth and breath cannot, or only partially, be covered. The corresponding status of the coverage gives an indication of the risk influence of the uncertainties of the conceptual design.

An executed example of the uncertainties and corresponding coverage by dimensioning during the conceptual design of the trusses of the Maeslant storm surge barrier is listed in table 17.4.

17.6.4 Reserves and optimisations

Reserves can be intentionally incorporated or are the result of rounding up to the nearest standardised product dimensions. Occasionally, an

Risk analysis conceptual design trusses Maeslant storm surge barrier Dimensioning and cost weighting			
Description	Weight [ton]	Costs indication [mhr/ton]	Weight × Costs
Top chord members ∅ 1500 × approx. 30 mm 1 splice-weld/member	2550	15	12%
Bottom chord members ∅ 1500 × approx. 70 mm 1 splice-weld/member	5790	10	18%
Web members ∅ 900 × approx. 20 mm 2 splice-welds/member	2830	35	31%
Hollow section joints in top chord 1 splice-weld/joint	600	60	11%
Hollow section joints in bottom chords 1 splice-weld/joint	1170	45	17%
Ball joint ∅ 10 m	1010	35	11%
Performed analysis: 3-D framework analysis with post-processing on stress level			

Table 17.3: Risk analysis: dimensioning and cost weighting

optimisation of requirements during the conceptual design phase can result in a reserve.

Foreseen, but time-consuming cost optimisations can be postponed to basic design, but registered as a potential reserve with regard to the completed conceptual design.

Risk analysis conceptual design trusses Maeslant storm surge barrier Uncertainties and coverage by dimensioning	
Uncertainties	Coverage by dimensioning
Top chord, bottom chords and web members:	
Decisive load combination head + wave based on 5% of the members	Average influence based on known member forces
Fatigue	No specific coverage, anticipation not decisive
Geometrical non-linearity	Average influence based on framework analysis of one load case
Serial effect	Assumption based on TNO report B-89-455
Global buckling	Average influence through enlargement factor
Hollow section joints in top and bottom chords:	
Decisive load combination unknown	No specific coverage
Fatigue	No specific coverage, anticipation not decisive
Net area due to man holes	10% extra plate thickness with a length of 3 m per hole
Ball joint:	
Load life cycle with FEM model stress distribution	Order of magnitude
Friction coefficient of bearing material	0.3

Table 17.4: Risk analysis: uncertainties and coverage by dimensioning

An executed example of the incorporated reserves and possible optimisations during the conceptual design of the trusses of the Maeslant storm surge barrier is listed in table 17.5.

Risk analysis conceptual design trusses Maeslant storm surge barrier Reserves and optimisations	
Reserves	Optimisations
3% reserve bottom chords Unity Check axial stress Ultimate Limit State Slight over-dimensioning of some top chord and web members due to minimum plate thickness Weight reduction of 830 ton due to head reduction (14-07-89) and disposed inlet slides (11-10-89)	Cross-section members Geometry web members with emphasis on length of chord members versus number of joints Eccentricity connections

Table 17.5: Risk analysis: reserves and optimisations

17.7 Further optimisations during basic design

17.7.1 Basic design

During conceptual design, the functional requirements and contractual conditions are evolved into a materialised overall system geometry, principal details included, with approximated modelling-based dimensions, quantities of materials, and corresponding risk analysis.

The subsequent basic design phase is characterised by a thorough analysis-based optimisation of the structural action and a thorough overall structural system check.

Furthermore, the basic design is assisted by testing of the ductility of the thermo-mechanical steel and the cast steel, the pre-stressed injection bolts, and the friction coefficient of the coating of the sliding surfaces.

Corresponding gained insight can reveal a need for adjustment and apparently even an addition of structural elements.

17.7.2 Extensive modelling

During basic design, the stiffness influence of both retaining wall and ball joint on the structural action in the trusses is refined within the overall finite elements model of the barrier door:

Retaining wall An accurate stiffness contribution of the combined bending plate and longitudinal stiffener, the combined shear plate and longitudinal stiffener, and the transverse stiffener, is individually modelled and condensed into the model of the retaining wall. Subsequently, the tubular bracings and partition bulkheads are included.

Ball joint The effects of slip-stick, variation of the friction coefficient, and actual rolling instead of slipping is determined with numerous life cycle calculations with partial models and the condensed overall model.

17.7.3 Coupling truss

During basic design, the refined modelling of the retaining wall in combination with more refined load cases and more load combinations, revealed unacceptable high longitudinal stresses in the retaining wall. These high stresses were mainly caused by the transversing wave load, with a top and bottom of approximately ± 3 metres.

For a more uniformly distributed support of the retaining wall, coupling of both supporting multi-planar trusses was required. Addition of an extra multi-planar coupling truss resulted in the necessary reduction of these longitudinal stresses.

17.7.4 Geometry and Section dimensions

During basic design, a more detailed code check was executed, including all individual failure mechanisms of the individual members, and particularly, their connection to the canned joints.

For the trusses, welding in situ was inevitable. Large lengths of the chord members could be welded on ground level and hoisted in place afterwards. However, all connections of the web members had to be welded in their permanent position.

Because of the difficultly accessible and laborious welding, a profound optimisation of geometry in combination with weld profile, volume, process, tolerances, and position was executed during basic design. This resulted in a reduction of 10% of the total weld volume and corresponding costs.

As a result of this optimisation of strength and welding of the tubular joints, the diameter of the web members was altered to 800 mm, and the diameter of the chords to 1800 mm.

Chapter 18

Training programme

18.1 Training programme outline

18.1.1 Load path design

Due to the huge degree of freedom, conceptual structural designing requires simple and clear modelling. This is merely possible on the level of axial forces, directly, with a truss-analogy or an arch depending on the structural form. Load path design on system level as discussed in section 13.2 is included in this training programme.

18.1.2 Programme outline

The required knowledge, skills, and professional attitude have to be achieved by a mix of the learning methods lecture, training, and project work. Lectured theories and trained engineering practice can be applied during project work.

The training programme consists of a series of trainings, including a zero measurement training, as listed in table 18.1. All trainings have approximately the same entry professional on graduate master level, with a balanced complexity of geometry and modelling:

- Complex 3-D geometry combined with simple modelling.
- Medium 2-D geometry combined with medium modelling.
- Simple 1-D geometry combined with abstract modelling.

Training programme outline			
No.	Training	Geometry	Modelling
0	Spatial struts and ties	3-D	Simple
1	Spatial trusses	3-D	Simple
2	Box truss with torsion		
3	Combined truss and arch	2-D	Medium
4	Second-order effect arch		
5	Truss analogy concrete beam	1-D	Abstract
6	Truss analogy steel beam		

Table 18.1: Training programme outline

The training requirements can be identified by utilising a zero measurement; the zero measurement training 0 can be applied as a self-check at the beginning of the learning process.

18.2 Training 0 - Spatial struts and ties

18.2.1 Intermediate bracing system with struts and ties

Given a 3-D bracing system with struts and ties as shown in figure 18.1. Such a structure typically can be utilised as an intermediate load distribution layer between a new and an underlying existing building.

For this training, the following requirements are applicable:

- Cross-sectional dimensions have to be determined for the ULS, the SLS is not applicable.
- The load-bearing capacity of the 4 bearings is limited to 400 kN each.

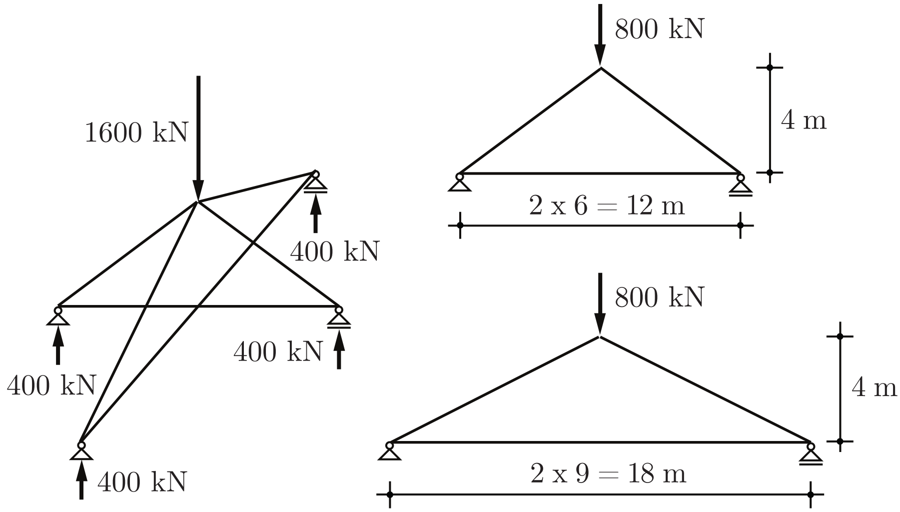


Figure 18.1: Bracing system with struts and ties

- The 4 struts consist of structural steel CHS with wall-thickness $t = 10$ mm and normal stress strength $f_y = 355$ N/mm^2. The section properties of a CHS, with approximations based on a relatively small wall-thickness, are given in figure 17.4 on page 198.
- The 2 crossing ties consist of structural steel round bars with normal stress strength $f_y = 355$ N/mm^2.
- The structural self-weight may be neglected in comparison with the loading of 1600 kN.

Analyse and determine the cross-sectional dimensions of this statically indeterminate structure with the following problem-solving approach:

1. Consider the separate statically determinate simply supported 2-D subsystems and determine the individual member forces and corresponding cross-sectional dimensions.
2. Now, model the complete 3-D system and determine the cross-sectional dimensions of the ties, assuming the struts are infinitely

stiff.

3. With this 3-D model, determine the cross-sectional dimensions of the struts, this time assuming the ties are infinitely stiff.

4. Determine the cross-sectional dimensions of the overall system and the corresponding displacement at the location of the loading of 1600 kN.

5. Analyse and optimise the total amount of structural material for this system.

6. Perform a redundancy analysis for the optimised system and conclude whether the optimisation is still valid when a second method of support is required.

18.2.2 Strength design separate 2-D struts and ties

For the statically determinate simply supported 2-D subsystems, the individual member forces for both the short and the long span can be determined as shown in figure 18.2.

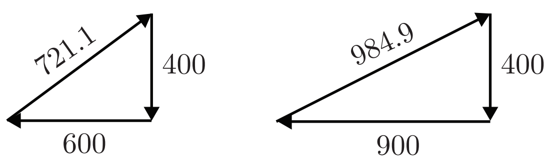


Figure 18.2: Load distribution 2-D struts and ties

For the short span, the cross-sectional dimension of the tie amounts to:

$$A \geq \frac{N_d}{f_y} = \frac{600 \cdot 10^3}{355} = 1690\,\text{mm}^2$$
$$\Rightarrow \varnothing\, 50\text{ mm } (1963\,\text{mm}^2) \quad (18.1)$$

For the short span, the cross-sectional dimension of the strut amounts to:

$$A \geq \frac{N_d}{f_y} = \frac{721.1 \cdot 10^3}{355} = 2031\,\mathrm{mm}^2 \quad \text{and}$$
$$I \geq N_d \cdot 1.7 \cdot \frac{L_{cr}^2}{\pi^2 E} = 721.1 \cdot 10^3 \cdot 1.7 \cdot \frac{7211^2}{\pi^2 \cdot 2.1 \cdot 10^5} = 3076 \cdot 10^4\,\mathrm{mm}^4$$
$$\Rightarrow \varnothing\, 200 \times 10\,\mathrm{mm}\ (6283\,\mathrm{mm}^2) \qquad (18.2)$$

For the long span, the cross-sectional dimension of the tie amounts to:

$$A \geq \frac{N_d}{f_y} = \frac{900 \cdot 10^3}{355} = 2535\,\mathrm{mm}^2$$
$$\Rightarrow \varnothing\, 60\,\mathrm{mm}\ (2827\,\mathrm{mm}^2) \qquad (18.3)$$

For the long span, the cross-sectional dimension of the strut amounts to:

$$A \geq \frac{N_d}{f_y} = \frac{984.9 \cdot 10^3}{355} = 2774\,\mathrm{mm}^2 \quad \text{and}$$
$$I \geq N_d \cdot 1.7 \cdot \frac{L_{cr}^2}{\pi^2 E} = 984.9 \cdot 10^3 \cdot 1.7 \cdot \frac{9849^2}{\pi^2 \cdot 2.1 \cdot 10^5} = 7836 \cdot 10^4\,\mathrm{mm}^4$$
$$\Rightarrow \varnothing\, 280 \times 10\,\mathrm{mm}\ (8796\,\mathrm{mm}^2) \qquad (18.4)$$

The cross-sectional dimensions based on the strength design of the separate 2-D subsystems are listed in table 18.2.

