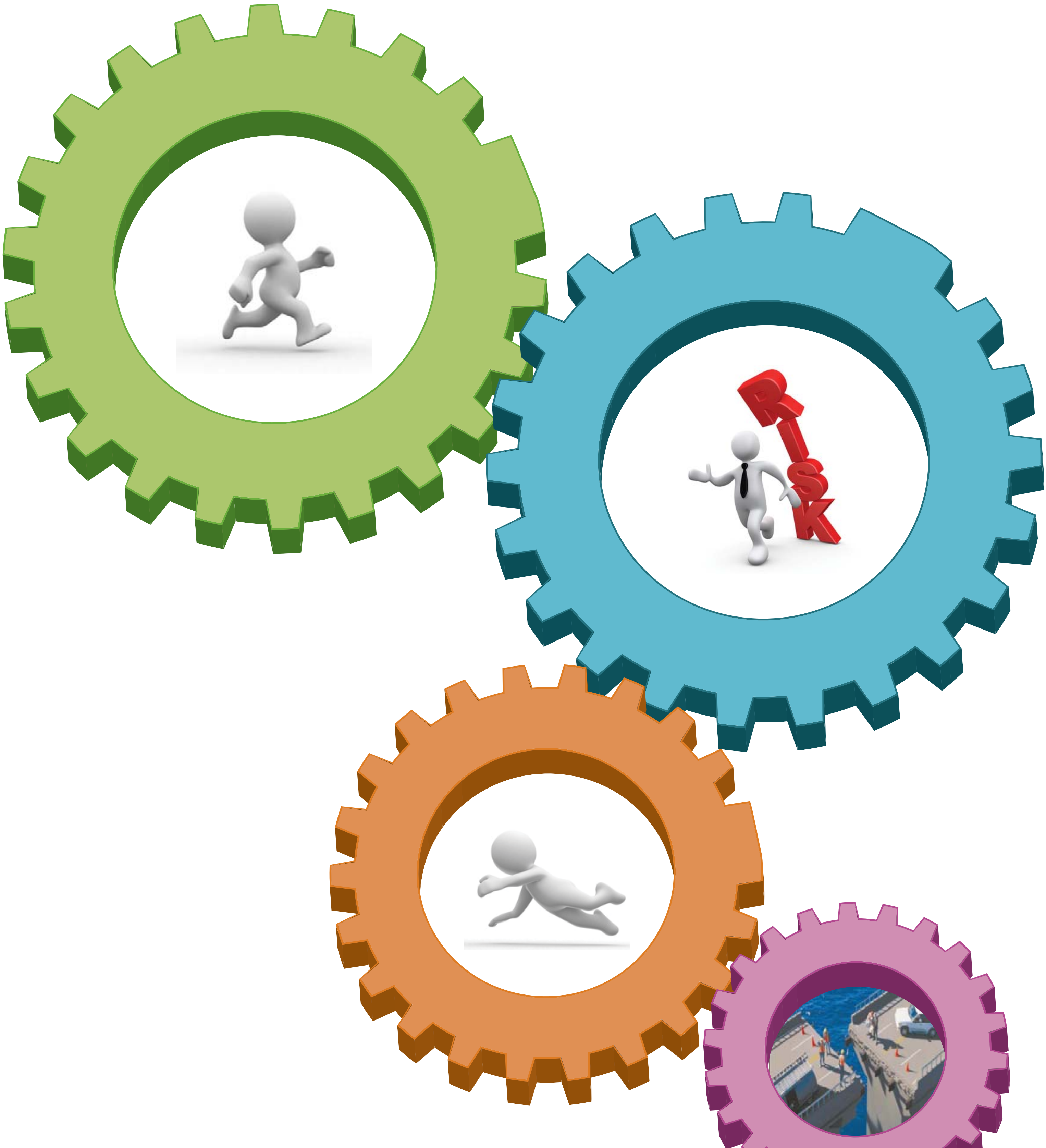

HUMAN ERROR IN STRUCTURAL ENGINEERING

The design of a Human Reliability Assessment method for Structural Engineering

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ABSTRACT

In the recent past a number of buildings collapsed in the Netherlands under apparent normal circumstances. The causes of these failures are predominantly human error within the design or construction of the building. Examples of this are the collapse of five balconies of an apartment building in Maastricht in 2003, and the partial collapse of a roof structure under construction of a football stadium in Enschede in 2012.

Based on these developments it is of importance to investigate the current building practice concerning the occurrence of human error. The objective of this research is to investigate the effect of human error within the design process on the reliability of building structures. Based on this, the following research question is defined:

> *What are the consequences of human error within the design process on the structural reliability of a typical building structure?*

The research question is answered by proposing a Human Reliability Assessment method and subsequently analyse the effect of selected human actions within the design process. For this, two consecutive activities are performed within the research. Firstly a literature study is performed to examine the current knowledge concerning human error in structural engineering. Secondly, based on the literature findings, a model for Human Reliability Assessment in structural engineering processes is proposed. This model is subsequently used to investigate the effect of human error within a specified design process.

LITERATURE STUDY

The literature study focusses on four aspects: the occurrence of structural failure, the basic aspects of human error, the basics of Human Reliability Assessments and probabilistic quantification methods.

Concerning the occurrence of structural failure, it can be concluded that the majority of the failures are caused by human error (Fruhwald et al., 2007). In most researches a value of eighty to ninety percent is mentioned (Ellingwood, 1987; Stewart, 1993; Vrouwenvelder, 2011). Based on the researches of Fruhwald et al. (2007), Boot (2010) and ABC-meldpunt (2011) it can be concluded that the occurrence or errors are of the same order of magnitude for design and construction, with slightly higher frequencies for the design phase.

An important aspect of failure is that in general multiple causes can be identified (CUR, 2010), and that taking away one of these causes usually mitigates the undesired situation. A useful model to represent error causation is the "Swiss cheese" model (Reason, 2000; Reason et al., 2001). The

model exists of several defensive layers between an hazard and an undesired situation. In an ideal world these layers would be intact. However in the real world holes are occurring, making an undesired situation possible. Another relevant aspect of failure is the cognitive level on which an error is made. A subdivision of this is given by Reason (1990): a skill-based level, rule-based level and knowledge-based level. This subdivision is roughly based on the complexity of the task at hand and the level of attention.

One method to investigate human error within design is by means of an Human Reliability Assessment (HRA). These techniques mostly contain three basic techniques (Kirwan, 1994): identify which errors can occur, deciding how likely the errors are to occur and enhancing human reliability by reducing this error likelihood. Most of the HRA techniques are aimed towards subdividing a process in a task sequence, and subsequently analyse these task sequences on human error. An example is the 'Cognitive Reliability and Error Analysis Method' (CREAM), which is used within the main research.

The last aspect discussed in the literature study is the use of probability analysis techniques for quantifying human error probabilities. A frequently used technique is reliability analysis methods which focus on relative effect of failures on the global reliability index of the structure. Another technique is scenario analysis, in which scenarios for errors are investigated to quantify relative consequences associated with these errors. A useful computation method for these kinds of analysis is Monte Carlo analysis, which uses repeated random sampling to calculate results for the analysis.

MAIN RESEARCH

In order to investigate the effect of human error in design tasks, a HRA method for specific use within engineering tasks is proposed. A simplified flow chart of this methodology is presented in figure 1. The model encompasses basically four elements: A qualitative analysis, a human error quantification stage, a design simulation stage and a probabilistic analysis.

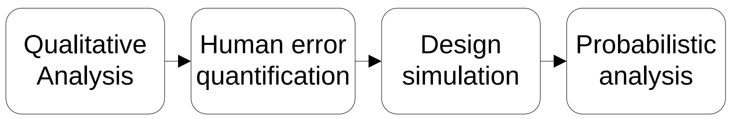

Figure 1: Basic steps within the HRA model

The first step in the HRA model is to define the process of interest and its boundaries (qualitative analysis). Furthermore, a selection of the most error prone processes within the overall process is required in order to focus the HRA efforts. The selected process is a structural design process of a beam element within a common office building. The office building is envisioned as a framework of concrete beams and columns supporting a slab floor. The overall stability is arranged by means of a concrete core. Within the analysis two beam types are considered: a statical determined beam element and a statical undetermined beam element. Furthermore two scenarios for specific analysis are selected: the level of professional knowledge and the level of design control.

The second step within the HRA method is to quantify the probability of failure within an individual design task. This probability of failure is represented by a probability distribution function expressed by two parameters: a Human Error Probability (HEP) and an Error Magnitude (EM). The EM is a parameter which describes the severity of an error. The procedure for determining HEPs consists of two methods: a basic HEP method and an extended HEP method. The extended method is labour intensive and requires quite some knowledge concerning human factors. The simplified method requires considerate less efforts and knowledge, however this method is only applicable for standard design tasks. The simplified method distinct seven basic design tasks, each subdivided in three cognitive levels: a rule-, a skill- and a knowledge based task level.

The third step is to combine the task probability distributions to obtain an overall probability distribution of the element strength due to errors in the process. For this, a Monte Carlo simulation procedure is proposed. Within this simulation process, each design task is modelled with an algorithm which models the design task at hand and the occurrence of failure. Furthermore design control is modelled as well in order to investigate the proposed scenarios. For this a subdivision is made between self-checking (by the designer) and normal supervision. Based on the analysis performed in the case study it can be concluded that the proposed simulation method is useful for combining task probability distributions into an overall probability distribution. However improvements are required for practical use of the model.

The last step in the model is to determine the probability of failure of the engineered structure. For this a probabilistic analysis method based on plastic limit state analysis is proposed. The overall probability distributions found in step three combined with probabilistic loading conditions are used to determine the structural failure probability. Based on the analysis is can be concluded that the structural failure probability can increase considerable.

Finally it can be concluded that the proposed HRA model has the potential to quantify the effect of human error within carefully defined boundary conditions. However further research is required to increase the accuracy of the model and its practical use. From the case study it can be concluded that the statical determined beam element is slightly more susceptible to structural failure. Within both structural types, the influence of design experience on the structural failure is limited. Furthermore, the effect of normal supervision on the failure probability in comparison to a process with only self-checking is about a factor 2,4. A design process without supervision and self-checking results in an unrealistic failure probability. However the occurrence of this seems not logical as self-checking is always present, mostly in a subconscious manner.

Error is a hardy plant;
it flourishes in every soil.

— Martin F. Tupper

ACKNOWLEDGMENTS

This report is the final result of a research on human error in structural engineering. The research is intended as the final step of two independent master studies: Construction Management and Engineering at the University of Twente and Structural Engineering at the Technical University of Delft. For this two separate reports are composed, both discussing common aspects and university specific aspects. This report is the final report for the Technical University Delft, focussing on technical aspects of human error quantification.

Almost 12 months of work have come to an end with the finalization of this report. At the end of this journey, and looking back to the process, it can be concluded that it was an interesting and educational process. Especially conducting research without intensive guidance on an iterative and discovering basis was very interesting.

I would like to thank the members of my graduation committees: Prof. Halman, Prof. Vrouwenvelder, Mr. Al-Jibouri, Mr. Terwel, Mr. Hoogenboom and Mrs. Rolvink. I would like to thank Prof. Halman and Mr. Al-Jibouri for there guidance from a process perspective, which enabled me to consider the problem outside its technical boundaries. Furthermore my gratitude goes to Prof. Vrouwenvelder by helping me to focus on the things which mattered. This has undoubtedly saved me a considerate amount of time. Finally I would like to thank Mr. Terwel, Mr. Hoogenboom and Mrs. Rolvink for there guidance throughout the process. This guidance did provide me with new ideas when I needed it, and has inspired me throughout the process.

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1 INTRODUCTION

1.1 INTRODUCTION SUBJECT

Designing and building an engineered structure in the Netherlands is bound to strict building regulations. Building codes, codes of practise, education, risk control measurements, etc., are all aimed towards minimizing the risks of structural failure. Despite these efforts, structural collapses within the Netherlands have illustrated the inadequacy of the current building practise. This will be demonstrated with two recent examples.

Balconies Maastricht
On the 23th of april 2003 five balconies of an apartment building collapsed due to sudden column loss, resulting in two deadly casualties. The triggering cause of the accident was insufficient strength in a concrete ridge, which was meant to transfer the column forces to the building foundation. The underlying cause was a design error of the structural engineer. Another important contributing cause of the collapse was the design of the balcony which lacked robustness [1] as no 'second carriage way' existed. (CUR, 2010).

Football stadium Enschede
On the 7th of July 2011 during construction activities for expansion of the football stadium in Enschede, the stadium roof partly collapsed. The accident resulted in two deathly casualties and nine wounded. The accident was (among others) a consequence of the lack of sufficient stability element in the truss system (for the loading conditions at that moment). The accident was mainly caused by a series of malfunctions in the building process concerning the safeguard of structural safety (OVV, 2012).

Both examples show the cause and consequence of errors in design and construction of building structures. An interesting aspect is the presence of human error within both examples, which is far from a coincidence. Researchers such as Ellingwood (1987), Kaminetzky (1991), Stewart (1993), Fruhwald, Serrano, Toratti, Emilsson & Thelandersson (2007) and Vrouwenvelder (2011) have all concluded that most of the structural failures are caused by human errors.

The problem with human errors within design is that they are not readily quantifiable. Numerous researchers have investigated this problem, by means of for instance Human Reliability Assessments (HRA). However quantifying the probability of human error inevitable leads to unreliable and subjective results (Swain, 1990; Kirwan, 1996; Hollnagel, 1998; Reason, 2000; Grozdanovic & Stojiljkovic, 2006). Despite these set-backs, further research in the effect of human error seems necessary due to the alarming

1 Defined as the ability of a structure to withstand events like the consequences of human error, without being damaged to an extent disproportionate to the original cause

failure numbers.

1.2 PROBLEM DEFINITION

Human Error has proven to be a problematic issue within the building industry, as is shown in the introduction. Especially quantitative error prediction and error causation are issues of concern. To summarize the problem analysis, the practical problem statement and the scientific problem statement are formulated as follows:

Practical problem statement	Recent collapses of building structures in the Netherlands have shown the lack of control of the occurrence of human error within the design- and construction- process
Scientific problem statement	In the existing literature the knowledge concerning human error prediction within engineering types of tasks is incomplete.

1.3 RESEARCH OBJECTIVE

The practical problem definition pinpoints an important aspects of human error within design from an engineering point of perspective: "the lack of control". This lack of control is something which is worrying every engineer, as most designed systems are based on extensively investigated assumptions leaving no space for unanticipated deviations. Furthermore building engineers need human error approaches that are simple and efficient to use, and which produce results that are practically valuable. From this perspective, this thesis focusses on the practical aspects of human error by considering human error from a human reliability perspective. By doing so, it also provides insights for theoretical aspect related to human reliability. Based on this assumption the objective for the research is defined as follows:

> The objective of this research is to investigate the effect of human error within the design process on the reliability of building structures. (*objective of the research*)
>
> Doing so by
>
> proposing a Human Reliability Assessment method and subsequently analyse the effect of selected human actions within the design process on structural reliability. (*objective in the research*)

1.4 DEMARCATION OF THE PROBLEM

The problem definition already focussed the research on human error within building structures. Furthermore three restrictions concerning the boundaries of the research are pointed out beneath in order to focus the research further.

Firstly, the research proposes a method for human error diagnosis rather then human error management. It is acknowledged that in order to control the occurrence of error, human error management is required. This, and the use of the diagnosis method within human error management is left for further research.

Secondly, the research focusses on the design process within building processes, which entails that the construction process is not considered. It is acknowledged that human errors within the construction process are important contributors to structural failure. However, limitation of the scope of the research is required to acquire sufficient depth within the research to attain a relevant result.

Finally, the research is meant as an explorative research on the possibilities to quantify human error within structural engineering processes. Due to this, the probabilities of human error are determined within a large margin.

1.5 RESEARCH QUESTIONS

After defining the problem, clarifying the objective and stating the problem demarcation, the research question is stated as follows:

What are the consequences of human error within the design process on the structural reliability of a typical building structure?

Three sub-questions are defined to answer the research question. For every sub-question, the research methodology is shortly discussed.

- *What is the current scientific knowledge concerning the assessment of human error in structural engineering?* in order to answer this question, a literature study is performed on relevant subjects.

- *What is the configuration of a Human Reliability Assessment method specifically aimed towards quantifying the probability and consequences of human error in typical design processes within structural engineering?* In order to answer this question, a model for a Human Reliability Assessment (HRA) in structural engineering is proposed.

- *What is the effect and consequence of human error within a design process of a typical building structure on the structural reliability of the structure?* In order to answer this question, the proposed HRA method is used to analyse a design case.

1.6 STRUCTURE OF THE REPORT

This thesis is primarily a research report consisting of two consecutive parts: a literature study and a main research. An overview of the subjects discussed in the two parts is shown in figure 2.

Literature study
The literature study starts with a chapter which introduces the aspect of structural failure and elaborates on the causes of failure (chapter 2). Chapter 3 elaborates on basic aspects of human error by discussing characteristics of human error and the cognition of human error. Chapter 4 discusses Human Reliability Assessment (HRA) techniques which are used to assess human effects within systems. Chapter 5 finally considers several aspects of probability analysis which are used in quantitative HRA analysis. The literature study is concluded in chapter 6 by considering the relevant literature findings and there use within the main research.

Main research
The main research elaborates on human error quantification in structural design. For this a Human Reliability Assessment tool for use in design tasks is proposed and subsequently used to analyse a design case. The model is set-apart in chapter 7. Chapter 8 discusses the HRA context and goal selection and introduces the case study. Chapter 9 discusses the Human Error Probability (HEP) and Error Magnitude (EM) quantification required to find a failure probability distribution for each design task. Chapter 10 elaborates on the simulation process to convert these probability distributions on a task level to an overall design probability distribution. Chapter 11 finally elaborates on a probabilistic analysis method to find a structural failure probability based on the design failure probability. The research is concluded in chapter 12. Furthermore recommendations for further research are stated in chapter 12 as well.

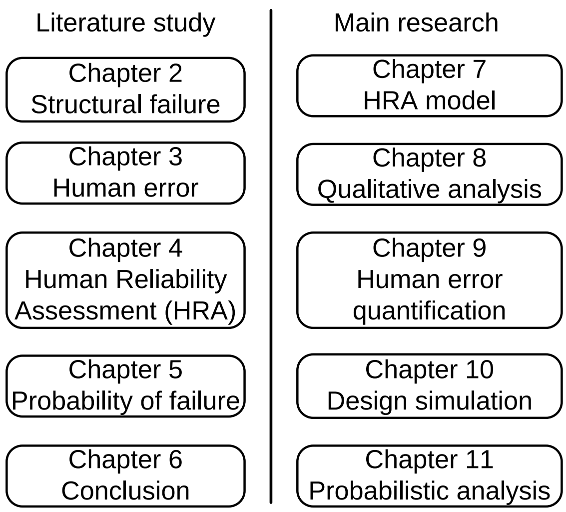


Figure 2: Thesis structure

Part I

LITERATURE STUDY

The literature study discusses aspects of human error within structural engineering. The objective of this study is to assess the current knowledge within scientific literature concerning the assessment of human error in structural engineering. Four topics are considered: the causes of structural failure, the technical and psychological aspects of human error, Human Reliability Assessment (HRA) in engineering and finally probabilistic modelling methods for human error/failure.

2 STRUCTURAL FAILURE

INTRODUCTION

Failure of structures or parts of structures are occurring throughout the world. Within the Netherlands their numbers are limited due to strict regulations and sufficient building knowledge. However a world without failure seems impossible, slips and lapses and gross-errors will always occur.

In line with van Herwijnen (2009) failure of a structure is defined as the unsuitability of the structure to serve the purpose where it was built for. The collapse of (parts of) a structure is the heaviest form of failure (van Herwijnen, 2009). The author classifies four basic ways of failure:

- the collapse of (parts of) a building;
- unequal settlements;
- strong deformations of construction parts;
- noticeable and disturbing vibration of walkable surfaces.

This chapter will examine the literature on failure of structures. It starts with outlining the findings on structural failure worldwide, followed with some information on failure statistics in the Netherlands specifically. This chapter concludes with a short review on the cost of structural failure.

2.1 STRUCTURAL FAILURES WORLDWIDE

A number of surveys on structural failures have been reported during the years. The purpose of these studies is to quantify sources of failure and to indicate their relative importance in the building process. A general conclusion from such studies is that failure without exception occur due to human error (see Fruhwald et al., 2007).

Fruhwald et al. (2007) cites several other researches concerning the causes of failure. Fruhwald refers Walker (1981) on this topic: "inappropriate appreciation of loading conditions and of real behaviour of the structure was found to be the prime cause in almost one third of the failure cases investigated." From a research of Matousek & Schneider (1976), an investigation of 800 cases of failure from different sources, Fruhwald concludes: "[...] a majority of mistakes is related to conceptual errors and structural analysis. Incorrect assumptions or insufficient consideration of loads and actions was found to be a common type of error." The causes of failure and the phase in which the failure is made are discussed in further detail in the following sections.

The research of Fruhwald et al. (2007) is specifically aimed at timber structures, containing 127 failure cases. The most common cause of failure found in the investigated cases is poor design or lack of strength design (41%), in total half of the failures were due to design errors. About 27% was caused during construction. Wood quality, production -methods and -principles only caused 11% of the failures. The outcomes of this research on the causes of failure are presented in table 1, together with similar information on steel and concrete structures received from literature. From this it can be concluded that design errors are also a common cause of failure within steel- and concrete- structures.

Table 1: Failure causes (in % of cases) for different building materials (Fruhwald et al., 2007, page 26)

Failure cause	Timber %	Steel %	Concrete %
Design	53	35	40
Building process	27	25	40
Maintenance and re-use		35	
Material	11		
Other	9	5	20

Ellingwood & Dusenberry (2005) compiled results from a series of investigations during the years 1979-1985, to identify where in the building process errors occur. This list is expanded in the research of Fruhwald et al. (2007). This list is given in table 2 to provide an indication of where in the design and construction process failures occur.

Based on table 2, Fruhwald et al. (2007) concludes: "the occurrence of errors are of the same order of magnitude for design/planning and construction respectively, with slightly higher frequency for the design phase. Failures due to material deficiencies or maintenance are relatively uncommon."

It should be noted that the classification of failures is not consistent between different investigators. Also, the results are incomplete and biased. For example only failures resulting in severe damage may be reported and much of the available data are reported voluntary and are not a random sample (Ellingwood & Dusenberry, 2005). Also information about errors and mistakes are difficult to get, since the involved parties often have a strong interest to conceal facts (Fruhwald et al., 2007). However this failure data provides in general an idea about technical and organizational defects in the design and construction process.

Table 2: Percentage of failures by the phase in which they were made (Fruhwald et al., 2007, page 6)

Reference	Planning /design %	Construc- tion %	Use /main- tenance %	Other %	Total %
Matousek	37	35	5	23	98[d]
Brand and Glatz	40	40	-	20	100
Yamamoto and Ang	36	43	21	-	100
Grunau	40	29	31[a]	-	100
Reygaertz	49	22	29[b]	-	100
Melchers et al.	55	24	21	-	100
Fraczek	55	53	-	-	108[c]
Allen	55	49	-	-	104[c]
Hadipriono	19	27	33	20	99

[a] Includes cases where failure cannot be associated with only one factor and may be due to several of them.
[b] Building materials, environmental influences, service conditions.
[c] Multiple errors for single failure case.
[d] Error in report Fruhwald, should be 100 %

2.2 Structural failures in the Netherlands

Two recent (and ongoing) studies in the Netherlands have shed some light on the occurrence of failure in the Dutch building Industry.

The first study is the graduation thesis conducted by W.F. Boot in 2010 (Boot, 2010). This study presents 151 cases of structural damage of various kinds and identifies their causes. The source of the data is Dutch case law (decisions of the 'Raad van Arbitrage voor de Bouw', the 'Stichting Arbitrage-Instituut Bouwkunst' and the 'Commissie van Geschillen' of the KIVI NIRIA).

Boot (2010) analyses the type of failure as an attempt to pinpoint the causes of failure. The author concludes that most failures are related to design errors, execution errors or a combination of both. 34% of the structural failures is caused by design errors. These failures include calculation errors, failure to consider relevant loads and drawing errors. 32% of the structural failures is caused by construction errors. These failures include unwanted removal of temporary supports, non-conformance to design intent and inadequate assembly by construction workers. 20% of the structural failures is caused by a combination of design and construction errors. The remaining 11% are due to material deficiencies (6%), improper use (3%), circumstances beyond ones control (1%) and errors in manufacturing (1%).

Boot (2010) also discussed the phase in which the failures were made. 26 % of the failures were made in the design phase, 23 % in the construction phase, 18 % in both the design and construction phase and 17 % of the failures were made during renovation or expansion.

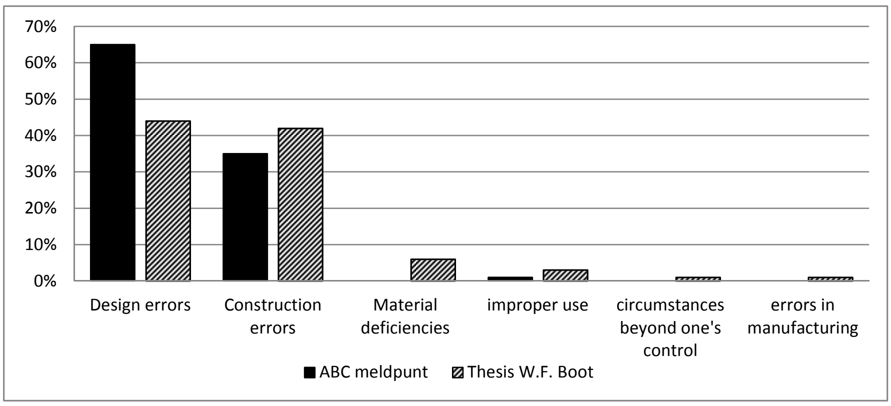
There is a critical note on the relevance of the research of Boot. Only cases which are presented in Dutch case law are investigated. The question remains how representative these cases are for the failures of the entire construction industry.

The second study is based on the findings of the 'ABC-meldpunt', set-up by TNO in commission of the 'Platform Constructieve Veiligheid'. The 'ABC meldpunt' registers structural failures which did lead, or could have led, to structural damage. Typical structures included in the database are houses, office buildings, bridges and tunnels. The registrations are based on voluntary and confidential reporting of the construction industry by means of an on-line enquiry (www.abcmeldpunt.nl). (ABC-meldpunt, 2009).

From the period 2009 till 2011, 189 reports are received. An analysis of these reportings is presented in ABC-meldpunt (2011). In line with the findings of Boot, design and construction errors are the dominant types of causes. 65% of the failures are design errors and 35% are production errors. Improper use of the construction has occurred in one case, the usage of new materials and circumstances beyond ones control did not occur in a single case.

The two main causes for design errors are insufficient knowledge/ qualification for the project (25%) and incorrect schematic representation of the force balance, or not including the force balance (21%). Production errors are mainly a consequence of incorrect composition of materials/ incorrect construction phasing (34%) or improper measurements (19%).

The phase in which the failures were made is also presented. 61% of the failures were made during design and detailed engineering, 31% were made during construction and 7 % during renovation/expansion.



* Adaptation ABC meldpunt: design- and construction errors are evenly divided between design errors and construction errors.

Figure 3: Comparison ABC meldpunt and Thesis Boot concerning the causes of failure

A comparison between both researches on the topic 'causes of failure' is presented in figure 3. From this figure it can be seen that the design and construction failures are the main causes for the occurring of errors, varying from 99% (ABC meldpunt) to 86% (thesis Boot). However, the subdivision between the design- and construction- phase differs considerable between both researches. Within the ABC research 65% of the errors are caused by design errors and only 35% due to construction errors. In the thesis of Boot the distribution is almost equal. Finally the percentage of failures originating from other failures the design- and construction- process differs considerable between both researches (11% in thesis Boot against 1% in ABC- meldpunt).

The findings on failure causes within the Netherlands differ considerable with the findings worldwide. The table of Fruhwald et al. (2007), as presented in table 1, states that 20% of the causes of failure is originated outside the design- and construction process. Boot (2010) concludes that 11% of the failure has a cause outside the design- and construction- process and the ABC meldpunt reports only 1% on this aspect.

The differences between the separate investigations could be a consequence of the small number of respondents. Within the thesis of Boot and the 'ABC meldpunt' the number of respondents was 151 and 189 respectively, and the number of respondents in Fruhwald et al. (2007) is 127. Another possibility could be the limited variety in the background of the respondents. For example within the thesis of Boot and the 'ABC meldpunt', only the construction industry is involved and not the end users or other relevant parties. And within the construction industry only observed noticeable cases are reported. Despite the differences between the discussed research, there results are very well useful as they provide basic insights in the aspects of structural failure.

2.3 COSTS OF FAILURE

Within the Netherlands some researchers have attempted to quantify the failure costs of the construction industry. USP marketing consultancy BV has conducted several researches based on opinions of directors of industry in the Netherlands (Busker, Busker, 2010). The survey of 2008 shows a total failure cost of 11,4 % as percentage of the turnover. This was 7,7 % and 10,3 % in 2001 and 2005 respectively (general failure costs), with an average of 10%. USP concludes from this two possibilities: the failure costs in the Netherlands are rising or the awareness among directors of industry has increased.

An in-depth research on the failure costs of two projects as part of a broader research of 15 projects has been performed by Mans et al. (2009). The term failure costs in this research is limited to the following definition: costs of recovery of the (structural) failures, before completion of the project. So only failures which were discovered during construction were included, leaving out construction failures which were discovered in later stages. The two projects show failure costs of 6 and 8 % in comparison with the structural construction costs. It is concluded by Mans et al. (2009) that the failure costs of the 15 projects vary from 0 to 8 % with an roughly estimated average of 4 % (structural failure costs). This costs could be prevented with only a minor extra investment of 0,8% of the total construction costs.

From these two researches it can be concluded that the general failure costs are somewhat around 10 % and the structural failure costs are approximately 4 %. It should be noted that these numbers are a rough estimate with considerable uncertainty.

3 HUMAN ERROR

INTRODUCTION

Chapter 2 provided some general information on failure statistics within the building industry. An interesting aspect noted in chapter 2 is the occurrence of human error and its effect on structural failure (human error is seen as the basic cause of failure). Especially within the modern technological systems, the consequences of human error can be devastating. Accidents within the nuclear industry such as Three Mile Island [1] and Chernobyl [2] have shown this devastating potential. This chapter discusses more in detail the background of structural failures and human errors. Three aspects are considered. Firstly section 3.1 elaborates several subdivisions of human failure. Section 3.2 several human error models. Finally, section 3.3 discusses the nature of human error by focussing on its cognitive aspects.

3.1 ASPECTS OF HUMAN ERROR

As mentioned in chapter 2, most failures are caused by human error. Results from numerous investigations of structural failures have led Kaminetzky (1991) to conclude that all failures are human errors and that they can be divided into three categories:

1. Errors of knowledge (ignorance)
2. Errors of performance (carelessness and negligence)
3. Errors of intent (greed)

Other researchers, such as Ellingwood (1987),Stewart (1993) and Vrouwenvelder (2011), do recognise the human error as the main cause of structural failures as well. But unlike Kaminetzky, they do not underline that all failures are human failures, restricting it to a maximum of 90%.

Another division of human errors, more based on operational task analysis is shown beneath (Swain as cited in Melchers, 1984). This division is frequently used within Human Reliability Analysis related to plant operations:

1 The Three Mile Island accident was a partial nuclear meltdown on March 28, 1979. The accident initiated with a failure in the secondary, non-nuclear section of the plant, followed by a stuck open relief valve in the primary system, which allowed large amounts of nuclear reactor coolant to escape. The mechanical failures were worsened by the failure of plant operators to recognize the situation due to inadequate training and human factors (USNRC, 2009)

2 The Chernobyl disaster was a nuclear accident that occurred on 26 april 1986 at the Chernobyl Nuclear Power Plant in Ukraine. Lack of human factors considerations at the design stage is one of the primary causes of the Chernobyl accident. Furthermore, human error and problems with the man machine interface were attributing to the disaster (Meshkati, 1991)

- Errors of omission (e.g. failure to perform a task)
- Errors of commission (e.g. incorrect performance of a task)
- Extraneous acts.
- Sequential errors
- Time-limit errors (e.g. failure to perform within allocated time)

Based on his research in structural engineering, Melchers (1984) concludes: "the limited available evidence suggests that the first two categories are probably of most importance for structural-engineering projects, with the last item being of only minor importance."

Besides categorising human errors, categorizing the factors which influence human errors is of interest. These factors originate from aspects within the person, the organization or within the environment. Vrouwenvelder (2011) elaborates on this by presenting six factors which influence the probability of human error:

1. Professional skill.
2. Complexity of the task, completeness or contradiction of information.
3. Physical and mental conditions, including stress and time pressure.
4. Untried new technologies.
5. Adaptation of technology to human beings.
6. Social factors and organisation.

Van Herwijnen (2009) gives 5 important factors which underline the factors given by Vrouwenvelder. Furthermore the author recalls the economic development as a relevant factor. Mans, Rings, van den Brand & Derkink (2009) also underlines that the factor 'completeness or contradiction of information' is an important aspect of structural safety within Dutch building projects: "It is concluded that parties in an individual building project do not always have the same view of structural safety, that in most projects no specific decisions are made about the level of structural safety [...]".

The question remains on which level of the organization errors, and more specifically human errors, occur. Is it on a personal level or on a more broader based organizational level? In order to provide insight in this question, a case study on structural failure held in the Netherlands (CUR, 2010) proposes to classify the causes of errors in three levels:

- Micro level: causes such as failures by mistakes or by insufficient knowledge of the professional.
- Meso level: causes arising from the organization of the project or the management.
- Macro level: causes arising from the rules, the culture within the sector or other external circumstances.

This classification is used within the CUR report to categorise 15 case studies of collapses within constructions. This report was set-up after major collapses occurred in the Netherlands, which started many initiatives by both government as well as building industry to avoid similar events in the future. It concludes that in general multiple causes can be identified for the appearance of a failure. These causes are based in all three levels; micro-, meso- and macro-level. The removal of only one of the causes can be sufficient to mitigate the failure.

3.2 MODELS OF HUMAN ERROR

Humans have always sought for means to find the causes of failure. Within the literature several models and metaphors are available to assist with this search. Within this section some of these models will be discussed.

A basic model which simplifies causal effects to a single chain is Heinrich‘s domino model (see figure 4, Hudson (2010)). Within this model each domino presents a factor in the accident sequence such as the social environment and the unsafe act himself. These factors are arranged in a domino fashion such that the fall of the first domino results in the fall of the entire row. If one of the domino’s is removed, the sequence is unable to progress and the undesired situation will not occur (Storbakken, 2002). Hudson (2010) criticises this model as it is not able to present accident causation in a non-linear fashion and it fails to model the non-deterministic character of error causation (error causation is not deterministic but rather more probabilistically).

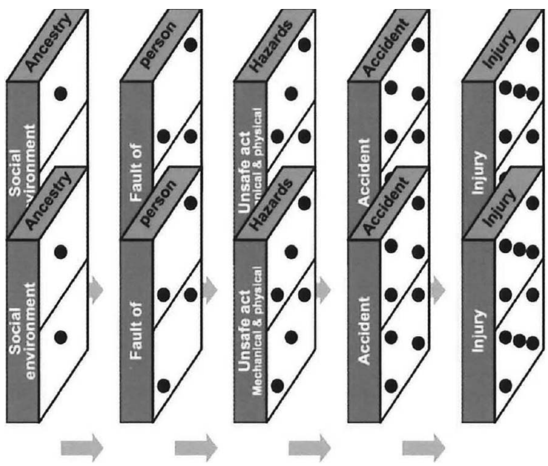


Figure 4: Heinrich‘s domino model. removing one domino prevents subsequent domino’s from falling (Hudson, 2010, page 6)

A more sophisticated model is developed by the psychologist James Reason (Reason, 2000; Reason, Carthey & de Leval, 2001). This model is generally termed the “Swiss cheese“ model. Williams (2009) refers to the same model by calling it the ‘Window of opportunity model of causation‘.

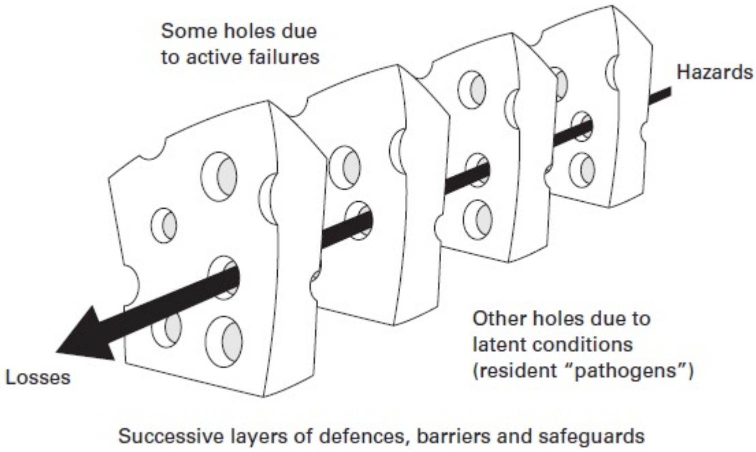


Figure 5: The "Swiss cheese" model of accident causation. (Reason et al., 2001, page ii21)

Reason (2000) presents general knowledge about causes of major accidents, and presents tools and techniques for managing the risk of organizational accidents. Reason distincts two kinds of accidents: those that happen to individuals and those that happen to organizations. The "Swiss cheese" model is of particular interest for the organizational accidents. The model exist of several defensive layers as presented in figure 5, representing the defensive functions within a company. These layers can be 'Hard' defences (e.g. personal protection equipment, alarms, etc.) or 'Soft' defences (e.g. legislation, training, etc.).

In an ideal world all the defensive layers would be intact, allowing no penetration by possible accidents. In the real world, however, each layer has weaknesses and gaps as represented in figure 5. These 'holes' are not fixed and static, they are constantly moving due to the changing conditions. Particular defences can be removed deliberately, or as the result of errors and violations.

These holes are created in two ways: active failures and latent conditions. Nearly all adverse events involve a combination of these two sets of factors. Active failure are the unsafe acts committed by people who are in direct contact with the project. They have several different forms: slips, lapses, fumbles, mistakes and procedural violations. Latent conditions arise from decisions made by designers, builders, procedure writers and top level management. All such strategic decisions can lead to unsafe conditions in the system. Latent conditions have two kinds of adverse effects: firstly they can translate into error provoking conditions within the workplace (for instance time pressure, under-staffing and inexperience), secondly they create long standing holes of weakness in the defences (for instance unworkable procedures and design- and construction- deficiencies). Latent conditions can be present within the organization for years, before they combine with active failures and local triggers to create an accident opportunity. Active failures are often hard to foresee, while latent conditions can be identified and repaired before an accident occurs.

Hudson (2010) commented on the "Swiss cheese" model concerning causal mechanisms the following: "the causal mechanisms by which latent failures or conditions create unsafe acts could be quite different from the causal mechanisms operating once the hazard was lined up and the unsafe acts ready to be carrier out." Hudson (2010) noted that the "Swiss cheese" model is deterministic as well but no longer linear. Furthermore the model misses the possibility to address common effects of higher order causes and lower order barriers. Concerning this latter Hudson (2010) has collected evidence within commercial aviation that the nature of the outcome can be predicted by factors well off the line of direct causality.

Reason elaborates further on failure theorem in a paper based on accident investigation in various hazardous domains (Reason et al., 2001). This investigation suggests that a group of organizational properties, called the vulnerable system syndrome (VSS), renders some organizations more vulnerable for failure. The authors state: "VSS has three interacting and self-perpetuating elements: blaming front line individuals, denying the existence of systematic error provoking weaknesses, and the blinkered pursuit of productive and financial indicators."

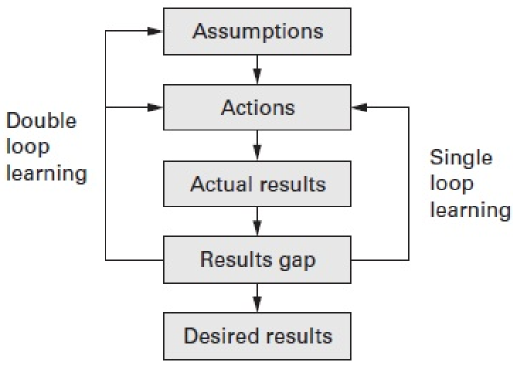


Figure 6: Single loop versus double loop learning (Reason et al., 2001, page ii25)

The investigation further states that these systematic shortcomings are present in all organizations, to a certain extend: "recognising its presence and taking remedial actions is an essential prerequisite of effective risk management." For this two types of organizational learning are recognised: 'single loop' learning that fuels and sustains VSS and 'double loop' learning. The solution lies in 'double loop' learning: "a crucial remedial step is to engage in 'double loop' organizational learning that goes beyond the immediate unsafe actions to question core assumptions about human fallibility and to identify and reform the organizational conditions that provoke it." A schematic visualization of 'single loop' and 'double loop' learning is presented in figure 6.

An aspect closely related to the 'Swiss cheese model' and the Vulnerable System Syndrome, is the so called Performance Shaping Factors (PSF), a term frequently used within the field of Human Reliability Assessment (HRA). More details on HRA is given in chapter 4, PSF will be discussed below.

According to DiMattia, Khan & Amyotte (2005), PSFs are those parameters influencing the ability of a human being to complete a given task. Examples are stress, complexity, training, experience and event factors. From this perspective PSFs are the factors that may cause human error incidents, and PSF analysis is meant to prevent the occurrence of future human errors by means of error management in the organization (Grozdanovic & Stojiljkovic, 2006).

A categorization of PSFs based on the process of human information handling and decision making is given by Grozdanovic & Stojiljkovic (2006). This categorization is presented in figure 7. According to this figure, a decision is influenced by three groups of PSFs: the available information, the knowledge base and the skills/experience. After the decisions are made, another set of PSFs influences the final outcome: the environment, the available equipment, communication and organization.

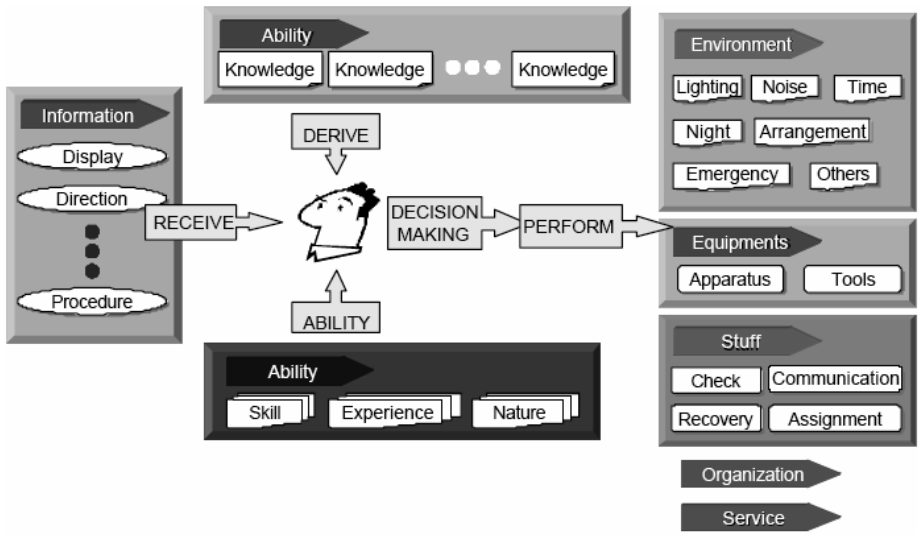


Figure 7: Categories of Performance Shaping Factors (PSF) from Okada (Grozdanovic & Stojiljkovic, 2006, page 135)

Prevention techniques for improving PSFs can be subdivided in two categories, quite similar to the earlier mentioned categories of active/latent condition of Reason. This subdivision acknowledges department based and department exceeding tasks:

- Immediate PSFs: the improvement of immediate PSFs cannot be expected to prevent other trouble occurrence. The prevention strategy against the immediate PSFs tends to depend on the specification of object task, so the strategy cannot be applied for tasks in other departments.

- Latent PSFs: as latent PSFs exist in other departments, the prevention strategy against the latent PSFs is expected to reduce the possibility of human error occurrence.

Finally it can be concluded from above literature findings that errors are an important source of unpredictability within the design- and construction-process. Ellingwood & Kanda (2005) writes that this can be managed in two ways: by maintaining an efficient and effective quality control process and by managing risk through monitoring secondary indicators of potential trouble.

Quality control requires (among others) independent checking procedures and a traceable documentation system. Quality assurance has a significant limitation: it assures the quality of management systems, but not of their content. Secondary indicators of potential trouble are for instance the experience, financial and political climate. The idea is to monitor the indicators and use them as a warning of increased risk within the process. Within the engineering practice, quality assurance is (among others) ensured by using extensive checking of the design by the designer, the supervisor or third parties. Concerning this Stewart (1993) states that the majority of the engineers believe that the best safeguard against structural failure are extensive checking of designs and close supervision of construction.

3.3 Cognition of human error

One of the questions remaining unanswered is what the exact nature of human error is. This is not an easy question, simple design tasks involves experience and insight which are hard to measure. Furthermore, it was found that even a simple design involves quite complex cognitive [3] tasks (Melchers, 1984). Within this section an attempt is made to pinpoint some characteristics of human error based on findings within the field of psychology. A psychological definition of human error is given by Reason (1990), page 9:

> Error will be taken as a generic term to encompass all those occasions in which a planned sequence of mental or physical activities fails to achieve its intended outcome, and when these failures cannot be attributed to the intervention of some change agency.

A more pragmatical definition of human error is given by Swain and Guttmann (sited from Kirwan (1994), page 1), which is adopted in this research:

> Any member of a set of human actions or activities that exceeds some limit of acceptability, i.e. an out of tolerance action [or failure to act] where the limits of performance are defined by the system.

For understanding human error, it is important to understand the factors which give rise to the production of errors. There are three major elements in the production of an error: the nature of the task and its environmental circumstances, the mechanisms governing performance and the nature

3 Cognition is defined as a group of mental processes by which input is transformed, reduced, elaborated, stored, recovered and used (OED, 2012)

of the individual. Also the notion of intention is an important aspect of error, defining human error begins with a consideration of the intentional behaviour (Reason, 1990). Another important factor which is described in Hollnagel (1993) is the distinction between competence and control as separate aspects of performance: "the competence describes what a person is capable of doing, while the control describes how the competence is realised, i.e., a person's level of control over the situation."

Error classification, which is necessary for error prediction, can be done on different levels. Reason (1990) distinguishes three levels at which classifications are made: the behavioural, contextual and conceptual levels. These corresponds approximately to the 'what? ', 'where? ' and 'how? ' questions about human errors.

At the behavioural level of classification, errors may be classified according to some easily observable aspects of the behaviour. These can include either the formal characteristics of the error (omission, commission, etc.), or its more immediate consequences (damage, injury). The contextual level of classification goes beyond the basic error characteristics and includes also assumptions about causality. Such categorizations are valuable as it draws attention to the complex interaction between 'local' triggering factors and the underlying error properties. However even these contextual factors cannot explain why the same or very similar circumstances not always lead to the same type of error. The conceptual level of classification is based on the cognitive mechanisms involved in producing the error. These classification is potentially the most promising because it tries to identify the underlying causal mechanisms (Reason, 1990).

Another distinction made by Reason (1990) is based on the origins of the basic human error types. This distinction is related to the process of the particular action (planning, storage and execution). These error types are conceptually tied to underlying cognitive stages or mechanisms. This distinction (or conceptual framework) consists basically out of two types of mistakes: slips/lapses and mistakes. Slips and lapses are actions which deviate from current intention due to execution failures and/or storage failures. Mistakes are actions which may run according to plan, but where the plan is inadequate to achieve its desired outcome.

In line with the level of cognitive operation in error production, mistakes occur at the level of intention formation, whereas slips and lapses are associated with failures at the more subordinate levels of action selection, execution and intention storage. As a consequence, mistakes are more complex than slips and lapses.

If the division in two (slips and mistakes) is used, differentiating based on cognitive mechanisms is not possible. Both slips and mistakes can take 'strong-but-wrong' forms, where the wrongful behaviour is more in keeping with past practise than the current circumstances demand. On way of resolving these problems is to differentiate two kinds of mistakes: rule based mistakes and knowledge based mistakes.

So finally to summarise the argumentation of Reason on error types, there are three error types distinguished: skill-based slips and lapses, rule-based mistakes and knowledge-based mistakes. These three error types may be differentiated by a variety of processing, representational and task related factors. Some of these are summarized in table 8, to provide some insight in the difference between the three error types. It should be noted that these error types are based on the cognitive task level, and as such generally applicable.

Figure 8: Summary of the distinctions between skill-based, rule-based and knowledge-based errors. (page 62 Reason, 1990, abbreviated)

Dimension	Skill-based errors	Rule-based errors	Knowledge-based errors
Type of activity	Routine action	Problem-solving activities	
Focus of attention	On something other than the task in hand	Directed at problem-related issues	
Control mode	Mainly by automatic processors (schemata)	(stored rules)	limited, consious processes
Predictability of error type	Largely predictable `strong but wrong` errors (action)	(rules)	Variable
Ease of detection	Detection usually fairly rapid and effective	Difficult, and often only achieved through external intervention	

To illustrate the usage of knowledge and the relation with errors in a daily working situation, Reason (1990) developed a Generic Error-Modelling System (GEMS). This system is depicted in figure 9.

The model visualises the dynamical system of errors by illustrating the cognitive steps which are required to reach a goal. The first step is made on the skill-based level, by the performance of a highly routinised activity in familiar circumstances. The rule-based level is engaged when an attentional check detects a deviation from the planning. The idea behind this is that humans always try to solve a problem with 'rules' they already know. If these rules are not sufficient, the far more effort-full knowledge-based level is used to solve the problem. It should be noted that the lines between the three levels are not as clear-cut as envisioned, iteration between the different levels are constantly occurring. Another final note is that within one specific task all three levels could be active at the same moment. An example is car driving; watching the traffic and switching the indicator on are activities on two different levels, occurring at the same moment.

GEMS is based on a recurrent theme in the psychological literature, given by Rouse (1983): "humans, if given a choice, would prefer to act as context-specific pattern recognizers rather than attempting to calculate or optimize." This means for the GEMS model that humans are strongly biased to find a solution on the rule-based level before going to the far more effort-full

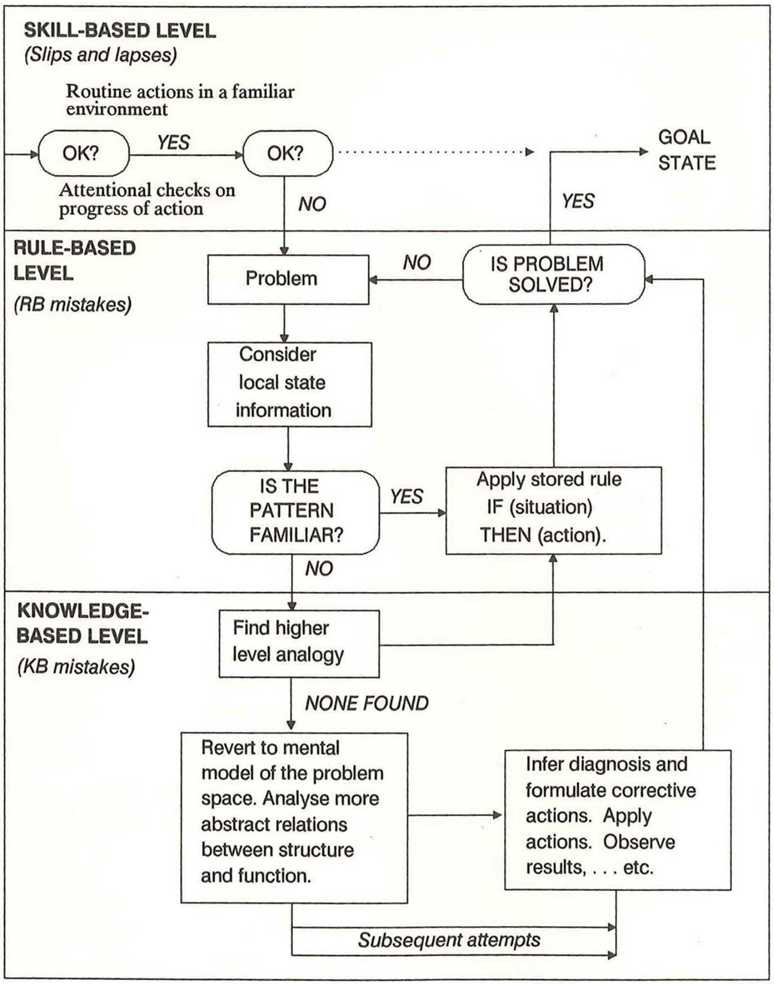


Figure 9: Outlining the dynamics of the generic error-modelling system (GEMS) (Reason, 1990, page 64)

knowledge-based level.

The idea of different kinds of errors on different cognitive levels is an interesting notion from a probability perspective. Quantifying probabilities are mostly based on the behavioural level of classification as will be outlined in the following chapters. In order to increase the accuracy of the probabilities and to be able to propose usable improvements, cognitive aspects such as discussed in this section should be incorporated in the risk analysis.

4 HUMAN RELIABILITY ASSESSMENT

INTRODUCTION

The probabilistic risk assessments which deal with human error are generally termed Human Reliability Assessments (HRA). This chapter discusses HRA in detail as it will be used extensively in the main research of this thesis. Within this chapter, the basics of HRA are discussed in section 4.1. The HRA process is discussed in section 4.2. Section 4.3 sets-apart three different HRA methodologies to get some idea about the technique. Section 4.4 finally discusses particular HRA application within structural design.

4.1 BASICS OF HUMAN RELIABILITY ASSESSMENT

Human Reliability Assessment (HRA) deals with the assessment of human potential in a system. HRA can be used to estimate the quantitative or qualitative contribution of human performance to system reliability and safety (Swain, 1990). The majority of work in human error prediction has come from the nuclear power industry through the development of expert judgement techniques such as HEART (Human Error Assessment and Reduction Technique), CREAM (Cognitive Reliability and Error Analysis Method) and THERP (Technique for Human Error Rate Prediction) (DiMattia et al., 2005).

According to Kirwan (1994), HRA has three basic functions which are:

- Identifying which errors can occur (Human Error Identification)
- deciding how likely the errors are to occur (Human Error Quantification)
- enhancing human reliability by reducing this error likelihood (Human Error Reduction)

Swain (1990) denotes the first function as the qualitative part and the second function as the quantitative part. The third function is not mentioned by Swain:

> The qualitative part of HRA largely consists of identifying the potential for human error, i.e. error like situations. The basic tool for identifying this error potential consists of task analysis [...] by observation, interviews, talk-troughs, error records, etc. [...] The quantitative part of HRA includes the estimation of time-dependent and time-independent Human Error Probabilities (HEP) [...] Such estimations are made by the use of human performance data, human performance models, and analytical methods associated with the HRA methods used.

HRA is an interdisciplinary discipline, involving reliability engineers, engineers, human-factors specialists and psychologists. According to Kirwan

(1994) two reasons are fundamental for this: it requires an appreciation of the nature of human errors and it requires some understanding of the engineering of the system.

One of the basic features of HRA is the allocation of so called Performance Shaping Factors (PSF). PSFs are the factors that affect human behaviour, and human error is also an aspect of human behaviour. From this perspective, PSFs within the human error characteristics are regarded as causes of human error. These considerations have a close link with the aspects of human error discussed in chapter 3, in which PSFs are discussed in more detail.

Origin HRA

Grozdanovic & Stojiljkovic (2006) and Kirwan (1994) provide both an overview of the development of HRA from the origin in the 60s. A short summary is presented in this paragraph to show the trends that have driven human reliability over the past decades. During the first decade of HRA, there was a desire to create human-error data-banks. There was also the need to consider Performance Shaping Factors (PSF), to guide the analyst in deriving the human-error rate for a particular task. After a while, it was being realised that the data-bank approach was not working. This is largely due to the now-obvious reason that humans are not, and never will be, the same as simple components, and should never be treated as such.

In the 70s, three strands of HRA research were evolving. The first involved a continuation of the database approach (on a more semi-judgemental basis). The second strand was the simulation approach, which uses distributions of performance times combined with Monte Carlo simulation to simulate the human reliability. The third strand was the time-reliability approach, involving the modelling of the 'mean-time-to-failure' ratio for hardware components. In the first half of the 1980s, there was a drive towards understanding human error at a deeper, psychological level. The predominance of engineering approaches was now going into reverse, and greater credibility was being given to more psychological and expert-opinion-based quantification approaches. A recent development is the shift from quantitative HRA towards qualitative HRA insights.

There are now a vast amount of HRA tools available, each of them focused on a particular usage or based on particular information sources. The comprehensive Safety Methods Database of Everdij, Blom & Kirwan (2006) provides an overview of 726 techniques, methods, databases, or models that can be used during a safety assessment. 142 of these tools are related to human performance analysis techniques.

Categorization HRA tools

Several categorizations within the HRA tools are available in order to require insight into the use of it. Most HRA tools are based on the usage within operation functions within a particular hazardous industry such as the nuclear industry, aviation and offshore industry. According to Kirwan (1996) HRA techniques can be divided according to their particular information source into two categories: those using a database and those using expert opinions. Examples of database based methods are Technique for Human Error Rate Prediction (THERP), Human Error Assessment and Reduction Technique (HEART), Justification of Human Error Data Information (JHEDI) and Cognitive Reliability and Error Analysis Method (CREAM), examples of expert opinion based methods are Succes Likelihood Index Method (SLIM), Absolute Probability Judgement (APJ) and Paired Comparisons (PC).

Hollnagel (1998) distinct first and second generation HRA. First generation HRA are based on the limited university of the binary event tree, which causes a low level within the classification schemes due to this limited university. Examples are THERP and HEART. Second generation HRAs use enhanced event trees which extend the description of error modes beyond the binary categorisation of success-failure. A second property of the second generation is that it explicitly states how the performance conditions affect performance. An example of a second generation HRA tool is CREAM.

Harrison (2004) distincts two approaches of HRA which is closely related to the distinction of Hollnagel (1998). The writer distinct an 'engineering' approach and a 'cognitive' approach. The engineering approach is based on a quantitative decomposition, the base assumption is: "human performance can be adequately described by considering individual elements of the performance. Total performance is an aggregate of the individual performance elements." Within this approach, the human is treated as components within a complex system. This approach is the dominant approach to HRA, and is comparable to the first generation approaches mentioned by Hollnagel. The cognitive approach is based on the explicit use of models and theories of cognitive functions which underlie human behaviour (comparable with Hollnagels second generation). This method is used to a lesser extent, the cognitive psychology is still immature and the human cognition is not directly observable.

4.2 The HRA process

The HRA process is depicted in figure 10. Not all of these steps are important within the scope of this research. For instance error reduction will not be discussed into detail as this research focusses primarily on Human Error Identification and Quantification. Based on the guidebook on HRA of Kirwan (1994), some components are briefly outlined below.

Task Analysis (at 2) refers to methods of formally describing and analysing human-system interactions. There are many different kinds of task analysis techniques, which are usually fairly straightforward. Examples are the hierarchical task analysis, link analysis and tabular task analysis. Human Error Identification (at 3) deals with the question of what can go wrong in a system from the human error perspective. Some of the Human Error

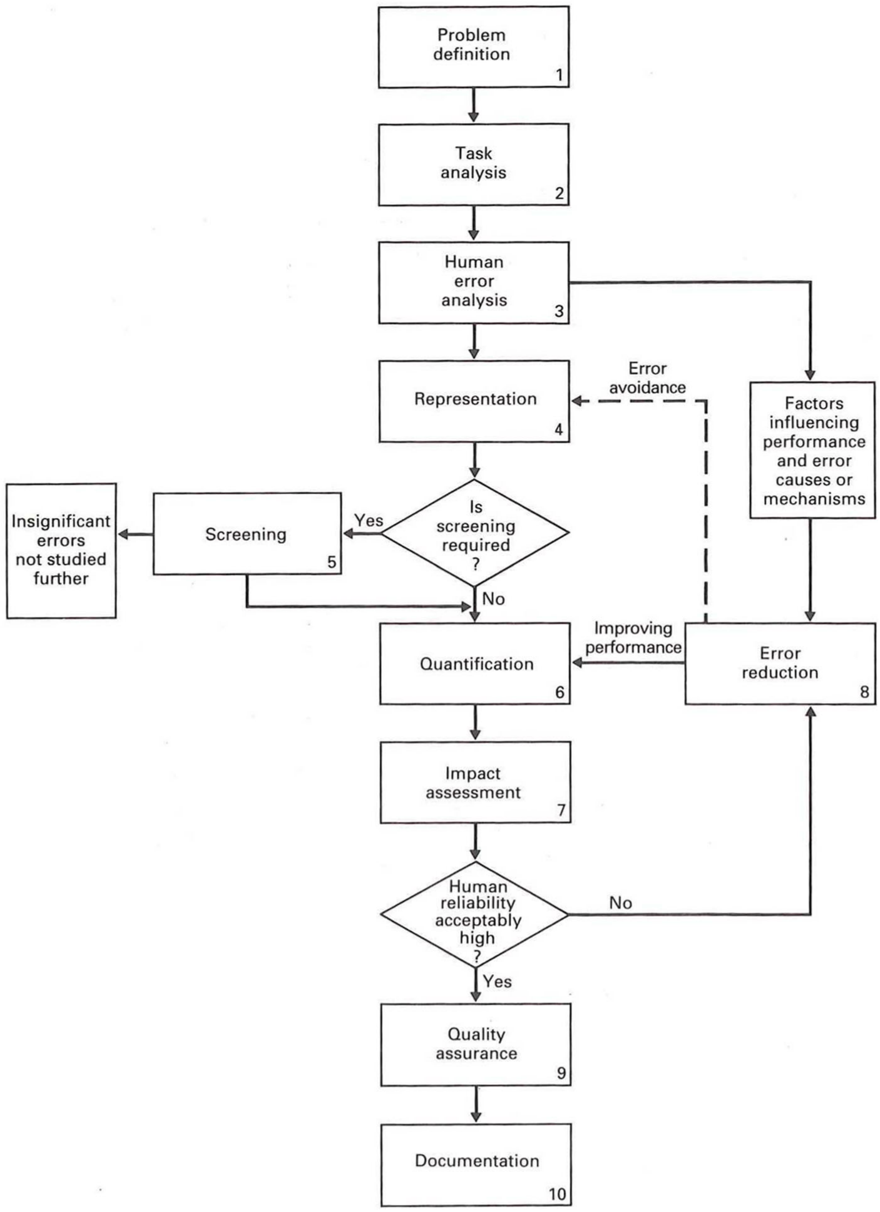


Figure 10: the HRA process (Kirwan, 1994, page 32)

Identification approaches are: Human error Hazard and Operability study and Murphy Diagrams. Human Error Quantification (at 6) is the most developed part of the HRA process, there are a number of sophisticated tools available. All Human Error Quantification techniques involve the calculation of a Human Error Probability, which is the measure of Human Reliability Assessment. Human Error Probability is defined as follows:

$$\mathsf{HEP} = \frac{\text{The number of times an error has occurred}}{\text{The number of opportunities for that error to occur}} \tag{1}$$

Examples of Human Error Quantification techniques are: APJ, PA, HEART, THERP and CREAM. Some of these techniques will be explained in section 4.3. Once the errors have been presented and quantified, the overall risk level can be determined in the Impact Assessment (at 7). One of the last steps within the model is to determine if the calculated/estimated human reliability is acceptable. After this step error reduction or quality assurance follows as a last step.

Limitations HRA

HRA is not commonly accepted as a reliable risk assessment tool. There are a couple of reasons for the fact that fully adequate HRAs do not exist. Swain (1990) provides two reasons. Firstly there are real problems in performing a HRA regardless of the HRA method used, and there is a wide variety in the effectiveness of existing HRA methods. Secondly, design and operation of many complex systems are inadequate for reliable human performance. Kirwan (1996) supports this statement by criticizing the validity of HRAs based on two counts: firstly, the quantitative prediction of human performance is seen as doubtful. And secondly, the theoretical underpinnings of the technique is seen as too crude to be plausible.

To illustrate the shortcomings of HRAs, Swain (1990) provides seven inadequacies within HRAs, which seem plausible at this moment as well:

- The scarcity of data on human performance that are useful for quantitative predictions of human behaviour in complex systems.
- Because of the lack of real-world data on human performance, less accurate methods are used like stop-gap models and/or expert judgement
- Methods of expert judgement have not demonstrated satisfactory levels of consistency and much less accuracy of prediction.
- The issue of calibrating human performance data from training simulators has not been seriously addressed.
- Demonstrations of the accuracy of HRAs for real-world predictions are almost non-existent.
- Some HRA methods are based on highly questionable assumptions about human behaviour.
- The treatment of some important performance shaping factors (PSF) are not well addressed.

Also producing reliable quantitative HEP values is regarded as difficult. Grozdanovic & Stojiljkovic (2006) states that there are three major technical problems for generating HEP data. The first problem is the degree of specificity inherent in the data for the particular situation, as large variations exist between each situation. The second problem is the usefulness of the data on how to improve human reliability, as the HEP data does not give information on this. The third problem with purely quantitative data is that such data only state the external form, or observable manifestation, of the error.

Despite the limitations of HRA, it still represents a very useful tool for designing complex systems and for assessing the human-induced risks of such systems to the public (Swain, 1990). Harrison (2004) ads to this that analysing and measuring dependability without assessing human reliability is at best incomplete and at worst misleading. Hollnagel (1998) supports this: "The need is, however, present whenever or wherever an interactive system is being designed, since the reliability of the system must be a concern."

Concerning the reliability of the HRA methods, Kirwan, Kennedy, Taylor-Adams & Lambert (1997) has validated three HRA techniques: THERP, HEART and JHEDI using 30 active HRA assessors. This resulted (among others) in the following conclusion "the results were that 23 of the assessors showed a significant correlation between their estimates and the real HEPs." Based on this validation it becomes clear that HRA has the potential to predict human error probabilities.

4.3 EXAMPLES HRA METHODOLOGIES

In this section three HRA methodologies (THERP, HEART and CREAM) will be discussed in some detail to give an idea of the methods used in practise. These methodologies are selected for further research as they are quite influential within the HRA theory and information concerning these technologies is readily available.

THERP

The Technique for Human Error Rate Prediction (THERP) is a methodology for assessing human reliability, developed by Swain & Guttmann (1983). THERP is a total methodology. It deals with task analysis, error identification, representation and quantification of HEPs (Kirwan, 1994).

For the quantification part, THERP provides several subsequent functions. A database of human errors can be modified by the assessor to reflect the impact of PSFs on the scenario. Then a dependency model calculates the degree of dependence between two actions. This is followed by an event-tree model that combines HEPs calculated for individual steps into an overall HEP. Finally error-recovery paths are assessed.

THERP's strengths are that it has been well-used in practice and that it offers a powerful methodology which can be altered by the assessor. THERP's disadvantages are that it can be resource-intensive and that it does not offer enough guidance in modelling both scenarios and the impact of PSFs on errors (Kirwan, 1994). Another disadvantage of THERP is that it is relatively psychologically opaque, dealing only with external error modes rather than psychological error mechanisms (Hollnagel, 1998).

HEART

The Human Error Assessment and Reduction Technique (HEART) is developed by J.C. Williams (Williams, 1986, 1988), a British ergonomic with experience of many hazardous technologies. According to Williams (1988), HEART is developed not only to assess the likelihood and impact of human unreliability but to apply human factors technology to optimise overall systems design.

Within the HEART methodology, a couple of Generic Task Types (GTT) are given. To each of these GTTs, a range of human unreliability values as well as a suggested nominal value is assigned (table 3). Besides these GTTs, a set of error producing conditions (EPS) is given, to which a multiplication factor (E) is assigned (table 4). Finally a weighting factor (P) should be assigned to each error producing condition based on the judgement of the designer (Hyatt, 2006). The Human Error Probability (HEP) is then computed from:

$$\mathrm{HEP} = \mathrm{GTT} \prod_{1}^{n} \{(E_i - 1)P_i + 1\}, \text{ and smaller than } 1 \tag{2}$$

GTT	General Task Type nominal value for human unreliability (table 3)
E_i	assigned EPC factor (table 4)
P_i	assigned weighting factor applied to individual EPC factor

A small example is presented to demonstrate the usage of HEART within the building industry. Say for instance we look at the calculation procedure of a fairly normal task, the design computations of a typical reinforced concrete beam. Say for instance the computation is executed under time pressure and as a consequence only minor independent checking is executed. The generic task description (GTT) would be D; a fairly simple task. The weighting factor (P) is set to 0.1. It should be noted that this factor is subjected to personal interpretation which leaves quite some space for variation. The error producing conditions are a shortage of time (2) and little or no independent checking (17) (see table 4). The HEP for this example is:

$$\mathrm{HEP} = 0.09 \cdot \{(11 - 1)0.1 + 1\} \cdot \{(3 - 1)0.1 + 1\} = 0.228 \tag{3}$$

The methodology is highly questionable as regards to determining accurate human error probabilities but can be valuable for comparing situations (Hyatt, 2006). Reason (2000) criticises the method on the absent of agreement between different assessors: "When people are asked to make absolute probability estimates of a particular kind or error type, their judgements may vary by orders of magnitude from person to person." However an extensive survey of the human factors literature has revealed that the effects of various kinds of manipulation upon error rates show a high degree of consistency across a wide variety of experimental situations (Reason, 2000). Despite these set-backs, the methodology is regarded as the best available account of the factors promoting errors and violations within the

Task type	Generic tasks	Nominal error probabilities (5th-95th percentile bounds)
A	Totally unfamiliar, performed at speed with no idea of likely consequence	0.55 (0.35 to 0.97)
B	Shift or restore system to a new or original state on a single attempt without supervision or procedures	0.26 (0.14 to 0.42)
C	Complex task requiring high level of comprehension and skill	0.16 (0.12 to 0.28)
D	Fairly simple task performed rapidly or given scant attention	0.09 (0.06 to 0.13)
E	Routine, highly practised, rapid task involving relatively low level of skill	0.02 (0.007 to 0.045)
F	Restore or shift system to original or new state following procedures, with some checking	0.003 ($8 \cdot 10^-4$ to $7 \cdot 10^-3$)
G	Very familiar, highly practised, time to correct, job aids	0.0004 ($8 \cdot 10^-5$ to $9 \cdot 10^-4$)
H	Respond correctly to system even when supervisory system provides accurate interpretation on system state	0.00002 ($6 \cdot 10^-6$ to $9 \cdot 10^-5$)
M	Miscellaneous task for which no description can be found	0.03 0.008 to 0.11

Table 3: Proposed nominal human unreliability (GTT) (Williams, 1988, page 439)

workplace according to Reason (2000). Reason declares: "the fact that they can be ranked reliably- so that we can assess the relative effects of the different factors- represents a major advance and an extremely valuable addition to the safety-concerned manager's tool box."

CREAM

The Cognitive Reliability and Error Analysis Method (CREAM) is developed by Erik Hollnagel and extensively described in Hollnagel (1998). This method is a representative method of the second generation HRA methods. CREAM has two main features: it emphasizes the important influence of the context on human performance and has a useful cognitive model and framework which could be used in analysis. The core of CREAM is that human error is not stochastic, but more shaped by the context of the task.