Cross-sectional dimensions 2-D subsystems design		
Element	Short span	Long span
Tie	∅ 50 mm	∅ 60 mm
Strut	∅ 200 × 10 mm	∅ 280 × 10 mm

Table 18.2: Dimensions 2-D struts and ties

18.2.3 Deformation-driven design of the ties

For such a statically indeterminate and thus deformation-driven 3-D system, the required stiffness and corresponding cross-sectional dimensions of the ties can be determined assuming the struts are infinitely stiff.

For both the short and the long span, the elongation Δl of the tie and corresponding displacement δ at the location of the loading can be determined as shown in figure 18.3.

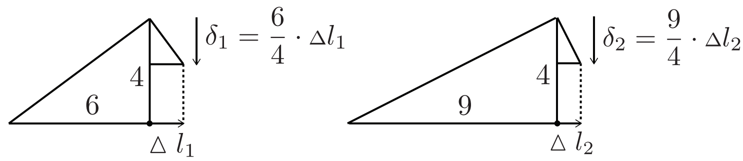


Figure 18.3: Deformation of the ties

For the long span, the cross-sectional dimension of the tie amounts to:

$$
\begin{aligned}
&\delta_1 = \delta_2 \quad \text{with tie } 1 \varnothing\, 50\,\text{mm } (1963\,\text{mm}^2) \\
&\delta_1 = \frac{6}{4} \cdot \Delta l_1 = \frac{6}{4} \cdot \frac{F_1 \cdot l_1}{E \cdot A_1} = \frac{6}{4} \cdot \frac{600 \cdot 10^3 \cdot 6000}{2.1 \cdot 10^5 \cdot 1963} = 13.1\text{ mm} \\
&\delta_1 = 13.1 = \delta_2 = \frac{9}{4} \cdot \Delta l_2 \Rightarrow \Delta l_2 = \frac{4}{9} \cdot 13.1 = 5.8\text{ mm} \\
&\Delta l_2 = \frac{F_2 \cdot l_2}{E \cdot A_2} \Rightarrow A_2 = \frac{F_2 \cdot l_2}{E \cdot \Delta l_2} = \frac{900 \cdot 10^3 \cdot 9000}{2.1 \cdot 10^5 \cdot 5.8} = 6650\text{ mm}^2 \\
&\qquad\qquad\qquad\qquad \Rightarrow \text{tie } 2 \varnothing\, 95\text{ mm } (7088\,\text{mm}^2)
\end{aligned} \tag{18.5}
$$

18.2.4 Deformation-driven design of the struts

For such a statically indeterminate and thus deformation-driven 3-D system, the required stiffness and corresponding cross-sectional dimensions of the struts can be determined assuming the ties are infinitely stiff.

For both the short and the long span, the shortening Δl of the strut and corresponding displacement δ at the location of the loading can be determined as shown in figure 18.4.

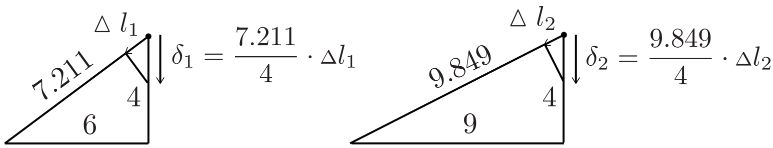


Figure 18.4: Deformation of the struts

For the long span, the cross-sectional dimension of the strut amounts to:

$$\delta_1 = \delta_2 \quad \text{with strut } 1 \varnothing\, 200 \times 10\,\text{mm}\ (6283\,\text{mm}^2)$$

$$\delta_1 = \frac{7.211}{4} \cdot \Delta l_1 = \frac{7.211}{4} \cdot \frac{721.1 \cdot 10^3 \cdot 7211}{2.1 \cdot 10^5 \cdot 6283} = 7.1\,\text{mm}$$

$$\delta_1 = 7.1 = \delta_2 = \frac{9.849}{4} \cdot \Delta l_2 \Rightarrow \Delta l_2 = \frac{4}{9.849} \cdot 7.1 = 2.9\,\text{mm}$$

$$\Delta l_2 = \frac{F_2 \cdot l_2}{E \cdot A_2} \Rightarrow A_2 = \frac{F_2 \cdot l_2}{E \cdot \Delta l_2} = \frac{984.9 \cdot 10^3 \cdot 9849}{2.1 \cdot 10^5 \cdot 2.9} = 15928\,\text{mm}^2$$

$$\Rightarrow \text{strut } 2 \varnothing\, 510 \times 10\,\text{mm}\ (16022\,\text{mm}^2) \quad (18.6)$$

18.2.5 Design of the overall system

The actual system behaviour combines the deformations and corresponding cross-sectional dimensions of both ties and struts.

These cross-sectional dimensions based on the deformation-driven design of the 3-D system are listed in table 18.3.

Cross-sectional dimensions 3-D system design		
Element	Short span	Long span
Tie	∅ 50 mm	∅ 95 mm
Strut	∅ 200 × 10 mm	∅ 510 × 10 mm

Table 18.3: Dimensions 3-D spatial struts and ties

For the actual displacement at the location of the loading of 1600 kN, both deformations of ties and struts have to be combined serially:

$$\delta_{tot} = \delta_{1,tie} + \delta_{1,strut} = \delta_{2,tie} + \delta_{2,strut} = 13.1 + 7.1 = 20.2\,\text{mm} \quad (18.7)$$

18.2.6 Material optimisation

The cross-sectional dimensions based on the strength design of both separate 2-D subsystems, as listed in table 18.2 on page 219, must not be undershooted.

However, the extra material for the required stiffness and corresponding displacement δ_2 of the long span, as listed in table 18.3 on page 221, is interchangeable:

$$\delta_{2,tie} = \frac{9}{4} \cdot \frac{900 \cdot 10^3 \cdot 9000}{2.1 \cdot 10^5 \cdot A_{2,tie}} = \frac{86786}{A_{2,tie}} \tag{18.8}$$

$$\delta_{2,strut} = \frac{9.849}{4} \cdot \frac{984.9 \cdot 10^3 \cdot 9849}{2.1 \cdot 10^5 \cdot A_{2,strut}} = \frac{113736}{A_{2,strut}} \tag{18.9}$$

The tie appears $\frac{113736}{86786} = 1.31$ more effective than the strut.

So, when strut 2 remains $\varnothing$ 280 $\times$ 10 mm $(8796\,\text{mm}^2)$ instead of the required $15928\,\text{mm}^2$,
then tie 2 has to be increased with $\frac{15928 - 8796}{1.31} = 5444\,\text{mm}^2$,
resulting in $A_{2,tie} = 6650 + 5444 = 12094\,\text{mm}^2 \Rightarrow \varnothing\,125\,\text{mm}$.

The cross-sectional dimensions based on this material optimisation of the 3-D system are listed in table 18.4.

Cross-sectional dimensions optimised 3-D system design		
Element	Short span	Long span
Tie	∅ 50 mm	∅ 125 mm
Strut	∅ 200 × 10 mm	∅ 280 × 10 mm

Table 18.4: Dimensions material optimisation

18.2.7 Redundancy analysis

The cross-sectional dimensions as listed in table 18.2 on page 219 are based on a statically determinate strength design and consequently have none residual strength. Only the extra material for the required stiffness of the long span within the statically indeterminate system design does have residual strength.

In case of an overload of the short span subsystem and corresponding yielding of tie 1, the following redundancy can generally be generated by the long span subsystem:

Short span The short span subsystem will maintain its full load-bearing capacity but with zero stiffness, whereby the short span struts still stabilise the top joint.

Long span Remains a statically determinate force driven 3-D system with residual strength due to the extra material of the long span subsystem, but with an overload of the corresponding bearings.

The non-optimised system with extra material for both struts and tie of the long span subsystem, as listed in table 18.3 on page 221, can generate such a redundancy.

However, the material-optimised system has no residual strength at all due to a lack of extra material for the struts of the long span subsystem, as listed in table 18.4 on page 222. So material optimisation and redundancy perform like communicating vessels.

Chapter 19

3-D geometry combined with simple modelling

19.1 Training 1 - Spatial trusses

19.1.1 Intermediate bracing system with trusses

Given a 3-D bracing system with trusses as shown in figure 19.1. Such a structure typically can be utilised as an intermediate load distribution layer between a new and an underlying existing building.

For this training, the following requirements are applicable:

- Cross-sectional dimensions have to be determined for the ULS, the SLS is not applicable.
- The load-bearing capacity of the 4 bearings is limited to 400 kN each.
- The truss members consist of structural steel square RHS with wall-thickness $t = 5$ mm and normal stress strength $f_y = 355$ N/mm^2. The section properties of a square RHS, with approximations based on a relatively small wall thickness, are given in figure 19.2.

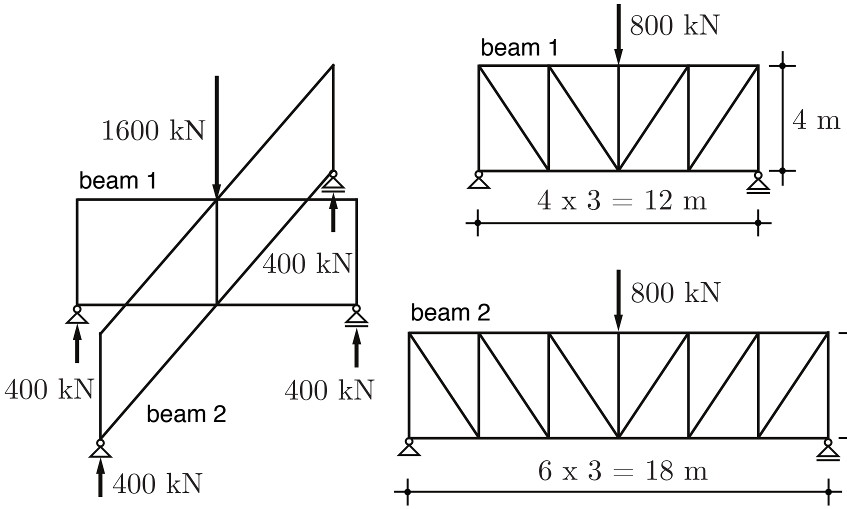
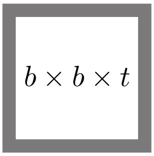


Figure 19.1: Bracing system with trusses

- The top joints of the trusses are laterally supported by a concrete floor slab.
- The structural self-weight may be neglected in comparison with the loading of 1600 kN.



$$A \approx 4bt$$
$$I \approx \tfrac{2}{3}b^3t \approx \tfrac{1}{6}Ab^2$$
$$W \approx \tfrac{4}{3}b^2t \approx \tfrac{1}{3}Ab$$

Figure 19.2: Section properties of a square RHS

Analyse and determine the cross-sectional dimensions of this statically indeterminate structure with the following problem-solving approach:

1. Consider the separate statically determinate simply supported 2-D subsystems and determine the individual member forces.

2. Then, with these member forces, determine the corresponding cross-sectional dimensions.

3. Now, model the complete 3-D system and determine the cross-sectional dimensions of the chord members, assuming the web members are infinitely stiff.

4. With this 3-D model, determine the cross-sectional dimensions of the web members, this time assuming the chord members are infinitely stiff.

5. Determine the cross-sectional dimensions of the overall system, based on standardised square RHS sections.

19.1.2 Load distribution in the separate 2-D trusses

For the statically determinate simply supported 2-D subsystems, the individual member forces for both the short and the long span can be determined as shown in figure 19.3.

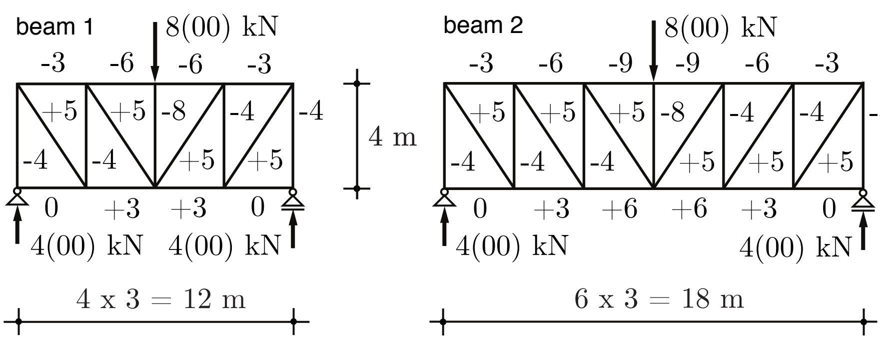


Figure 19.3: Load distribution 2-D trusses

19.1.3 Strength design of the separate 2-D trusses

For the short span, the cross-sectional dimension of the chord members amounts to:

$$A \geq \frac{N_d}{f_y} = \frac{600 \cdot 10^3}{355} = 1690\,\text{mm}^2 \quad \text{and}$$
$$I \geq N_d \cdot 1.7 \cdot \frac{L_{cr}^2}{\pi^2 E} = 600 \cdot 10^3 \cdot 1.7 \cdot \frac{3000^2}{\pi^2 \cdot 2.1 \cdot 10^5} = 443 \cdot 10^4\,\text{mm}^4$$
$$\Rightarrow \square\, 120 \times 5\,\text{mm} \qquad (19.1)$$

For the short span, the cross-sectional dimension of the web members amounts to:

$$A \geq \frac{N_d}{f_y} = \frac{500 \cdot 10^3}{355} = 1408\,\text{mm}^2 \quad \text{and}$$
$$I \geq N_d \cdot 1.7 \cdot \frac{L_{cr}^2}{\pi^2 E} = 400 \cdot 10^3 \cdot 1.7 \cdot \frac{4000^2}{\pi^2 \cdot 2.1 \cdot 10^5} = 525 \cdot 10^4\,\text{mm}^4$$
$$\Rightarrow \square\, 120 \times 5\,\text{mm} \qquad (19.2)$$

For the long span, the cross-sectional dimension of the chord members amounts to:

$$A \geq \frac{N_d}{f_y} = \frac{900 \cdot 10^3}{355} = 2535\,\text{mm}^2 \quad \text{and}$$
$$I \geq N_d \cdot 1.7 \cdot \frac{L_{cr}^2}{\pi^2 E} = 900 \cdot 10^3 \cdot 1.7 \cdot \frac{3000^2}{\pi^2 \cdot 2.1 \cdot 10^5} = 664 \cdot 10^4\,\text{mm}^4$$
$$\Rightarrow \square\, 130 \times 5\,\text{mm} \qquad (19.3)$$

For the long span, the cross-sectional dimension of the web members amounts to:

$$A \geq \frac{N_d}{f_y} = \frac{500 \cdot 10^3}{355} = 1408\,\text{mm}^2 \quad \text{and}$$
$$I \geq N_d \cdot 1.7 \cdot \frac{L_{cr}^2}{\pi^2 E} = 400 \cdot 10^3 \cdot 1.7 \cdot \frac{4000^2}{\pi^2 \cdot 2.1 \cdot 10^5} = 525 \cdot 10^4\,\text{mm}^4$$
$$\Rightarrow \square\, 120 \times 5\,\text{mm} \qquad (19.4)$$

The cross-sectional dimensions based on the strength design of the separate 2-D subsystems are listed in table 19.1.

Cross-sectional dimensions 2-D subsystems design		
Member	Short span	Long span
Chord	□ 120 × 5 mm	□ 130 × 5 mm
Web	□ 120 × 5 mm	□ 120 × 5 mm

Table 19.1: Dimensions 2-D trusses

19.1.4 Deformation driven design of the chord members

For such a statically indeterminate and thus deformation-driven 3-D system, the required stiffness and corresponding cross-sectional dimensions of the chord members can be determined assuming the web members are infinitely stiff.

For both the short and the long span, the bending deformation is curved due to axial deformation of the chord members and with a shared displacement δ at the location of the loading as shown in figure 19.4.

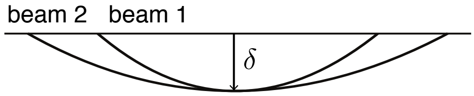


Figure 19.4: Bending deformation of the truss beams

The required stiffness and corresponding cross-sectional dimensions of the chord members amount to:

$$\delta_1 = \delta_2 \Rightarrow \frac{F \cdot 12^3}{48 \cdot EI_1} = \frac{F \cdot 18^3}{48 \cdot EI_2} \Rightarrow \frac{I_1}{I_2} = 0.30 \quad \text{and}$$
$$I_{truss} = 2 \cdot A_{chord} \cdot \left(\tfrac{1}{2}h\right)^2 \Rightarrow I_{truss} \sim A_{chord} \quad (19.5)$$

$$(h \times t)_{chord,1} = 120 \times 5\,\text{mm}$$
$$\Rightarrow (h \times t)_{chord,2} = \left(\frac{120}{0.30}\right) \times 5 \approx 400 \times 5\,\text{mm} \quad (19.6)$$

19.1.5 Deformation driven design of the web members

For such a statically indeterminate and thus deformation-driven 3-D system, the required stiffness and corresponding cross-sectional dimensions of the web members can be determined assuming the chord members are infinitely stiff.

For both the short and the long span, the shear deformation is linear due to axial deformation of the web members and with a shared displacement δ at the location of the loading as shown in figure 19.5.

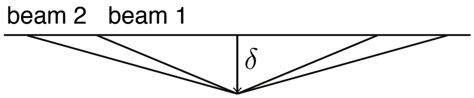


Figure 19.5: Shear deformation of the truss beams

For both the short and the long span, the linear shear deformation, underlying axial deformation of the web members, and corresponding displacement δ at the location of the loading are shown in figure 19.6.

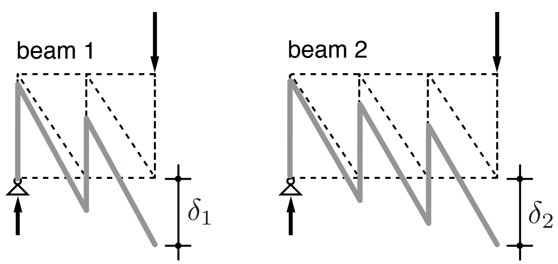


Figure 19.6: Deformation of the web members

The required stiffness and corresponding cross-sectional dimensions of the web members amount to:

$$\delta_1 = \delta_2 \Rightarrow 2\delta_{v,1} + 2\delta_{d,1} = 3\delta_{v,2} + 3\delta_{d,2} \Rightarrow \frac{A_1}{A_2} = \frac{2}{3} \tag{19.7}$$

$$(h \times t)_{web,1} = 120 \times 5\,\mathrm{mm}$$
$$\Rightarrow (h \times t)_{web,2} = \left(\frac{3}{2} \cdot 120\right) \times 5 \approx 180 \times 5\,\mathrm{mm} \quad (19.8)$$

19.1.6 Design of the overall system

The truss beams, with a slenderness between slender and stocky, are subject to both significant bending and shear deformation. The actual system behaviour combines these deformations and corresponding cross-sectional dimensions of both chord and web members.

These cross-sectional dimensions based on the deformation-driven design of the 3-D system are listed in table 19.2.

Cross-sectional dimensions 3-D system design		
Member	Short span	Long span
Chord	□ 120 × 5 mm	□ 400 × 5 mm
Web	□ 120 × 5 mm	□ 180 × 5 mm

Table 19.2: Dimensions 3-D spatial trusses

The final cross-sectional dimensions based on standardised square RHS sections in accordance with the required strength and stiffness ratios are listed in table 19.3.

Cross-sectional dimensions optimised 3-D system design		
Member	Short span	Long span
Chord	□ 120 × 5 mm	□ 200 × 10 mm
Web	□ 120 × 6.3 mm	□ 140 × 8 mm

Table 19.3: Standardised square RHS sections

19.2 Training 2 - Box truss with torsion

19.2.1 3-D square box truss with eccentric loading

Given a cantilevered 3-D square box truss with eccentric loading F as shown in figure 19.7. The square frames are connected with chords and the top, bottom, and side planes are stabilised with X-bracings (not all bracings are shown in figure 19.7).

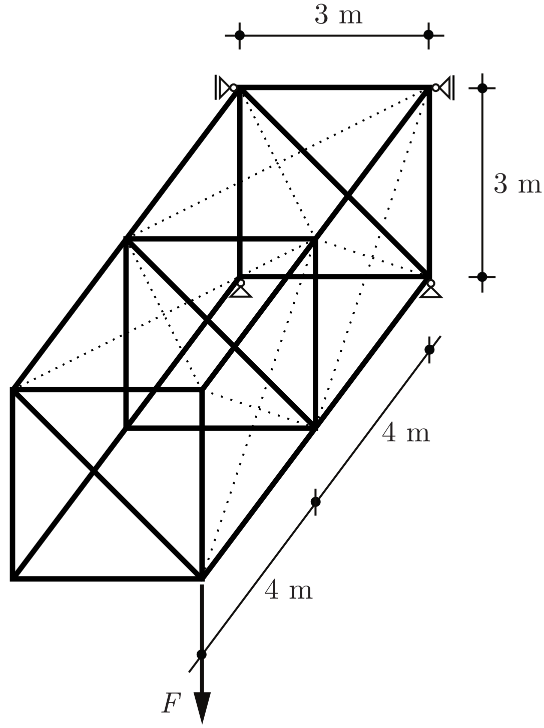


Figure 19.7: 3-D square box truss with eccentric loading

For this training, the following requirements are applicable:

- The load-bearing capacity F has to be determined both for the ULS with $F = F_d/1.5$ and the SLS with $F = F_k$.

- For the SLS the displacement at the location of the loading F is limited to $l/125$.
- The square frames and chords consist of structural steel RHS $\square 120 \times 120 \times 8$ mm ($A = 3515$ mm^2 and $I = 725.8 \cdot 10^4$ mm^4) with normal stress strength $f_y = 275$ N/mm^2.
- The X-bracings consist of high strength steel round bars $\varnothing$ 20 mm ($A = 314.2$ mm^2) with normal stress strength $f_y = 1200$ N/mm^2.
- The structural self-weight may be neglected in comparison with the loading F.

Analyse and determine the load-bearing capacity F of this statically determinate structure with the following problem-solving approach:

1. Consider only the right side plane as a simplified 2-D model for the load-bearing capacity of the complete 3-D system. With this 2-D model, determine the individual member forces expressed in F and then determine the load-bearing capacity F for the ULS.
2. With this 2-D model, determine the displacement at the location of the loading F and then determine the load-bearing capacity F for the SLS.
3. Now, model the complete system with a 3-D model. Split the eccentric loading F in a centric force and a torsion couple and then determine the load-bearing capacity F for the ULS.
4. With this 3-D model, determine the displacement at the location of the loading F and then determine the load-bearing capacity F for the SLS.
5. Analyse and explain the difference in load-bearing capacity between the simplified 2-D model and the 3-D model.
6. Explain why the transition slenderness of $l/h = 1.5$ for cantilevered stocky to slender beams as shown in figure 15.3 on page 168 is not applicable for this case.

19.2.2 Simplified 2-D modelling ULS

Only the right side plane is considered as a simplified 2-D model for the load-bearing capacity of the complete 3-D system.

With this 2-D model, the individual member forces expressed in F can be determined as shown in figure 19.8.

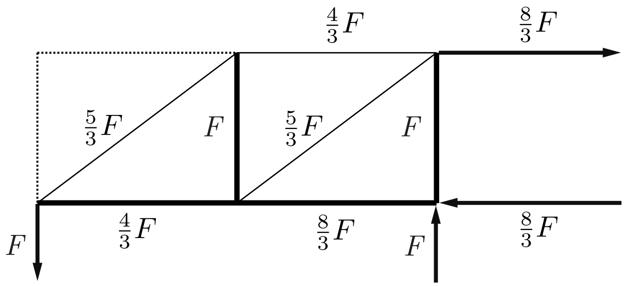


Figure 19.8: Individual member forces

Axial strength:

$$N_u = A \cdot f_s = 314.2 \cdot 1200 \cdot 10^{-3} = 377.0\,\text{kN}$$
$$\leq 3515 \cdot 275 \cdot 10^{-3} = 966.6\,\text{kN} \quad (19.9)$$

Buckling strength:

$$N_u = \frac{\pi^2 EI}{1.7 L_{cr}^2} = \frac{\pi^2 \cdot 2.1 \cdot 10^5 \cdot 725.8 \cdot 10^4}{1.7 \cdot 4000^2} \cdot 10^{-3}$$
$$= 553.1\,\text{kN} \quad (19.10)$$

Then, the load-bearing capacity F for the ULS can be determined:

$$F = \frac{F_d}{1.5} = \frac{1}{1.5} \cdot \frac{3}{8} \cdot 553.1 = \underline{\underline{138.3}}\,\text{kN}$$
$$\leq \frac{1}{1.5} \cdot \frac{3}{5} \cdot 377.0 = 150.8\,\text{kN} \quad (19.11)$$

19.2.3 Simplified 2-D modelling SLS

With equal normal stiffness modulus E and more than an order of magnitude difference between A_{tube} (3515 mm^2) and A_{bar} (314.2 mm^2), a displacement at the location of the loading F will be largely caused by elongation of the bars.