EPC Type	Error producing condition (EPC)	Multiplying factor (E)
1	Unfamiliar situation, potentially important, only occurs infrequently or is novel	17
2	A shortage of time available for error detection and correction	11
3	Channel capacity overload, e.g., by flooding with new information	6
10	The need to transfer specific knowledge from task to task without loss	5.5
16	Poor quality of info conveyed by procedures and person-to-person interaction	3
17	Little or no independent checking or testing of output	3
20	Mismatch between educational level of individual and requirements of task	2
25	Unclear allocation of function and responsibility	1.6
38	Age of personnel performing perceptual tasks	1.02

Table 4: Error producing conditions (E) (Williams, 1988, page 438-439) (abbreviated)

The main advantage of CREAM is its emphasises on the complex interactions between human cognition, and the situations or context in which the behaviour occurs. This model of cognition is an alternative to the traditional information processing models. Rather than describing the human mind as an information processor, the focus is on how actions are chosen. It does not define specific routes of human information processing, but rather describes how a sequence of actions can develop as the result of the interaction between competence and context. The basic notion of this is that the degree of control that a person has over his actions may vary, and that this to a large extent determines the reliability of performance (Hollnagel, 1998).

A second advantage is the useful cognitive model and framework elaborately presented in Hollnagel (1998), which can be easily used within quantifying probabilities. From this perspective CREAM is considerably simplified in comparison with other methods, as CREAM focuses on the level of the situation or working conditions rather than on the level of the individual actions.

The CREAM methodology proposes a two step approach to quantifying error probabilities. The first step is an initial screening (basic method) of tasks, followed by a second step to analyse only the most important probabilities (extended method). A deficit for use of this model within engineering is that it is primarily focussed on operator tasks within hazardous industries such as aviation and nuclear power industry. Despite this set-back, CREAM is usable within engineering types of tasks as Hollnagel has attempted to keep the methodology generally applicable. The CREAM method is selected as a basis for the HRA model for (design) engineering tasks defined in this thesis. The CREAM method is used as it emphasises the complex interaction between human cognition and the situation or context in which the behaviour occurs. Further elaboration on the extended quantification method in CREAM can be found in chapter 9.

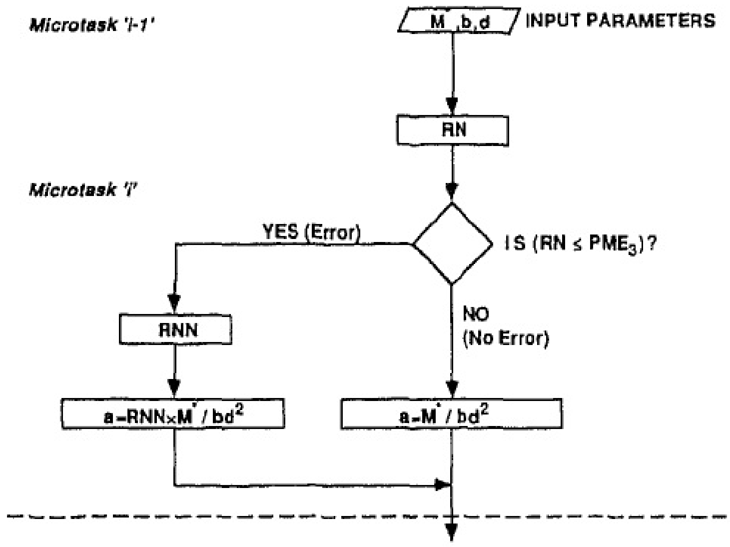
4.4 HRA IN DESIGN

Most Human Reliability Assessment (HRA) literature and techniques are directed towards operational tasks, which differs from designing tasks considered in this thesis. A particular research program aimed towards HRA in design situations is commenced in the eighties/nineties in Australia at the University of Newcastle. This research program is published in several papers: Melchers (1984), Melchers (1989), Stewart & Melchers (1988), Stewart (1992a), Stewart (1992b) and Stewart (1993). This section summarises some of the relevant results of this research program.

The HRA method proposed is based on Monte-Carlo simulation. The mathematical model used to simulate human error was developed from event-tree methodology. This methodology enables a design task (or macro-task) to be divided into successive individual components (or micro-tasks) (Stewart, 1992b). On this way first understanding the influence of human error on the 'micro' (or single task) level is obtained, which is then translated to understanding on the macro-level (Stewart, 1992a). The procedure comprises the following steps: at each micro-task a random variable is generated and compared with the given Error Rate for the represented operation. This enables a binary decision (either "error-included" or "error-free") to be made. If the error is included, the output of the micro-task is multiplied with an Error magnitude. This Monte-Carlo simulation process is presented in figure 11.

This Monte-Carlo procedure requires two sets of parameters originating from human behaviour. These parameters are the probability of human error (HEP) within a micro-task and the error magnitude (EM) if such an error has occurred. One of the main difference between HRA within operational types of tasks and within design tasks is the occurring Error Magnitude. Within design this is defined as a numerical deviation within a design parameter from the expected numerical value. According to Melchers (1989), these probabilities of human error are clearly related to task complexity, and cannot be represented by Poisson process models in which error are modelled as random events over some tasks. In the remainder of this section three micro-tasks will be discussed concerning the two sets of parameters. These tasks are: simple calculation, table look-up and table interpolation.

Simple calculation
A calculation is defined as a discrete number of mathematical operations on a set of numbers leading to a recorded result. For simple calculations



RN = Random Number
RNN = Error magnitude for calculation error
PME_3 = error rate for calculation micro-task

Figure 11: Section of event tree for calculation micro-task $a = \frac{M^*}{bd^2}$ (Stewart, 1993, page 282)

it was found in a study among engineering students, that the error rate was about 0,01 to 0,015 with considerable scatter (Melchers, 1989). A more intensive study found that the error rate increased directly with the number of mathematical operations to be performed (Melchers, 1989). In this study a higher error rate for an one-step calculation of about 0,025 was observed. The results could be categorized in three groups: computation errors, decimal error and round-off errors. The number of round-off errors was found to be small, and was ignored in further analysis (Melchers, 1984). An overview of the other two categories is shown in table 5.

Table 5: Error frequency within a one-step and two-step calculation task (Melchers, 1984)

	One-step S.S.[a] = 1244 P_E	Two-step S.S.[a] = 1211 P_E	Combined S.S[a] = 2455 P_E
Computation	0,0072	0,0157	0,0114
Decimal	0,0056	0,0049	0,0053
Overall	0,0128	0,0206	0,0167

[a] Sample Size

Based on the calculation results presented above, Melchers & Harrington (1984) (as sited from Stewart, 1992b) have proposed a distribution curve for the error magnitude in an one-step calculation task. This distribution curve is shown in figure 12. Visual interpretation of this curve reveals that the distribution consist of two separate curves: a log-normal distribution with a standard deviation of 1,35 representing the computation errors and a discrete distribution representing the decimal errors.

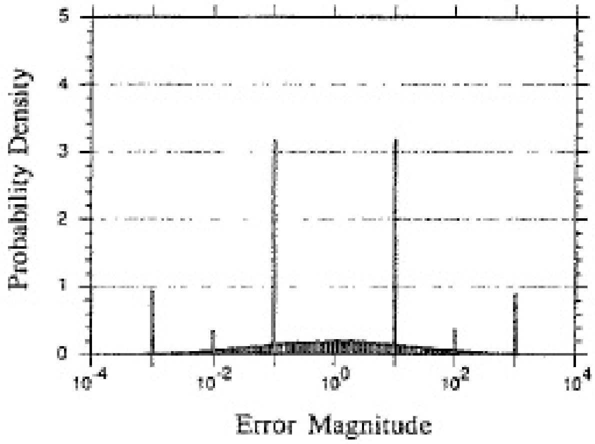


Figure 12: Distribution curve of the error magnitude of a one-step calculation task (Melchers & Harrington (1984), sited from Stewart (1992b))

Table look-up
This micro-task is defined as the ability of a designer to look-up a specific value from a table of values. The average error rate for a table look-up is estimated to be 0,0126. (Melchers, 1984). However this can increase to 0,06 if a more complex table was used, and if there was a high degree of task repetition (Melchers, 1989). The error magnitude of the table look-up task is presented in figure 13. Visual interpretation of this figure based on relevant tables for look-up tasks reveals that 10 % of the errors have approximately a value of 2 to 4 times the correct value. A Normal distribution with a standard deviation of 0,80 (if the correct value is 1) would approximate this result.

Table interpolation
This task is defined as comparing tabulated values and then selecting the correct value corresponding to a specific ranking instruction. The average error rate from this micro-task is 0,0135 (Melchers, 1984) for rather simple tasks. An error rate of up to 0,08 was obtained for more realistic tables (Melchers, 1989).

Conclusion
Based on the findings in Melchers (1984, 1989), Stewart (1992b) presented a summary on the average error rate of the micro-tasks discussed above. This summary is presented in table 6.

Error variation
It might be expected that error rates for individuals (for a specific task) will

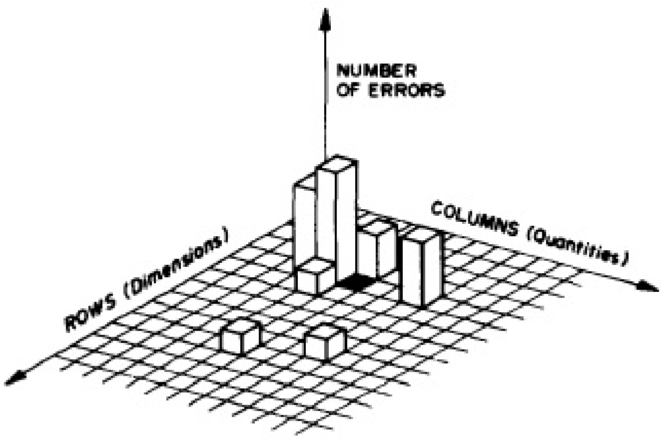


Figure 13: Distribution curve of the error magnitude of a table look-up task. The shaded location represents the correct results. (Melchers, 1984)

Table 6: Summary of average micro-task error rates (Stewart, 1992b)

Microtask	Average error rate
Table ranking	0,0135
Table look-up	0,0126
Chart look-up	0,0200
One-step calculation	0,0128
Two-step calculation	0,0256
Three-step calculation	0,0384
Five-step calculation	0,0640
Six-step calculation	0,0768

vary due to differing ability, personal characteristics, work environments and other factors that affect task performance. Within the Cognitive Reliability and Error Analysis Method (CREAM) presented in Hollnagel (1998), this is accounted for by adjusting the nominal Cognitive Failure Probabilities (CFP). Stewart (1993) proposes to represent this variation in error rates by a log-normal distribution, based on findings of Swain & Guttmann (1983). The mean of the log-normal distribution is equal to the average error rate as presented in table 6. A usable measure of variance is the Error Factor (EF), as expressed in equation 4.

$$EF = \sqrt{\frac{Pr(E_{90^{th}})}{Pr(E_{10^{th}})}} \tag{4}$$

Within this formula $Pr(F_{10^{th}})$ and $Pr(F_{90^{th}})$ are the error rates corresponding to the 10^{th} and 90^{th} percentiles respectively (Apostolakis, 1982). Stewart (1992b) proposes to use an Error Factor of EF = 3 for design tasks.

5 PROBABILITY OF FAILURE

INTRODUCTION

In this section the probability of structural failure is discussed. Analysing probabilities is used within Human Reliability Assessments to quantify human error probabilities. Furthermore probability analysis is commonly used in risk [1] assessment techniques. Concerning this Baker, Schubert & Faber (2008) wrote that an ideal design is the one having minimal risk, achieved by balancing the reduction of risks against the cost of the risk reducing measurements. Within human factors, reduction of risks can be translated to taking human error prevention measurements which are realistically and applicable within the current engineering practice. This section will first answer the question how to quantify probabilities (section 5.1) and then elaborate on how to execute a risk assessment (section 5.2). The use of Monte Carlo analysis methods is discussed in section 5.3, followed by a short discussion on robustness in probabilistic analysis in section 5.4.

5.1 QUANTIFYING PROBABILITIES

There are some problems with quantifying failures and the chance of collapse. In 1987, Ellingwood noticed that most of the errors are difficult to quantify, as their source is human imperfection. Quantifying this is hard due to the difficulty to obtain complete and unbiased statistics (Fruhwald et al., 2007). Given this, the solution of the error problem is not strictly a matter of statistics, probability and reliability theory. However despite the challenges (assigning probabilities to the exposures), occurrence probabilities are required to efficiently allocate resources for risk reduction (Baker et al., 2008).

The question remains which analysis tool should be used to quantify the probabilities. Many Techniques and methods have been developed to support the safety assessment process of a particular operation, procedure or technical system. Everdij et al. (2006); Everdij & Blom (2006) have developed a database of well over 700 safety methods. The list contains methods for hazard identification, mathematical models, etc. The methods come from several domains of application, such as nuclear power industry, aviation, etc.

Most of the methods concerning the quantification of probabilities can be categorized as so called Probabilistic Risk Assessment (PRA) tools. These methods are characterized by a systematic methodology to evaluate risks associated with a complex system. In a PRA, risk is characterized by two quantities: the magnitude of the possible consequence and the probability of occurrence (Stamatelatos, 2000). A PRA can be quantitative as well as

1 Defined as the combination of the probability of occurrence of a hazard and the magnitude of the consequence of occurrence.

qualitative.

One method for analysing human reliability is a straightforward extension of Probabilistic Risk Assessment (PRA). In the same way that equipment can fail in a plant, so can a human operator commit errors. In both cases, an analysis would articulate a level of detail for which failure or error probabilities can be assigned. The analysis to assess human error is termed Human Reliability Assessment (HRA) and is already discussed in chapter 4. A shortfall of this method is its focus on operational types of activities instead of design activities, caused by its usual application in operational industries such as aviation, nuclear- and offshore- industries. Melchers (1984) wrote: "Virtually no information appears to exist about tasks such as those performed in design." It seems that this statement is still valid as not much has changed in the past decades concerning HRA in design.

5.2 RISK ANALYSIS

There are several tools developed within the literature to model human error and risk with the help of probability calculation. Some of these tools will be set-apart in this section. A basic model for describing the effect of human error on structural resistance is presented by Vrouwenvelder et al. (2009), by defining a multiplier on the resistance within the basic limit state function of $Z = R - S$. Within this limit state function, failure will occur if $Z \leqslant 0$. Furthermore S is defined as the loadings acting on the structure and R is defined as the resistance of the structure. The multiplier on the resistance is termed:

$$R = R_0 + \Delta \tag{5}$$

The term R_0 is the resistance based on the correct design, appropriate construction and use of a structure, unaffected by any error. Δ represents the effect of errors on the resistance. Within the paper, probability of occurrence is conservatively assumed to be 1.0 and the effect of an error is approximated by the normal distribution with a zero mean and a standard deviation of 0.15 or 0.30 μ_{R0}.

This basic formula can be enhanced by using more accurate numbers for the mean and standard deviation. The paper of El-Shahhat, Rosowsky & Chen (1995) elaborates on this by presenting two approaches for addressing the issue of human failure during engineering and construction. These methods are:

- Reliability analysis of human error; within this analysis, the relative effect of failures in load and resistance on the global reliability index of the structure is evaluated. This is done with the help of statistical information on the occurrence and magnitude of errors.
- Scenarios for failures during construction; different scenarios for errors during the construction are investigated to quantify relative consequences associated with these errors.

5.2.1 *Reliability Analysis*

The first method, assessing global reliability and human errors, is also examined in the paper of Nowak & Carr (1985). This paper terms this method as 'sensitivity analysis for structural errors'. According to the authors, this method provides engineers with a tool which can calculate the relative importance of different types of failure on structural reliability and concentrate on the most important failures. According to the paper frequencies of failures can be established from experience, and their consequence can be identified through sensitivity analysis. Establishing frequencies based on experience, however, is doubtful. The paper does not present these frequencies of errors as a proof that these numbers can be found easily. As mentioned before several researchers have doubted the possibility to quantify frequencies with sufficient accuracy (Fruhwald et al., 2007; Ellingwood, 1987; Ellingwood & Dusenberry, 2005).

Within this method, an error free reliability index is first computed, and is then modified to account for errors to provide an estimate of the global reliability index. Finally a sensitivity analysis is performed to investigate the relative effects of errors on the global reliability index (El-Shahhat et al., 1995). The method is based on the variables R and S mentioned before. Based on El-Shahhat et al. (1995), the probability of failure can be expressed as:

$$P_f = \int_0^x F_R(q) f_s(q)\,dq \tag{6}$$

Where F_R is cumulative distribution function of the resistance and f_s is the probability density function of the load. The second-moment reliability index can be expressed as:

$$\beta = \frac{\mu_R - \mu_Q}{\sqrt{\sigma_R^2 + \sigma_Q^2}} \tag{7}$$

Where μ_R and μ_Q are mean values of the resistance and load respectively. σ_R^2 and σ_Q^2 are the variances of the resistance and load respectively. If we consider only errors which affect the mean value, we can consider an error which modifies the resistance (μ_R) or an error which modifies the load (μ_Q). Say μ_R is modified by the factor λ and μ_Q by the factor τ, then the second-moment reliability index becomes:

$$\beta = \frac{\lambda\mu_R - \tau\mu_Q}{\sqrt{\sigma_R^2 + \sigma_R^2}} \tag{8}$$

Where the factors λ and τ are defined as: $\lambda = 1 \pm rV_R$ and $\tau = 1 \pm qV_Q$. r and q are positive numbers and V_R and V_Q are the coefficients of variation of R and Q respectively. This formula can be used to investigate the sensitivity of the global reliability index to changes in the error parameters λ and τ. As input of this formula, statistical information on the occurrence and magnitude of errors is required.

Nowak & Carr (1985) have applied the approach to the analysis of a concrete bridge slab, a timber bridge deck and a steel frame beam-to-column connection. The paper presents usable sensitivity functions, as example the reliability function of the concrete slab is presented in figure 14. In this reliability function, the concrete slab is investigated on the parameters effective depth, strength of the concrete, spacing between rebars (s), dead load (D), live load (L) and Impact (I). It can be seen from figure 14 that effective depth is the most sensitive parameter. Despite the interesting reliability indexes, Nowak & Carr (1985) provides no statistical data on the occurrence and magnitude of errors.

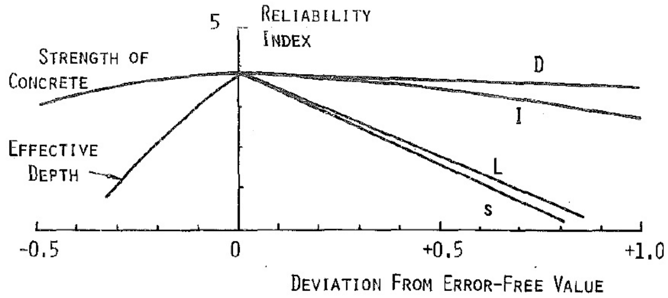


Figure 14: Sensitivity functions for the concrete slab (Nowak & Carr, 1985, page 1741)

Several other researchers have contributed to the development of probabilistic formulas for the analysis of human error. Nowak (1979) has developed a probabilistic model to provide for the risk of failure due to possible errors. This models is primarily based on the probability density function of Z, probability of failure P_F and the optimum cost of control. This model leans on the suggestion that socio-economic optimization is a valid procedure for setting both safety factors and levels of control. According to Melchers (1984), the main difficulties with this approach are the knowledge of the cost of failure of the structure, the cost of control and the probability of failure. Duffey & Saull (2004) have developed two formulas for the probability of human error based on the learning hypotheses as a function of experience. The formula consists of a double exponential function. According to Duffey & Saull (2004), this function plays a key role: "after the initial failure rate given by the purely Bayesian estimate, the subsequent failure rate is of a double exponential form, where the probability naturally follows a classic bathtub curve."

5.2.2 *Scenario Analysis*

The second approach, scenario analysis, assumes different error scenarios and calculates the corresponding failure probabilities. In comparison to reliability analysis, this method focusses on a the most relevant risks, omitting the need for statistical data on all risks. Ellingwood (1987) demonstrates this method in his basic form. According to Ellingwood (1987), scenarios for analysing errors and their effects should include the following elements:

- the identification of likely error-causing scenarios;
- the percentage of errors that are detected and corrected by quality assurance programs;
- the possibility that undetected errors cause structural defects, which may subsequently lead to damage or failure.

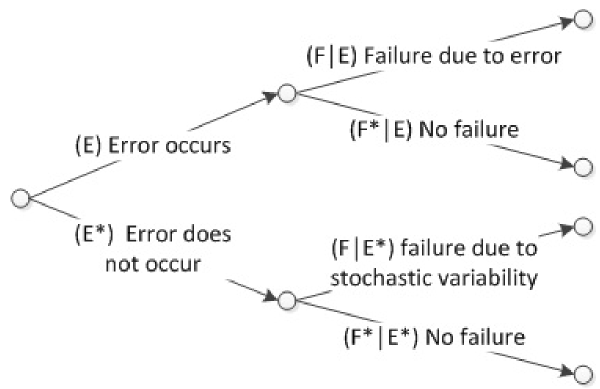


Figure 15: Event tree analysis of failure (Ellingwood, 1987, page 414)(abbreviated)

The mathematical model of the error effect on structural reliability can be developed by the event tree shown in figure 15. Let E be the event that an error occurs and is undetected. The probability of failure can be calculated as:

$$P(F) = P(F \mid E)P(E) + P(F \mid \bar{E})P(\bar{E}) \tag{9}$$

In which E is the event that a gross error does occur, and $\bar{E}$ is the event that a gross error does not occur. $P(F \mid E)$ is the probability of failure on the condition that event E occurs and $P(F \mid \bar{E})$ is the probability of failure due to stochastic variability. It is common to make the assumption that structures that contains errors never fail as a consequence of stochastic variability in loads and strengths. From this it can be concluded that structures fail either due to an error or due to a stochastic variability.

Baker et al. (2008) and Vrouwenvelder & Sorensen (2010) elaborate on this scenario based method, by presenting its usage for assessing structural robustness. Robustness is closely related to human error, as building robustness is the ability of a structure to withstand (among others) the consequences of human error. Robustness is further discussed in appendix A.

The basic event tree for robustness quantification is shown in figure 16. The assessment starts with the consideration of exposures (EX) that can cause damage to the components of the structural system. Exposures are for example extreme values of design loads, accidental loads, deterioration processes and human errors. After the exposure event occurs, the components of the structural system either remain in an undamaged state ($\bar{D}$) or change to a damaged state (D). Each damage state can then lead to the failure of the structure (F) or no failure ($\bar{F}$). Consequences are classified as

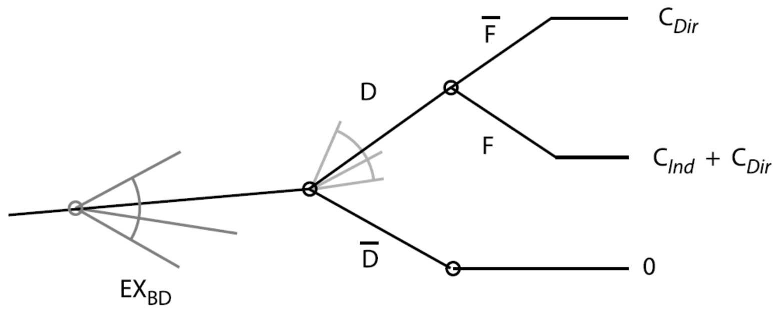


Figure 16: An event tree for robustness quantification (Baker et al., 2008, page 255)

either direct (C_{dir}) or indirect (C_{ind}). Direct consequences are considered to result from damage states of individual components and indirect are considered due to the loss of system functionality or failure caused by insufficient robustness.

With the event tree defined in figure 16 it is possible to compute the system risk due to each possible event scenario. This is done by multiplying the consequence of each scenario by its probability of occurrence, and then integrating over all of the random variables in the event tree. The risk corresponding to each branch is:

$$R_{dir} = \int_x \int_y C_{dir} f_{D|EX_{BD}}(y \mid x) f_{EX_{BD}}(x) dy dx \tag{10}$$

$$R_{indir} = \int_x \int_y C_{indir} P(F \mid D = y) f_{D|EX_{BD}}(y \mid x) f_{EX_{BD}}(x) dy dx \tag{11}$$

In which the parameters are defined as:

C_{dir}	consequence of local damage D due to exposure EX_{BD}
C_{indir}	consequence of comprehensive damage F given local damage D due to exposure EX_{BD}
$f_{EX_{BD}}$	probability of exposure EX_{BD}
$f_{D\|EX_{BD}}$	probability of damage D given exposure EX_{BD}
$P(F \mid D = y)$	probability of comprehensive damage F given local damage D due to exposure EX_{BD}

According to Baker et al. (2008), these integrals may be evaluated either through numerical integration or Monte Carlo simulation. Vrouwenvelder & Sorensen (2010) notes that an important step in the risk analysis is to define the system and the system boundaries.
A critical note on the scenario approach is the impossibility to detect and quantify unknown and undetected errors. By focussing on the risks which are produced by the scenarios the probability exists that unknown risks occur, causing a major failure.

5.3 MONTE CARLO ANALYSIS

The use of Monte Carlo analysis is mentioned in the previous section in combination with scenario analysis. These analysis are a class of computational methods that rely on repeated random sampling to compute their results. Within this section the results of two papers which model human error with the help of Monte Carlo simulation will be briefly mentioned. It concerns the papers of Epaarachchi & Stewart (2004) and Stewart (1993).

The first example concerns the modelling of human error within a construction by Epaarachchi & Stewart (2004). This paper discusses a human reliability model to estimate the probability of structural collapse during the construction of typical multi-storey reinforced-concrete buildings due to human error. The paper uses event-based Monte-Carlo simulation procedures to quantify this effect. This is achieved by incorporating human reliability models into an existing 'error-free' PRA model. To measure the effectiveness of error control measures, two cases are considered: system risks before engineering inspections and system risks after engineering inspections. About 100.000 simulation runs were done to reach a coefficient of variation of about 0.1. the probability of failure (p_f) is estimated as $p_f = n/N$ in which n is defined as the runs with a failure and N as the total number of Monte-Carlo simulation runs.

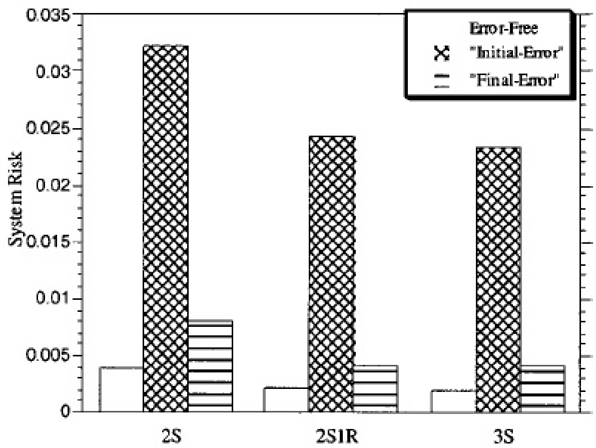


Figure 17: Effect of human error (before and after engineering inspections) on system risk (Epaarachchi & Stewart, 2004, page 18)

Figure 17 presents a diagram which compares the system risk of several building systems investigated within the paper of Epaarachchi & Stewart (2004). Three types of shoring systems were analysed: two floors shored (2S), two floors shored and one floor re-shored with no pre-compression (2S1R) and three floors are shored (3S). The results are given for the Error-Free situation (PRA model without human error), the system risk without inspection (Initial-Error) and the system risk after inspection (Final-Error). It can be seen that inspection has quite an influence, the Final-Error system risks have been reduced by about 70-80 % for all three shoring systems when compared to Initial-Error system risks.

The second example is provided by Stewart (1993). Within this paper, a Human Reliability Analysis (HRA) model is set forth to simulate the effect of human error on the design and construction of a reinforced concrete beam. The method consists of an event tree which is analysed using Monte-Carlo simulation techniques. The event tree represents a series of individual components (or micro-tasks) which are required for the successful completion of the system task (member design). The model incorporates the effect of designer checking, independent design checking, engineering inspections of construction works, and interaction between the designer and contractor (more information on the technical aspects of the model can be found in section 4.4).

The paper concludes that structural safety is more vulnerable to construction errors if there is no error control. Furthermore it concludes that structural reliability for two design checks is only marginally higher than that obtained for a single design check. Another paper of Stewart (Stewart & Melchers, 1988) provides more insight in the effect of control and checking on structural safety. According to this paper, self-checking detects only the small or minor errors that may occur in calculations, and that self-checking cannot adequately safeguard against error due to misconceptions, oversight or misunderstanding. With independent checking, larger errors are more easily detected than smaller ones. The paper concludes that independent detailed checking is a more effective control measure in comparison with self-checking.

The papers of Epaarachchi & Stewart (2004) and Stewart (1993) provide a good example of how to use a PRA/HRA Monte Carlo analysis within engineering, in order to receive realistic values for the effect of human error on the structural properties of a building. With regard to the content, both investigations focus (among others) on the effect of checking and inspection on the occurrence of errors. Both researches conclude that checking and inspection have quite a positive effect on the detection of errors.

5.4 ROBUSTNESS IN RISK ANALYSIS

Probabilistic risk assessment within structural engineering deals with the question if a building will collapse. An important aspect of this is the degree of robustness of the structure. Robustness of a building is the ability of a structure to withstand events like human error without being damaged to an extend disproportionate to the original cause. Faber et al. (2007) links robustness to risk assessment on the following manner: "the vulnerability of a give system [...] characterizes the risk associated with the direct consequences and the robustness characterizes the degree the total risk is increased beyond the direct consequences." Within this research emphasises is placed on characterizing the vulnerability of a structure to the occurrence of human error. The next step would be to broaden this research to indirect consequences by considering structural robustness. As an introduction to this subject, the theory of robustness is discussed in appendix A.

6 CONCLUSION

The literature study discussed aspects of human error within structural engineering. The objective of this study is to assess the current knowledge within scientific literature concerning the assessment of human error in structural engineering. In this chapter the results will be evaluated and the use of the literature findings in the main study will be discussed.

The literature study started with a chapter focussing on the causes of structural failure (chapter 2). Within this chapter general remarks on failure of structures and statistics of failure are given. Based on these findings it is concluded that most structural failures are caused by human error. Furthermore the occurrence of errors are of the same order of magnitude for design/planning and construction respectively, with slightly higher frequency for the design phase. This information pinpoints the problem of human error within structural engineering, which is used within the main research to focus on the relevant aspects of human error.

The following chapter (chapter 3) focusses on the basic aspects of human error from an engineering perspective. An important notion of human error are the so called models for accident causation, which enable conceptual thinking of error causation. An important model is the the 'swiss cheese' model of Reason et al. (2001) which consists of a number of defensive barriers after the unsafe act, once the hazards is introduced, and before the incident. Holes are occurring in these barriers due to latent failures or conditions, creating an opportunity for the incident to occur. Within the main research this information is used by modelling the design process with design steps in which an unsafe act can occur and with control/design steps which prevent the error from occurring (barriers). Another aspect is the non-linear and non-deterministic character of error causation which makes it hard to predict errors deterministically (Hudson, 2010). A solution for this is to represent errors by means of probability distributions instead of simple failures rates. This aspect is adopted in the model represented in the main research.

In order to understand human error, the cognitive behaviour of human error should be taken into consideration. An important aspect is error type distinction based on cognitive notions. Reason (1990) distinguishes three levels. Skill-based slips and lapses occurring during automatic behaviour which require little concious thought or when attention is being diverted. Rule-based mistakes occur when a known rule is incorrectly applied, or a situation is misinterpreted. Knowledge-based mistakes result from a deficit of knowledge. This analogy is used in the main research to distinguish three cognitive tasks levels within the proposed Human Error Probability quantification model.

Chapter 4 discusses the characteristics of Human Reliability Assessments (HRA). HRA deals with the assessment of human potential in a system. According to Kirwan (1994) it consists of three basic functions: identifying which errors can occur, decide how likely they will occur and reducing there error likelihood. Several aspects of HRA are discussed, such as quantitative HRA methods, HRA within design tasks and error magnitudes. Within the main research, these insights are extensively used within the proposed HRA model for design tasks.

The last chapter (chapter 5) discusses the possibilities to quantify human error and structural failure. For quantifying human error a reliability analysis method is discussed. Within this analysis failure will occur if the loadings acting on the structure are larger then the resistance of the structure. Human error is presented by means of a multiplier on the resistance. Another aspect is to use scenario analysis which assumes different error scenarios to calculate the corresponding failure probabilities. Both aspects, reliability analysis and scenario analysis are used in the main research to establish a structural failure probability based on human error probabilities.

Part II

MAIN RESEARCH

The main research focusses on presenting a Human Reliability Assessment model for use within engineering processes. The model is subsequently used to investigate the effect of human error in a predefined case. The main research consists of five chapters. The first chapter introduces the model, the following four chapters elaborates each on a subsequent part of the model and discuss the results of the case study.

7 HUMAN RELIABILITY ASSESSMENT MODEL

INTRODUCTION

Within this chapter a model for Human Reliability Assessment (HRA) within typical engineering tasks is proposed. This chapter consists of three sections. Section 7.1 states relevant model requirements. Section 7.2 discusses the application area of the model. Section 7.3 finally sets apart the model and discusses its basic form.

7.1 MODEL REQUIREMENTS

This section briefly discusses the model requirements by focussing on the functional demands of the model and the validity of the model. An elaborate discussion and evaluation of the model is presented in de Haan (2012).

The main functional demand of the Human Reliability Assessment model is to quantify human error within structural engineering in a realistic and useful manner. In line with Ale et al. (2012) the functional departure point is: "[...] that control systems need to be capable of at least as much variation as the body to control". This entails that the model should be suitable for representing human error in a structural engineering context, represent error causation, represent the cognitive aspects and represent the organizational context.

An important deficit of Human Reliability Assessments concerning accident causation is mentioned by P. Hudson in his inaugural speech (Hudson, 2010). According to Hudson, accident causation must be regarded as both non-linear and non-deterministic. Within the proposed model this is incorporated to a large extend. The non-linearity from organizational causes to effects is present in the focus of the model on the complex interactions between human cognition, rather then defining specific routes of human information processing. Concerning the non-deterministic character of accident causation, the model represents the variables in terms of probability distributions rather then simple failure rates. This is in line with Hudson (2010) and the distributions used in the CATS [1] model (Ale et al., 2009). Finally concerning the internal validity the question remains if the proposed model is advanced enough to cope sufficiently with the non-linear and non-deterministic properties of accident causation. Further research is required to verify this.

1 CATS, or Causal model for Air Transport Safety, is developed by a research consortium in the Netherlands in the beginning of this century. CATS is a causal model for air transport safety. Its purpose is to establish in quantitative terms the risks of air transport (Ale et al., 2009).

Another functional limitation of the model is its use for specific error types. The model is not capable of modelling all causes of human error in a structural design process. For instance extraneous acts and errors of intent cannot be modelled adequately. This deficit is a consequence of the focus of the model on tasks and within those tasks error probabilities from literature. As within the literature errors of intent and extraneous acts are not quantified, a structural failure probability caused by intentional actions cannot be obtained.

7.2 APPLICATION AREA

The Human Reliability Assessment method described in this chapter is developed for engineering processes, and more specifically structural design. These types of processes are characterised by an iterative procedure to determine the design of an engineered system. A common aspect of these processes is that the output of the process, and his intermediate processes, primarily consists of numerical design parameters. From this perspective it differs considerable from conventional HRA tools which are designed for operational types of activities.

The HRA model is designed for engineering tasks within the design-process. The model seems usable for tasks within the construction process as well. However re-evaluation of the model is required to adjust it to the specific needs within construction tasks, as design tasks differ somewhat from construction tasks. For this re-evaluation, adjustments based on operational types of HRA models can be useful as operational tasks and construction tasks have much in common.

7.3 MODEL

The model is among others based on two distinct HRA methods which are described in the chapters 4 and 5. The first method is the Cognitive Reliability and Error Analysis Method (CREAM) of Hollnagel (1998). The second method is the design HRA method proposed by Stewart and Melchers (Melchers, 1984, 1989; Stewart & Melchers, 1988; Stewart, 1992a,b, 1993).