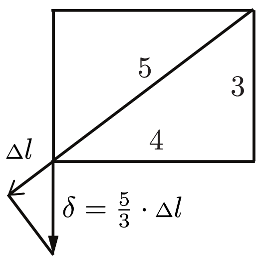


Figure 19.9: Displacement due to the elongation of a bar

The displacement due to the elongation of the bars, as shown for one bar in figure 19.9, amounts to:

$$\delta_{tot} = 2 \cdot \frac{5}{3} \cdot \frac{\frac{5}{3} F \cdot l}{E \cdot A} \leq \frac{2 \cdot 4000}{125} = 64 \,\text{mm} \tag{19.12}$$

Then, the load-bearing capacity F for the SLS can be determined:

$$F = 64 \cdot \frac{1}{2} \cdot \left(\frac{3}{5}\right)^2 \cdot \frac{E \cdot A}{l} = 64 \cdot \frac{1}{2} \cdot \left(\frac{3}{5}\right)^2 \cdot \frac{2.1 \cdot 10^5 \cdot 314.2}{5000} \cdot 10^{-3}$$
$$= 152.0 \,\text{kN} \; (\geq 138.3 \,\text{kN}) \tag{19.13}$$

19.2.4 3-D system modelling ULS

Now, the complete system is 3-D modelled.

The eccentric loading F is split in a centric force and a torsion couple as shown in figure 19.10.

Geometry, section properties, and loading pattern of top, bottom, and side planes are completely identical. Only magnitude and direction of the loading varies.

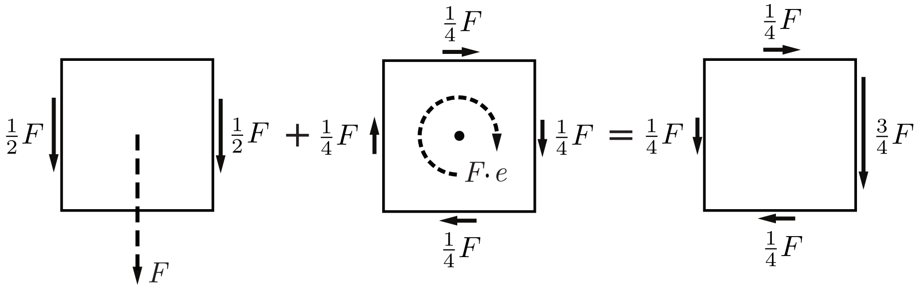


Figure 19.10: Centric force and torsion couple

Maximum load and strength of frames and chords:

$$N_{max} = \frac{3}{4} \cdot \frac{8}{3} F \,(\text{compression}) - \frac{1}{4} \cdot \frac{4}{3} F \,(\text{tension}) = \frac{5}{3} F$$
$$\text{and} \quad N_u = 553.1\,\text{kN} \quad (19.14)$$

The maximum loaded rear bottom right chord is in compression by a downwards force $\frac{3}{4}F$ on the right side plane and in tension by a leftwards force $\frac{1}{4}F$ on the bottom plane.

Maximum load and strength of X-bracings:

$$N_{max} = \frac{3}{4} \cdot \frac{5}{3} F = \frac{5}{4} F \quad \text{and} \quad N_u = 377.0\,\text{kN} \quad (19.15)$$

Then, the load-bearing capacity F for the ULS can be determined:

$$F = \frac{F_d}{1,5} = \frac{1}{1.5} \cdot \frac{3}{5} \cdot 553.1 = 221.2\,\text{kN}$$
$$\geq \frac{1}{1.5} \cdot \frac{4}{5} \cdot 377.0 = \underline{\underline{\underline{201.1}}}\,\text{kN} \quad (19.16)$$

19.2.5 3-D system modelling SLS

The governing displacement of the right side plane amounts to:

$$\delta_{tot} = 2 \cdot \frac{5}{3} \cdot \frac{\frac{5}{4} F \cdot l}{E \cdot A} \leq \frac{2 \cdot 4000}{125} = 64\,\text{mm} \quad (19.17)$$

Then, the load-bearing capacity F for the SLS can be determined:

$$F = 64 \cdot \frac{1}{2} \cdot \frac{3}{5} \cdot \frac{4}{5} \cdot \frac{E \cdot A}{l} = 64 \cdot \frac{1}{2} \cdot \frac{3}{5} \cdot \frac{4}{5} \cdot \frac{2.1 \cdot 10^5 \cdot 314.2}{5000} \cdot 10^{-3}$$
$$= 202.7\,\mathrm{kN} \;\; (\geq 201.1\,\mathrm{kN}) \qquad (19.18)$$

19.2.6 2-D versus 3-D modelling

The load-bearing capacity F of the simplified 2-D model versus the 3-D system model is listed in table 19.4.

Load-bearing capacity F		
Limit state	Simplified 2-D model	3-D system model
ULS	$F = 138.3$ kN	$F = 201.1$ kN
SLS	$F = 152.0$ kN	$F = 202.7$ kN

Table 19.4: 2-D versus 3-D modelling

The doubling of load-bearing material with an extra left side plane has not resulted in a doubling of the load-bearing capacity F, due to an eccentricity of the load and corresponding introduction of torsion in the 3-D modelling.

However, the actual additional material will generally generate more strength and stiffness as quantified in table 19.4.

19.2.7 Stocky or slender system behaviour

The transition slenderness of $l/h = 1.5$ for cantilevered stocky to slender beams, as shown in figure 15.3 on page 168, is not applicable for this case.

As a result of the high strength steel, the X-bracings are an order of magnitude more flexible than the frames and chords. So, shear deformation of the 3-D square box truss is governing, resulting in a stocky behaviour of the structure despite the fact that $l/h = 2.7 \geq 1.5$.

Chapter 20

2-D geometry combined with medium modelling

20.1 Training 3 - Combined truss and arch

20.1.1 Truss girder bridge building combined with arch

Given a truss girder bridge building combined with a tied arch as shown in figure 20.1. The truss girder is combined with an arch out of architectural considerations.

For this training, the following requirements are applicable:

- The floor and roof loading amounts to $q_d = 1.2 \cdot 15 + 1.5 \cdot 10 = 33\ \mathrm{kN/m^1}$ respectively $q_d = 1.2 \cdot 10 + 1.5 \cdot 4 = 18\ \mathrm{kN/m^1}$, the structural self-weight may be neglected in comparison with this loading.
- For reasons of redundancy both the truss girder and the tied arch have to bear half the load.
- For the SLS the displacement is limited to $l/500$.
- The truss members and arch consist of structural steel CHS $\left(20 \leq \frac{d}{t} \leq 30\right)$

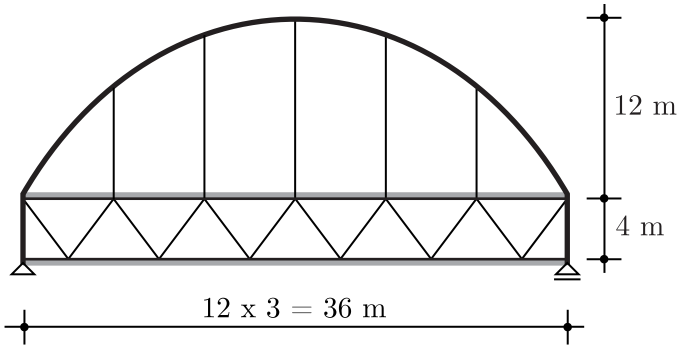


Figure 20.1: Truss girder bridge building combined with tied arch

with normal stress strength $f_y = 460$ N/mm^2. The section properties of a CHS, with approximations based on a relatively small wall-thickness, are given in figure 17.4 on page 198.

- The hangers consist of structural steel round bars with normal stress strength $f_y = 460$ N/mm^2.
- For in-plane buckling of the arch, a buckling length of $0.5l_{arch} \approx$ 23 m is appropriate. Out-of-plane buckling is prevented by lateral bracing.
- The deformation of the arch is negligible relative to the elongation of the hangers.

Analyse and determine the cross-sectional dimensions of this statically indeterminate structure with the following problem-solving approach:

1. Consider the separate statically determinate simply supported truss girder subsystem with half the load and determine the member forces, corresponding cross-sectional dimension of the chord members, and the displacement in the ULS.
2. Consider the separate statically determinate simply supported tied arch subsystem with half the load and determine the member forces, corresponding cross-sectional dimensions, the influence of

an eccentric loading, and the displacement in the ULS.

3. Now, model the complete statically indeterminate system with combined truss girder and tied arch and determine the actual load distribution between the truss girder and the tied arch.
4. With this modelled system, analyse and determine how the utilisation of high strength steel can realise the required equal load distribution between the truss girder and the tied arch.
5. For the chord members of the truss girder, analyse and optimise the amount of structural material on system level.
6. With this modelled system, determine and check the displacement in the SLS.
7. For the web members of the truss girder, determine the actual member forces on system level and corresponding cross-sectional dimension.

20.1.2 Strength design of the separate truss girder

For the separate statically determinate simply supported truss girder subsystem with half the load, the maximum chord and web member forces amount to:

$$N_{chord,\,max} = \frac{\frac{1}{8} q_d l^2}{h} = \frac{\frac{1}{8} \cdot 25.5 \cdot 36^2}{4} = 1032.8\,\text{kN} \quad \text{and}$$
$$N_{web,\,max} = \frac{\frac{1}{2} q_d l}{\cos\alpha} = \frac{5}{4} \cdot \tfrac{1}{2} \cdot 25.5 \cdot 36 = 573.8\,\text{kN} \quad (20.1)$$

The cross-sectional dimension of the chord members amounts to:

$$A_{chord} \geq \frac{N_d}{f_y} = \frac{1032.8 \cdot 10^3}{460} = 2245\,\text{mm}^2 \quad \text{and}$$
$$I_{chord} \geq N_d \cdot 1.7 \cdot \frac{L_{cr}^2}{\pi^2 E} = 1032.8 \cdot 10^3 \cdot \frac{1.7 \cdot 6000^2}{\pi^2 \cdot 2.1 \cdot 10^5} = 3050 \cdot 10^4\,\text{mm}^4$$
$$\Rightarrow \varnothing\, 200 \times 10\,\text{mm}\,\left(A_{chord} = 6283\,\text{mm}^2\right) \quad (20.2)$$

The deformation of the separate truss girder in the ULS amounts to:

$$\delta = \frac{5}{384} \cdot \frac{q_d l^4}{EI_{truss}} \qquad \text{and } I_{truss} \approx 0.8 \cdot 2 \cdot A_{chord} \cdot \left(\tfrac{1}{2}h\right)^2$$
$$\Rightarrow \delta_{truss} = \frac{5}{384} \cdot \frac{q_d l^4}{E \cdot 0.8 \cdot 2 \cdot A_{chord} \cdot \left(\tfrac{1}{2}h\right)^2}$$
$$= \frac{5}{384} \cdot \frac{25.5 \cdot 36000^4}{2.1 \cdot 10^5 \cdot 0.8 \cdot 2 \cdot 6283 \cdot 2000^2} = 66.0\,\text{mm} \quad (20.3)$$

20.1.3 Strength design of the separate tied arch

For the separate statically determinate simply supported tied arch subsystem with half the load, the tie and maximum arch forces amount to:

$$H = \frac{\frac{1}{8} q_d l^2}{h} = \frac{\frac{1}{8} \cdot 25.5 \cdot 36^2}{12} = 344.3\,\text{kN} = N_{tie} = N_{arch,\,mid}$$
$$\text{and} \quad N_{arch,\,max} = \sqrt{\left(\tfrac{1}{2} q_d l\right)^2 + H^2} = 573.8\,\text{kN} \quad (20.4)$$