The CREAM method is used to quantify Human Error Probabilities (HEPs), while the design HRA of Stewart and Melchers is used to couple these HEPs to derive an error probability on element level. The CREAM method is used as it emphasises the complex interaction between human cognition, and the situation or context in which the behaviour occurs. This is deemed necessary to meed the functional requirements. The design HRA of Stewart and Melchers provide a useful flow chart to transform HEPs of individual design tasks to an error probability on the structural level.

The model starts with a general idea about the particular engineering task, of which insights on the effect of human error is required. Through four distinctive HRA steps a failure probability of the structure due to human error is obtained. A stepwise overview of this model is shown in figure

18. The four steps are briefly set apart in the remainder of this section. The chapters 8 to 11 elaborate each on a separate step of the model in detail.

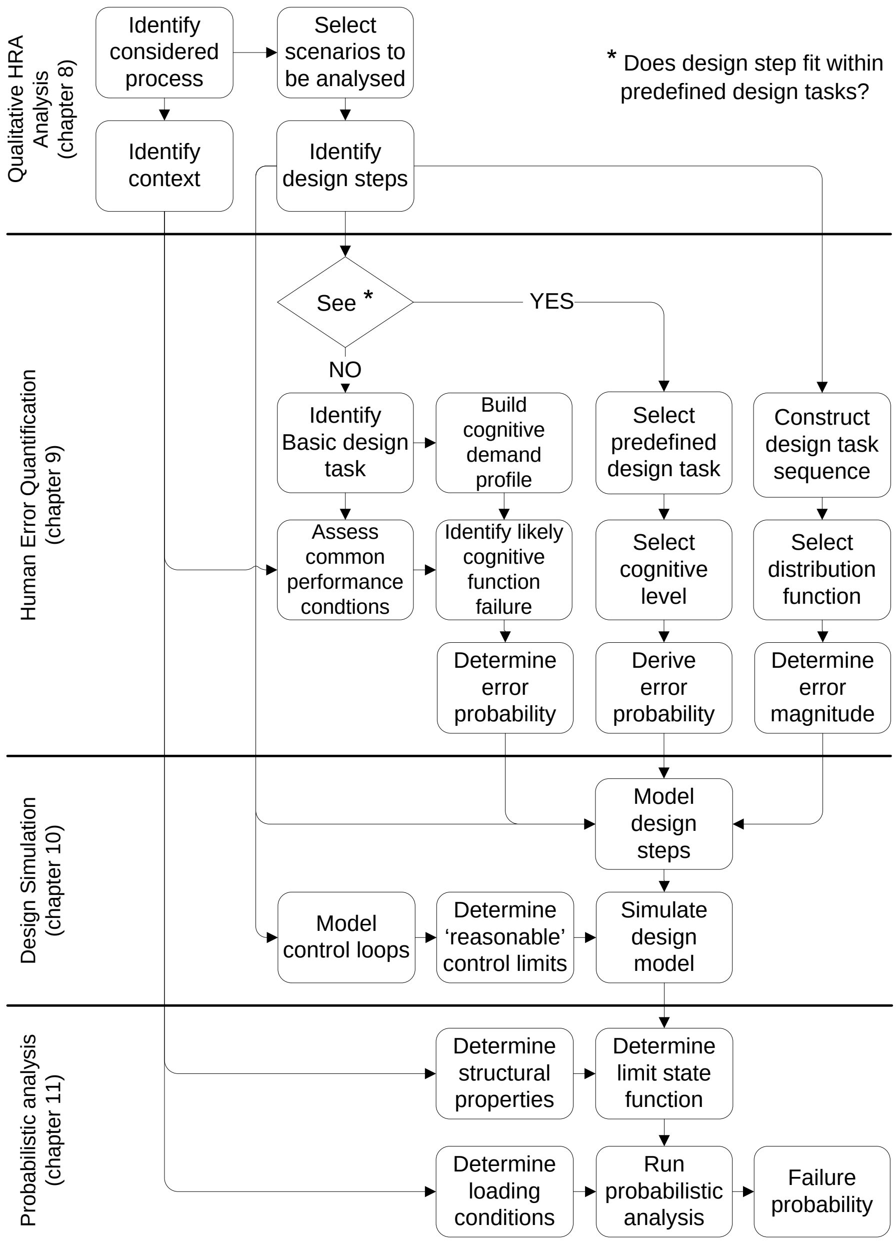


Figure 18: Basic model of Human Reliability Assessments within structural design tasks

The first step is a qualitative analysis of the design system/situation of interest. It starts with identifying the considered process and determining its boundary conditions. This is followed by selection of scenarios for further analysis, the design context and the required design steps. This process is required in order to focus on the design aspects, which are worthwhile considering with a quantitative HRA analysis. This is deemed necessary as a quantitative HRA analysis is very labour intensive. Furthermore, this step serves as a basis for the remainder of the HRA. The qualitative analysis is worked out in detail in chapter 8.

The second step is the Human Error Quantification (HEQ) within the HRA procedure. Based on the identified design steps and design context, a Human Error Probability (HEP) and Error Magnitude (EM) for each design task is determined. These HEPs and EMs together form a probabilistic representation of human error within a design task. The underlying method to quantify human error is based on the assumption that within each design task, basic cognitive tasks can be distinguished. The second step is discussed further in chapter 9.

The third step is the design simulation process. Within this process the individual HEPs and EMs are combined by means of a Monte-Carlo simulation. This process results in a probabilistic representation of the strength on structural element level. The third step is worked out in chapter 10.

The last step is a probabilistic analysis. Step 3 resulted in a probabilistic strength distribution. These distributions together with relevant loading distributions and material characteristics are used within a probabilistic analysis to determine the probability of failure of the structure. In total two structural beam types are considered: a statically determined beam and a statically undetermined beam within a frame structure. A detailed elaboration on this last step is presented in chapter 11.

8 QUALITATIVE HRA ANALYSIS

INTRODUCTION

The first step in the HRA model is to define the process of interest and its boundaries. This process consists of four steps which are shown in figure 19. The first step is to identify the design process of interest (section 8.1). Based on this, the design context and scenarios for research are selected (section 8.2 and 8.3 respectively). The last step is to identify all design steps within the considered process, which is discussed in section 8.4.

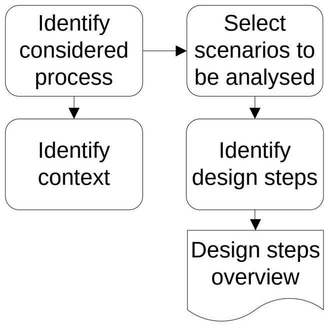


Figure 19: Basic model of Qualitative HRA analysis

8.1 PROCESS IDENTIFICATION

The process of interest is a structural engineering process within a typical engineering company. Two restrictions are imposed on the boundaries of the process: only the design process is considered and only a common structure is used as design object (office building). Furthermore with design process is meant the steps which are required to establish structural dimensions and other structural properties. Design activities left outside the research boundary are: selecting a structural type, establishing boundary conditions, site research, authority improvement, etc. These activities are of importance for finding the exact failure probability of a structure. However for demonstrating the use of the HRA model, restricting the case study to a few design tasks is sufficient. Further focus within the design process is based on the findings in the remainder of this chapter.

8.2 DESIGN CONTEXT

The context of the process under consideration consists of two parts: the design object and the design organisation. Within the case study an imaginary design object and organisation are used which is set-apart in this section.

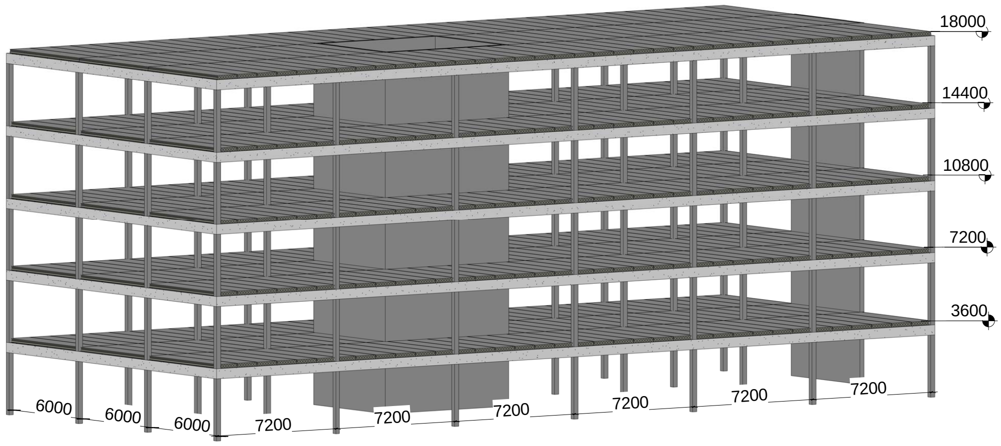


Figure 20: Overview of the office building used within the case study.

The design object is a beam element within an office building of the type shown in figure 20. This office building consists of a framework of concrete beams and column elements. The floor element (which are supported by the beam elements) consist of hollow core slabs. The overall stability of the frame structure is ensured by means of a stabilizing core. A technical drawing of the office building is given in appendix B. The material properties, dimensions of the structure and loading conditions on the beam element are given in table 7. It should be noted that wind loads and other horizontal loads are not considered in the HRA as they are predominantly carried by the stabilizing core. Finally two distinctive beam types are considered: a statically determined beam (which is frequently occurring in these types of structures) and a statically undetermined beam within the frame structure of the office building.

The following assumptions are made concerning the design organization:

- The design is executed by a 'standard' engineering company within the Netherlands. The organization of the company is adequate and sufficient support is available.
- The design is performed by a professional engineer, capable of doing the particular design task.
- Detailed design of a single element within the design is performed by a single engineer, coordination and communication between several designers is required to design the whole structure.

8.3 SCENARIO IDENTIFICATION

The HRA tool presented in chapter 7 is mainly set-up as a performance prediction HRA (in contrast to retrospective HRA analysis, see Hollnagel (1998) for more details). Within HRA for performance prediction, selection of the scenario or event sequence for further analysis is required. This typically involves drawing up a complete list of all potential system failures that can reasonably be expected. From this list one particular scenario at a time must be selected as the target for the HRA (Hollnagel, 1998). Tech-

Table 7: Basic design assumptions within the case study

Materials	Concrete	c35	
	Reinforcing steel	FeB 500	
Dimensions	Beam length	7,2	[m]
	Beams	250 x 500	[mm^2]
	Columns	250 x 250	[mm^2]
	Slab length	6,0	[m]
	Stabilizing walls	d = 250	[mm]
	floor slabs	d = 200 [a]	[mm]
	Column length	3,6	[m]
Loads	Dead load floors [b]	3,0	[kN/m^2]
	live load floors [c]	3,1	[kN/m^2] (ψ = 0,5)

[a] 150 mm hollow core slab and 50 mm finishing floor.
[b] Class B: office use.
[c] Hollow core slabs with finishing floor.

niques which can be used for this are fault tree analysis (FTA) or a failure mode and effect analysis (FMEA or FMECA). More details of the usage of these techniques within the proposed HRA method is presented in de Haan (2012).

Within this research, two scenarios are selected for further investigation. Selection of these two scenarios is based on analysis of failure information available within the literature by means of a desk research. This section elaborates on this literature analysis. First an overview of the investigated literature is given. After that the selection of the two scenarios is set-apart.

Eight papers which present quantitative information about the types of failures occurring throughout the world are used in this analysis. These papers are: Boot (2010), ABC-meldpunt (2011), Fruhwald et al. (2007), Matousek & Schneider (1976), Walker (1981), Eldukair & Ayyub (1991), Allen (1979) and Hadipriono (1985). It should be noted that Walker is cited from Fruhwald. This section summarises the result of this desk research. An elaborate description of the desk research methodology and the analysing methodology is given in appendix B.

To select relevant information, three research questions are selected for further research:

- What type of error did lead to the failure?
- How could these errors have occurred?
- Which building elements were involved in the failure?

The first two questions are closely related to the distinction in error classification given by Reason (1990). Reason differentiated three levels: the behavioural level consisting of the easy observable aspects of behaviour (question 1), the contextual level which also includes assumptions about causality (question 2) and the conceptual level which is based on the cognitive mechanisms involved in producing the error (this level is not considered in the available literature).

Type of error

The first research question is: ‘What type of error did lead to the failure? ‘. Within the analysis methodology (see appendix B), a subdivision in main category and sub-category is made. The three most relevant risks on main category level are in decreasing order:

1. Error in design (in general).
2. Error in communicating the design to others.
3. Error in system‘s schematics.

The seven most relevant risks on sub-category level are in decreasing order:

1. Error in mechanical schematization / force balance.
2. Calculation error.
3. Error in structural / mechanical system choice.
4. Error in document coordination among disciplines.
5. Error in drawing (wrong measurements etc.).
6. No calculation update or missing detailed calculation.
7. Error in determining loading scenarios.

From above main- and sub- category analysis it can be concluded that ‘Error in design (in general)‘ is the most error prone activity in the design process. Errors within this main category are typically occurring due to schematization, calculation and coordination activities. Exploring design concepts and understanding the functional requirements (both main categories) is found to be of minor importance.

Causes of error

The second research question is: ‘How could these errors have occurred? ‘. The analysis method of this question is presented in appendix B. Based on this research question, the five most important error causes are in decreasing order:

1. Insufficient knowledge / education / qualification.
2. Ignorance.
3. Communication error.

4. Insufficient task division / overview.
5. Reliance on other parties.

Comparing this short list with similar results in literature seems useful. One such a list is presented in chapter 3 based on findings from Vrouwenvelder (2011), van Herwijnen (2009) and Mans & Derkink (2009). This list is: professional knowledge, complexity of the task, physical and mental conditions, untried new technologies, adaptation of technology to human beings and social factors and organisation. The first category, insufficient knowledge, is directly mentioned. Communication errors and insufficient task division / overview is indirectly mentioned in the category 'completeness or contradiction of information'. The last two categories, ignorance and reliance on other parties, are not directly mentioned but originate from the category 'Social factors and organization'. From this considerations it can be concluded that the short list is not completely corresponding to the literature. However there is some correlation between both lists.

Finally the causes of error can be subdivided in errors on Micro level (knowledge, communication and ignorance) and on Meso level (Insufficient task division and reliance on other parties). This subdivision on levels is acknowledged, but will not be discussed further.

Elements affected by failure

The third research question is: 'Which building elements were involved in the failure? '. The analysis of this question is presented in appendix B. The five elements which were most affected by failure in a decreasing order are:

1. Beams and trusses
2. Slabs and plates
3. Vertical elements
4. Foundations
5. Connections

Failure in vertical elements (Columns and walls) is only causing approximately 12 % of the collapses, while failure in horizontal elements (beams, trusses, slabs and plates) cause approximately 44 % of the collapses (see appendix B). This is quite in line with the remark given by Vrouwenvelder (2011) based on other literature, concerning the triggering event to progressive collapse: "[...] the column failure is only responsible for about 10% of the structural failures [...]". Based on the research within this thesis, another more likely triggering event would be progressive collapse due to impact loading of failing beams/slabs. Within this research only the probability of occurrence of this impact load is considered.

8.3.1 *Conclusions scenario identification*

A last step is to select relevant scenarios within these defined limits. As mentioned before, the information concerning the causes of failure is potentially the most powerful. This level of abstractness does go beyond the project boundary concerning its relevance. This makes the results of the scenario analysis on this level valuable for other design processes then the particular process concerned in the scenario. Furthermore the causes of failure are on a deeper cognitive level, which is beneficial for analyses of the real causes of failure.

Based on this, two 'causes of failure' are selected as scenarios for further research. The selection is based on relevance according to the desk research and the possibilities to model these scenarios with the HRA method. These scenarios are:

1. Level of professional knowledge
2. Level of design control

Professional knowledge

Professional knowledge is a property of the designer, based on skills, experience, nature and abilities. Within this research professional knowledge is limited to engineering knowledge, leaving out other types of knowledge such as communication- and social knowledge.

Professional knowledge is primarily based on the Micro-level or individual level, but is strongly influenced by aspects on the Meso-level such as organizational knowledge management. Another important aspect of the required knowledge within a design process is the cognitive demand level.

The link between professional knowledge on one hand and cognitive level on the other hand is not straight forward, and needs some clarification. Reason elaborates extensively on this subject in his book on Human Error (Reason, 1990). Reason distinguishes three levels on which a human mind is performing: skill-based, rule-based and knowledge-based level. Concerning the effect of expert knowledge on failure at the Knowledge-based level, Reason remarks (Reason, 1990, page 58):

> Experts, then, have a much larger collection of problem-solving rules than novices. They are also formulated at a more abstract level of representation. Taken to an unlikely extreme, this indicates that expertise means never having to resort to the Knowledge Based mode of problem solving. [...] the more skilled an individual is in carrying out a particular task, the more likely it is that his or her errors will take 'strong-but-wrong' forms at the Skill-based and Rule-based levels of performance.

Above citation does not directly provide information on the link between professional knowledge and the probability of failure. However knowledge influences the level on which a person executes a task. An expert will execute most engineering tasks on a rule-based level, while a young engineer entangles the engineering question for the first time, inevitable leading to a knowledge-based type of action. Knowledge-based actions have another cognitive demand profile in comparison to Rule-Based actions, leading to different probabilities of failure.

Internal control

One of the measures to prevent failures within a engineering firm is control. Due to the generally accepted assumption that humans are the 'weakest link' in the design of an engineered structure, engineers place a very high importance on error control measures (Stewart, 1993).

Within a design process several types of control can be distinguished. The level and profundity of control is process dependent, and differs widely. In an attempt to simplify this, three levels of control will be distinguished in accordance with the design supervision levels given in Annex B of NEN-EN-1990 (2002):

Self checking
Checking performed by the person who has prepared the design.

Normal supervision
Checking by different persons than those originally responsible and in accordance with the procedure of the organization.

Third party checking
Checking performed by an organization different from that which has prepared the design.

As this thesis focusses on human error within the organizational boundaries, no further attention will be paid to third party checking.

8.4 DESIGN PROCESS

The proposed HRA model is based on failure probabilities within each design task. As such an overview of all tasks within the design process is required. The considered design process is the detailed design of a beam element. For this the design is subdivided in a global design phase and a detailed design phase. The global design phase provides information for the detailed design phase by means of communication, while the detailed design phase is the process which is considered in the case study. An flowchart of all design steps is shown in appendix C. This flow chart consists of design tasks and parameters which are obtained from the considered design tasks. This overview will be used in chapter 9 to quantify human error on a task level.

Based on the above mentioned assumptions it can be noted that the overall design process is not considered into depth. This is an boundary condition applied to the research in order to fit the research within its time limits.

An interesting topic for further research is to model the decision process leading to an overall design.

9 HUMAN ERROR QUANTIFICATION

INTRODUCTION

The second step within the HRA method is to quantify the error probability within typical engineering tasks. This error probability is represented by a probability distribution function described by two parameters. These parameters are a Human Error Probability (HEP) and an Error Magnitude (EM). The procedure for determining HEPs consists of two methods: a basic HEP method and an extended HEP method. (see figure 21). The extended method is basically the extended quantification method defined by Hollnagel (1998, chapter 9). This method is however labour intensive and requires quite some knowledge concerning human factors/psychology. In order to make the HEP quantification accessible for use by engineers, a simplified method is proposed for standard engineering tasks. This simplified method is basically a predefined extended method, based on standard engineering tasks.

The second parameter, the Error Magnitude, consist of a method based on information from literature (Melchers, 1984, 1989; Stewart, 1992b). This methodology consists of three steps, which are quite easy to perform. The HEPs are determined on the basic task level, while the EM are determined on the micro-task level. This is due to the fact that a micro-task is defined at the parameter level, while each micro-task consists of multiple basic tasks. As EMs can only be given on a parameter level, and HEPs are defined on the basic task level, the distinction in both levels is applied (shown in figure 21).

This chapter consists of five sections. Section 9.1 discusses the extended HEP method. Section 9.2 discusses the simplified HEP method, and section 9.3 discusses the EM method. The results of the human error quantification process are discussed in section 9.4. Finally section 9.5 discusses the link between the HEPs and EMs and the overall design process.

9.1 EXTENDED HEP METHOD

The extended HEP method is meant to determine the Human Error Probability (HEP) of each basic design task. This requires the completion of four distinctive steps. The basic idea behind these steps is to subdivide each basic task into basic cognitive tasks, on which standard failure probabilities are defined. The input of this procedure consists of the identified design steps, the context of the design and classification diagrams from the CREAM methodology (see appendix D). The four steps of the method are given beneath.

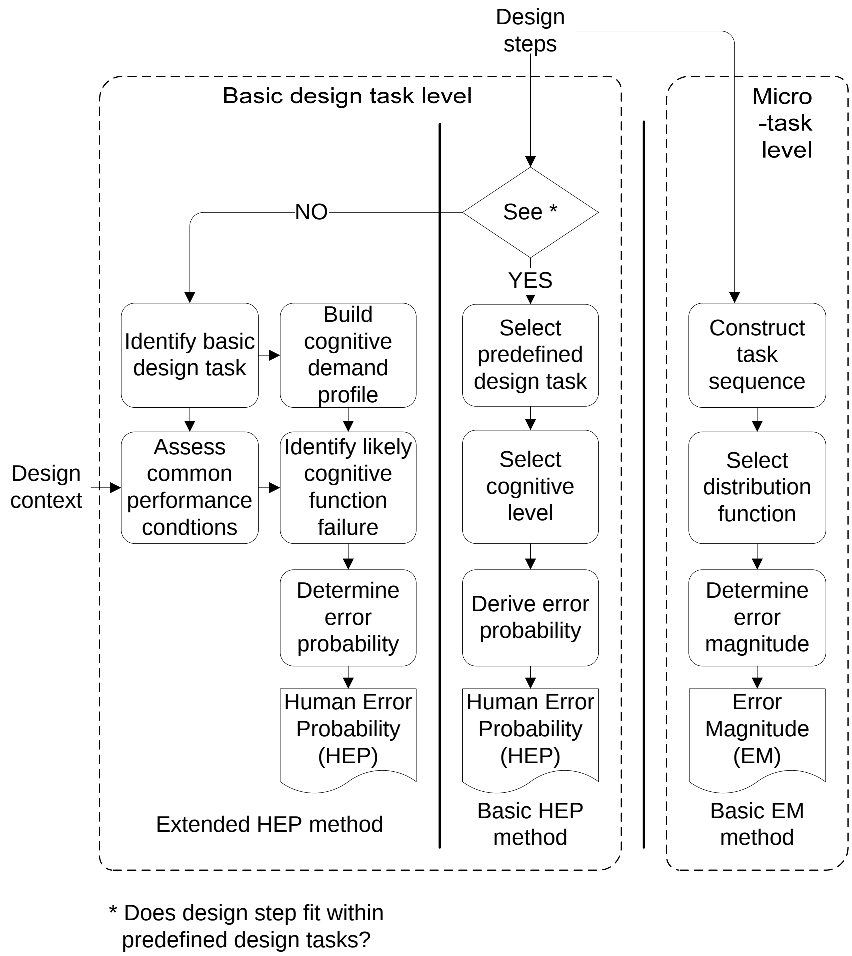


Figure 21: Basic model of human error quantification within the HRA model

Build a cognitive demand profile.
In this step the description of the basic task is refined by identifying a list of cognitive activities that characterise each basic task. The cognitive activities are then used to build a cognitive demand profile of the basic task, based on the functions described by the CREAM model.

Identify likely cognitive function failure.
In this step the cognitive function failures that could occur, are determined. This action transforms the cognitive demand profile into a cognitive failure function, and subsequently generates a value for each cognitive function failure.

Assess Common Performance Conditions (CPC).
In this step the conditions under which the performance is expected to take place is characterised. This characterisation is expressed by means of a combined CPC score, resulting in a weighting factor in the HEP calculation.

Determine error probability.
In this step the basic failure probabilities of each cognitive activity is multiplied with the corresponding weighting factor. The overall failure probabil-

ity of the basic task is subsequently calculated by adding these basic failure probabilities based on full independence between the cognitive activities $(1 - \prod_{i=1}^{n}(1 - P_i))$.

More details on the Extended HEP quantification method, and a small case study concerning its use by engineers, can be found in de Haan (2012).

9.2 Simplified HEP method

Within the structural design process typical design tasks are frequently returning. Examples are consulting norm requirements and calculating a formula. These typical design tasks are not readily covered by a single cognitive activity within the extended HEP method, hence subdivision of these tasks is required to match these design tasks within the extended HEP method. Subdivision of frequently occurring design tasks on an independent basis is not very efficient, and requires quite some human factor/psychological judgement which is not always available to the HEP assessor.

For this reason a simplified HEP method is proposed consisting of a matrix based on the type of design activity and the required cognitive level. seven basic design tasks are identified, of which the HEPs are calculated. These seven basic design tasks are typically more thorough then the cognitive activities of the extended HEP method. For example "communication" is mentioned within both methods. However within the extended HEP methodology it involves passing on or receiving person-to-person information, while within simplified method "communication" is thought of as a thorough discussion on design aspects.

These seven basic design tasks serve as a basis for all HEPs within the considered design process. Selection of these seven basic tasks is based upon an assessment of al task types within typical design processes. It should be noted that this list is not intended as a complete list and extra addition may be required in the light of other design processes. For instance if the construction process is of interest as well, a logical addition would be to add "instruct" as a basic task. However the operational types of cognitive tasks mentioned in the CREAM methodology are quite suitable for direct usage within the construction process. For example "execute" and "monitor" are typical construction tasks. The seven basic tasks and their definitions are given beneath.

Consult
Reading and interpreting guidelines or norm requirements. "Consult" typically is more advanced then "obtain".

Obtain
Adopting a design parameter from a resource such as a drawing. Typically for tasks in which thorough interpretation of the resource is not required.

Derive
Selecting a value from a range of values based on previous determined se-

lection criteria.

Determine
Taken a decision based on engineering judgement and available design parameters.

Calculate
Calculating a parameter based on available design values. This task typically involves inserting values in a calculation program, calculator or hand calculation, and retrieving the outcome.

Insert
Placing a calculated/derived parameter in a design program / design document. "Insert" is opposite to "obtain".

Communicate
Thorough discussion on basic design parameters, the design or other aspects. This task typically involves passing on or receiving person-to-person information, interpretation of the information and reasoning about the implications of the information.

Another subdivision is made on the level of complexity which can be distinguished within each basic task. This subdivision is made in order to tackle the problem of task difficulty within the seven basic tasks. For this, three different levels of cognitive operations are distinguished: a skill-based, a rule-based and a knowledge-based level. This division is in line with the cognitive stages presented by Reason (1990, chapter 3). Each level requires another set of cognitive activities resulting in another HEP value. It should be remarked that not all basic tasks are acting on all three cognitive levels, as the knowledge based level is deemed unrealistic within obtain- and insert activities. The definition of the three cognitive levels are given beneath.

Skill-based level
Comprising of highly routinised activities in familiar circumstance. Errors are typically occurring when persons actions are different to their intentions. They often occur during automatic behaviour which require little concious thought, or when attention is being diverted.

Rule-based level
Comprising of problem solving activities by means of previous established if-then-rules. Errors occur when a known rule is incorrectly applied, or a situation is misinterpreted.

Knowledge-based level
Comprising of problem solving activities based on a higher level analogy. Errors results from a deficit of knowledge. A person may intend to implement a plan or action, but the plan or action is incomplete or flawed by a lack of knowledge and does not result in the desired outcome.

The simplified HEP method based on the seven basic tasks and three cognitive levels is shown in figure 22. In total 19 distinctive HEPs are distinguished within the simplified HEP. Derivation of these numbers is presented in appendix E. From figure 22 it can be concluded that skill-based activities have generally a HEP-value of $1,25 \cdot 10^{-5}$ to $2,25 \cdot 10^{-3}$, rule-based activities have HEPs from $7,75 \cdot 10^{-4}$ to $1,25 \cdot 10^{-2}$ and the HEPs for knowledge-based activities vary from $1,1 \cdot 10^{-2}$ to $3,0 \cdot 10^{-2}$. These values seems quite logical, however they are slightly lower then the values given by Stewart (1993). Which also presents HEPs for calculation and derive activities (the results of Stewart are compared with the knowledge-based level).

Basic task	Skill-based	Rule-based	Knowledge-based
Consult	2,25E-03	1,25E-02	2,24E-02
Obtain	1,28E-05	2,50E-03	
Derive	5,13E-04	7,63E-04	2,06E-02
Determine	5,13E-04	1,03E-02	3,00E-02
Calculate	2,56E-05	7,75E-04	2,02E-02
Insert	1,28E-05	2,50E-03	
Communicate	7,68E-04	1,02E-03	1,10E-02

Figure 22: Human Error Probabilities of the simplified HEP method

9.3 Error magnitude method

Determining the Error Magnitude (EM) of a error probability requires the completion of three distinctive steps. The EMs are determined on the Micro-task level and are based on the task characteristics. The three steps are given beneath. The EM is basically a distribution function in which the standard deviation represents the deviation from the design value. Furthermore the mean value equals the error free design value.

construct task sequence
A task sequence is defined on the micro-task level consisting of several basic tasks. Each micro-task represents a sequence of basic tasks required to deduce a design parameter.

Select distribution function
The characteristics of the task are assessed in order to link a distribution function to the micro-task. three distribution functions are distinguished: Log-Normal functions for calculation tasks, normal functions for the remaining six basic tasks and a discrete function for special situations.

Determine error magnitude.
In this step the standard deviation of the distribution function is determined. This is based on two characteristics of the task: task complexity and task overview. This results in an Error Magnitude (EM) for the micro-task of concern.

Determining the error magnitude is based on selecting a standard deviation from table 8. The HRA assessor couples a complexity level (given in row one) to a task sequence. If the task sequence lacks a clear overview, the task complexity should be increased with one level. In case of a controllable situation, the task complexity should be decreased with one level. This results in a standard deviation for the selected distribution function.

Table 8: Subdivision Error Magnitudes

Task complexity	Normal distribution	Log-normal distribution
very complex	1,4826	1,0277
complex	0,9648	0,6688
neutral	0,7803	0,5409
simple	0,6080	0,4219
very simple	0,4299	0,2980

Furthermore, calculation EMs consists of a combined distribution function: a Log-normal function and a discrete function. This is based on the findings of Melchers (1984). The first distribution consists of computational errors, with the following properties:

- $\frac{2}{3}$ of the errors are caused by computational errors.
- Assumed is that negative and positive errors are equally occurring. This entails that the median of the distribution curve should be equal to 1, which results in a mean value of 0 ($e^{\mu} = median$).

The second distribution consist of decimal errors with the following properties:

- $\frac{1}{3}$ of the errors are caused by decimal mistakes.
- The order of magnitude of decimal errors are $10^{(-)1}$, $10^{(-)2}$ and $10^{(-)3}$, comprising $\frac{1}{3}$, $\frac{1}{18}$ and $\frac{1}{9}$ of the errors respectively.

Most of the other EM are based on logical values. Some 'determine' activities are based on choices between two calculated parameters. An example is to determine if the calculated reinforcement is lower than the maximum allowable reinforcement. In these cases the EM consisted of choosing the wrong value. Furthermore, some of the EM are for $\frac{1}{2}$ of the errors based on logical values, in order to approach a realistic EM (see Appendix F for details).

9.4 RESULTS HUMAN ERROR QUANTIFICATION

In total 111 basic design tasks are modelled for designing the reinforced concrete beam. The design tasks are presented in appendix F. Table 9 presents an overview of the division of these activities between the seven basic tasks specified within the simplified HEP method. The 'calculate' task is by far the most occurring design activity, followed on a distance by the 'determine', 'derive' and 'consult' design activities. The last three tasks: 'Obtain', 'insert' and 'communicate' are occurring to a lesser extend within the design tasks. These results do comply to the expectations as one specific design activity is considered: the design of a simple beam element. On this detailed design level few communication is required. Furthermore most tasks consists of consulting the norm requirements, and applying them by means a calculation. If more parts of the overall design, or a more diverse design activity was selected, a more equivalent distribution among the basic tasks is expected.

Table 9: Occurrence of the basic tasks within the case study

Basic task	No. of design activities
consult	30
obtain	14
derive	21
determine	25
calculate	46
insert	7
communicate	12

Within the type of designer a difference is made between experienced and inexperienced designers. Based on this division, the 111 basic tasks are coupled to the three cognitive levels. An overview of the division of design activities as a function of cognitive levels and professional experience is shown in table 10. It can be seen from this figure that an experienced engineer executes the engineering task on a lower cognitive level then an inexperienced engineer. A remarkable thing is that an experienced designer is almost not acting on the knowledge-based level, while a inexperienced designer does act on a skill-based level. This occurrence is a consequence of the fairly easy design task within the case study. With a complicated design tasks, these figures will shift towards a higher cognitive level.

Table 10: Applied cognitive level in the case study as a function of experience

Cognitive level	Experienced designer	Inexperienced designer
Skill-based	62	34
Rule-based	82	93
Knowledge-based	3	20

9.5 LINKAGE WITH DESIGN SIMULATION

Within this chapter the Human Error Probabilities (HEP) and Error magnitudes (EM) are introduced. These parameters are linked to the design process within the case study on the task level of the process. This is presented in appendix F. The first parameter, HEP, is coupled to the basic task level. The design activities consists of 111 tasks obtained from the design process described in appendix C. Each design activity is coupled to one of the seven basic tasks defined with the simplified HEP method. Furthermore a cognitive level is assigned based on the experience of the designer.

The second parameter, EM, is coupled to the micro-task level. This is due to the fact that EMs must be coupled to a parameter in order to be of use. Each micro-task consists of several basic tasks which are required to obtain the parameter. Each EM has a mean value which equals the design value. Furthermore one or two distribution functions are assigned to the EM based on the method described in section 9.3. Based on the results given in appendix F, a design simulation can be performed, which is presented in chapter 10.

10 DESIGN SIMULATION

INTRODUCTION

The next step in the analysis is to combine the individual distribution functions of the micro-tasks into an overall distribution function on the element level of a structure. Hollnagel (1998) advises to use the structure of an underlying fault tree to calculate the overall Failure Probability. However within this research the underlying system is not a fault tree but a design sequence existing of tasks which are linked together through calculation sequences. This problem is tackled by using a Monte-Carlo simulation process to determine the overall failure probability of a design process. For this a micro-task simulation structure is used, which will be explained in detail beneath.

An important notion of human behaviour which is not addressed in chapter 9 is the level of control a person has over the tasks he or she performs. This notion will be addressed in this chapter as it is incorporated in the Monte-Carlo simulation process. Hollnagel (1993) describes control as the person's level of control over the situation. The level of control is influenced by the context as it is experienced by the person, by knowledge or experience of dependencies between actions and by expectations about how the situation is going to develop (Hollnagel, 1998). Within engineering type of tasks, the effect of control is considerable as calculations and decisions are regularly checked on correctness and applicability. Concerning control in design, the Eurocode has developed an informative annex (NEN-EN-1990, 2002) on the management of structural reliability for construction works. Within this annex three possible design supervision levels are distinguished (presented in chapter 8). These levels will be used within the simulation procedure presented in this chapter.

Within this chapter, section 10.1 discusses the simulation procedure. Section 10.2 discusses the results of the simulation procedure and section 10.3 finally goes into the link with the probabilistic analysis.

10.1 SIMULATION PROCEDURE

If the simulation procedure is simplified to its very basics, four distinctive elements are remaining which are depicted in figure 23. The first element is a list of micro-tasks which represent the activities within the design process. The second element is the internal control by the designer, which is termed in line with Annex B of NEN-EN-1990 (2002) as Self-checking. The combination of both these elements is termed the overall task (element 3). The final element is the internal control by the supervisor, which is termed normal supervision in line with Annex B of NEN-EN-1990 (2002). These elements will be set-apart in this section in more detail. The code script for modelling this simulation procedure is presented in appendix H.

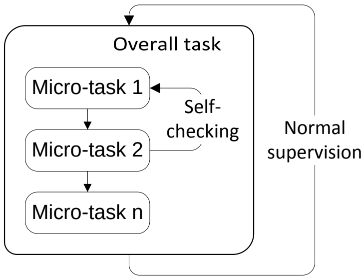


Figure 23: Basic simulation procedure of the Monte-Carlo simulation

Micro-tasks
The procedure for simulating each micro-task is based on the task-cycle approach presented in Stewart (1993). The micro-task procedure is given in figure 24 for the typical micro-task "calculate reinforcement". The procedure starts with input parameters which can be the output of a preceding micro-task or an input parameter from outside the considered design process. The next step is to generate a Random Number (RN) between 0 and 1, and to obtain a Human Error Probability (HEP) for the micro-task at hand (from the list given in appendix F). If the Random Number (RN) is smaller then the HEP-value, a failure occurs and subsequently the output of the micro-task is multiplied with a Error Magnitude (EM). If the Random Number is equal or larger then the Failure Probability no error occurs and subsequently the output of the micro-task is not multiplied with an Error Magnitude.

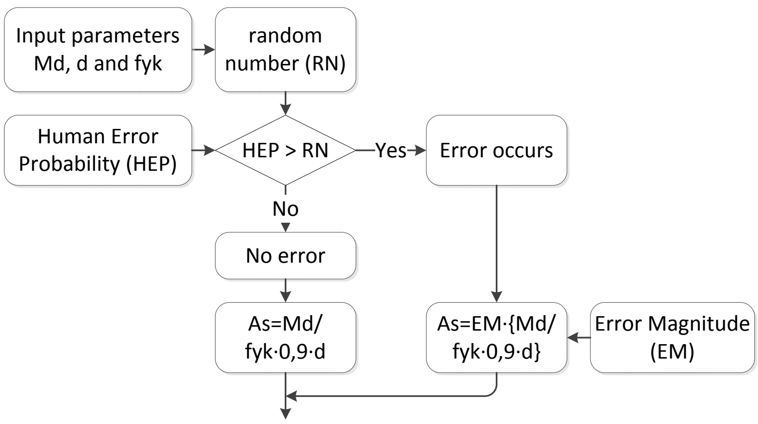


Figure 24: Basic procedure of a micro-task

Self-checking
The lowest level of control described in annex B of NEN-EN-1990 (2002) is self checking: "Checking performed by the person who has prepared the design." The level of control within a design task is particularly dependent of the knowledge of the designer and his ability to appraise the results of a micro-task. This entails that the level of control of a designer over each micro-task depends on the task within the micro-task and the knowledge

level of the designer of this particular task.

Within the Monte-Carlo simulation, self-control is based on the notion that a designer uses his previous experience as a reference for assessing the correctness of the results. The adopted process is shown in figure 25. Within this process, the output of a series of micro-tasks is compared with the correct output of the series of micro-tasks. If the output is within predefined bounds, the output is deemed correct and the design process continues. If the output is not within these predefined bounds, reconsidering of the series of micro-tasks is performed. If the output is not within the predefined bounds after one reconsiderations, the design process is continued with the incorrect output. This process is very basic but encompasses some very basic aspects of self-checking: comparison of the results with an output which the designer deems realistic and reconsidering for a finite number of times if the designer suspicions in-correctness. The predefined bounds are presented in appendix H. The limits are different for experienced and inexperienced designers.

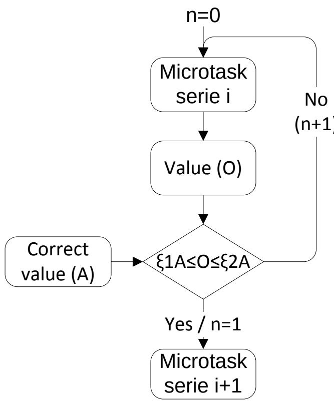


Figure 25: Procedure of self checking

Overall process
The overall process consists of all micro-tasks and all self-checking loops performed by the designer. Besides basic micro-tasks, the process consists of two control loops. Figure 26 presents the steps within the overall process. The micro-tasks are bundled in segments on which a self-checking loop is envisioned. For instance "Calculate beam dimensions" consists of six micro-tasks, after which a control loop is performed on the beam height and beam width. It should be noted that column design is not considered in the case of a statically determined beam, but only in the statically undetermined beam case.

Normal supervision
The final element of the procedure is the internal control by the supervisor, which is termed normal supervision. Within this process parts of the process are recalculated by the supervisor on a basic manner, subsequently the results are compared with the results obtained by the designer. If these results differ considerable from the results obtained by the designer, the complete process is re-evaluated. This process has much in common with

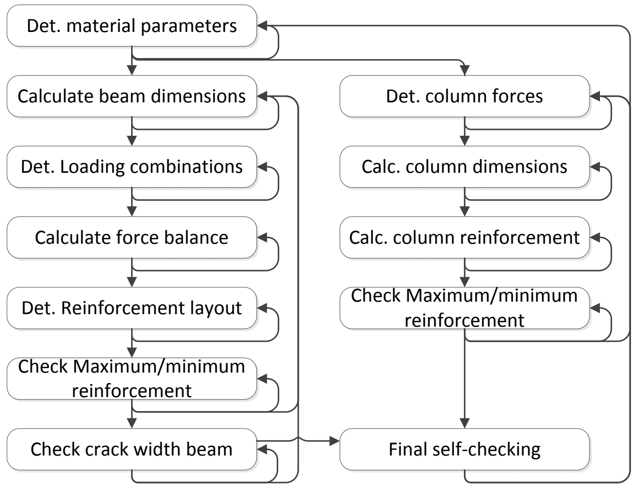


Figure 26: Overview of the steps within the overall process

control based on an independent concurrent design, with the difference that the process is simplified and that the same basic assumptions are used within the design process and the normal supervision.

10.2 RESULTS DESIGN SIMULATION

In the previous section, the Monte Carlo procedure is set-apart in detail. This section will clarify this procedure by presenting some intermediate results of the process. The results of a case executed by an inexperienced designer are used in this section.

The first result obtained from the analysis is the result of a single micro-task, which is presented in figure 27. This figure presents the scatter in the beam length (L_y) by means of a histogram. This result is only depending of the single micro-task "determine beam length" as the input parameters consists of external information. It can clearly be seen that the result of this operation equals the correct results in most of the cases. Furthermore, the error rate [1] equals 0,0015, which equals the HEP value of the considered micro-task. In fact this histogram shows the probability distribution of a single micro-task.

The second result is the output of a series of micro-tasks. Depending on the number of micro-tasks required to obtain a certain parameter, the error probability will increase, as presented in figure 28. This histogram presents the outcome of the micro-task to calculate the distributed load on the beam. The input from this micro-task is depending on 13 other micro-tasks, which is causing the scatter in the histogram. The error rate within the distributed load parameter is 0,38. It should be noted that most of the errors lie within an acceptable margin from the correct value. This is a consequence of the

1 defined as the fraction of cases in which the design parameter deviates from the correct value

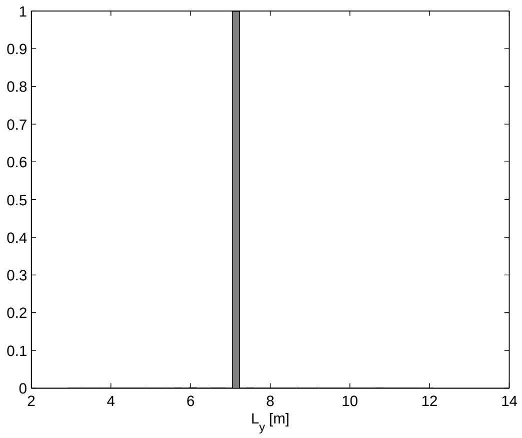


Figure 27: Histogram of the Beam length (Ly)

Error Magnitude definition given in section 9.3.

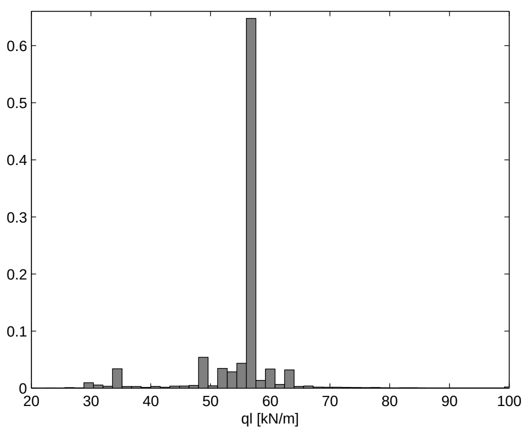


Figure 28: Histogram of the distributed load (ql)

The designer checks regularly the correctness of a design parameter based on experience or logical assumptions. This process is termed self-checking. The effect of self-checking is presented in figure 29 which shows the distributed loads on the beam before and after self-checking. It can be seen from this figure that the error rate is reduced from 0,38 to 0,30. Also the scatter within the error is reduced somewhat. From this it can be concluded that self-checking is an important aspect of human error prevention.

Within the process, three checking mechanisms are incorporated: minimum reinforcement control, maximum reinforcement control and crack width control. If the design exceeds the calculated control values an action sequence is initiated in order to satisfy the control value. In case of the minimum reinforcement check, the minimum reinforcement is adopted. Within the maximum reinforcement check, the design is recalculated one iteration and within the subsequent iterations the beam height is increased with 50 mm until the maximum reinforcement check is satisfied or the beam height

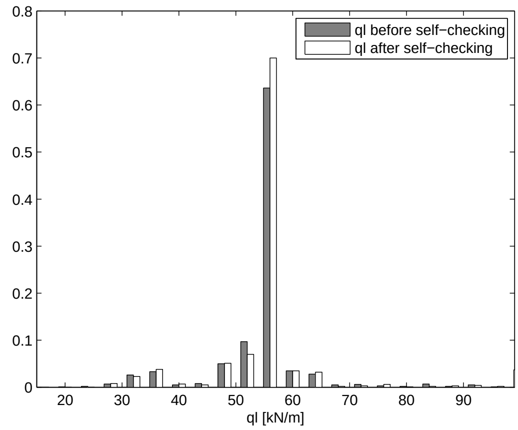


Figure 29: Histogram of the distributed load (ql) before and after self-checking

equals 800 mm. Within the crack with control the complete design is reconsidered one iteration. Within the subsequent iterations the reinforcement area is increased until the crack with is satisfied or the maximum of three iterations is reached.

As an example of the usability of the control loops, the effect of checking the maximum allowable reinforcement is shown in an histogram in figure 30. It can be concluded that this control loop reduces the undesired beam heights (300 to 400 mm) with about 75 %. However this goes at a certain cost as the overall beam height is increased with 5 mm and the error rate is increased with 0,007 due to an error in the control loop.

Overall the effect of minimum/maximum reinforcement checking does not have a major influence on error control. The reason for this is probably twofold:

- The minimum and maximum reinforcement values differentiate considerable from the correct reinforcement value. This makes is less suitable for error detection.
- The reinforcement checks are both based on parameters within the design process. An error in these parameters results in a comparable error in the minimum/maximum reinforcement value, hence the error will not be detected based on these values.

The last step performed by the designer is checking the complete design on correctness. This is performed by recalculating a couple of micro-tasks again and compare the answer with the previous found answer. If both answers deviate considerable, redesign is performed. This process results in an outcome for the top reinforcement as presented with a histogram plot in figure 31. The figure reveals that the negative errors are reduced slightly and that the error rate is reduced from 0,30 to 0,28.

After the design is finished by the designer an independent engineer checks the results. This is performed by reconsidering small parts of the design again and subsequently compare the results with the results of the

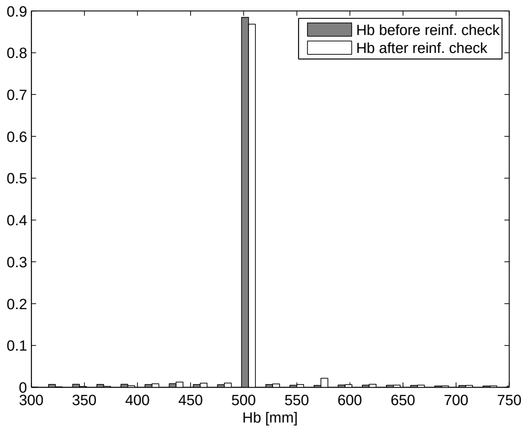


Figure 30: Histogram of the beam height (Hb) before and after maximum reinforcement control

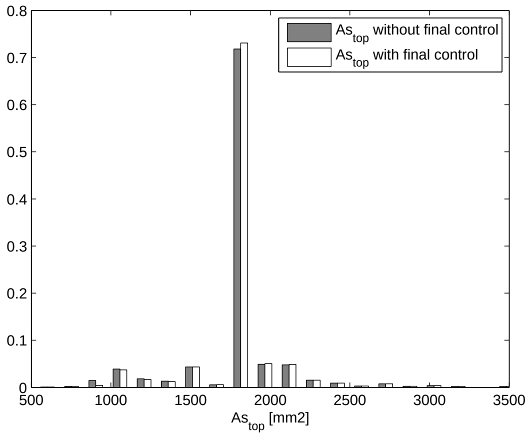


Figure 31: Histogram of top reinforcement (As_{top}) before and after final design check

designer. The effect of this process is shown in figure 32. The figure shows that the negative errors leading to a top reinforcement lower then 1000 mm are almost completely disappeared. Furthermore the error rate is reduced from 0,28 to 0,22.

10.3 linkage with probabilistic analysis

Within the design simulation the overall failure probability of several design parameters is determined. Within the probabilistic analysis, these design parameters will be used as input for the reliability analysis. The calculated design parameters can roughly be divided in: loading parameters, material parameters and geometric parameters. Only the geometric parameters and material characteristics are of importance for the probabilistic analysis. Loading parameters are separately determined in the probabilistic analysis as the real-time loading conditions are not depending of the loading condi-

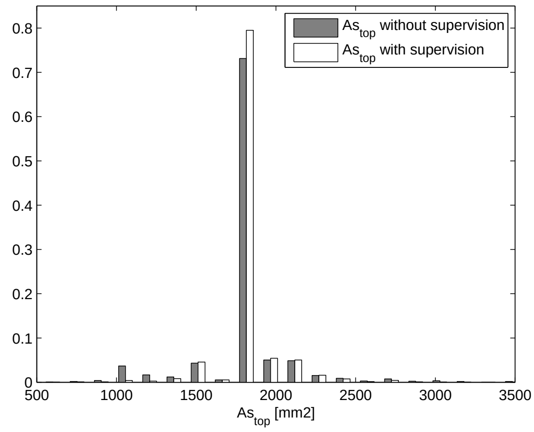


Figure 32: Histogram of top reinforcement (As_{top}) before and after superior control

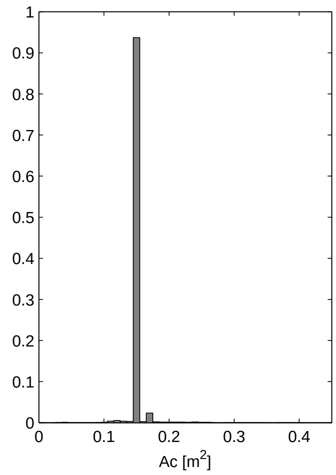


Figure 33: Histogram of the concrete area (Ac)

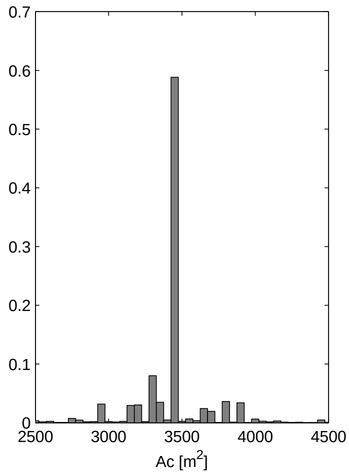


Figure 34: Histogram of the reinforcement area (As)

tions used in the design.

The overall error probability of the design parameters is a distribution function. As an example two distribution functions of the concrete area and reinforcement area are shown in the figures 33 and 34 respectively. It is noticed that the human error is defined as the deviation from intend. This entails that the human error did lead to a deviation in the design, but it is not sure if this error will lead to an undesired situation: structural failure. For instance it can be seen from the figures 33 and 34 that also positive errors are occurring and most errors are within a certain bandwidth. Another interesting thing to notice is the difference in the error rate of the concrete area and the reinforcement area, caused by the effect of required number of micro-tasks.

11 PROBABILISTIC ANALYSIS

INTRODUCTION

The last step within the HRA method is to determine the probability of failure of the engineered structure. These probabilities are determined with basic probability analysis for the reliability on element level. The probabilistic analysis is based on plastic limit state analysis. Within these probabilistic analysis two beam types are considered: a statical determined beam and a statical undetermined beam within a frame element. Both beams are envisioned as part of the overall design presented in chapter 8. Section 11.1 sets apart the derivation of the reliability function. Section 11.2 elaborates on the probabilistic procedure to incorporate the loading conditions and resistance properties. Section 11.3 finally discusses the results of the probabilistic analysis.

11.1 RELIABILITY FUNCTION

A limit state is defined in CUR (1997) as the state just before failure occurs. Furthermore, the reliability is defined as the probability that this limit state is not exceeded (CUR, 1997). The general form of a reliability function is:

$$Z = R - S \tag{12}$$

In which:

- R is the resistance to failure of the structure;
- S is the load acting on the structure.

This section elaborates on the reliability function of beam elements. This function is based on plastic limit state analysis, which is a method based on physical non-linear and geometrical linear behaviour of the structure. Within this method failure is defined as loss of static equilibrium of the structure or any part of it (Vrouwenvelder, 2003).

The plastic limit state analysis consists of two consecutive parts: a upper bound analysis and a lower bound analysis. The reliability function is determined with the upper bound analysis. This analysis is easily to perform, however can be unsafe as it is not sure if the dominating failure mechanism is found. In order to investigate this, a lower bound analysis is performed. The advantage of a lower bound solution is that it is always at the safe side. If the lower bound coincides with the upper bound the correct failure mechanism is found, and the correct reliability function is defined.

Some simplifications were required to analyse the two beam elements with the upper bound and lower bound analysis. The beam elements are modelled as a simple beam carrying a slab floor within an office building.

The effect of tying reinforcement in the supports and positive effects of the slab floors are neglected. Furthermore errors within the shear reinforcement are not taken into account in the calculations.

Within the reliability calculation two construction types are considered. The first structural type is a prefab single beam element, which is modelled as a statically determined beam. The second analysis is a statically undetermined beam element. This element is envisioned as a beam within a frame structure supported by the concrete core in the office building. A schematic representation of this is given in figure 35. Both beam layouts are envisioned as an element of the office building presented in chapter 8.

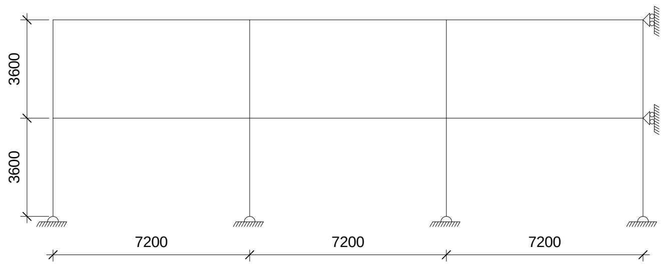


Figure 35: Basic layout of the frame structure

Upper bound calculation
The general formulation of the upper bound theorem is given by Vrouwenvelder (2003):

> Starting from an arbitrary mechanism, the corresponding equilibrium equation will provide an upper-bound solution for the limit load

The corresponding equilibrium equation is defined as a function of the plastic capacity of the beam/column cross section and the forces acting on the cross section:

$$\sum_{k=1}^{m} M_{pk}\vartheta_k = \lambda \sum_{i=1}^{q} F_i u_i \tag{13}$$

The upper bound calculations are based on the mechanisms defined in appendix G. This analysis is based on 1-D beam elements loaded by distributed loads.

For the hinges within the beam elements, no additional Normal force is considered as these forces are relatively small. However for the hinges within the column elements, the Normal force is considered. As a consequence, the cross-sectional plastic capacity becomes a function of the Moment (Mp) and the Normal force (Np) acting on the cross section. This is shown in figure 36. The effect of Normal force on the plastic capacity can be positive and negative. In the case within the statically undetermined beam, a positive effect is found.

In order to form a mechanism within the statically determined beam, one plastic hinge is required. The formula for the upper bound mechanism of

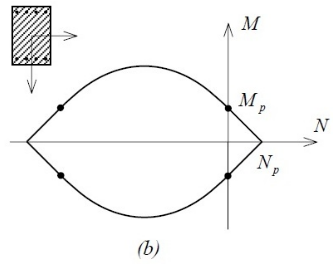


Figure 36: Yield contour as a function of moment and normal force in a symmetrical reinforced concrete cross section (Vrouwenvelder, 2003, blz. 83).

the statically determined beam is given in equation 14. In this formula M_{Pf} is the plastic capacity in the middle of the beam, q the loads acting on the beam and L_b is the length of the beam. The derivation of this formula is given in appendix G.

$$M_{Pf} \leqslant \frac{1}{8} q L_b^2 \tag{14}$$

The reliability function for the statical undetermined beam is not that straightforward, as 19 hinges are required to form a mechanism. This seems rather unrealistic and as a consequence failure by partial mechanisms is governing. These partial mechanisms are in line with the progressive collapse theorem: partial collapse of a beam, column or parts of the structure. This thesis focusses on partial collapse of a beam resulting in two realistic mechanisms, which are given in appendix G. Beams excluded in this analysis are the roof top beams. An extra mechanism which can occur besides the plastic collapse mechanism is local buckling of the columns. This collapse mechanism is not taken into account as the upper bound analysis is based on geometrical linear behaviour, while buckling is a geometrical nonlinear behaviour. The calculations resulted in the following formulas for the upper bound mechanism of the statical undetermined beam element:

$$M_{Ps} + M_{Pf} \leqslant \frac{1}{8} q L_b^2 \tag{15}$$

$$M_{Ps} + 2 \cdot M_{Pf} + 2 \cdot M_{Pc} \leqslant \frac{1}{4} q L_b^2 \tag{16}$$

Within this formula M_{Pf}, M_{Ps} and M_{Pc} is defined as the plastic moment capacity in the cross section of the middle of the beam, the support of the beam and the column cross section respectively. The maximum plastic moment capacity is calculated through basic equilibrium equations of a reinforced concrete cross section. As no strain is considered in the plastic analysis, the stress-strain relations given in figure 37 are used. The plastic capacity of the beam elements and the columns is given in appendix G.

The load combination on the beam element consists of self-weight loads of the slab and the beam and live loads on the slab. Further details of the

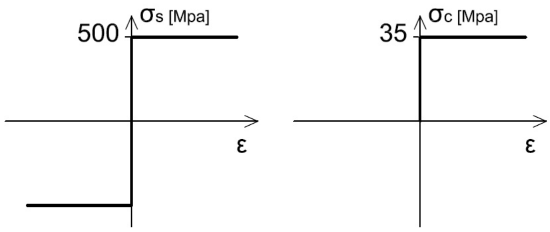


Figure 37: Stress-strain curve of reinforcement steel (left) and concrete (right) within the upper bound analysis

order of magnitude of the load factors are given in section 11.2. For the reliability function it suffices to provide the formula for q_b:

$$q_b = \rho_b H_b B_b + \rho_s H_s L_s + (q_{long} + q_{short}) L_s \tag{17}$$

Combining the formulas of the upper bound mechanisms, cross sectional capacity and loading conditions results in a formulation of the reliability functions. The exact form of these reliability functions are presented in appendix G.

Lower bound calculation
The lower bound theorem is formulated as follows (Vrouwenvelder, 2003):

> Each arbitrary moment distribution, that is in equilibrium with the external load and for which nowhere the yield condition is violated, delivers a lower bound for the limit load.

The lower bound is calculated with the engineering program SCIA Engineer. The structure is modelled with 1-D elements. Some of the outcomes of the lower bound analysis are presented in appendix F. Beneath relevant technical properties and the results are discussed.

Table 11 list relevant properties of the calculation. The calculation is executed with geometrical and physical non-linearity. The geometrical non-linearity is modelled by geometrical deviations in the mesh points of the 1-D elements. The physical non-linearity is modelled by iterating towards equilibrium over the composite cross section. For this SCIA engineer uses the following procedure: the member is discretized in a number of sections. During the calculation process stiffness is modified for the sections where cracking takes place (SCIA, 2012). This is done by the same equations as for the upper bound analysis. As a result only σ_{xx} and ε_{xx} can be evaluated.

In order to approximate the results of the upper bound analysis, the material properties within the lower bound calculation are kept almost similar. For this reason steel tension stiffening and concrete tensional forces are not considered. The stress/strain curves of these material properties are shown in figure 38. Another assumption is that failure occurs if the ultimate stress is reached within the cross section, and not the ultimate strain.

The failure curves found with the upper bound analysis are checked on a number of discrete point on the failure curve. These discrete points and

Table 11: Numeric properties SCIA calculation

Elements	1-D elements
Mesh length	50 mm
Solver type	Direct solver
Numeric solver	Modified Newton-Raphson
No. geometrical increments	5
cross-sectional iterations	50

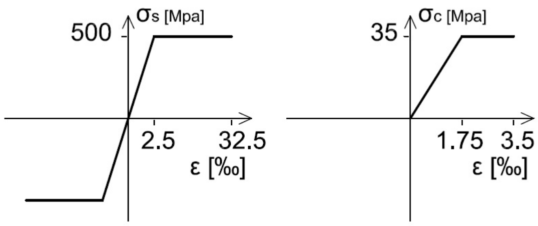


Figure 38: Stress-strain curve of reinforcement steel (left) and concrete (right) within the lower bound analysis

there outcomes are given in table 12. The results for the statically determined beam are reasonable coinciding, as expected. For the statically undetermined beam this is somewhat different. In cases with a normal reinforcement layout and a design with a low amount of top reinforcement / high amount of bottom reinforcement, the values for the upper and lower bound reasonable coincide. In case of a high amount of top reinforcement / low amount of bottom reinforcement, the outcome differentiates considerable. It is not clear why this is occurring.

Table 12: Comparison of the results of the lower bound and the upper bound analysis

Beam parameters				Analysis results	
Structural type	Bb	Ast	Asb	h_b upper bound	h_b lower bound
Stat. determ.	400	226	1808	417	420
Statically	250	1808	1520	233	235
undetermined	250	628	2463	288	290
beam	250	2463	628	372	<350

Robustness analysis

After plastic yielding, collapse will probably not occur due to beneficial phenomena such as force redistributions, catenary actions, concrete arching and strain hardening. These aspects are of particular interest for robustness analysis, as this entails the search for the real ultimate strength of the structure. This effect is not further investigated in this research.

11.2 PROBABILISTIC PROCEDURE

.

The reliability calculations are based on a Monte-Carlo simulation procedure. The input for this analysis is based on the results of the HRA simulation process described in chapter 10 and the JCSS probabilistic model code (JCSS, 2001). This is shown in figure 39, in which a subdivision is made in geometric properties, material characteristics and loading parameters.

The mean value of the geometric properties, such as beam dimensions and reinforcement layout, result from the HRA simulation process. It should be noted that these resistance parameters are not a fixed value as they are subjected to human error in the design. As a result they are presented by the probability functions given in chapter 10. Deviation in the geometric properties originating from construction activities are considered by using a standard deviation based on the probabilistic model code.

The material characteristics (f_y and f_c) are dominantly based on the probabilistic models given in JCSS (2001). Only the mean value of the concrete strength is based on the HRA simulation process. This is due to the common design practise that concrete strength is determined in the design phase, which is mostly adapted in the construction phase. As such, an error in the design phase will be adopted in the construction phase as well. Concerning reinforcement steel strength this is somewhat different. Due to the highly standardized steel strengths within especially normal applications, an error in design will probably not lead to an alteration in the strength of the steel in the construction. Based on this considerations, the choice is made to keep the reinforcement as a function of the probabilistic model code alone. It should be noted that deviation in the concrete strength due to construction activities is incorporated by using a standard deviation based on the probabilistic model code.

The loading conditions (self weight and distributed load) are functions of the distributions found in the model code. Loading conditions are also calculated in the design process. However these are the design parameters, and not the real occurring loading conditions. As such they are not relevant for the reliability analysis.

11.2.1 *Loading conditions*

The loading conditions of the structure are based on the action models defined in the probabilistic model code (JCSS, 2001). Only self weight and distributed loads are considered. Wind loads are not considered as they are primarily of concern for the stability core in this particular structural design. Besides the loading conditions, the geometrical properties and material characteristics are partly based on the probabilistic model code.

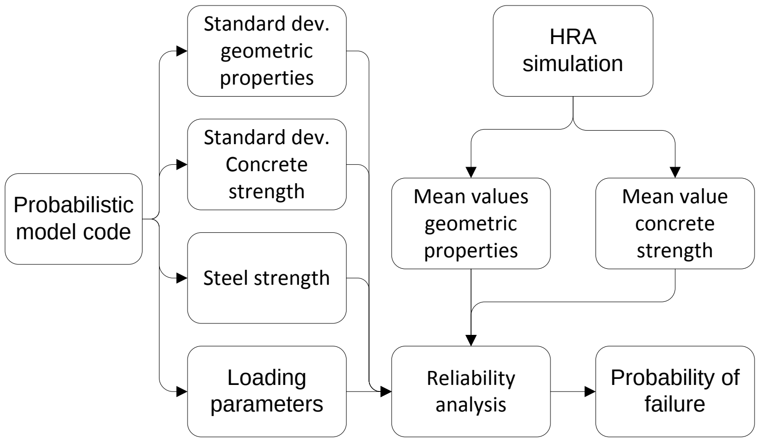


Figure 39: Input variables of the reliability function

The following two paragraphs elaborate on how the model code is interpreted. The model code provides a basic formula for the calculation of the Variance of the live loads:

$$Var = \sigma_V^2 + \sigma_U^2 \frac{A_0}{A} \kappa \tag{18}$$

The long term live load is presented with a Gamma function, with a mean value of 0,5 kN/m^2 (office use) and an average renewal time of 5 years. The floor area equals two bays ($7,6 \cdot 6,0 = 45,6m^2$). The parameters for formula 18 are: $\sigma_v = 0,3kN/m^2$, $\sigma_u = 0,6kN/m^2$ and κ = 2,0. This results in a standard deviation of 0,64 kN/m^2. The short term live load can be presented with a Gamma function, with a mean value of 0,2 kN/m^2 and an renewal time of 1 year. The parameters for formula 18 are: $\sigma_v = 0kN/m^2$, $\sigma_u = 0,4kN/m^2$ and κ = 2,0. This results in a standard deviation of 0,37 kN/m^2.

A good approximation of the concrete compressive strength distribution is the log-normal distribution. The mean value of the concrete strength is a function of the outcome of the HRA simulation. The standard deviation is defined as:

$$\sigma_c = s\sqrt{\frac{n}{n-1}\frac{v}{v-2}} \tag{19}$$

The parameters for this formula are: n = 3, s = 0,09 and v = 10. This results in a coefficient of variation of 0,12. An extra factor is the long term reduction factor for concrete which is defined with a normal distribution with mean 0,85 and variance 0,10. The yield strength of the reinforcement steel can be presented with a normal distribution. In this case the standard deviation equals 30 N/mm^2 and the mean value equals the nominal value plus two standard deviations which is 560 N/mm^2. The mass density of concrete is modelled with a normal distribution with a mean of 2500 kg/m^3 and standard deviation 96 kg/m^3. Finally two model factors are applied: one for the resistance and one for the loading effects. An overview of the used loading conditions is presented in table 13.

Table 13: Probabilistic models for the loading conditions and material properties (JCSS, 2001; Vrouwenvelder, 2002)

X	Parameter	Distr.	μ	V	λ	Unit
Hs	Slab height	Normal	160	0.004		$[mm]$
Hb	Beam height	Normal	HRA	0.004		$[mm]$
Bb	Beam width	Normal	HRA	0.004		$[mm]$
Hc/Bc	column dimensions	Normal	HRA	0.004		$[mm]$
ρ_c	Mass density concrete	Normal	2500	0,04		$[kg/m^3]$
f_c	concrete strength	Logn.	HRA	0,12		$[N/mm^2]$
α	long term reduction factor	Normal	0,85	0,10		[-]
f_y	yield strength	Normal	560	0,05		$[N/mm^2]$
q_{long}	Long term live load	Gamma	0,50	1,27	0,2/year	$[kN/m^2]$
q_{short}	Short term live load	Gamma	0,20	1,85	1,0/year	$[kN/m^2]$
m_R	Model factor resistance	Normal	1,0	0,05		[-]
m_E	Model factor load effect	Normal	1,0	0,10		[-]

HRA: mean value is output value of the design simulation.

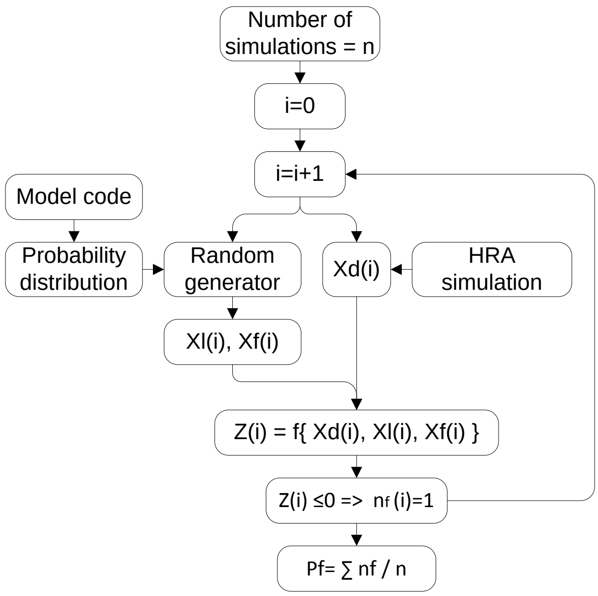
Table 14: Basic assumptions in the Euro-code for the representation of loading conditions (NEN-EN-1990, 2002)

Load property	Euro-code assumption
Material properties	characteristic value is equal to 5-95 % confidence bound
Self-weight loading	Characteristic value based on nominal dimensions and mean unit mass
Variable actions	characteristic value with a exceeding probability of 0,02 for the reference period of one year

The loading conditions in the design are based on the assumptions presented in table 14. The loading conditions in the design and the probabilistic analysis should be of the same order of magnitude. The live or imposed load is defined as a characteristic value of 3,0 $[kN/m2]$. The imposed load in the probabilistic analysis is presented by a short term and a long term live load of which the characteristics are given in table 13. simulating of both live loads results in an exceedence probability of 0,798 in the design life, which is somewhat lower then the requirements in the Euro-code presented in table 14 (exceedence probability of 1,0 in the design lifetime). From this it is concluded that the loading conditions within the probabilistic analysis are somewhat lower then the values used in the design, however they are deemed applicable for use within this research.

11.2.2 *Monte Carlo simulation*

The probability of failure is calculated with a Monte Carlo procedure. Within this procedure, the loading conditions and material characteristics are modelled by using a random generator as a function of the distribution functions defined in table 13. The mean values of the geometric properties and the concrete strength are based on the values obtained within the design simulation process. This entails that these resistance parameters differ within each simulation due to human errors. By running a sufficient number of simulations a reliable result can be obtained. This process is shown in figure 40.



Xd = mean values dimensions and concrete strength
Xf = Other dimensional and material parameters
Xl = loading parameters

Figure 40: Monte Carlo procedure

The failure probability is calculated for a design lifetime of 50 years which is modelled on the following way. The permanent loading conditions, material properties and model factors are randomly determined once in each simulation. The live loads differentiate from this as they are renewed within each simulation based on the renewal time of the load and the lifetime of the structure. The renewal time of the long term live load is 5 years. This entails that 10 consecutive long term live loads are acting during a life time of 50 years. The short term live load has a renewal time of 1 year resulting in 50 consecutive short term live loads during the lifetime of the structure. Subsequently, the first long term live load is successively combined with the first five short term live loads. This process is represented in figure 41, which shows a random live load history during the lifetime of a structure.