The cross-sectional dimension of the arch amounts to:

$$A_{arch} \geq \frac{N_d}{f_y} = \frac{\frac{1}{2}\,(344.3 + 573.8) \cdot 10^3}{460} = 998\,\text{mm}^2 \quad \text{and}$$
$$I_{arch} \geq N_d \cdot 1.7 \cdot \frac{L_{cr}^2}{\pi^2 E} = \tfrac{1}{2}\,(344.3 + 573.8) \cdot 10^3 \cdot \frac{1.7 \cdot 23000^2}{\pi^2 \cdot 2.1 \cdot 10^5}$$
$$= 19918 \cdot 10^4\,\text{mm}^4 \Rightarrow \varnothing\, 320 \times 16\,\text{mm} \quad (20.5)$$

And the cross-sectional dimension of the hanger amounts to:

$$A_{hang} \geq \frac{N_d}{f_y} = \frac{25.5 \cdot 6000}{460} = 333\,\text{mm}^2$$
$$\Rightarrow \varnothing\, 21\,\text{mm}\,\left(A_{hang} = 346\,\text{mm}^2\right) \quad (20.6)$$

A bending moment within the arch, due to an eccentric loading of half the variable uniform loading, appears not to be decisive:

$$M_{arch,\,max} \approx \tfrac{1}{64} q_d l^2 = \tfrac{1}{64}\left(\frac{1.5 \cdot 10}{2}\right) \cdot 36^2 = 151.9\,\text{kNm}$$

$$W_{arch} \geq \frac{M_{arch,\,max}}{f_y} = \frac{151.9 \cdot 10^6}{460} = 330.2 \cdot 10^3\,\text{mm}^3$$

$$\Rightarrow \text{Buckling is decisive } \varnothing\, 320 \times 16\,\text{mm} \quad (20.7)$$

The deformation of the arch is stated negligible relative to the elongation of the hangers, so the deformation of the separate tied arch in the ULS is equal to the elongation of the middle hanger and amounts:

$$\delta_{arch} = \delta_{hang} = \frac{N_d l}{EA} = \frac{(25.5 \cdot 6000) \cdot 12000}{2.1 \cdot 10^5 \cdot 346} = 25.3\,\text{mm} \quad (20.8)$$

20.1.4 Load distribution combined truss girder and arch

The complete system with combined truss girder and tied arch is a statically indeterminate structure and therefore the load distribution is deformation driven, whereby the deformation of the truss equals the elongation of the middle hanger as shown in figure 20.2.

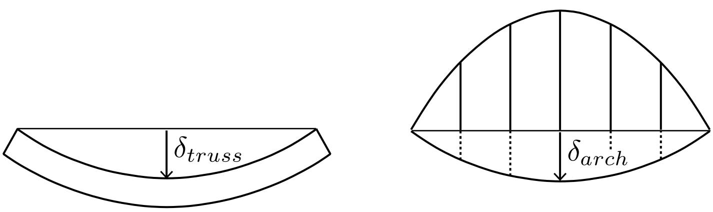


Figure 20.2: Displacement combined truss girder and tied arch

The distribution of the total load of $q_d = 51\,\text{kN/m}^1$ between the truss girder and the tied arch amounts:

$$q_{truss} = \frac{\delta_{arch}}{\delta_{truss} + \delta_{arch}} \cdot q_{d,tot} = \frac{25.3}{66.0 + 25.3} \cdot 51$$

$$= 14.1\,\text{kN/m}^1 \quad (20.9)$$

$$q_{arch} = \frac{\delta_{truss}}{\delta_{truss} + \delta_{arch}} \cdot q_{d,tot} = \frac{66.0}{66.0 + 25.3} \cdot 51 = 36.9\,\mathrm{kN/m^1} \quad (20.10)$$

20.1.5 Equal load distribution with high strength steel

For reasons of redundancy both the truss girder and the tied arch have to bear half the load, so the actual difference in stiffness between these two subsystems can possibly be corrected with another steel grade.

A lower steel grade for the truss girder is useless due to the decisive buckling strength.

A higher steel grade for the hangers of the tied arch is highly effective, whereby the round bars can also be replaced by cable strands:

$$\delta_{arch} = \delta_{hang} = \frac{Nl}{EA} = \delta_{truss} \Rightarrow A_{hang} = \frac{N \cdot l}{E \cdot \delta_{truss}} \quad (20.11)$$

$$A_{hang} = \frac{N}{f_y} \Rightarrow f_y = \frac{N}{A_{hang}} = \frac{N}{\frac{N \cdot l}{E \cdot \delta_{truss}}} = \frac{E \cdot \delta_{truss}}{l} = \frac{2.1 \cdot 10^5 \cdot 66.0}{12000} = 1155\,\mathrm{N/mm^2} \quad (20.12)$$

20.1.6 Material optimisation on system level

On system level the interaction between the top chord member in compression and the arch tie in tension can lead to a material optimisation of the chord members:

$$I_{chord} \geq (N_{chord} - N_{tie}) \cdot 1.7 \cdot \frac{L_{cr}^2}{\pi^2 E} = (1032.8 - 344.3) \cdot 10^3 \cdot 1.7 \cdot \frac{6000^2}{\pi^2 \cdot 2.1 \cdot 10^5} = 2033 \cdot 10^4\,\mathrm{mm}^4 \Rightarrow \varnothing\, 200 \times 7\,\mathrm{mm}\,\left(A_{chord} = 4398\,\mathrm{mm}^2\right) \quad (20.13)$$

For the bottom chord member in full tension this cross-sectional area is still sufficient:

$$A_{chord} = 4398\,\text{mm}^2 \geq A_{min} = 2245\,\text{mm}^2 \tag{20.14}$$

Then, the higher steel grade and corresponding cross-sectional area of the hangers amount to:

$$\begin{aligned}\delta_{truss} &= \frac{5}{384} \cdot \frac{q_d l^4}{E \cdot 0.8 \cdot 2 \cdot A_{chord} \cdot \left(\frac{1}{2}h\right)^2} \\ &= \frac{5}{384} \cdot \frac{25.5 \cdot 36000^4}{2.1 \cdot 10^5 \cdot 0.8 \cdot 2 \cdot 4398 \cdot 2000^2} = 94.3\,\text{mm}\end{aligned} \tag{20.15}$$

$$\begin{aligned}f_y = \frac{E \cdot \delta_{truss}}{l} = \frac{2.1 \cdot 10^5 \cdot 94.3}{12000} = 1650\,\text{N/mm}^2 \\ \Rightarrow A_{hang} = \frac{N_d}{f_y} = \frac{25.5 \cdot 6000}{1650} = 93\,\text{mm}^2\end{aligned} \tag{20.16}$$

20.1.7 Displacement in the SLS

Both Parallel subsystems truss girder and tied arch bear half the load:

$$q_{k,arch} = q_{k,truss} = \frac{(15+10)+(10+4)}{2} = 19.5\,\text{kN/m}^1 \tag{20.17}$$

Both Parallel subsystems have the same corresponding displacement and satisfy the requirement for the SLS:

$$\begin{aligned}\delta = \delta_{arch} = \delta_{truss} &= \frac{5}{384} \cdot \frac{q_{k,truss} l^4}{E \cdot 0.8 \cdot 2 \cdot A_{chord} \cdot \left(\frac{1}{2}h\right)^2} \\ = \frac{5}{384} \cdot \frac{19.5 \cdot 36000^4}{2.1 \cdot 10^5 \cdot 0.8 \cdot 2 \cdot 4398 \cdot 2000^2} &= 72\,\text{mm} \\ \leq \frac{l}{500} = \frac{36000}{500} &= 72\,\text{mm}\end{aligned} \tag{20.18}$$

The tied arch, with equal displacement as the truss girder, can serve as a verification:

$$\delta_{arch} = \delta_{hang} = \frac{N_k l}{EA} = \frac{(19.5 \cdot 6000) \cdot 12000}{2.1 \cdot 10^5 \cdot 93} = 72\,\mathrm{mm} \qquad (20.19)$$

20.1.8 Strength design web members on system level

The truss load distribution in the web members can be determined by a transformation of the uniform floor and roof loading into point loads onto the truss joints as shown in figure 20.3.

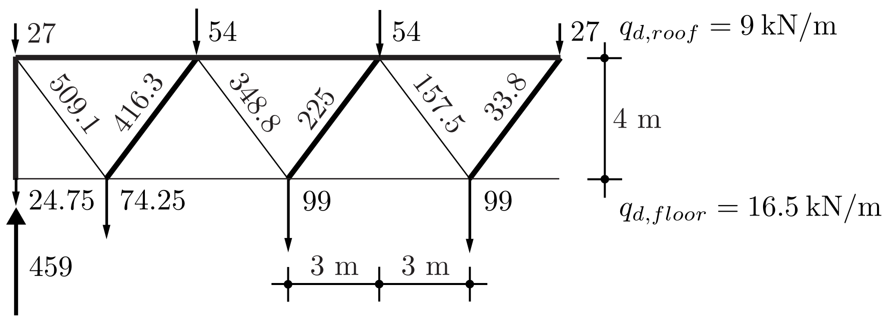


Figure 20.3: Truss load distribution web members

The arch load distribution in the web members can be determined by a transfer of the floor loading via the web members into the hangers as shown in figure 20.4.

The actual load distribution in the web members is a combination of the truss load distribution and the arch load distribution, with a maximum tension force of $509.1 + 61.9 = 571.0$ kN and a maximum compression force of $416.3 - 61.9 = 354.4$ kN.

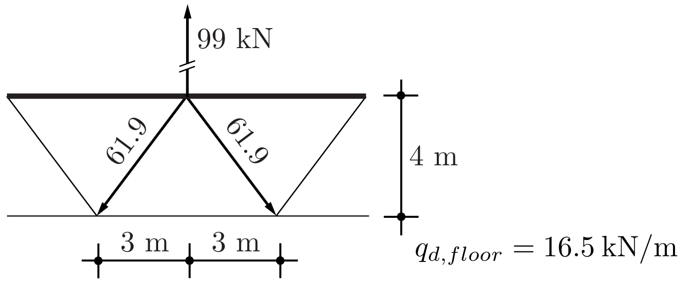


Figure 20.4: Arch load distribution web members

The corresponding critical cross-sectional dimension amounts to:

$$A_{web} \geq \frac{N_d}{f_y} = \frac{571.0 \cdot 10^3}{460} = 1241\,\text{mm}^2 \quad \text{and}$$

$$I_{web} \geq N_d \cdot 1.7 \cdot \frac{L_{cr}^2}{\pi^2 E} = 354.4 \cdot 10^3 \cdot 1.7 \cdot \frac{5000^2}{\pi^2 \cdot 2.1 \cdot 10^5} = 727 \cdot 10^4\,\text{mm}^4$$

$$\Rightarrow \varnothing\, 150 \times 6\,\text{mm} \quad (20.20)$$

20.2 Training 4 - Second-order effect arch

20.2.1 Three-hinged truss arch

Given a three-hinged truss arch as shown in figure 20.5. The point load is transferred via a semi-circular arch out of architectural considerations.

For this training, the following requirements are applicable:

- Cross-sectional dimensions of the chord members have to be determined for the ULS, the SLS is not applicable.
- The semi-circular truss arch has a radius $r = 8$ m and truss height $h = 1$ m with web members at a 45 degree angle.
- The point load amounts to $F_d = 1.2 \cdot 360 + 1.5 \cdot 270 = 837\,\text{kN}$ and the structural self-weight may be neglected in comparison with this

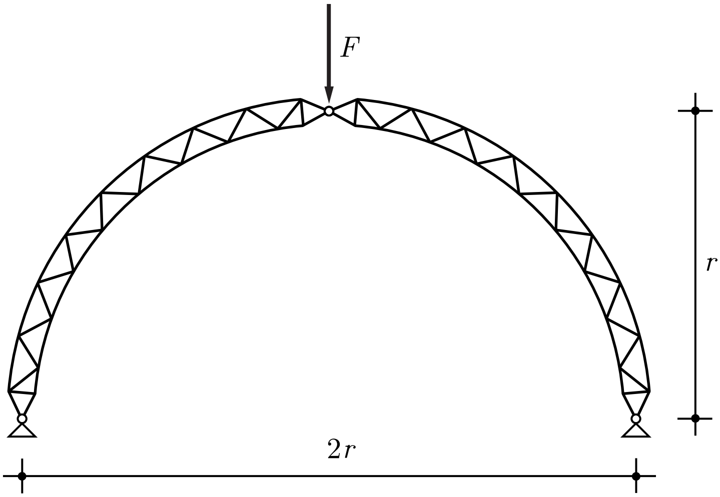


Figure 20.5: Three-hinged truss arch

load.