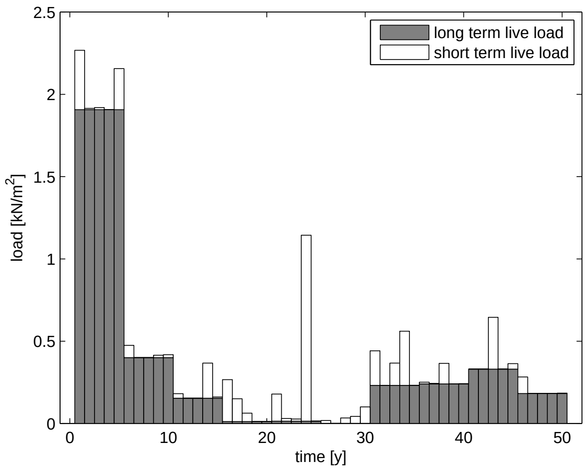


Figure 41: Load histories for short and long term distributed load

The next step within the Monte-Carlo procedure is to run the simulation of the model. The probability of failure is estimated with the following formula:

$$P_f \approx \frac{n_f}{n} \tag{20}$$

In which n is the total number of simulations and n_f is the number of simulations for which $Z < 0$. The required simulation runs is a function of the probability of failure, which is formulated as (CUR, 1997):

$$n > \frac{k^2}{E^2}\left(\frac{1}{p_f} - 1\right) \tag{21}$$

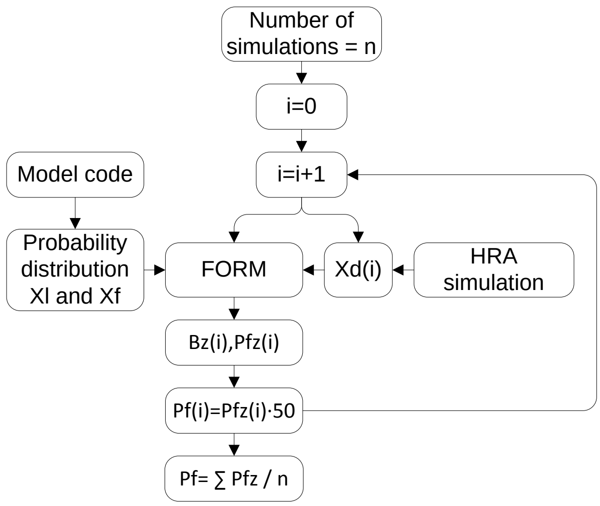
The required reliability is set to 90 % (k=1,65) with a relative error (E) of 0,1. The only unknown in equation 21 is the probability of failure. This value is not known in advance. An indication is:

- if the model is simulated with no human errors, a error probability of $0,723 \cdot 10^{-4}$ ($\beta = 3,8$) is expected. A reliable failure probability is obtained with $3,7 \cdot 10^6$ simulations.
- if the model is simulated with human errors, a error probability in the range $5 \cdot 10^{-4}$ to $5 \cdot 10^{-3}$ is expected. A reliable failure probability is obtained with respectively $5,5 \cdot 10^5$ and $5,5 \cdot 10^4$ simulations.

$3,7 \cdot 10^6$ simulation runs requires considerable calculation capacity, which is not available with the present means. This entails that checking the simulation procedure based on the norm requirements is infeasible. A solution would be to apply importance sampling, which reduces the number of simulations whit sufficient accuracy. Within this research only an indicative simulation is performed with $2,5 \cdot 10^5$ simulations. This resulted in a failure probability of $0,8 \cdot 10^{-5}$ which corresponds to a β-value of 4,3 which is somewhat on the safe side. This could be a consequence of the previous discussed suitability of the probabilistic loading conditions (these are somewhat to low in comparison with the design loading conditions). Within the remainder of the analysis, the number of simulations is 100.000. This is sufficient to obtain a coefficient of variation of the failure probability of 0,1 or lower.

11.2.3 *Mixed FORM - Monte Carlo simulation*

In order to check the reliability and correctness of the Monte-Carlo simulation, the same procedure is repeated, but in this case the random generator (of the Monte Carlo simulation) is replaced by the First Order Reliability Method (FORM, CUR, 1997, chapter 5). This results in a FORM analysis which is repeated for every simulation within the HRA simulation process. This is required to convert the probability distributions of the geometric properties and material characteristics within the HRA to deterministic values within the FORM analysis. This process is shown in figure 42.



Xd = mean values dimensions and concrete strength
Xf = Other dimensional and material parameters
Xl = loading parameters

Figure 42: Monte Carlo procedure

Within this method the probability function is linearised in a carefully selected point. The method approximates the probability distribution of each variable by a standard normal distribution. In order to fit the current limit state function within this method two transformations are required: a transformation for the non-linear reliability function and one transformation for the non-normally distributed base variables. The base variables are ought to be independent, which entails that the Rosenblatt-transformation is not required.

The FORM analysis consists of an iterative procedure to determine the reliability index (β) around the design point. A useful reliability index is obtained if the design point and reliability index is sufficiently converged. The iterative procedure consists of determining the standard deviation and mean value of the reliability function, calculating an α-factor and β-value, and subsequently using these values to determine a new design point. This design point is used in the following iteration as starting point. The standard deviation and the mean value of the reliability function are defined in formula 22 and 23 respectively (CUR, 1997). A visual check of the convergence of the calculation procedure is presented in appendix H.

$$\sigma_Z = \left\{ \sum_{i=1}^{n} \left(\frac{\partial Z(X_i^*)}{\partial X_i} \sigma_{X_i} \right)^2 \right\}^{1/2} \tag{22}$$

$$\mu_Z = Z(X_i^*) + \sum_{i=1}^{n} \frac{\partial Z(X_i^*)}{\partial X_i} \left(\mu_{X_i} - X_i^* \right) \tag{23}$$

In these formulas X_i^* is defined as follows:

$$X_i^* \approx \mu_{X_i} + \alpha_i \beta \sigma_{X_i} \tag{24}$$

In which α_i is defined as:

$$\alpha_i = \frac{-\frac{\partial Z(X_i^*)}{\partial X_i} \sigma_{X_i}}{\left\{ \sum_{i=1}^{n} \left(\frac{\partial Z(X_i^*)}{\partial X_i} \sigma_{X_i} \right)^2 \right\}^{1/2}} \tag{25}$$

In order to fit the non-normally distributed base variables, these variables have to be transformed to normally distributed base variables. A transformation proposed by Rackwitz & Fiessler (1977) is based on the assumption that the values of the real and the approximated probability density function and probability distribution function are equal in the design point. These transformation provides the following formulas for the standard deviation and mean. A check of the applicability of these formulas is presented in appendix H.

$$\sigma_X^{'} = \frac{\varphi\left(\Phi^{-1}\left(F_X(X^*)\right)\right)}{f_X(X^*)} \tag{26}$$

$$\mu_X^{'} = X^* - \Phi^{-1}\left(F_X(X^*)\right)\sigma_X^{'} \tag{27}$$

Comparison of both probabilistic methods results in a deviation in the reliability index of about 0 to 0,1. A deviation is expected as the Monte-Carlo method is based on simulation instead of the exact method and the FORM analysis simplifies non-normal distribution functions to a normal distribution function. It can be concluded that the Monte Carlo analysis and the mixed FORM - Monte Carlo analysis coincide nicely. This entails that the Monte Carlo analysis concerning the application of the probabilistic model code by means of a random generator seems reliable.

11.3 Results probability analysis

Within this section the results of the probabilistic analysis are discussed. The results are presented for the two beam types: the statical determined and the statical undetermined beam element.

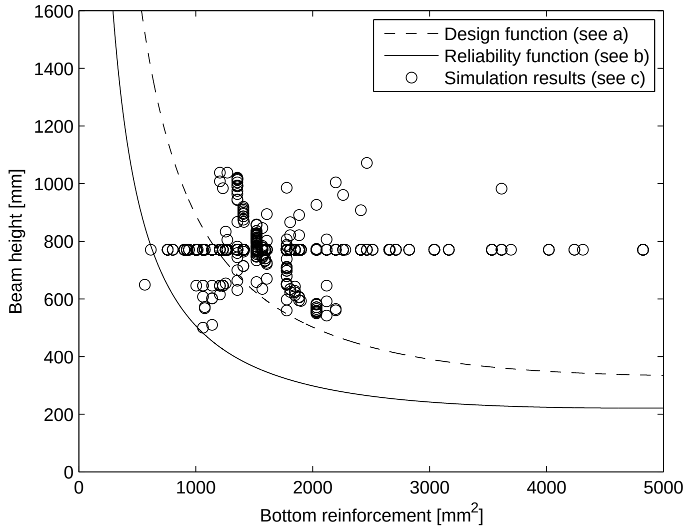
11.3.1 *Statical determined beam*

Table 15: Dimensional parameters static determined beam in case of no error occurrence

X	Parameter	μ	Unit
Lx	Beam length	7.20	[m]
Hb	Beam height	0.75	[m]
Bb	Beam width	0.40	[m]
Asb	Bottom reinforcement	1570	[mm^2]
Ast	Top reinforcement	226	[mm^2]
Asshear	Shear reinforcement	335 (ø8-300)	[mm^2/m]

The first analysis concerns the case of a prefab beam modelled as a statically determined beam. The dimensional parameters in case of no error occurrence are presented in table 15. A visual representation of the results is given in figure 43. Within this figure the reliability function is simplified considerable in order to represent some characteristics of the function. Only two parameters are depicted (beam height and reinforcement area), while the other parameters are kept deterministically. Nevertheless figure 43 provides some useful insights into the results of the analysis and the used reliability function. The shape of the reliability function encompasses two important aspects of failure in reinforced concrete beams: a lower reinforcement ratio results in a higher collapse probability and the reinforcement ratio has an optimum due to brittle concrete failure.

Within figure 43 two trends can be distinguished. Firstly, there is a horizontal line at a fixed beam height of 750 mm. This represents errors which only affect the reinforcement area, and not the beam dimensions. Secondly there is a trend-line parallel to the design function. This represents errors which affect both the beam height and reinforcement area. Both trend lines seem logical results of the simulation process.



[a] Reliability function based on ULS partial factor method (elastic analysis).
[b] Reliability function based on the plastic limit state analysis.
[c] Single simulation result out of a total of 20.000 runs.

Figure 43: Results of a statical determined beam simulation as a function of beam height and bottom reinforcement.

The results of the Monte-Carlo is presented in table 16. The results for a statically determined beam are not completely in agreement with the expectations. First of all, the failure probability decreases slightly if the design is executed by an experienced designer. This suggests that the experience of the designer has only a minor influence on the structural reliability. Secondly, the failure probability decreases with a factor of approximately 2,4 [1] if self-checking and normal supervision is applied instead of only self-checking.

Earlier analysis within the statical determined beam suggested somewhat other values. this is shown in table 16 as well. These values are based on other values within the self-checking and supervision loops. Comparison of the results of both analysis results in two conclusions. Firstly, the values of the reliability index differ too much. Secondly, the relative effect of the scenarios do not differ considerable (influence of experience is limited while control has a larger effect). From this it can be concluded that the final results should be used as relative results and not as absolute results.

1 Factor is defined as SC/NS in which SC is the failure probability of a design with only self-checking and NS is the failure probability of a design with self-checking and normal supervision.

Table 16: Results of the Monte-Carlo simulation of the statically determined beam.

Scenario	failure probability	Reliability index	Earlier analysis
Experienced designer with self-checking	$2.30 \cdot 10^{-3}$	2.83	3.02
Experienced designer with normal supervision	$9.50 \cdot 10^{-4}$	3.11	3.48
Inexperienced designer with self-checking	$2.27 \cdot 10^{-3}$	2.81	3.00
Inexperienced designer with normal supervision	$7.75 \cdot 10^{-4}$	3.16	3.35

11.3.2 *Statical undetermined beam*

The second analysis is a statical undetermined beam element. The dimensional parameters are presented in table 17. In this case there is not a single reliability function but two reliability functions as two failure mechanisms are governing the failure domain. Depending on the numerical values of the design parameters, one of these curves will determine the probability of failure. If the domain is restricted to three parameters: Hb, Asb, Ast and the other parameters are kept deterministic, the failure domain consists of two curves as shown in figure 44. It can be seen from this figure that both reliability functions are governing in a part of the solution domain.

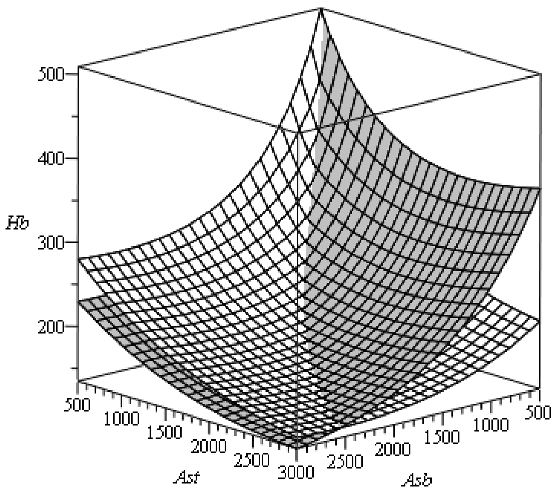


Figure 44: Governing failure curves on the failure domain Ast, Asb, Hb.

A visual representation of the reliability function is given in figure 45. The same properties as found in figure 43 for the statical determined beam are visible in this figure as well. Remarkable is that the effect of concrete crushing is becoming relevant at a lower reinforcement ratio. Furthermore it can be seen that the same two trend-lines are visible within the results

Table 17: Results of the Monte-Carlo simulation of the statically undetermined beam.

X	Parameter	μ	Unit
Lx	Beam length	7.20	[m]
Hb	Beam height	0.50	[m]
Bb	Beam width	0.25	[m]
Asb	Bottom reinforcement	1520	[mm^2]
Ast	Top reinforcement	1808	[mm^2]
Asprac	Practical reinforcement	628	[mm^2]
Lc	Column length	3.60	[m]
Bc	Column width	0.25	[m]
Asc	Column reinforcement	1256 ($2 \cdot 628$)	[mm^2]
Asshear	Shear reinforcement	335 (ø8-300)	[mm^2/m]

(horizontal and parallel trend-line). Finally it can be concluded that the scatter within the results in the statical undetermined case is higher then in the statical determined case. It should be noted that the total number of simulations with an error however is almost similar.

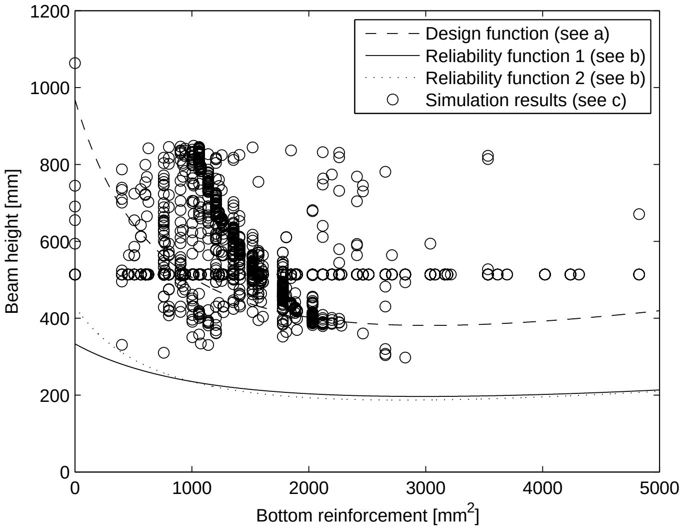
An interesting thing to mention is that the scatter within the statical undetermined beam is higher, but the final structural failure probability is lower. From this it can be concluded that the statical undetermined beam is somewhat safer. This is supported by the observation that the simulation results within the statical undetermined beam are in general positioned at a larger distance from the plastic reliability functions.

The results from the probability analysis are depicted in table 18. Overall, the failure probabilities are lower in comparison to the statical determined case. The same results as within the statical determined case are found. Experience has only minor influence, an experienced engineer decreases the structural failure probability with a factor 1,15 [2]. Design control has again quite some influence, as the failure probability decreases with a factor 2,4 [3] in case of normal supervision. Furthermore the earlier results deviate again in absolute values but they support the relative conclusions as well.

The conclusion from both beam types is that normal supervision has influence on the reliability of a structure while the effect of experience is not clear. Another point of interest is the use off the results. The absolute values of the calculations are not reliable, however the relative values seems reliable. This entails that the method is suitable for comparing different design

2 Factor is defined as EXP/INEXP in which EXP is the failure probability of a design performed by an experienced designer and INEXP is the failure probability of a design performed by an inexperienced designer.

3 Factor is defined as SC/NS in which SC is the failure probability of a design with only self-checking and NS is the failure probability of a design with self-checking and normal supervision.



[a] Reliability function based on ULS partial factor method (elastic analysis).
[b] Reliability function based on the plastic limit state analysis.
[c] Single simulation result out of a total of 20.000 runs.

Figure 45: Results of a statical undetermined beam simulation as a function of beam height and bottom reinforcement.

Table 18: Results of the Monte-Carlo simulation of the statical determined beam

Scenario	failure probability	Reliability index	Earlier analysis
Experienced designer with self-checking	$1,50 \cdot 10^{-3}$	2,97	3,33
Experienced designer with normal supervision	$6,40 \cdot 10^{-4}$	3,22	3,55
Inexperienced designer with self-checking	$1,72 \cdot 10^{-3}$	2,93	3,09
Inexperienced designer with normal supervision	$7,2 \cdot 10^{-4}$	3,19	3,27

process configurations.

11.3.3 *Comparison results*

Within the literature study a paper (Stewart, 1993) is discussed which simulates the effect of human error on the design and construction of a reinforced concrete beam (simply supported, without compressive and shear reinforcement) This case is comparable with the case study for the statically determined beam presented in this thesis. Stewart (1993) distinguishes two control measurements: 'designer check' and 'design check'. These are comparable with 'self-checking' and 'normal supervision' defined in this research. A comparison of the results is given in table 19.

Table 19: Comparison results Stewart (1993) and case study

	Stewart (1993)	Case study
Self-checking	$0,381 \cdot 10^{-3}$	$2,27 \cdot 10^{-3}$
Normal supervision	$0,586 \cdot 10^{-4}$	$7,75 \cdot 10^{-4}$

Comparing the results shows that the results of Stewart are systematically lower. Furthermore supervision is slightly more effective within the model of Stewart. Despite these numerical differences, there is quite an agreement in the general picture of the results: normal supervision has quite an influence of the structural reliability. The numerical differences can be explained by the large margins on the failure probabilities in any Human Reliability Assessment. from this, and the analysis in the previous section, it can be concluded that the results are only valuable as relative numbers. This is in line with the shortcomings of HRA formulated by Swain (1990) (presented in chapter 4) and the conclusions made in the previous section.

12 CONCLUSIONS AND RECOMMENDATIONS

This research considers the effect of human error within structural engineering. The objective of this research is to investigate the effect of human error within the design process on the reliability of building structures. In this chapter conclusions and recommendations for further research are presented. The main research question is defined as:

> *What are the consequences of human error within the design process on the structural reliability of a typical building structure?*

To answer the main research question, a Human Reliability Assessment (HRA) method for structural design tasks is proposed. This model is subsequently used to investigate the consequences of human error within the detailed design of a reinforced concrete beam in a building structure.

The HRA model basically encompasses four steps, which are presented in figure 46. The model starts with a general idea about the particular engineering task, of which insights on the effect of human error is required. Through the four HRA steps a failure probability of the engineered structure is obtained.

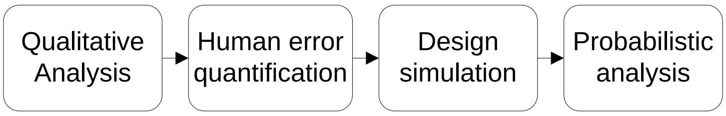


Figure 46: Basic steps within the HRA model

12.1 CONCLUSIONS

The qualitative analysis is used to determine the context of the situation. Furthermore a prior analysis (scenario selection) is used to select potential hazardous design tasks, on which a HRA analysis should be performed. The following conclusion is formulated concerning this step:

- It is found that using the HRA model for a complete design process is unrealistic. To tackle this, minimizing the HRA to the most hazardous design steps is required.

Within the human error quantification, a Human Error Probability (HEP) and an Error Magnitude (EM) is calculated for each task within the design. For the HEP quantification two methods are proposed: an extended and a simplified HEP method. These parameters present the occurrence probability and consequence of a human error in a single design task. Concerning this part of the HRA process, the following conclusions are stated:

- Quantifying HEPs within design is sensitive to subjectivity due to the inherent aspect to assess possible human failure modes. In order to minimize these negative effects, careful selection of the boundary conditions and starting points is required.

- The extended method is useful, however also labour intensive and requires quite some knowledge concerning human factors. The simplified method requires considerate less efforts and knowledge, however this method is only applicable for standard design tasks.

The next step, design simulation, is used to derive an error probability on the structural element level. This simulation process is based on a task step analogy of the design process. Within this analogy, a task HEP and EM is combined in a so called micro-task. All micro-tasks combined form the design process. Besides this, design control is modelled with a algorithm where some or all prior micro-tasks are re-evaluated if the initial results are not within 'reasonable' limits. This analysis resulted in the following conclusions:

- The micro-task analogy is useful for modelling human error in design. Two important elements of a design process are encompassed in the model: a task is depending on a finite number of previous design steps and errors are modelled as a deviation from intend. This latter deviates from the simple success-failure notion often used in HRA.

- the control loop analogy is useful, however very crude. This entails that further research is required to increase the accuracy of control loops in design. Despite this it encompasses the ability to check the results based on previous experience.

The last step is to determine the failure probability on element level. This analysis is performed with a probabilistic Monte-Carlo method. The error probabilities found in the previous step combined with probabilistic loading conditions are used to determine the structural failure probability. This process is deemed useful for determining structural failure probabilities based on human error probabilities.

Concerning the overall process, it can be concluded that the HRA model has the potential to quantify the effect of human error within carefully defined boundary conditions. However further research is required to increase the accuracy of the model and its practical use.

Case study

The HRA model is used to analyse a simple design process, consisting of the design of a reinforced concrete beam element within a building structure. For this analysis two scenarios are selected: the level of design control and the level of professional knowledge of the designer. Furthermore two beam types are considered: a statical determined and a statical undetermined beam. Conclusions based on the performed case study are:

- Human error has the potential to reduce the reliability index of a beam element from 3,8 to approximately 3,5 to 2,5.

- The influence of design experience on the structural failure probability is limited. The failure probability decreases slightly if an experienced designer performs the design instead of an inexperienced designer.
- There is quite some effect of normal supervision on the structural failure probability. In comparison to a process with only self-checking, the effect is about a factor 2,4.
- A design process without design supervision (self-checking and normal supervision) results in an unrealistic failure probability. Due to the inherent presence of (mostly subconscious) self-checking within design, this is deemed an unrealistic design practice.
- The results are only usable as relative results. This entails that the method can only be used to compare different design configurations defined within the HRA. Comparison with results outside the HRA is doubtful due to the lack of real time validation.
- Overall the structural failure probability within the statical undetermined case is slightly lower then within the statical determined beam.

12.2 Opportunities for further research

The conducted research is an explorative research concerning the use of Human Reliability Assessment methods within structural engineering design. Based on this, several opportunities for further research are formulated.

Verification and calibration of the model
The proposed Human Reliability Assessment method is only worked out in basic form. A important step is to verify and calibrate the model with a real-time structural engineering process. Based on this, the model can be improved and altered for practical use.

Improvement of the Human Error probabilities
The Human Error Probabilities (HEPs) are primarily based on values available within literature. A deficit of these values is that they are primarily based on operational types of actions. Further research is required to verify the applicability of the HEPs or to find reliable and applicable HEPs.

Improvement of the Error Magnitude
The Error Magnitude (EM) within design tasks is only vaguely known. These EMs are required together with the HEPs to obtain a probability distribution of the effects of human error within a single design task. Research is required to attain EMs of professionals conducting relevant engineering tasks, which can be used as realistic input within the model.

Control mechanisms
The control mechanisms within the process are modelled on a very basic manner within the proposed HRA-method. They are only depending of the knowledge of the engineer and the person's level of control over the design parameter. This simple model is deemed to be to crude for use within realistic applications. As such, modifications of this is required. Improvements

can be made by incorporating the available control time, task complexity and designer experience on a more sophisticated manner.

Non-linear character failure causation
Within the literature (Hudson, 2010) it is discussed that accident causation must be regarded as both non-linear and non-deterministic. A Human Reliability Assessment method must cope with these properties in order to estimate failure probabilities accurately. It is discussed that the presented model comprises some features to model these non-linear and non-deterministic aspects. However further research is required to investigate the limitations of the proposed model on these aspects and to propose improvements to the model.

Safety barriers
Within the design process more (often subconscious) safety barriers are present. Furthermore safety barriers are often interrelated caused by common organizational factors. This could mean that barriers might fail after an incident much easier al of a sudden. Within the model this is applied on a basic manner. Investigation on these safety barriers and the effect of the interrelation between these barriers is required to pinpoint there effect on human error prediction

Model for construction tasks
The HRA model is designed for typical design tasks within structural engineering. Expansion of the model with typical construction tasks would increase the applicability of the model for use within basic engineering tasks.

Improve calculation accuracy
Within the probabilistic analysis up to 250.000 simulation runs were used. This was sufficient for the results within this research. However investigating of less error prone situations or small parts of the process requires extra calculation capacity. Due to the lack of this in most situations, adaptive importance sampling can be useful to increase the accuracy of the probabilistic analysis. Further research on the particular use of this in the model is required.

13 DEFINITIONS

13.1 LIST OF DEFINITIONS

Cognitive process	A group of mental processes by which input is transformed, reduced, elaborated, stored, recovered and used.
Construction process	The process by which resources such as manpower, material and equipment is used to construct a facility or product based on the design plan.
Design process	The process (often iterative) in which a plan or scheme is created for the realization of a stated objective to create a product.
error rate	The fraction of cases in which the performance deviates from the limits of performance defined by the system.
Failure	In general defined as the unsuitability of the structure to serve the purpose where it was built for, regardless of cause. Within this thesis narrowed to failure caused by the collapse of (parts of) a building.
Human error	Any member of a set of human actions or activities that exceeds some limit of acceptability, i.e. an out of tolerance action or failure to act where the limits of performance are defined by the system.
Limit state	The state just before failure occurs.
Micro-task	A task sequence consisting of one or more cognitive activities to acquire one single design parameter.
Reliability	The probability that the limit state is not exceeded.
Risk	The combination of the probability of occurrence of a defined hazard and the magnitude of the consequences of the occurrence.
Robustness	The ability of a structure to withstand events like fire, explosions, impact or the consequences of human error, without being damaged to an extent disproportionate to the original cause.
Structural safety	The absence of harm due to an unexpected chance of failure due to structural collapse of (part of) the building.

13.2 LIST OF ABBREVIATIONS

APJ	Absolute Probability Judgement
CATS	Causal Model for Air Transport Safety
CPC	Common Performance Condition
CREAM	Cognitive Reliability and Error Analysis Method
EM	Error Magnitude
EPC	Error Producing Condition
FORM	First Order Reliability Method
FMEA	Failure Mode and Effect Analysis
FTA	Fault Tree Analysis
GTT	General Task Type
HEART	Human Error Assessment and Reduction Technique
HEP	Human Error Probabilities
HEQ	Human Error Quantification
HRA	Human Reliability Assessment
IA	Impact Assessment
JHEDI	Justification of Human Error Data Information
PC	Paired Comparisons
PRA	Probabilistic Risk Assessment
PSF	Performance Shaping Factor
RPN	Risk Priority Number
SLIM	Success Likelihood Index Methodology
THERP	Technique for Human Error Rate Prediction

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Part III

APPENDICES

A

ROBUSTNESS OF STRUCTURES

Contents

introduction

Probabilistic risk assessment within structural engineering deals with the question if a building will collapse. An important aspect of this is the degree of robustness of the structure. Faber et al. (2007) links robustness to risk assessment on the following manner: "the vulnerability of a give system [...] characterizes the risk associated with the direct consequences and the robustness characterizes the degree the total risk is increased beyond the direct consequences." These indirect consequences occur if a structure is not robust, that is when some local damage can trigger a chain reaction of failures causing collapse of the whole structure or of a major part of it (Val & Val, 2006).

Within the main research emphasises is placed on characterizing the vulnerability of a structure to human error. The next step would be to broaden this research to indirect consequences by considering structural robustness. As an introduction to this subject, the theory of robustness is discussed in this appendix. First definitions of robustness are given followed by some general considerations of progressive collapse. After that the difficulties of robustness design, the applicability of the Euro code concerning robustness design and some robustness design methods are discussed. The appendix is concluded with a few remarks on the calculation details of robustness design.

definition robustness

There are several definitions of robustness available within the literature. Starossek (2009) defines robustness as the insensitivity to local failure. 'local failure' is associated to the 'assumable cases of initial local failure' and 'insensitivity' is stated as no more than the 'acceptable total damage'. The quantitative definition of these terms is rooted in the design objectives that have to be predetermined in a decision-making process. This definition pinpoints a peculiarity of robustness; the interpretation differs within each project.

Val & Val (2006) provides two different definitions of robustness. The first is: "the robustness of a structure may be defined as the ability of a structure to absorb, with an appropriate level of reliability, the effect of an accidental event without suffering damage disproportionate to the event that caused it." The authors criticise this definition by stating that it is an ambiguous definition: "The main problem is to determine the meaning of 'disproportionate', that is to establish a relationship between accidental events and acceptable levels of damage." With this critical note in mind, Val & Val (2006) states a second definition: Robustness is the ability of a structure to withstand local damage, not depending on particular accidental events. From this definition, robustness is solely defined as an internal property of a structure, and the problem of defining a relationship between accidental events and levels of damage is omitted.

A broader definition is used in the European Code (NEN-EN1991-1-7, 2006). This definition is given as: "the ability of a structure to withstand events like fire, explosions, impact or the consequences of human error, without being damaged to an extend disproportionate to the original cause." A final remark on robustness can be based on a remark of Baker et al. (2008) concerning the properties of robustness: "The property of robustness depends upon system properties such as redundancy, ductility, load redistribution and damage detection, but it also depends upon failure consequences." In this thesis we will use the robustness definition given by the NEN-EN 1991-1-7 as a basis.

From above definition some general conclusions can be drawn. First of all, defining a unilateral definition for all types of structures is hard, defining on a project basis is required. Secondly, defining external events and the relation between external events and acceptable levels of damage is difficult. A good example of the latter is the collapse of the WTC. FEMA (2002) reports that the original design of the WTC was based on an impact of a Boeing 707 while low on fuel and at landing speed. The real impact was a Boeing 767 with 37.000 litres of fuel on flight speed . From this perspective FEMA (2002) concludes: "The fact that the structures were able to sustain this level of damage and remain standing for an extended period of time is remarkable."

progressive collapse

When a collapse commences with the failure of one or a few structural components, it is defined as 'progressive collapse' (Starossek, 2009; Nair, 2004). Nair (2004) ads to this: "a building's susceptibility to progressive collapse should be of particular concern only if the collapse is also disproportionate, i.e., the collapse is out of proportion to the event that triggers it." Starossek (2009) also distinct the term 'disproportionate' from 'progressive'.

Although all progressive collapses share the disproportion between cause and effect, the mechanisms to produce such progressive collapses vary considerable. Starossek (2009) distinguishes six types of progressive collapses:

- Pancake-type collapse; a vertical propagating collapse characterized by the separation of structural elements, the release of gravitational energy and the occurrence of impact forces.
- Zipper-type collapse; a horizontal propagating collapse characterized by the redistribution of forces into alternative paths, the impulsive loading due to sudden element failure and the concentration of static and dynamic forces.
- Domino-type collapse; a horizontal propagating collapse characterized by the overturning of individual elements and the horizontal pushing force that leads to the overturning of the next element.
- Instability-type collapse; caused by compression in stabilised elements, the subsequent failure of stabilising elements and then stability failure of the compressed elements in a progressive manner.

- Mixed-type collapse; occurs if a particular collapse exhibits features of more than one of the basic four collapse types.

- Section-type collapse; when a part of the cross-section is cut, the corresponding stress increase at some locations can cause the rupture of further parts of the cross-section. This kind of failure is usually called brittle fracture of fast fracture.

Above mentioned classification of six basic types of progressive collapse are helpful to recognise different collapse mechanisms, and to classify the different design requirements in the design codes. Classifying these design requirements required for progressive collapse can be hard as will be mentioned below.

DESIGN DIFFICULTIES

Modern design codes and verification procedures are based on the probabilistic theory of reliability. Actions and resistances are determined statistically from empirical data obtained from observations and measurements and represented by their respective probability density functions. According to Starossek (2009), such an approach is based on mathematically sophisticated and sound foundations and is, as it seems, correct. Unfortunately, that idea is illusory because such an approach fails with regard to the identification of progressive collapse. There are three reasons for this failure (Starossek, 2009):

1. Design equations are defined at a local level only. Structural safety, therefore, is also accounted for only at a local level. The structural safety at a global or system level is a function of not only the safety against local failure of all load-bearing elements but also of the structural response of the entire system to local failure.

2. Accidental circumstances, such as events that are unforeseeable or whose probability of occurrence is very low, are neglected.

3. The underlying probabilistic concept requires the specification of an acceptable probability of failure. Considering the huge losses that can result from progressive collapse, it is difficult to reach an informed and true societal consensus.

Several design strategies and methods to ensure collapse resistance are described in the literature. Most of them are also prescribed in modern design codes such as the European Norm NEN 1991-1-7. Burnett (as cited in Ellingwood & Dusenberry (2005)) suggests several design strategies to control the frequency and the severity of the relevant abnormal loading that might lead to progressive collapse:

- by eliminating the cause;

- by reducing the effect of abnormal loading;

- by protecting the structure;

- by adjusting the structure to accommodate some form of abnormal loading.

Ellingwood and Leyendecker (in Ellingwood & Dusenberry (2005)) examine the development of design criteria to control progressive collapse. The authors consider three approaches to prevent progressive collapse:

- Event control; which refers to protecting against incidents that might cause progressive collapse.
- Indirect design; to prevent progressive collapse by specifying minimum requirements with respect to strength and continuity.
- Direct design; considering resistance against progressive collapse and the ability to absorb damage as a part of the design process.

The subdivision given by Ellingwood and Leyendecker is adopted by several other researchers and design codes. For instance Starossek (2009) uses almost three identical approaches. The European Code (NEN-EN1991-1-7, 2006) gives a number of similar design strategies for accidental design situations, which will be discussed in section A.

European code

The topic of robustness is essentially covered by two Eurocodes, these are:

- NEN-EN-1990 (2002): Eurocode, Basis of structural design; this document provides the higher level principles for achieving robustness.
- NEN-EN1991-1-7 (2006): Eurocode 1, Actions on structures, part 1-7, accidental actions; this document provides strategies and methods to obtain robustness and the actions to consider.

According to Gulvanessian & Vrouwenvelder (2006) the principal design principle behind the assessment of accidental actions in buildings in the NEN-EN 1991-1-7 is that local damage is acceptable, on condition that it will not endanger the structure and that the overall load-bearing capacity is maintained during an appropriate length of time to allow necessary emergency measures to be taken.

The code makes a distinction between identified and unidentified accidental actions. Typical identified accidental actions are fire, explosion, earthquake, impact, floods avalanches, landslides and so on. Unidentified actions are human errors, improper use, exposure to aggressive agencies, failure of equipment, terrorist attacks, and so on. Strategies for identified accidental actions are mostly based on classical (advanced) structural analysis, while the strategies for unidentified actions are based on more general robustness requirements.