- The truss members consist of structural steel RHS $\left(15 \leq \frac{b}{t} \leq 25\right)$ with normal stress strength $f_y = 355\ \mathrm{N/mm^2}$. The section properties of a RHS, with approximations based on a relatively small wall-thickness, are given in figure 19.2 on page 226.

Analyse and determine the cross-sectional dimension of the chord members, including the deformation driven second-order effect of this statically determinate 2-D system, with the following problem-solving approach:

1. Consider the statically determinate semi-circular three-hinged arch and analyse the load distribution.

2. Then, determine the corresponding N- and M-lines of the arch.

3. With these N- and M-lines, determine the cross-sectional dimension of the chord members of the truss arch.

4. Analyse the second-order effect and determine whether this effect

has to be taken into account with respect to the final cross-sectional dimension of the chord members.

20.2.2 Analysis load distribution in the 2-D system

The load distribution of the point load F into the semi-circular three-hinged arch brings about a substantial first order moment $M = \frac{F}{\sqrt{2}} \cdot e$ due to the large eccentricity e as shown in figure 20.6.

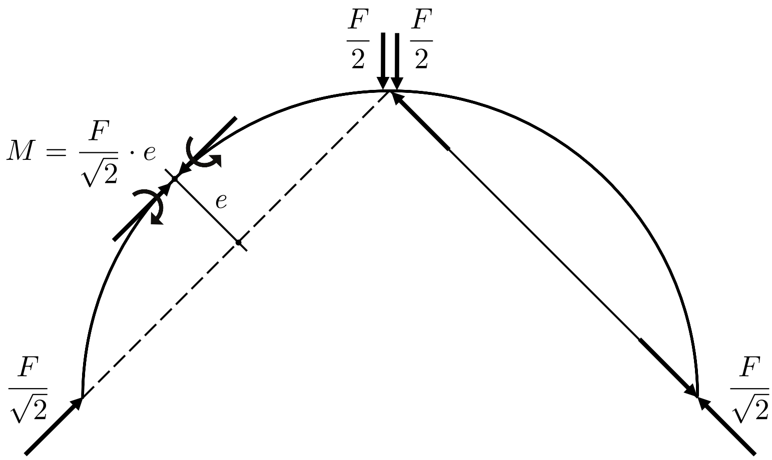


Figure 20.6: Load distribution three-hinged arch

20.2.3 N- and M-lines of the arch

For getting grip on the more complex N- and M-lines of a semi-circular arch, the N-, M-, and, V-lines of enclosing simplified structures can be exercised as shown in figure 20.7.

Now, the actual N- and M-lines of the semi-circular arch can be determined as shown in figure 20.8.

The corresponding quantification of the maximum values for the axial

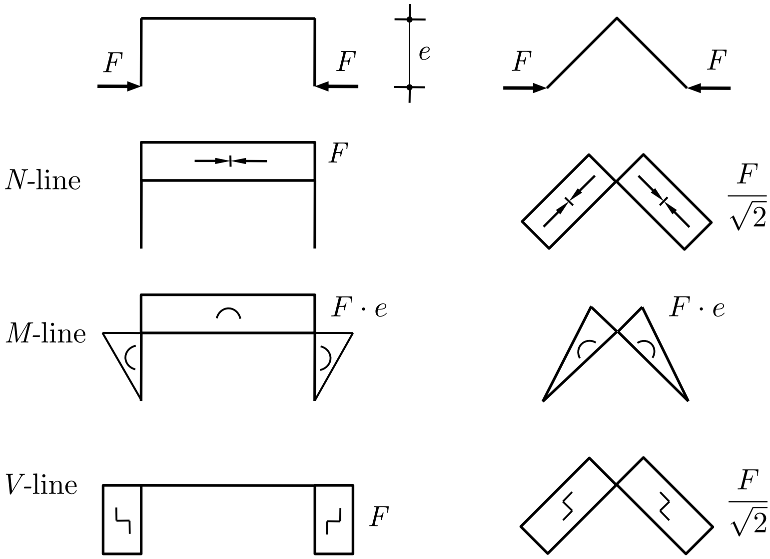


Figure 20.7: N-, M-, and, V-lines of enclosing simplified structures

force N and the bending moment M amount to:

$$N = \frac{F}{\sqrt{2}} = \frac{837}{\sqrt{2}} = 591.8\,\text{kN} \quad \text{and} \quad e = 8 - 4\sqrt{2} = 2.343\,\text{m}$$
$$\Rightarrow M = N \cdot e = 591.8 \cdot 2.343 = 1386.7\,\text{kNm} \quad (20.21)$$

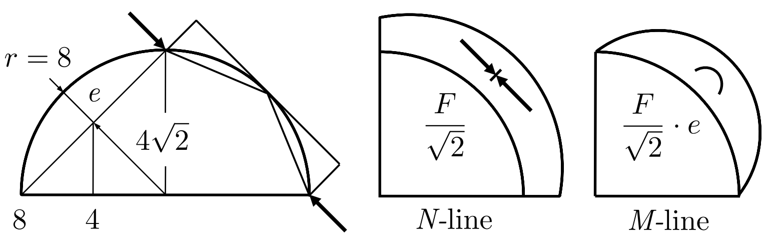


Figure 20.8: N- and M-lines of a semi-circular arch

20.2.4 Dimensioning chord members of the truss arch

The critical axial force in the chord members, as a result of the combined axial force in the arch and the bending moment due to the eccentricity of this axial force, amounts to:

$$N_d = \frac{N}{2} + \frac{M}{h} = \frac{591.8}{2} + \frac{1386.7}{1} = 1682.6\,\mathrm{kN} \tag{20.22}$$

The corresponding cross-sectional dimension amounts to:

$$\begin{aligned} A &\geq \frac{N_d}{f_y} = \frac{1682.6 \cdot 10^3}{355} = 4740\,\mathrm{mm}^2 \quad \text{and} \\ I &\geq N_d \cdot 1.7 \cdot \frac{L_{cr}^2}{\pi^2 E} = 1682.6 \cdot 10^3 \cdot 1.7 \cdot \frac{2000^2}{\pi^2 \cdot 2.1 \cdot 10^5} = 552 \cdot 10^4\,\mathrm{mm}^4 \\ &\Rightarrow \square\, 160 \times 8\,\mathrm{mm}\, \left(A_{chord} = 5120\,\mathrm{mm}^2\right) \end{aligned} \tag{20.23}$$

20.2.5 Analysis second-order effect

The first-order deformation of the hinged arch section can be approximated with the deformation of a fictive uniformly loaded simply supported beam and amounts to:

$$q_{fictive} = \frac{8M}{l^2} = \frac{8 \cdot 1386.7}{\left(8\sqrt{2}\right)^2} = 86.7\,\mathrm{kN/m} \tag{20.24}$$

$$\begin{aligned} \delta &= \frac{5}{384} \cdot \frac{q_{fictive} \cdot l^4}{E \cdot 0.8 \cdot 2 \cdot A_{chord} \cdot \left(\frac{1}{2}h\right)^2} \\ &= \frac{5}{384} \cdot \frac{86.7 \cdot \left(8000 \cdot \sqrt{2}\right)^4}{2.1 \cdot 10^5 \cdot 0.8 \cdot 2 \cdot 5120 \cdot 500^2} = 43\,\mathrm{mm} \end{aligned} \tag{20.25}$$

There are second-order effects when the first-order deformation due to loading affect the distribution of internal forces.

The approximate first-order deformation of 43 mm is more than an order of magnitude smaller than de eccentricity of 2343 mm. So, the second-order deformation has a negligible influence on the load distribution and corresponding dimensioning.

Chapter 21

1-D geometry combined with abstract modelling

21.1 Training 5 - Truss analogy concrete beam

21.1.1 Cantilevered reinforced concrete beam

Given a cantilevered reinforced concrete beam on two supports as shown in figure 21.1.

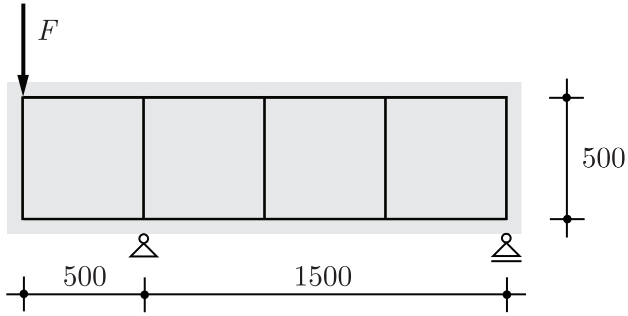


Figure 21.1: Cantilevered reinforced concrete beam

For this training, the following requirements are applicable:

- Shear strength and corresponding shear deformation of the cantilever have to be determined for the ULS, the SLS is not applicable.
- The structural self-weight may be neglected in comparison with the point load.
- Within a truss analogy, the concrete diagonal struts have an effective cross-sectional area $A_c = 5000\,\text{mm}^2$ and normal stress strength $f_c = 20\,\frac{\text{N}}{\text{mm}^2}$.
- The steel stirrup reinforcements have a cross-sectional area $A_s = 200\,\text{mm}^2$ and normal stress strength $f_s = 435\,\frac{\text{N}}{\text{mm}^2}$.

Analyse and determine the shear strength and corresponding shear deformation of the (externally) statically determinate 1-D cantilevered concrete beam on two supports, modelled by a (internally) statically determinate 2-D strut-and-tie model, with the following problem-solving approach:

1. Consider a truss analogy with a 2-D strut-and-tie model and determine the load distribution.
2. Determine the shear strength of the separate cantilever.
3. Then, determine the corresponding shear deformation of this separate cantilever.
4. Analyse the shear deformation caused by the concrete struts of the right span and determine the displacement of the cantilever, including the contribution of this right span.
5. Appoint the contribution of the other elements within the strut-and-tie model which have then to be analysed.

21.1.2 Load distribution strut-and-tie model

The member forces of the statically determinate 2-D strut-and-tie model, and corresponding V- and M-lines of the statically determinate 1-D concrete beam, are shown in figure 21.2.

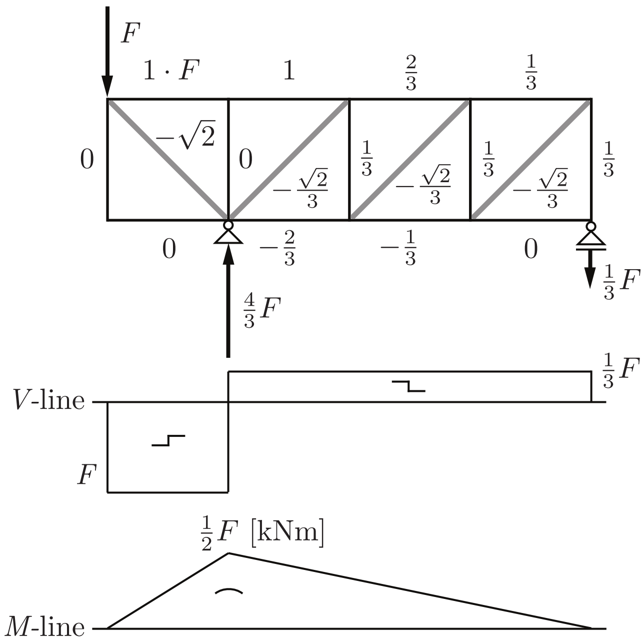


Figure 21.2: Load distribution concrete beam

21.1.3 Shear strength of the concrete cantilever

The axial strength of the concrete strut amounts to:

$$N_{u,c} = A_c \cdot f_c = 5000 \cdot 20 \cdot 10^{-3} = 100\,\mathrm{kN} \tag{21.1}$$

The axial strength of the steel tie amounts to:

$$N_{u,s} = A_s \cdot f_s = 200 \cdot 435 \cdot 10^{-3} = 87\,\mathrm{kN} \tag{21.2}$$

The shear strength of the cantilever is completely dependent on the axial strength of the concrete strut and amounts to:

$$F_u = \frac{N_{u,c}}{\sqrt{2}} = \frac{100}{\sqrt{2}} = 70.7\,\mathrm{kN} \tag{21.3}$$

21.1.4 Shear deformation of the concrete cantilever

The shear deformation of the cantilever is completely dependent on the axial deformation of the concrete strut as shown in figure 21.3.