In this study we will further examine three strategies for providing a building with an acceptable level of robustness. The first two are mentioned in NEN-EN1991-1-7 (2006), the last one is mentioned in Starossek (2009):

1. enhancing redundancy
2. design of key elements
3. isolation by segmentation

Enhancing redundancy

Enhancing redundancy is in the eurocode mainly provided by ensuring alternative load paths. The following instruction is given in NEN-EN 1991-1-7, annex A.4, page 35:

> the building should be checked to ensure that upon the notional removal of each supporting column and each beam supporting a column, or any nominal section of load-bearing wall as defined in A.7 (one at a time in each storey of the building) the building remains stable and that any local damage does not exceed a certain limit.

Removing a supporting column is a popular design tool for unforeseen actions (Vrouwenvelder, 2011). However there are some critical notes on this method. Vrouwenvelder (2011) states that only 10% of the investigated structural failures is related to column loss and it is hard to define the likelihood for such an event. Furthermore, Starossek (2009) states: "the continuity required for the formation of alternative load paths may, in certain circumstances, not prevent but rather promote collapse progression." This is an consequence of the underlying demand to provide sufficient strength within the alternative path to carry the extra static and dynamic loads. If this extra strength is not available within the construction, the structure will collapse. Starossek (2009, 2006) illustrates this critical note with an example: the Confederation Bridge in Canada (see figure 47). To provide alternative load paths, the initial failure of one bridge pier would require designing a prestressed concrete frame with the length of 500 metre. This was deemed economically not attractive and technically not possible within the adapted design.

Another critical note can be found in Nair (2004). The author writes: "the problem with the redundancy approach (or alternative load paths) is that it does not account for differences in vulnerability." This becomes clear if we compare different column layouts. Say for instance we have a row of 200 x 200 mm columns with an in-between distance of 4 meters or a row of 900 x 900 mm columns with an in-between distance of 12 meters. An explosion that will take out the 900 x 900 column would likely destroy several 200 x 200 columns. This loophole is not tackled in the current eurocode.

Finally Izzuddin et al. (2008) gives a critical remark concerning the mechanical modelling of 'notional member removal': "[a] shortcoming of the

Figure 47: The Confederation Bridge (Canada)

notional member removal provisions is the assumption of a static structural response, when the failure of vertical members under extreme events [...] is a highly dynamic phenomenon." The paper argues that sudden column loss represents a more appropriate design scenario: "Although such a scenario is not identical in dynamic effect to column damage resulting from impact or blast, it does capture the influence of column failure occurring over a relative short duration to the response time of the structure."

Key Elements

Another possibility stated in the Eurocode, are designing so-called key elements. The Eurocode writes about this in annex A.4, page 35 (NEN-EN1991-1-7, 2006):

> Where the notional removal of such columns and sections of walls would result in an extent of damage in excess of the agreed limit, or other such limit specified, then such elements should be designed as a "key element".

The Eurocode provides further insight in what a key element is in annex A.8, page 39:

> [...] a 'key element' [...] should be capable of sustaining an accidental design action of A_d applied in horizontal and vertical directions (in one direction at a time) to the member and any attached components having regard to the ultimate strength of such components and their connections. [...] The recommended value of A_d for building structures is 34 kN/m².

Isolation by Segmentation

As discussed before in this section, the alternative load path method can sometimes not prevent but rather promote collapse progression. Another method, which is not widely acknowledged in the literature or design codes is the Isolation by Segmentation method. This is already demonstrated with the Confederation Bridge, in which dimensioning alternative load paths was not possible due to the long spanning of the bridge. Starossek (2009, 2006) not only describes the problem, but also presents a solution in the form of segmentation, which will be set forth in this subsection.

The isolation by segmentation is a design method which spatially limits local failure by isolating the collapsing section. Within the confederation bridge this was done by inserting a hinge in every second span, which interrupt the continuity in the concrete slab and the prestressed tendons. If a collapse of one of the bridge piers or one of the slabs would occur, the total damage would be limited to two segments. For further details see Starossek (2009). In more general sense, segmentation can be achieved by eliminating continuity, reducing local stiffness or through ductility. For instance by inserting joints, break-away hinges, structural fuses or providing plastic hinges at the segment border.

The segmentation method can be used instead of the alternative load path method, but also together with the alternative load path method. It is for instance possible to segment a structure, and within these segments provide alternative load paths. If this idea is applied to a building, the extend of collapse will be limited to one segment in case of very huge impacts, and only to a part of the segment in case of small impacts.

This method has also his application boundaries. For instance how could we apply this method to high-rise buildings? And is the collapse of a part of the structure socially acceptable? Even in the example of the Confederation Bridge, the question remains if the collapse of two segments is socially accepted. Especially if human lives are involved.

CALCULATION DETAILS

This section provides some information on the calculation details belonging to robustness calculations. Information is provided on quantifying probabilities of robustness, indexes to quantify the level of robustness and some particular mechanical aspects of robustness.

Robustness probabilities

In terms of risk, the probability of progressive collapse developing from an abnormal event can be expressed as:

$$P(F) = P(F \mid DA_i) \cdot P(D \mid A_i) \cdot P(A_i) \tag{28}$$

In which F is the event of structural collapse, $P[A_i]$ is the probability of hazard A_i, $P(D \mid A_i)$ is the probability of local damage, D, given that A_i occurs, and $P(F \mid Da_i)$ is probability of collapse, given that hazard and local damage both occur (Ellingwood & Dusenberry, 2005). This equation is also mentioned in Baker et al. (2008); Vrouwenvelder & Sorensen (2010).

The breakdown of the collapse probability into various events makes it possible to focus attention on strategies to prevent global failure of the structure. For instance Diamantidis (2009) elaborates on this by stating that the condition probability of local damage $P(D \mid A_i)$ given the event A_i, can be considered in two different ways: “In many cases local damage is accepted and consequently $P(D \mid A_i)$ is equal to 1.0 (modelling a failure in order to test the robustness of the structure). In other cases local strengthening is preferred in order to reduce the probability of local damage.“ A broader classification of measurements is also given by Diamantidis (2009): Prevention measures influence $P(A_i)$ and mitigation measures influence $F \mid DA_i)$, $P(D \mid A_i)$ and the consequence of collapse. Finally the risk of progressive collapse is defined as:

$$R_c = P(F) \cdot C_c \tag{29}$$

In which C_c is the consequence of collapse.

Robustness Index

The robustness definitions mentioned in section A do not specify any criteria for an engineer to use in measuring robustness or determining whether a system’s level of robustness is acceptable. Baker et al. (2008) writes that a measure of system robustness should contain three properties in order to be helpful for design and analysis application: it is applicable to general systems, it allows for ranking of alternative system choices and it provides a criterion for verifying acceptable robustness. This section provides some criterion’s for measuring robustness based on findings form Baker et al. (2008), Ellingwood & Dusenberry (2005), Vrouwenvelder & Sorensen (2010) and Biondini et al. (2008).

The first quantification method is based on the findings of Baker et al. (2008) on direct and indirect risk. The following index of robustness is proposed, which measures the fraction of total system risk resulting from direct consequences:

$$I_{rob} = \frac{R_{dir_1}}{R_{dir_1} + R_{ind_1}} \tag{30}$$

If the structure is completely robust and there is no risk due to indirect consequences, the I_{rob} is 1. If the structure is not robust, then I_{rob} is 0. Baker et al. (2008) states that this index provides insight in the following three elements of robustness. First, the index measures the relative risk due to indirect consequences (direct risk should be measured with the reliability criteria). Secondly the index depends on failure probabilities of the damaged structure and upon the relative probabilities of the various damage states occurring. This allows exposures causing the loss of two columns to be incorporated in the index. Third, the index accounts for both the probability of failure of the damaged system and the consequences of the failure. According to Vrouwenvelder & Sorensen (2010), this index should be considered as a possible helpful indicator. This conclusion is based on the opinion that minimizing risk should be based on minimizing total risk (R_{dir_1} and R_{ind_1}). This implies that the robustness index given in equation

30 is not always fully consistent with a full risk analysis.

The second quantification method is based on Fu & Frangopol (1990), which proposes a probabilistic measure related to structural redundancy (which is a measure of robustness). The redundancy index, RI, is defined as follows:

$$RI = \frac{P_{d(dmg)} - P_{f(sys)}}{P_{f(sys)}} \tag{31}$$

The parameters in the formula are defined as follows:

$P_{d(dmg)}$	probability of damage (i.e. component failure) occurrence to the system.
$P_{f(sys)}$	probability of system failure (i.e. the probability that any failure path will occur)

The difference between $P_{d(dmg)}$ and $P_{f(sys)}$ is assumed to describe the system residual strength. According to equation 31, a structure is considered non-redundant if RI is 0 ($P_{d(dmg)}$ is $P_{f(sys)}$). The structure is redundant if $P_{d(dmg)} > P_{f(sys)}$ ($RI > 0$), with completely redundant if RI is 1.

Another closely related index for redundancy is provided by Frangopol & Curley (1987). This paper uses the reliability index β of the intact (β_{intact}) and damaged ($\beta_{damaged}$) structural system:

$$\beta_R = \frac{\beta_{intact}}{\beta_{intact} - \beta_{damaged}} \tag{32}$$

Within this formula, $\beta_R = 0$ indicates a completely damaged structure and β_R = infinite indicates an intact structure.

A last method is proposed by Biondini et al. (2008), which proposes a dimensionless robustness index associated with the displacement of the system:

$$\rho = \frac{\|S_0\|}{\|S_d\|} \tag{33}$$

In which S is the displacement vector, ||.|| denotes the euclidean scalar norm, the subscript '0' refers to the intact state of the structure and 'd' refers to the damaged state of the structure.

Mechanical Modelling

Mechanical modelling related to robustness requirements differs in some aspects from modelling with reliability criteria. In this section some of these issues are briefly mentioned, in order to gain insight in the mechanical issues of robustness.

Robustness design involves geometrically non-linear mechanisms, including tensile catenary and compressive arching actions. Detailed modelling of the non-linear response typically involves the use of advanced non-linear finite elements (Izzuddin et al., 2008). Due to these advanced methods calculation demands/time can become quite high. In order to reduce this modelling effects could be reduced by considering only the affected bay of the structure (in case of column loss), or only the floors above the lost column, or ignoring the planar effects within the floor slabs in order to reduce calculation time.

Within these models, sophisticated or simplified models can be used to model the mechanical properties. An example of a simplified beam model with tensile catenary action for a steel beam is given by Izzuddin et al. (2008) and is presented in figure 48. From this figure it can be seen that catenary action have quite an influence on the final strength of a beam.

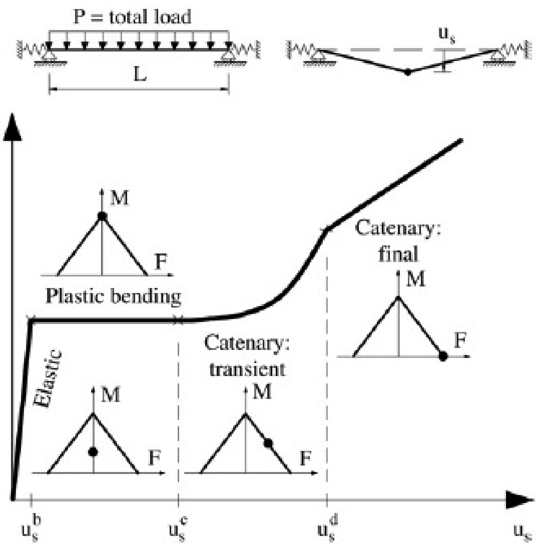


Figure 48: Simplified beam model with tensile catenary action (Izzuddin et al., 2008, page 1311)

Another important aspect of robustness design is the allowance of substantial local damage, causing impact of debris resulting from such a damage on other areas of the building. Present design codes, including the NEN-EN 1991-1-7 provide no provisions of this problem, which could potentially lead to progressive collapse of the structure (Izzuddin et al., 2008). Within the current calculation programs, this debris loading must be specifically modelled as a force on the remaining structure. This is a complex task as for instance determining the weight, acceleration and direction of the debris is required. It should be noted that some sophisticated programs are able to model debris loading. An example of this is given in Salem (2011), based on a program which allows element separation within the Final Element Model (FEM).

B SCENARIO IDENTIFICATION

Contents

introduction

This appendix consists of two parts. The first part elaborates on the research methodology within the desk research. Within the second part the results of the desk research are presented.

desk research

Scenario identification is based on analysis of failure information available within the literature. Eight papers which present quantitative information about the types of failures occurring throughout the world are used in this analysis. These papers are selected on the basis of availability and relevance. Some papers were recommended by the thesis supervisors, while others were found by analysing the references of the available papers. These papers are: Boot (2010), ABC-meldpunt (2011), Fruhwald et al. (2007), Matousek & Schneider (1976), Walker (1981), Eldukair & Ayyub (1991), Allen (1979) and Hadipriono (1985). It should be noted that Walker is cited from Fruhwald.

Some of the characteristics of these investigated literature is presented in table 20. From this table it can be seen that the literature is mainly focussing on building structures (in contrast to civil structures). Further more, the researches are conducted from quite recently (2011) to about 35 years ago (1976). Also the type of material differs considerable and the investigated region is worldwide. From this it can be concluded that the investigated literature has quite a broad basis within failures of building structures.

It should be noted that the numbers given in this desk research can only be used as a broad indication, as the number of surveys is limited, the scope of these researches differ and background information is lacking. However they can still be useful in order to select relevant scenarios for further research.

investigated aspects

Three research questions are selected for further research:

- *question 1;* What type of error did lead to the failure?
- *question 2;* How could these errors have occurred?
- *question 3;* Which building elements were involved in the failure?

analysis method question 1 (error type)

The first research question is: 'What type of error did lead to the failure? '. Analysing this question is performed in several steps. First a list of risks of each activity within the design process is composed. This list is based on the 'design facility' model proposed by Sanvido, Khayyal, Guvenis, Norton,

Table 20: Characteristics of investigated failure literature

Author	Year	No. of cases	Type of structures	Type of material	Country or Region
Boot	2010	151	Buildings[a]	Various	Netherlands
Nelisse and Dieteren	2011	189	buildings[a]	Various	Netherlands
Fruhwald et al.	2007	127	Buildings	Timber	Scandinavia [c]
Matousek and Schneider	1976	295	Buildings	Various	Europe
Walker[b]	1981	120	Unknown	Unknown	Unknown
Eldukair and Ayyub	1991	604	Various	Various	United States
Allen	1979	188	Various	Concrete	Canada
Hadipriono	1985	150	Various	Various	The world

[a] Mainly buildings, varying from 90 to 95 %
[b] Cited from Fruhwald et al.
[c] Also some worldwide cases.

Hetrick, Al-mualllem, Chung, Medeiros, Kumara & Ham (1990), and consists of six main risks which are subsequently subdivided in sub-groups. Secondly each error mentioned in the literature is classified according to this list. Finally the results are analysed with the help of three parameters:

- Numbers of authors which mention the risk category
- Ranking of error categories on sub-group level.
- Ranking of risk categories on main-group level.

Ranking of both the sub- and main categories is based upon a formula depending of the number of authors which mentioned the risk and the corresponding percentages. Ranking varies from 1 for the highest score to 5 or 10 for the lowest score. This formula is given in equation 34.

$$R = N \cdot \sum_{1}^{8} P_i \tag{34}$$

N	number of authors which mention the risk category
P_i	percentage of cases which mentioned the risk category within each research

There are six main categories identified, which were all mentioned within the literature. There are 36 sub-categories identified, of which 14 categories are mentioned within the literature.

analysis method question 2 (causes of error)

The second research question is: 'How could these errors have occurred? '. For analysing this question a slightly different approach is used. The list of risks are not based on the 'design facility' model of Sanvido et al. (1990). Instead, the list is composed out of categories mentioned in the eight investigated researches. Ranking of the categories is based on the same formula as presented in the analysis method of question 1 (equation 34). There are 12 categories identified within the literature.

analysis method question 3 (affected elements)

The third research question is: 'Which building elements were involved in the failure? '.

The research is based on categorization of building elements which are present within a standard building type, such as an office building. Ranking of the categories is based on equation 34. Within this question 8 categories identified, complemented with a category unknown for cases where no information about the type of affected element was provided.

If ranking was not applied but a percentage of the total number of cases, a slight different order would occur. This is caused by the fact that slabs and plates are not mentioned by Fruhwald et al. (2007), which is quite logical as this research is based on timber structures. However this does not have large consequences for the conclusion.

	Type of errors occurring	Boot (2010)		Nelisse & Dieteren		Fruhwald et al. (2007)[a]		Matousek & Schneider (1976)		Walker (1981)[b]		Eldukair & Ayyub (1991)		Allen (1979)[c]		Hadi-priono (1985)		Analyse		
		[No.]	[%]	[No.]	[%]	[No.]	[%]	[No.]	[%]	[No.]	[%]	[No.]	[%]	[No.]	[%]	[No.]	[%]	No. of authors[1]	Ranking sub groups[2]	Ranking main groups[3]
	Number of cases	151		189		127		295		120		604		188		150				
1	**Error in understanding functional requirements**							15	7									1		**5**
1 1 1	Error in analysing information																			
1 1 2	Error in defining requirements and limits																			
1 2 1	Error in selecting Objectives							15	7									1		
1 2 2	Conflicting objectives																			
1 3 1	Insufficient cost, schedules and quality demands																			
1 3 2	Conflicting cost, schedules and quality demands																			
2	**Error in exploring concept**			2	1													1		**4**
2 1 1	Error in design code reviews																			
2 1 2	Conflicting code requirements			2	1													1		
2 2 1	poor investigation of different system types																			
2 2 2	Poor investigation of material types																			
2 3 1	Poor concept coordination																			
2 4 1	Wrong selection of concepts																			
3	**Error in System's schematics**	33	38					134	50					9	5			3		**3**
3 1 1	Error in structural/mechanical system choice	33	38					100	34									2	3	
3 1 2	Error in technical parameters (ground etc.,)							34	16					9	5			2	8	
3 2 1	Poor coordination on system's schematics																			
3 2 2	Conflicts within system's schematics																			
3 3 1	Poor documentation of coordinated schemes																			
3 4 1	Poor selection of system's schematics																			
4	**Error in design (in general)**	46	53	91	48	90 d	71	100	34	60	50	299	50	103	55	43	29	8		**1**
4 1 1	Error in material selection																			
4 1 2	Error in analysing quality/quantity material			2	1									3	2			2		

Type of errors occurring	Boot (2010)		Nelisse & Dieteren		Fruhwald et al. (2007)[a]		Matousek & Schneider (1976)		Walker (1981)[b]		Eldukair & Ayyub (1991)		Allen (1979)[c]		Hadi-priono (1985)		Analyse		
	[No.	%]	[No.	%]	[No.]	[%]	[No.]	[%]	[No.	%]	[No.]	[%]	[No.	%]	[No.	%]	No. of authors[1]	Ranking sub groups[2]	Ranking main groups[3]
4 2 1 Calculation error	46	53	3	2					8	7	15	3	2	1			5	2	
4 2 2 Error in determining loading scenarios			20	11									22	12			2	7	
4 2 3 Error in mechanical schematization/force balance			45	24					52	43	284	47	27	14			4	1	
4 2 4 No calculation update or missing/error in detailed calculation			3	2									25	13	15	10	3	6	
4 3 1 Error in calculation documentation																			
4 4 1 Design does not meet code requirements			18	10									1	1			2	10	
4 4 2 Design does not meet owner requirements																			
5 Error in communicating design to others	7	8	22	12			56	19	11	9	274	45					5		**2**
5 1 1 Error in drawing (wrong measurements etc.)	7	8	5	3			56	19									3	5	
5 2 1 Error in defining contract responsibilities											142	24					1	9	
5 3 1 Error in document coordination among disciplines			17	9					11	9	132	22					3	4	
5 4 1 Insufficient Regulatory control on design																			
6 Error in maintaining design information / models									5	4							1		
6 1 1 Error in data collection																			
6 2 1 Data storage error																			
6 3 1 Insufficient awareness of available knowledge																			
6 4 1 Error in updating information																			
6 5 1 Error in information delivery									5	4							1		

Legend:

[a] Only timber structures considered.

[b] Cited from Fruhwald et al. (2007).

[c] the groups 'bridge decks and pile or pier are left outside this table.

d Sum of the failure categories Design (mechanical loading) and Design (environmental loading), respectively 54,3 and 16,5 percent of the failures,

[1] The number of authors which identified the concerned risk as a cause of failure (N)

[2] and [3] Ranking based on $N \cdot \sum Pi$. Pi is percentage of errors in the considered case in which the type of error occurred.

Scaling from 1 (most important) to 5 or 10 (less important)

	Causes of errors	Boot (2010)		Nelisse & Dieteren		Fruhwald et al. (2007)[a]		Matousek & Schneider (1976)		Walker (1981)[b]		Eldukair & Ayyub (1991)		Allen (1979)[c]		Hadi-priono (1985)		Analysis		
		[No.	%]	[No.	%]	[No.]	[%]	[No.]	[%]	[No.	%]	[No.]	[%]	[No.	%]	[No.	%]	No. of authors [1]	Percentage[2]	Ranking groups[3]
	Number of cases	151		189		127		295		120		604		188		150				
1	Communication errors	12	48									224	37					2	12,9	3
2	Insufficient task division/overview			20	11							199	33					2	12,0	4
3	Lack of authority/guidance											274	45					1	15,0	7
4	Reliance on other parties							21	10			175	29					2	10,8	5
5	Insufficient knowledge/education/qualification	3	12	53	28			76	36			403	67					4	29,3	1
6	Insufficient time	2	8	1	1													2	0,2	10
7	Ignorance	2	8					30	14			495	82					3	28,9	2
8	Mistakes / lapses /slips							28	13									1	1,5	
9	Underestimation (of for instance influences											436	72					1	23,9	6
10	Frugality	5	20					2	1									2	0,4	8
11	Error in software			9	5													1	0,5	
12	Unknown situations											201	33					1	11,0	9
13	Unknown																			

Legend:

[a] Only timber structures considered.

[b] Cited from Fruhwald et al. (2007).

[c] the groups 'bridge decks and pile or pier are left outside this table.

[1] The number of authors which identified the concerned risk as a cause of failure (N)

[2] and [3] Ranking based on $N \cdot \sum P_i$. P_i is percentage of errors in the considered case in which the type of error occurred.

Scaling from 1 (most important) to 5 or 10 (less important)

Building elements affected by error	Boot (2010)		Nelisse & Dieteren (2011)		Fruhwald et al. (2007)[a]		Matousek & Schneider (1976)		Walker (1981)[b]		Eldukair & Ayyub (1991)		Allen (1979)[c]		Hadipriono (1985)		Analyse		
	[No.]	[%]	[No.]	[%]	[No.]	[%]	[No.]	[%]	[No.]	[%]	[No.]	[%]	[No.]	[%]	[No.]	[%]	No. of authors [1]	Percentage[2]	Ranking main groups[3]
Number of cases		151		189		127		295		120		604		188		150			
1 Foundation (soil, raft footings)	20	13	44	23							37	6	13	7			4	9,0	**4**
1 1 Foundation pile			9	5													1	0,7	
1 2 Foundation beam			12	6													1	1,0	
1 3 Strip footing			3	2													1	0,2	
2 Vertical elements	22	15	43	23	5	4					64	11	18	10			5	12,1	**3**
2 1 columns	1	1	23 [d]	12	5	4							10	5			4	3,1	
2 2 piles																			
2 3 walls	21	14	20 [d]	11									8	4			3	3,9	
3 Beams and trusses	10	7	25	13	116	91					65	11	23	12			5	19,0	**1**
3 1 Beams	7	5	25 [d]	13	60	47							23	12			4	9,1	
3 2 Trusses	3	2			43	34											2	3,7	
3 3 Frames & Arches					13	10											1	1,0	
4 Slabs and plates	17	11	49 [d]	26							206	34	41	22			4	24,9	**2**
5 Stability elements			30	16	37	29											2	5,3	
5 1 Bracings			3	2	37	29											2	3,2	
5 2 Stability construction			27 [d]	14													1	2,1	
6 Roof structures	27	18	9	5													2	2,9	
7 Connections	3	2			29	23					53	8,8	28	15			4	9,0	**5**
8 Finishing structures	27	18	12 [d]	6													2	3,1	
9 Unknown	23	15	9 [d]	5							204	34					3	18,8	

Legend:

[a] Only timber structures considered.

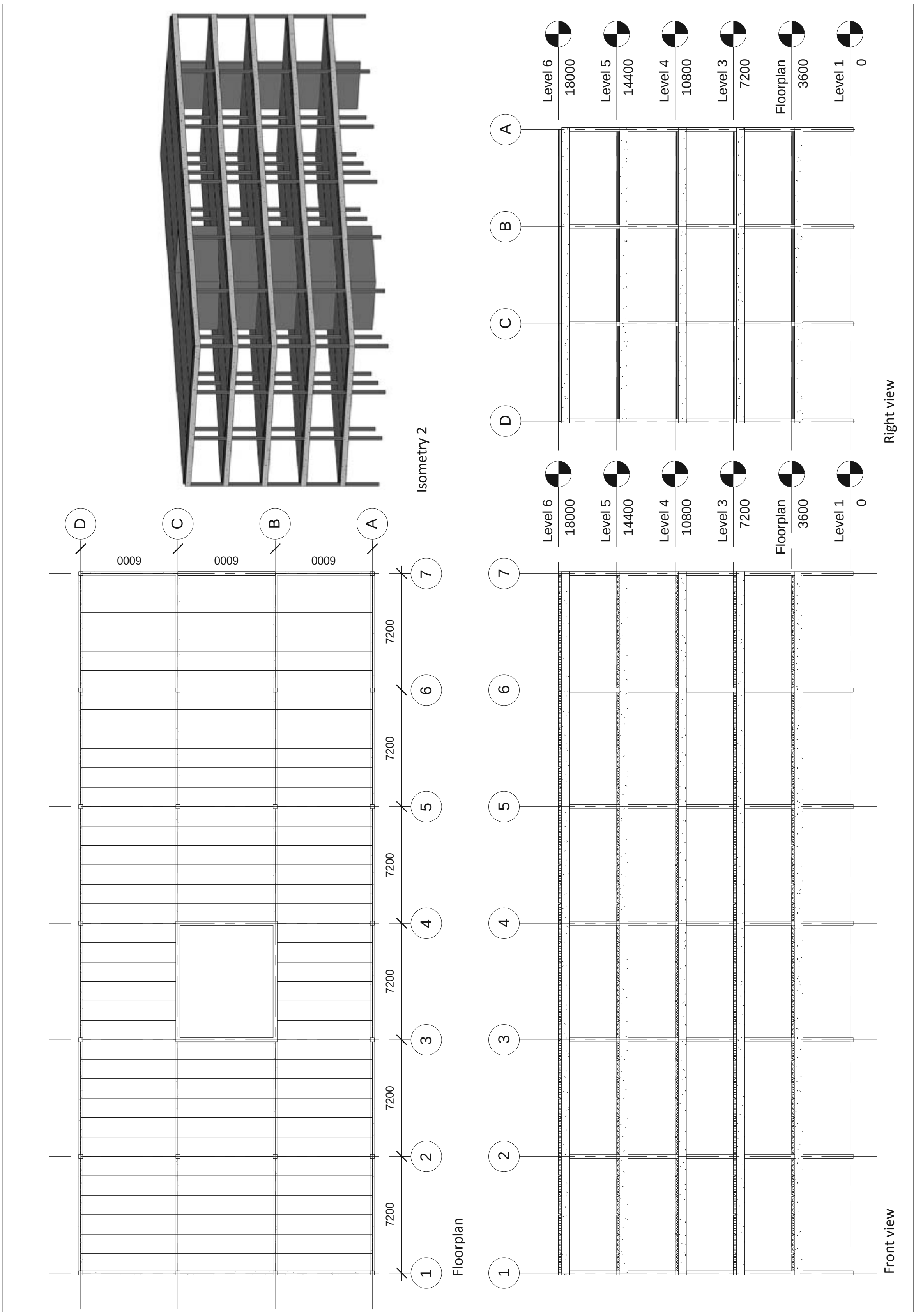
[c] the groups 'bridge decks and pile or pier are left outside this table.

[d] Subdivision within research is altered for use in this report.

[1] The number of authors which identified the concerned risk as a cause of failure (N)

[2] The percentage of cases which mentioned the error as a function of the total number of cases

[2] and [3] Ranking based on $N \cdot \sum Pi$. Pi is percentage of errors in the considered case in which the type of error occurred.



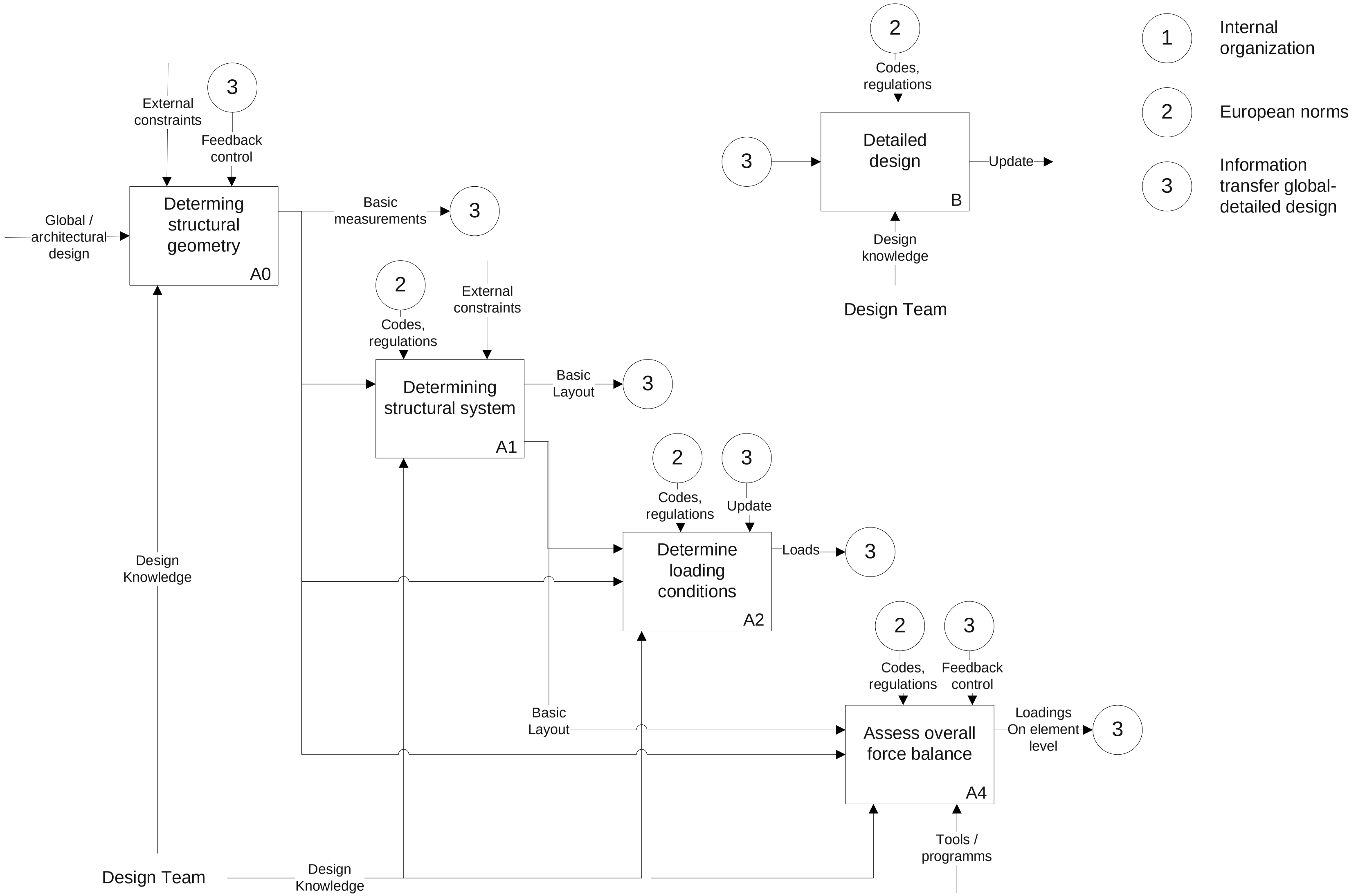
Isometry 2
Floorplan
Right view
Front view
Level 6 18000
Level 5 14400
Level 4 10800
Level 3 7200
Floorplan 3600
Level 1 0
A
B
C
D
1
2
3
4
5
6
7
6000
7200