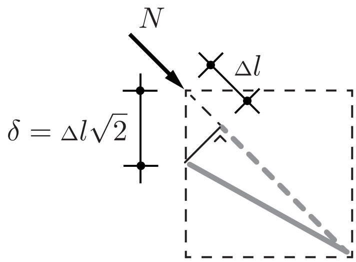


Figure 21.3: Shear deformation of the cantilever

Then the shear deformation of the cantilever amounts to:

$$\Delta l = \frac{N \cdot l}{E_c \cdot A_c} = \frac{100 \cdot 10^3 \cdot 500 \cdot \sqrt{2}}{\frac{20}{1.75 \cdot 10^{-3}} \cdot 5000} = 1.2\,\mathrm{mm}$$
$$\Rightarrow \delta = \Delta l\sqrt{2} = 1.2 \cdot \sqrt{2} = 1.7\,\mathrm{mm} \quad (21.4)$$

21.1.5 Displacement of the cantilever

Besides the cantilever, also the right span with corresponding axial deformation of the concrete struts is subject to shear deformation as shown in figure 21.4.

The right span is 3× longer, but the shear force is 3× smaller, so the shear deformation δ of the right span is equal to the shear deformation δ of the cantilever.

As a result of this shear deformation of the right span in combination with the right roller support, the displacement of the cantilever due to both the axial deformation of the concrete strut of the cantilever and the concrete struts of the right span amounts to $\delta_{tot} = \frac{4}{3}\delta$ as shown in figure 21.5.

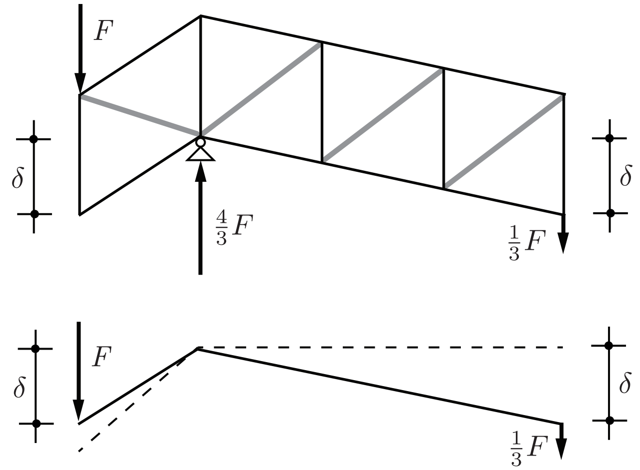


Figure 21.4: Contribution shear deformation right span

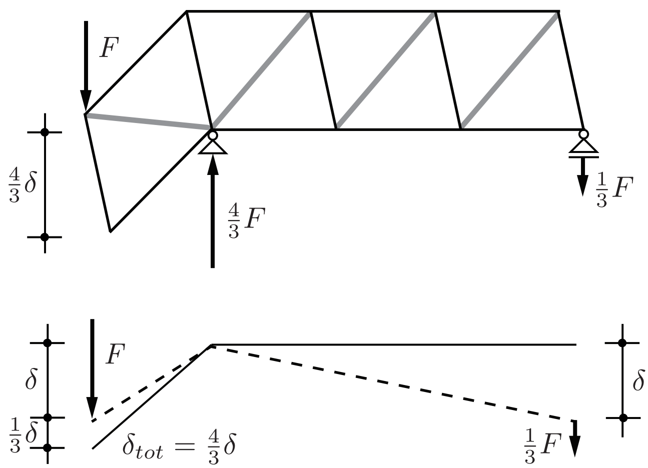


Figure 21.5: Displacement of the cantilever

21.1.6 Completion of the strut-and-tie modelling

This problem-solving approach first focussed on the contribution of the concrete diagonal struts to the deformation of the cantilever.

The contribution of the other members within the strut-and-tie model to this deformation of the cantilever has then to be analysed:

1. Deformation of the stirrup reinforcement of the right span.
2. Deformation of the longitudinal reinforcement.
3. Deformation of the longitudinal compressed concrete area.

21.2 Training 6 - Truss analogy steel beam

21.2.1 Cantilevered steel beam

Given a cantilevered steel beam on two supports as shown in figure 21.6.

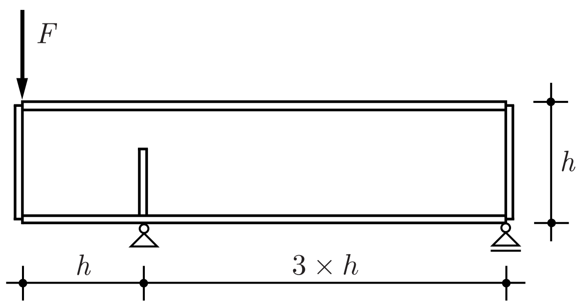


Figure 21.6: Cantilevered steel beam

For this training, the following requirements are applicable:

- Shear strength and corresponding shear deformation of the cantilever have to be analysed with a truss analogy and an effective width b_e modelling of the web members.
- The structural self-weight may be neglected in comparison with the point load.

Analyse and determine the shear strength and corresponding shear deformation of the (externally) statically determinate 1-D cantilevered steel beam on two supports, modelled by a (internally) statically determinate 2-D strut-and-tie model, with the following problem-solving approach:

1. Consider a truss analogy with a 2-D strut-and-tie model, whereby for the determination of both shear strength and shear deformation an effective width b_e modelling of the web members is applicable.
2. Determine the load distribution within this 2-D strut-and-tie model.
3. Determine the effective width b_e with regard to the shear strength of the separate cantilever.
4. Then, determine the effective width b_e with regard to the corresponding shear deformation of the separate cantilever.
5. Analyse the effective width b_e modelling with regard to both shear strength and shear deformation and conclude whether this modelling is viable.

21.2.2 Truss analogy with an effective width modelling

The cantilevered steel beam as shown in figure 21.6 on page 258 is subjected to a combined bending and shear deformation. For a slender beam the bending deformation is decisive and the shear deformation may be neglected as shown in figure 21.7.

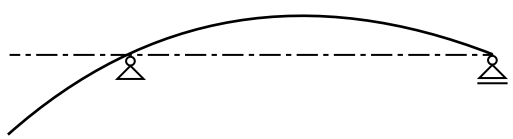


Figure 21.7: Bending deformation cantilevered beam

For a stocky beam the shear deformation is decisive and the bending deformation may be neglected as shown in figure 21.8. With a displacement of the cantilever due to both the shear deformation of the individual cantilever and the contributing shear deformation of the right span as elaborated in figure 21.5 on page 257.

Figure 21.8: Shear deformation cantilevered beam

A beam between slender and stocky dimensions can best be analysed with a truss analogy, where the load distribution completely consists of axial forces in the truss members and therefore not limited by bending formula based on slender beam theory. For the determination of both shear strength and shear deformation an effective width b_e modelling of the web members can be applied as shown in figure 21.9.

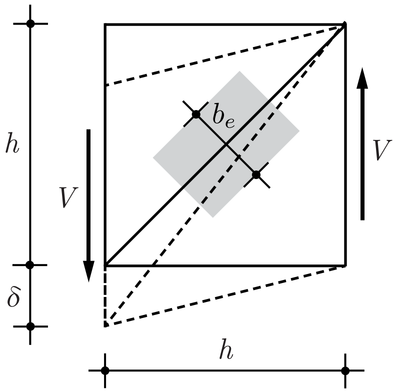


Figure 21.9: Effective width model

21.2.3 Load distribution truss analogy

Within the truss analogy the member forces of the statically determinate 2-D strut-and-tie model can be determined and are shown in figure 21.10.

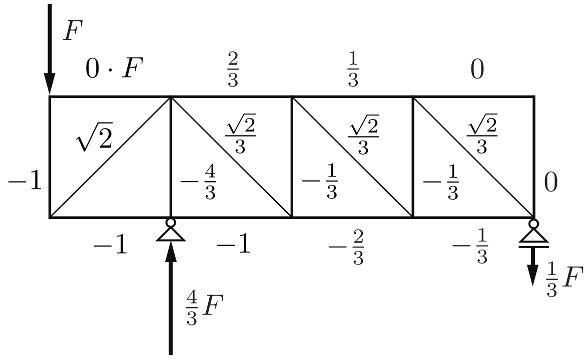


Figure 21.10: Load distribution truss analogy

21.2.4 Shear strength of the steel cantilever

The shear strength of the actual web plate of the beam can be modelled as follows:

$$V_u = A_v \cdot f_v = h \cdot t_w \cdot \frac{f_y}{\sqrt{3}} \tag{21.5}$$

The shear strength within a truss analogy can be modelled with an effective width b_e of the web members as follows:

$$V_u = \frac{b_e \cdot t_w \cdot f_y}{\sqrt{2}} \tag{21.6}$$

Then, with regard to shear strength the effective width b_e can be determined and amounts to:

$$V_u = h \cdot t_w \cdot \frac{f_y}{\sqrt{3}} = \frac{b_e \cdot t_w \cdot f_y}{\sqrt{2}} \Rightarrow b_e = \frac{\sqrt{2}}{\sqrt{3}} \cdot h \approx 0.8h \tag{21.7}$$

21.2.5 Shear deformation of the steel cantilever

The shear deformation of the actual web plate of the beam can be modelled as follows:

$$\delta_v = \frac{V \cdot h}{G \cdot A_v} = \frac{V \cdot h}{G \cdot h \cdot t_w} \tag{21.8}$$

The shear deformation within a truss analogy can be modelled with an effective width b_e of the web members as follows:

$$\delta_v = \frac{F \cdot l}{E \cdot A} \cdot \sqrt{2} = \frac{V \cdot \sqrt{2} \cdot h \cdot \sqrt{2}}{E \cdot b_e \cdot t_w} \cdot \sqrt{2} = 2\sqrt{2}\frac{V \cdot h}{E \cdot b_e \cdot t_w} \tag{21.9}$$

Then, with regard to shear deformation the effective width b_e can be determined and amounts to:

$$\delta_v = \frac{V \cdot h}{G \cdot h \cdot t_w} = 2\sqrt{2}\frac{V \cdot h}{E \cdot b_e \cdot t_w}$$
$$\Rightarrow b_e = 2\sqrt{2} \cdot \frac{G}{E} \cdot h = 2\sqrt{2} \cdot \frac{8.1 \cdot 10^4}{2.1 \cdot 10^5} \cdot h \approx 1.1h \tag{21.10}$$

21.2.6 Analysis effective width modelling

This problem-solving approach focussed on a representative model with a constant value for the effective width b_e. The difference between both effective width b_e values, $0.8h$ respectively $1.1h$, is too large for a viable effective width modelling.

Chapter 22

Epilogue

22.1 Complexity of the built environment

22.1.1 Complexity

Complexity describes the behaviour of a system or model whose components interact in multiple ways and follow local rules, meaning there is no reasonable higher instruction to define the various possible interactions [11].

These are typically large collections of connected elements that influence each other. Examples are the brain; society; traffic; the financial system; interacting institutions; climate; ecosystems; interacting atoms or molecules; the World Wide Web. These diverse examples have surprisingly many features in common. As a rule, they show various properties that make complex systems more than the sum of their parts.

Complexity is generally used to characterise something with:

1. Many components.
2. Where those components interact with each other in multiple ways.
3. Culminating in a higher order of emergence greater than the sum

of its parts.

However, many simple components interacting with each other in multiple simple ways cannot be characterised as a complex system. Therefore the definition of complex systems by Herbert Simon as a “large number of parts that interact in a non simple way. In such systems the whole is more than the sum of the parts” [22], clarifies the higher order characterisation.

22.1.2 Complex systems and processes

Design is an activity that plays a fundamental part in the creation of the built environment. The interdisciplinary design of the built environment consists of cyclic design processes, culminating in a physical system.

In the ISO 9000:2015 a system is defined as a “set of interrelated or interacting elements” and a process is defined as a “set of interrelated or interacting activities that use inputs to deliver an intended result” [10].

Complex system The traditional approach to dealing with complexity is to reduce or constrain it. Typically, this involves decomposition: dividing a large system into separate elements. The actual physical system itself - although consisting of numerous physical elements - is not complex.

The individual system elements of the built environment, however, can be classified as physical or non-physical. For example architectural demands can include non-physical elements, as aesthetics can neither be classified as physical, nor as a process.

Particularly the interacting of these non-physical elements such as aesthetics, load paths, and constructability becomes complex when the corresponding traditional disciplinary boundaries have to be crossed.

Complex process It is important to study how the design is organised in practice, and especially the ways in which designers with different disciplinary expertise are able to work together, collaboratively in teams. A motivation in these studies is not only to improve design processes but also the designed system.

22.1.3 Complexity of interdisciplinary interfaces

For an integral conceptual structural design, the main contributing disciplines and corresponding interfaces have to be considered. Architectural design is par excellence - besides construction engineering - an important and much discussed interface.

In order to get hold on the complexity of interdisciplinary interfaces in general, some fundamental characteristics of the interface between structural and architectural demand are researched; the creation of utilitarian space with materialised forms as a main influential architectural design interface with the structural form.

Fundamental architectural demand The following terms can serve as directional guidance for the research on architectural demand for conceptual structural design:

Architectural concept A guiding concept implies an idea or range of ideas, a design intent or a development approach. It resolves the issues of “what” and “how much” and begins to set the stage for understanding “how”.

Architectural imaging In architecture, imaging often stands for a physical or digital visualisation. On a more conceptual level, it is related to aesthetics, interpretations, and perspectives.

Structural form “The resistant virtues of the structures that we seek depend on their form; it is through their form that they are stable, not because of awkward accumulation of material” as stated by Eladio Dieste [7].

Space in architecture The search for a definition which covers the basic idea of architecture, and corresponds with the enclosure of space with three-dimensional structural forms, results in the following more or less common definitions:

Function, structure, and aesthetics Architecture is a combination of function, structure, and aesthetics; these factors moving together through time creates architecture. A building exists as a crystallisation of a given moment of society, technology, and art.

Utilitarian space Architecture is a conscious creation of utilitarian spaces and construction of materials in such a way that the whole is both technically and aesthetically satisfying.

Utilitarian art Frank Gehry and Santiago Calatrava design in three dimensions. They create art that is tailored to provide shelter. Their designs also serve certain programmatic needs.

An introduction to architecture and space in architecture is given by Frank Ching in his book "Architecture: Form, Space, and Order" [5].

Complexity of the interdisciplinary interface Architectural decisions are those that need to be made from an overall system perspective. Essentially, these decisions identify the key structural elements of the system and their externally visible properties and relationships.

Further, they define how the architecturally significant requirements will be achieved. During the conceptual design phase, the architectural designer should focus on the capacity to bear and resist, in addition to the architectural requirements.

Nowadays, both disciplines are seldom combined in one person, so mutual cooperation is necessary. Although organising the physical meeting in an early stage of the design process is a necessity, it is not enough; a mental meeting for mutual understanding has to be arranged as well.

An architectural design is often concept-based, whereas a structural design is form-based: a materialised structural form with the emphasis on the internal distribution of the loads. In order to connect these manifest differences in design attitude between architects and structural engineers, a recognisable intersection of both disciplines has to be defined.

Furthermore, consumer ideology has turned architecture into fast cycles of fashion and signature styles. Successive architectural movements claiming an avant-garde position have emerged, one after the other, including Minimalism, High Tech, Deconstructionism, and most recently, computer generated Blobitecture.

Consequently, a workable intersection of architectural concepts and structural forms can be characterised as a complex interface.

22.2 Recommendations for research

22.2.1 In-depth research

This textbook discusses a methodical approach on conceptual structural design on a high level of abstraction. However, a deepening research on conceptual structural design is valuable and feasible. Recommendations for research are given with respect to both understanding structural performance and conceptual structural design.

Out of the multitude of missing in-depth knowledge, the following most direct completing topics with respect to the discussed methodical approach are recommended for research:

- Conceptual design parameters built environment.
- Fundamental behaviour of structural materials.
- Transition of stocky to slender beam theory.
- Adjustment factor of buckling strength.

22.2.2 Conceptual design parameters built environment

A definition and collection of the fundamental conceptual design parameters of the influential object-related participating disciplines can serve as a joint breeding ground for a concurrent "shared knowledge-based conceptual design" as discussed in section 7.3.

Because of the complexity of the interdisciplinary interfaces between these object-related disciplines, concurrent engineering will be an appropriate solution as discussed in subsection 5.5.5 of the solution components.

As given in figure 7.6 on page 74 the T-shaped in-breadth understanding in general, and the conceptual structural design process control in particular, is qualitatively modelled in part II of this textbook. Subsequently the conceptual structural design parameters are approximately quantified in part III.

Both the individual sets of conceptual design parameters of the other influential disciplines and the integral process of interacting and control of these sets are in need of approximate quantitative modelling.

22.2.3 Fundamental behaviour of structural materials

Design approximations of load distribution are common property through standard applied mechanics textbooks. Design approximations of strength and stiffness behaviour of common structural materials are widely accessible through numerous textbooks and design codes of practice.

Both design approximations of load distribution, and material strength and stiffness of concrete as well as of structural steel are organised and supplemented into a set of conceptual structural design parameters in this textbook.

Approximated behaviour and strength of new structural materials, however, have to be modelled with care. Especially brittle material behaviour, and corresponding approximate conceptual modelling, is not widely accessible. Brittle material behaviour requires far more in-depth modelling to detect and prevent high-peak stresses with consequent progressive tearing failures.

In general, the relationship between the degree of ductility - quantified by the length of the ductile or plastic zone - and the required corresponding degree of in-depth modelling, has to be researched. In particular, the fundamental behaviour of such structural materials and an effective approximate modelling for conceptual design should become available.

22.2.4 Transition of stocky to slender beam theory

For slender beams, the bending deformation is decisive and the shear deformation may be neglected; for stocky beams, on the other hand, the shear deformation is decisive and the bending deformation may be neglected. The transition from "stocky" to "slender" depends on the bending versus shear stiffness of the material. This transition cannot be determined with the common abstraction of bending based upon slender beam theory.

For a better understanding and application of deformation in general, and shear deformation in particular, research on the transition slenderness with the help of proper constructed in-depth truss-analogy and/or finite element modelling, is advisable.

22.2.5 Adjustment factor of buckling strength

The mathematical formula of Euler gives a proper insight into structural behaviour but has to be adjusted by a factor k for accuracy reasons. Even then, buckling strength-related design approximations are difficult to capture and often have insufficient accuracy with regard to conceptual design.

Therefore, more thoroughly substantiating the material-dependent adjustment factor k, is desirable; first of all, an appropriate adjustment factor k for the buckling strength of concrete members, in combination with a corresponding stiffness modulus E_c; and subsequently, material-dependent adjustment factors for other common structural materials.

Bibliography

[1] Braam, C.R., P. Lagendijk. *Constructieleer Gewapend Beton (CB2)*. Boxtel: Aeneas Media, 2011.

[2] Braam, C.R. *Ontwerpen in Gewapend Beton (CB4)*. Boxtel: Aeneas Media, 2012.

[3] Büttner, O., E. Hampe. *Bauwerk, Tragwerk, Tragstruktur, Band 2: Klassifizierung, Tragqualität, Bauwerkbeispiele*. Berlin: Ernst & Sohn, 1985.

[4] Calaprice, A., F. Dyson, A. Einstein. *The New Quotable Einstein*. Princeton, NJ: Princeton University Press, 2005.

[5] Ching, F.D.K. *Architecture: Form, Space, and Order*. Hoboken, NJ: John Wiley & Sons, 2007.

[6] Couger, D. *Creative Problem Solving and Opportunity Finding*. Danvers, MA: Boyd & Fraser Publishing Company, 1995.

[7] Dieste, E. *La estructura cerámica*. Bogotá: Escala, 1987.

[8] *fib Bulletin 65: Model Code 2010, Volume 1*. International Federation for Structural Concrete fib "fédération internationale du béton", 2012.

[9] Horikx, M.P. *A Methodical Approach on Conceptual Structural Design*. Delft: Delft University of Technology, 2017.

[10] *ISO 9000:2015 Quality management systems - Fundamentals and vocabulary*. International Organization for Standardization, 2015.

[11] Johnson, S.B. *Emergence: The Connected Lives of Ants, Brains, Cities, and Software*. New York: Scribner, 2001.

[12] Kamerling, M.W., J.W. Kamerling. *Jellema deel 9 - Bouwmethoden Utiliteitsbouw*. Amersfoort: ThiemeMeulenhoff, 2011.

[13] Kickert, W.J.M. *Organisation of decision-making: a systems-theoretical approach*. Amsterdam: North Holland Publishing Company, 1979.

[14] *NEN-EN 1990+A1:2006+A1:2006/C2:2019 Eurocode: Basis of structural design*. Nederlands Normalisatie-instituut, 2019.

[15] *NEN-EN 1992-1-1+C2:2011 nl: Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings*. Nederlands Normalisatie-instituut, 2011.

[16] *NEN-EN 1992-1-1+C2:2011/NB:2016+A1:2020 nl: National Annex to NEN-EN 1992-1-1+C2 Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings*. Nederlands Normalisatie-instituut, 2020.

[17] *NEN-EN 1993-1-1:2006 en: Eurocode 3: Design of steel structures - Part 1-1: General rules and rules for buildings*. Nederlands Normalisatie-instituut, 2006.

[18] *NEN-EN 1993-1-1+C2+A1/NB:2016 nl: Eurocode 3: Design of steel structures - Part 1-1: General rules and rules for buildings*. Nederlands Normalisatie-instituut, 2016.

[19] *NEN-ISO 6707-1: 2017 en: Buildings and civil engineering works - Vocabulary - Part 1: General terms*. Nederlands Normalisatie-instituut, 2017.

[20] Nervi, P.L. *Structures*. Transl. G. Salvadori, M. Salvadori. New York: F.W. Dodge Corp., 1956.

[21] Ridder, H.A.J. de. *Design & Construct of Complex Civil Engineering Systems: A new approach to organization and contracts.* Delft: Delft University Press, 1994.

[22] Simon, H.A. *The Sciences of the Artificial - 3rd Edition.* Cambridge, MA: The MIT Press, 1996.

[23] Spekkink, D. *Castle or House of Cards?: Strengthening the structural safety chain.* The Hague: Ministry of Housing, Spatial Planning and the Environment, 2009.

[24] Spekkink, D. *Compendium Aanpak Constructieve Veiligheid.* Gouda: Betonvereniging, 2011.

[25] Terwel, K.C. *Structural Safety: Study into critical factors in the design and construction process.* Rotterdam, 2014.

[26] Timoshenko, S.P., J.M. Gere. *Theory of Elastic Stability.* New York: Dover Publications Inc., 2009.

[27] Torroja, E. *Philosophy of structures.* Transl. J.J. Polivka, M. Polivka. Los Angeles: University of California Press, 1958.

[28] Vinci, L. Da. *The Notebooks of Leonardo Da Vinci, Volumes I and II.* Transl. J.P. Richter. New York: Dover Publications Inc., 1970 (1883).

[29] Vitruvius. *Vitruvius Handboek bouwkunde.* Transl. T. Peters. Amsterdam: Athenaeum, 2003.

[30] World Bank. *World Development Report 1994: Infrastructure for Development.* New York: Oxford University Press, 1994.

Curriculum Vitae

Michiel Paul Horikx was born in 1956, in The Hague, the Netherlands. He attended the Lyceum Augustinianum in Eindhoven and completed his secondary education in 1976. Subsequently, he first studied Architectural, and later Structural Engineering at the Eindhoven University of Technology. He completed his master's thesis in 1983.

After completing his military service he worked with the Hollandsche Beton Groep, at that time the largest civil engineering contractor in the Netherlands. As a structural designer, he was involved in large scale projects, including offshore and bridge design. From 1988 up to 1992 he held the position of conceptual designer and engineering manager of the steel structures - retaining wall, trusses, and ball joint - of the Maeslant Storm Surge Barrier.

Since 1992 he has worked as a senior lecturer and manager at the Amsterdam University of Applied Sciences and has been responsible for the design, implementation and management of the following successful higher education programmes: Bachelor in Civil Engineering; Bachelor in Structural Engineering; and Master in Structural Engineering.

In 2019 he has been appointed as professor of the national Lectorate Structural Safety, commissioned by the national concrete association BV, the national steel association BmS, and the Amsterdam University of Applied Sciences, including their corresponding professional master's programmes in structural design.