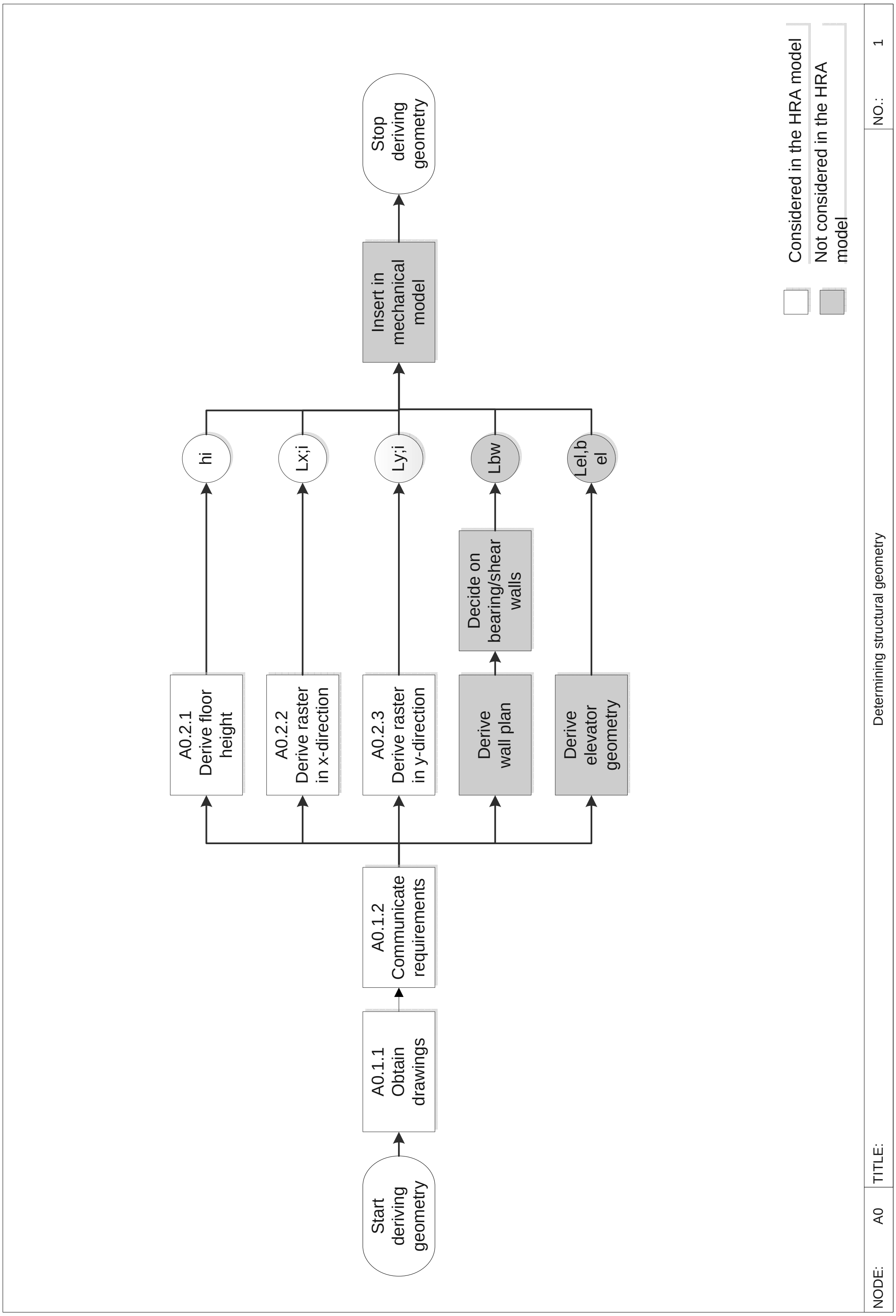
C

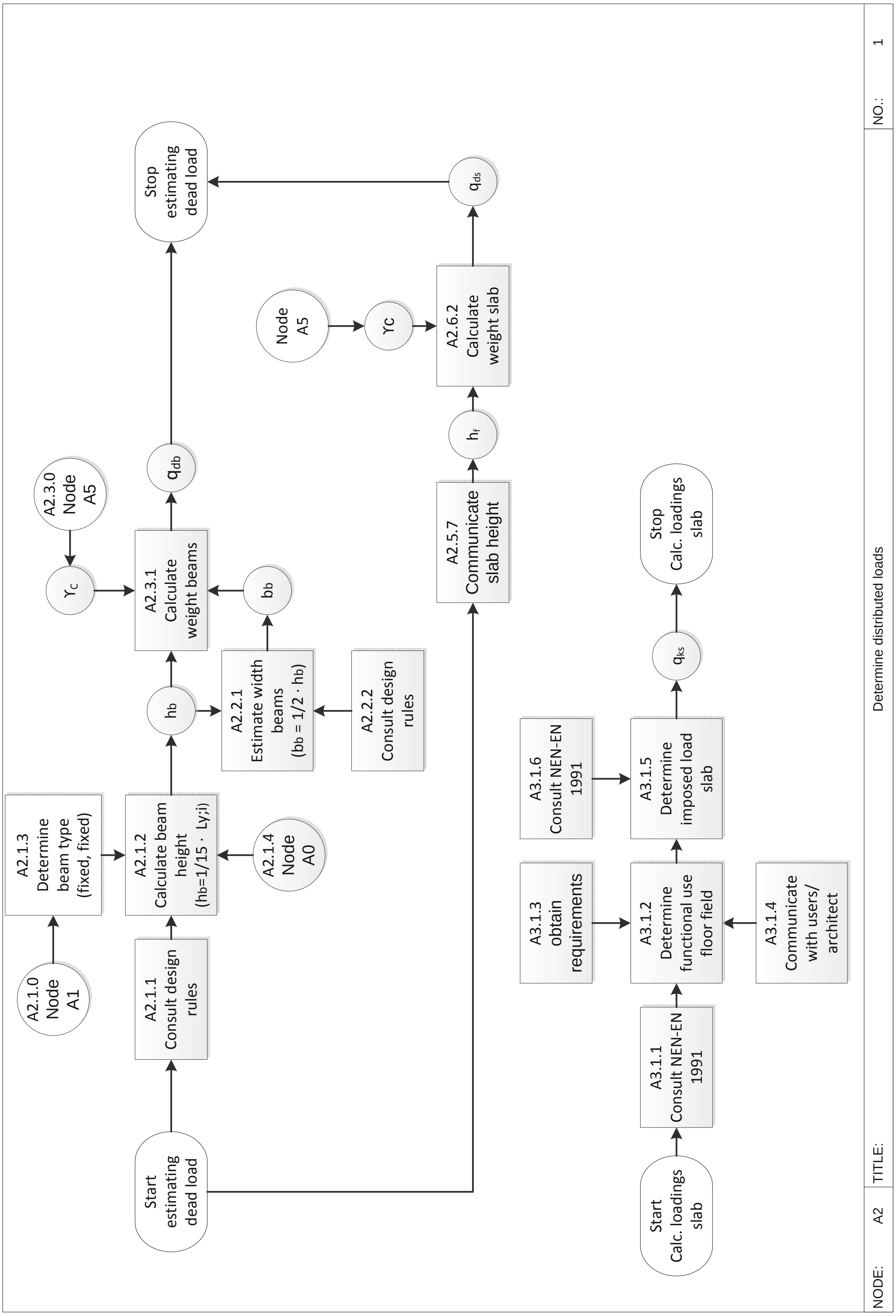
DESIGN PROCESS

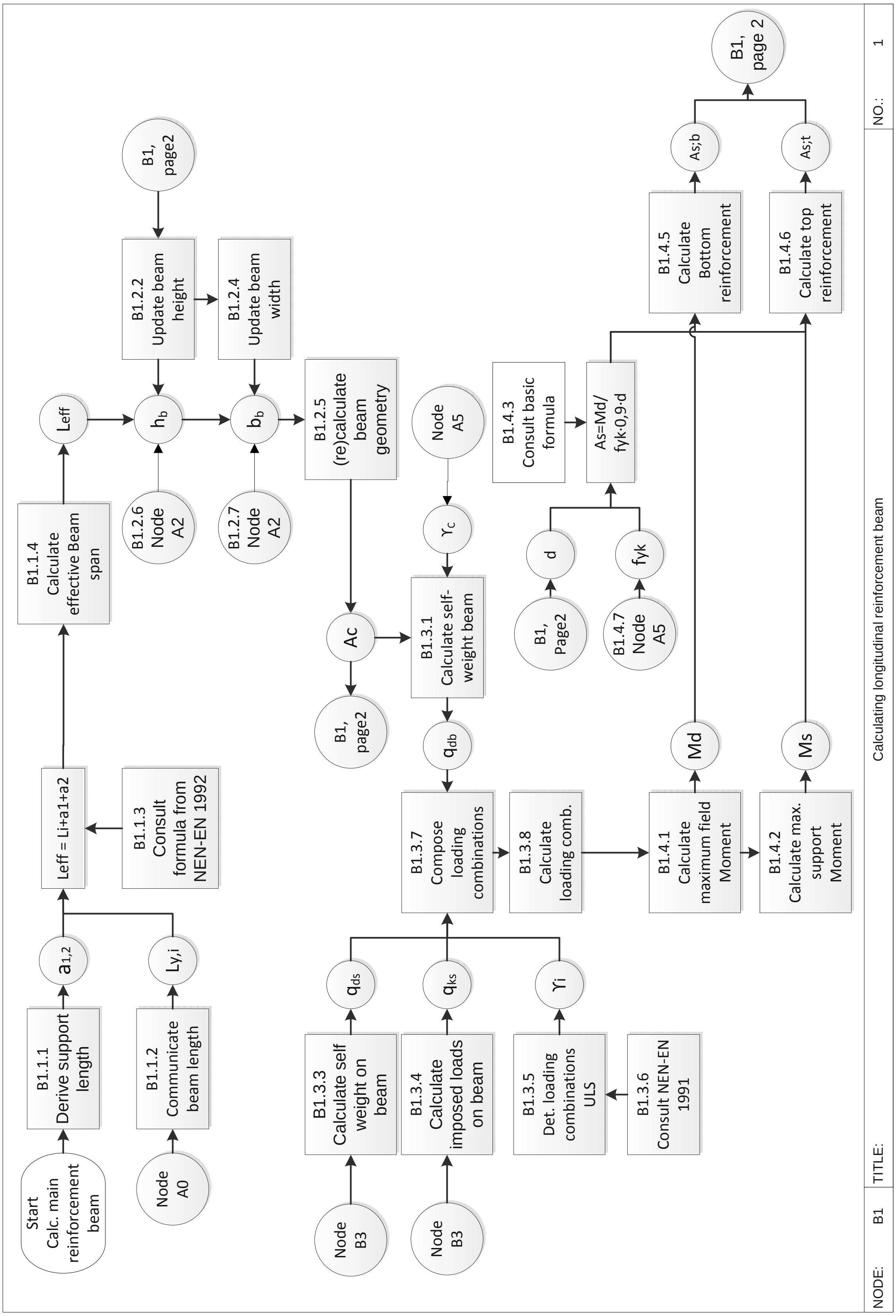
Contents

Global design (abbreviated)

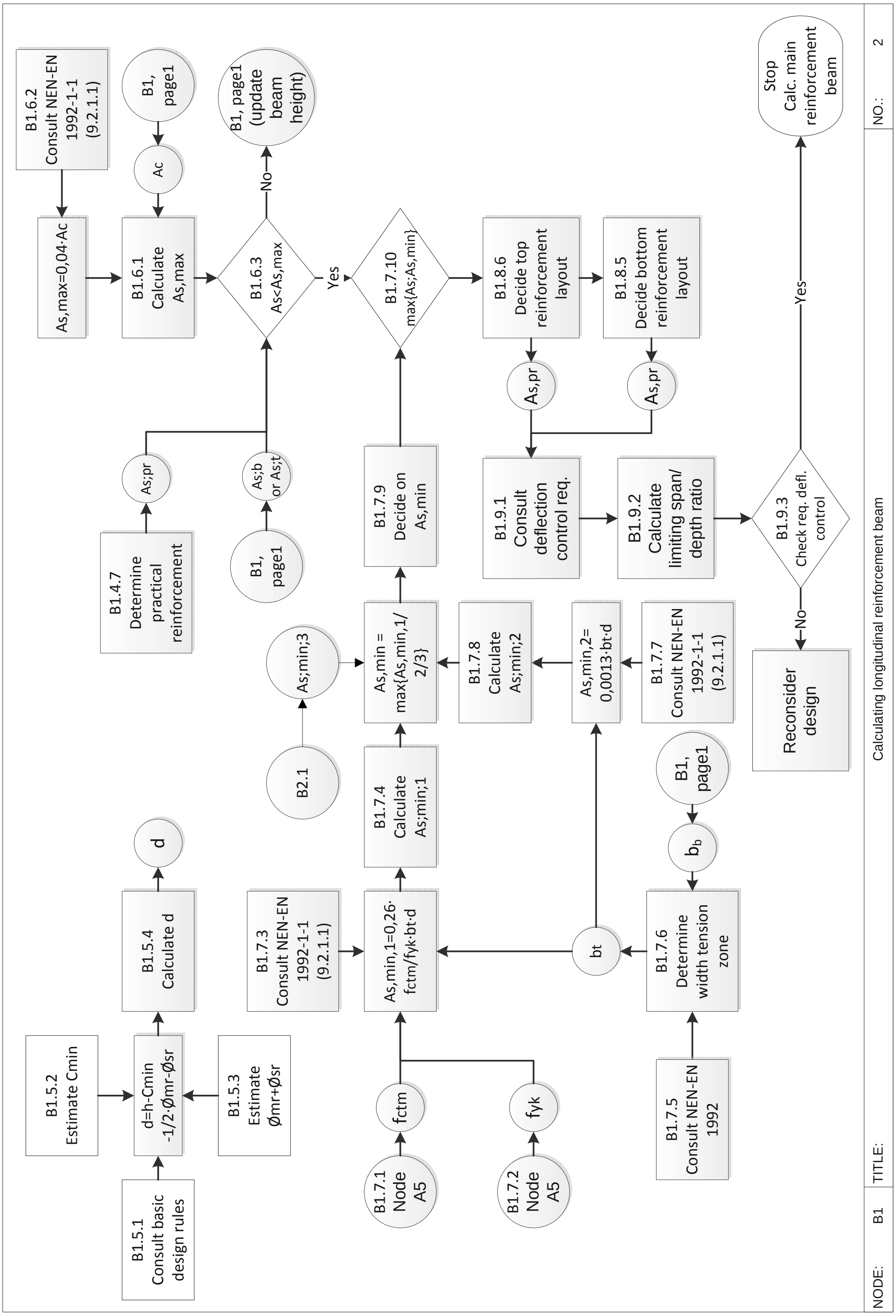



Start deriving geometry
A0.1.1 Obtain drawings
A0.1.2 Communicate requirements
A0.2.1 Derive floor height
A0.2.2 Derive raster in x-direction
A0.2.3 Derive raster in y-direction
Derive wall plan
Derive elevator geometry
Decide on bearing/shear walls
hi
Lx;i
Ly;i
Lbw
Lel,b el
Insert in mechanical model
Stop deriving geometry
Considered in the HRA model
Not considered in the HRA model
NODE:
A0
TITLE:
Determining structural geometry
NO.:
1

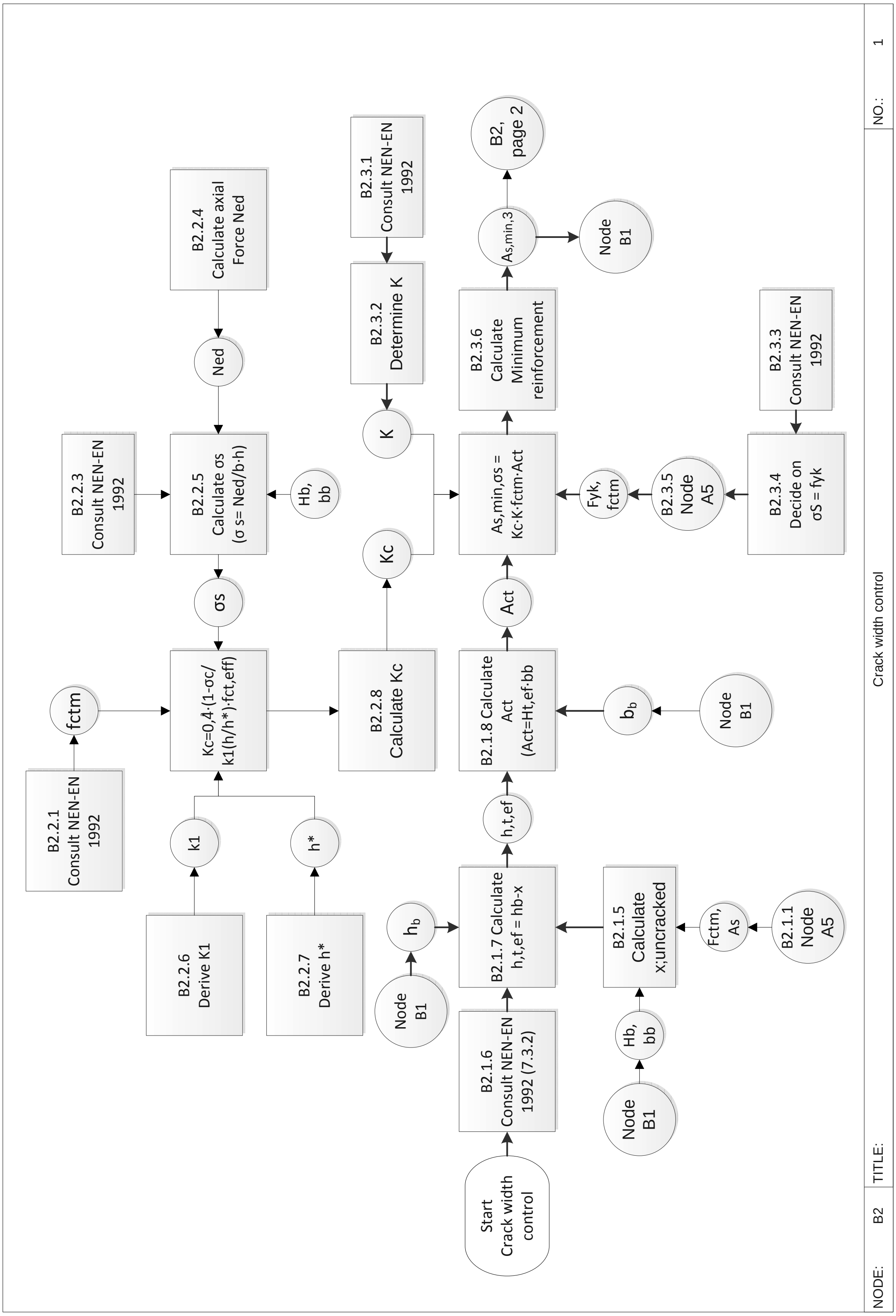

Start estimating dead load
A2.1.1 Consult design rules
A2.1.0 Node A1
A2.1.3 Determine beam type (fixed, fixed)
A2.1.2 Calculate beam height (hb=1/15 · Ly;i)
A2.1.4 Node A0
hb
A2.2.1 Estimate width beams (bb = 1/2 · hb)
A2.2.2 Consult design rules
bb
A2.3.0 Node A5
γC
A2.3.1 Calculate weight beams
qdb
Stop estimating dead load
A2.5.7 Communicate slab height
hf
A2.6.2 Calculate weight slab
Node A5
γC
qds
Start Calc. loadings slab
A3.1.1 Consult NEN-EN 1991
A3.1.3 obtain requirements
A3.1.2 Determine functional use floor field
A3.1.4 Communicate with users/ architect
A3.1.6 Consult NEN-EN 1991
A3.1.5 Determine imposed load slab
qks
Stop Calc. loadings slab
NODE: A2
TITLE: Determine distributed loads
NO.: 1

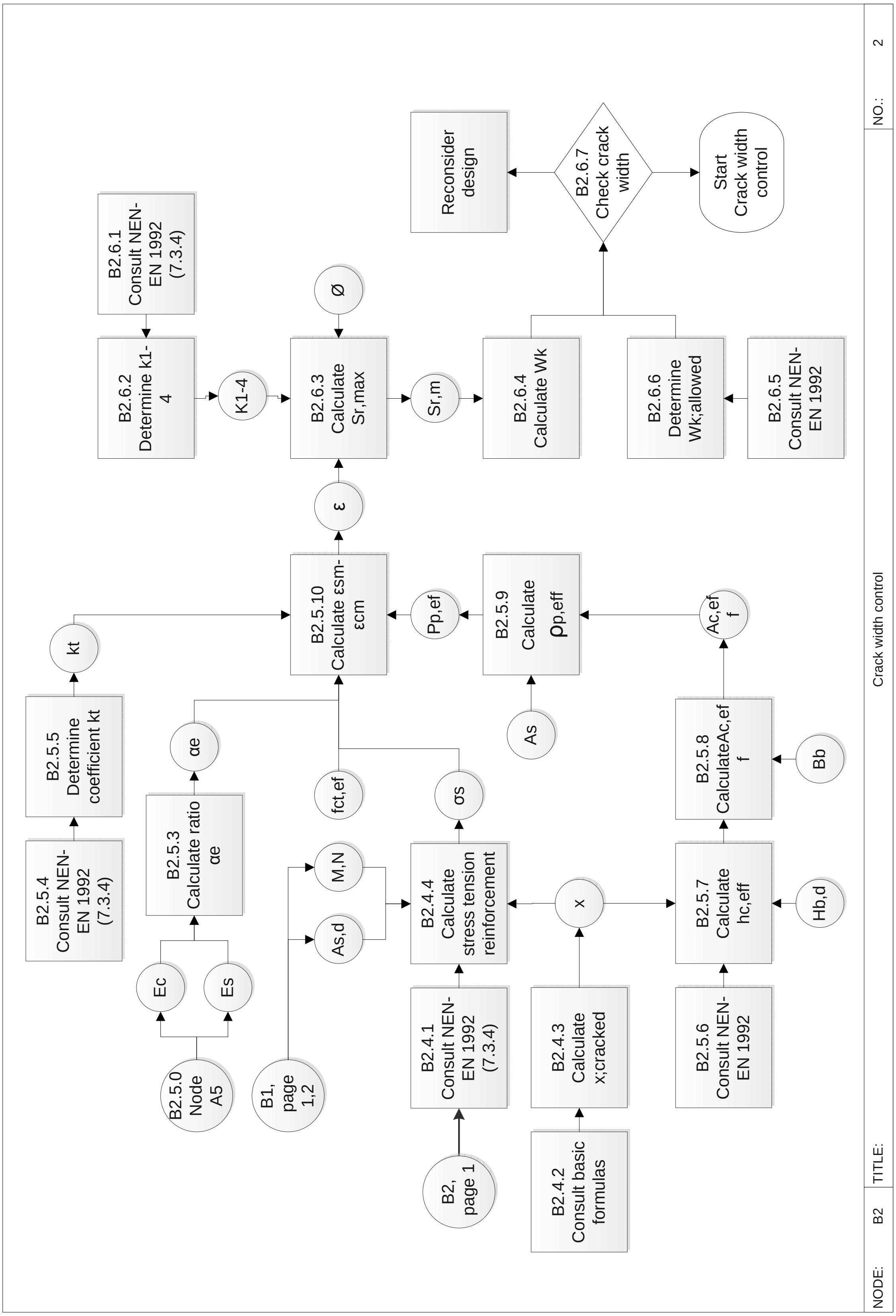


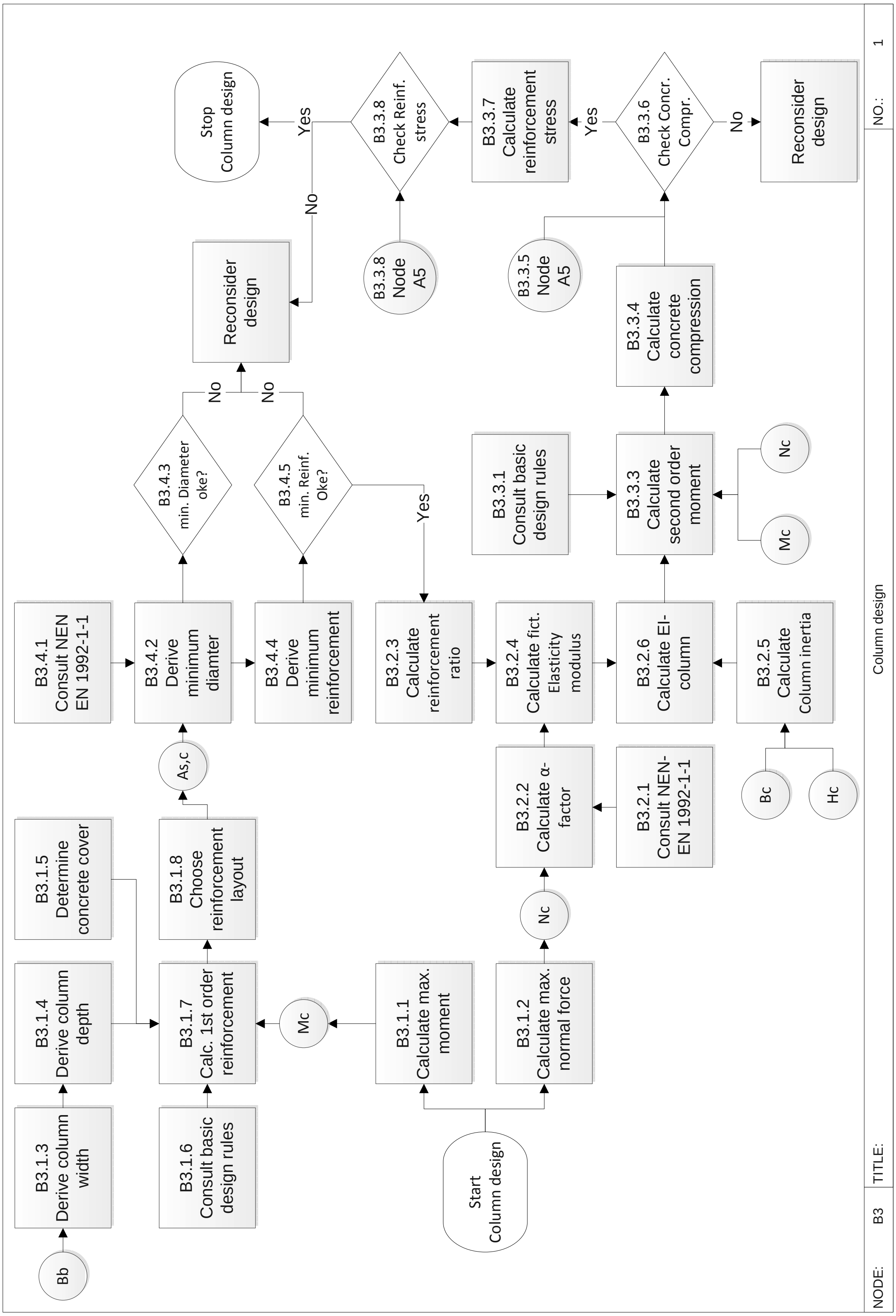
Start Calc. main reinforcement beam
B1.1.1 Derive support length
$a_{1,2}$
Node A0
B1.1.2 Communicate beam length
Ly,i
Leff = Li+a1+a2
B1.1.3 Consult formula from NEN-EN 1992
B1.1.4 Calculate effective Beam span
L_{eff}
h_b
b_b
B1.2.6 Node A2
B1.2.7 Node A2
B1, page2
B1.2.2 Update beam height
B1.2.4 Update beam width
B1.2.5 (re)calculate beam geometry
Ac
B1, page2
B1.3.1 Calculate self-weight beam
γ_C
Node A5
q_{db}
Node B3
B1.3.3 Calculate self weight on beam
q_{ds}
Node B3
B1.3.4 Calculate imposed loads on beam
q_{ks}
B1.3.5 Det. loading combinations ULS
γ_i
B1.3.6 Consult NEN-EN 1991
B1.3.7 Compose loading combinations
B1.3.8 Calculate loading comb.
B1.4.1 Calculate maximum field Moment
Md
B1.4.2 Calculate max. support Moment
Ms
B1, Page2
d
B1.4.7 Node A5
fyk
B1.4.3 Consult basic formula
As=Md/ fyk·0,9·d
B1.4.5 Calculate Bottom reinforcement
As;b
B1.4.6 Calculate top reinforcement
As;t
B1, page 2
NODE: B1
TITLE: Calculating longitudinal reinforcement beam
NO.: 1

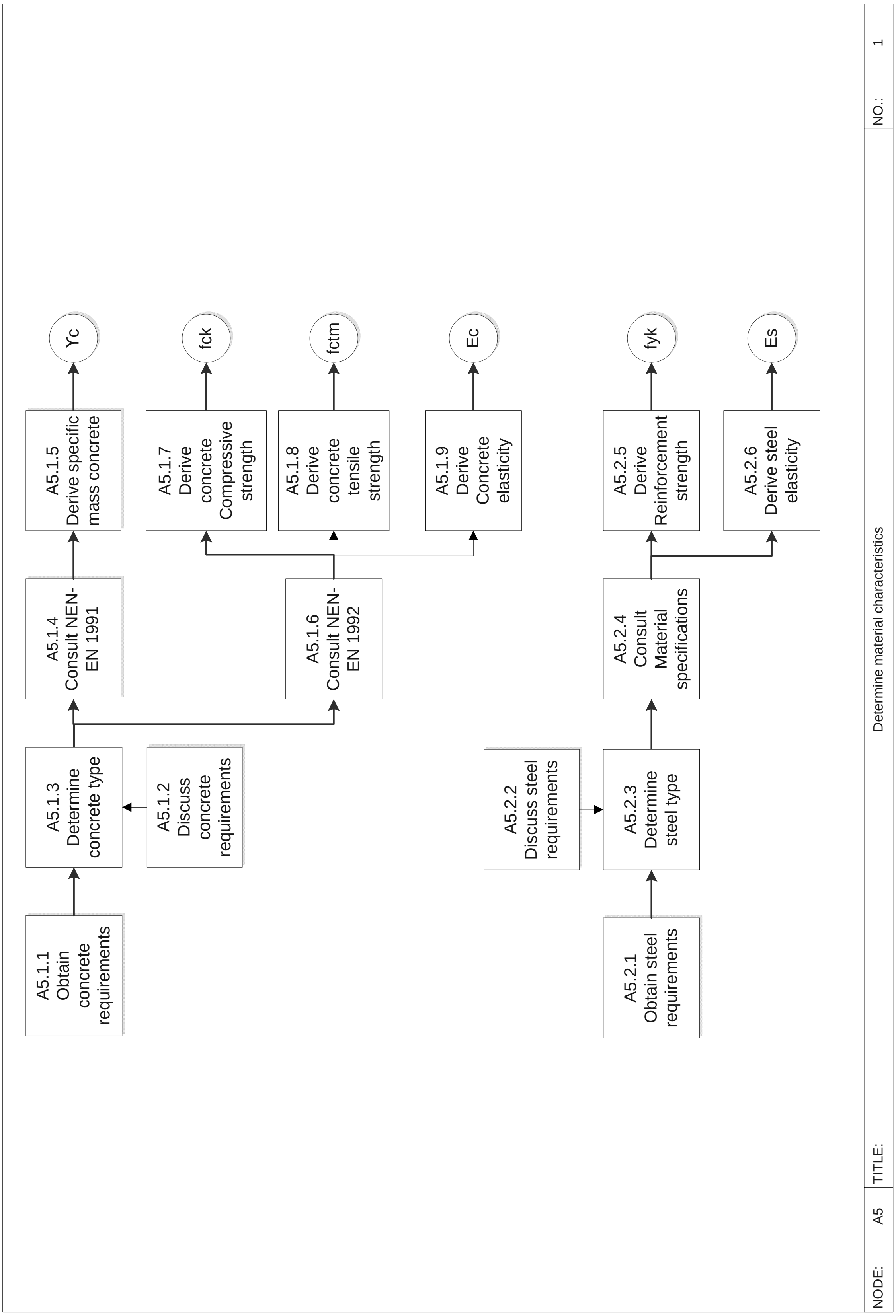


B1.6.2 Consult NEN-EN 1992-1-1 (9.2.1.1)
As,max=0,04·Ac
B1, page1
Ac
B1.6.1 Calculate As,max
B1.6.3 As<As,max
No
B1, page1 (update beam height)
Yes
B1.7.10 max{As;As,min}
B1.8.6 Decide top reinforcement layout
B1.8.5 Decide bottom reinforcement layout
As,pr
As,pr
Stop Calc. main reinforcement beam
B1.4.7 Determine practical reinforcement
As;pr
As;b or As;t
B1, page1
B1.7.9 Decide on As,min
B1.9.1 Consult deflection control req.
B1.9.2 Calculate limiting span/ depth ratio
B1.9.3 Check req. defl. control
Yes
No
Reconsider design
As;min;3
B2.1
As,min = max{As,min,1/ 2/3}
B1.7.8 Calculate As;min;2
As,min,2= 0,0013·bt·d
B1.7.7 Consult NEN-EN 1992-1-1 (9.2.1.1)
B1.7.4 Calculate As;min;1
B1, page1
b_b
d
B1.5.4 Calculate d
B1.7.3 Consult NEN-EN 1992-1-1 (9.2.1.1)
As,min,1=0,26· fctm/fyk·bt·d
bt
B1.7.6 Determine width tension zone
B1.5.2 Estimate Cmin
d=h-Cmin -1/2·Ømr-Øsr
B1.5.3 Estimate Ømr+Øsr
fctm
fyk
B1.7.5 Consult NEN-EN 1992
B1.5.1 Consult basic design rules
B1.7.1 Node A5
B1.7.2 Node A5
NODE: B1
TITLE: Calculating longitudinal reinforcement beam
NO.: 2


NODE: B2
TITLE: Crack width control
NO.: 1
Start Crack width control
B2.1.6 Consult NEN-EN 1992 (7.3.2)
Node B1
hb
B2.1.7 Calculate h,t,ef = hb-x
Node B1
Hb, bb
B2.1.5 Calculate x;uncracked
Fctm, As
B2.1.1 Node A5
h,t,ef
B2.1.8 Calculate Act (Act=Ht,ef·bb
bb
Node B1
Act
As,min,σs = Kc·K·fctm·Act
Fyk, fctm
B2.3.5 Node A5
B2.3.4 Decide on σS = fyk
B2.3.3 Consult NEN-EN 1992
B2.3.6 Calculate Minimum reinforcement
As,min,3
Node B1
B2, page 2
B2.3.1 Consult NEN-EN 1992
B2.3.2 Determine K
K
Kc
B2.2.8 Calculate Kc
B2.2.6 Derive K1
B2.2.7 Derive h*
k1
h*
Kc=0,4·(1-σc/ k1(h/h*)·fct,eff)
B2.2.1 Consult NEN-EN 1992
fctm
σs
B2.2.5 Calculate σs (σ s= Ned/b·h)
Hb, bb
B2.2.3 Consult NEN-EN 1992
Ned
B2.2.4 Calculate axial Force Ned


NODE:
B2
TITLE:
Crack width control
NO.:
2
B2, page 1
B2.4.1 Consult NEN-EN 1992 (7.3.4)
B2.4.2 Consult basic formulas
B2.4.3 Calculate x;cracked
x
B2.4.4 Calculate stress tension reinforcement
As,d
M,N
B1, page 1,2
σs
fct,ef
B2.5.0 Node A5
Ec
Es
B2.5.3 Calculate ratio αe
αe
B2.5.4 Consult NEN-EN 1992 (7.3.4)
B2.5.5 Determine coefficient kt
kt
B2.5.6 Consult NEN-EN 1992
B2.5.7 Calculate hc,eff
Hb,d
B2.5.8 CalculateAc,ef f
Bb
Ac,ef f
B2.5.9 Calculate ρp,eff
As
Pp,ef
B2.5.10 Calculate εsm-εcm
ε
B2.6.1 Consult NEN-EN 1992 (7.3.4)
B2.6.2 Determine k1-4
K1-4
Ø
B2.6.3 Calculate Sr,max
Sr,m
B2.6.4 Calculate Wk
B2.6.5 Consult NEN-EN 1992
B2.6.6 Determine Wk;allowed
B2.6.7 Check crack width
Reconsider design
Start Crack width control


Start Column design
B3.1.1 Calculate max. moment
B3.1.2 Calculate max. normal force
Mc
Nc
Bb
B3.1.3 Derive column width
B3.1.4 Derive column depth
B3.1.5 Determine concrete cover
B3.1.6 Consult basic design rules
B3.1.7 Calc. 1st order reinforcement
B3.1.8 Choose reinforcement layout
As,c
B3.2.1 Consult NEN-EN 1992-1-1
B3.2.2 Calculate α-factor
B3.2.3 Calculate reinforcement ratio
B3.2.4 Calculate fict. Elasticity modulus
B3.2.5 Calculate Column inertia
Bc
Hc
B3.2.6 Calculate EI-column
B3.3.1 Consult basic design rules
B3.3.3 Calculate second order moment
Mc
Nc
B3.3.4 Calculate concrete compression
B3.3.5 Node A5
B3.3.6 Check Concr. Compr.
Yes
No
Reconsider design
B3.3.7 Calculate reinforcement stress
B3.3.8 Node A5
B3.3.8 Check Reinf. stress
Yes
No
Stop Column design
Reconsider design
B3.4.1 Consult NEN EN 1992-1-1
B3.4.2 Derive minimum diamter
B3.4.3 min. Diameter oke?
B3.4.4 Derive minimum reinforcement
B3.4.5 min. Reinf. Oke?
Yes
No
No
NODE: B3
TITLE: Column design
NO.: 1


A5.1.1 Obtain concrete requirements
A5.1.2 Discuss concrete requirements
A5.1.3 Determine concrete type
A5.1.4 Consult NEN-EN 1991
A5.1.5 Derive specific mass concrete
Yc
A5.1.6 Consult NEN-EN 1992
A5.1.7 Derive concrete Compressive strength
fck
A5.1.8 Derive concrete tensile strength
fctm
A5.1.9 Derive Concrete elasticity
Ec
A5.2.1 Obtain steel requirements
A5.2.2 Discuss steel requirements
A5.2.3 Determine steel type
A5.2.4 Consult Material specifications
A5.2.5 Derive Reinforcement strength
fyk
A5.2.6 Derive steel elasticity
Es
NODE: A5
TITLE: Determine material characteristics
NO.: 1

D

EXTENDED CREAM METHOD

Contents

introduction

Within this appendix the extended HEP quantification method used within this research is set apart. The extended HEP quantification method proposed in this research is derived from the extended quantification method within the CREAM methodology. This method is presented in chapter 9 of Hollnagel (1998). The extended HEP method is meant to determine the Human Error Probability (HEP) of each basic design task.

basic methodology

The extended HEP quantification method requires the completion of four distinctive steps. The basic idea behind these steps is to subdivide each basic task into basic cognitive tasks, on which standard failure probabilities are defined. The input of this procedure consists of the identified design steps, the context of the design and classification diagrams from the CREAM methodology. The four steps of the extended HEP method are given beneath.

Build a cognitive demand profile.
In this step the description of the basic task is refined by identifying a list of cognitive activities that characterise each basic task. The cognitive activities are then used to build a cognitive demand profile of the basic task, based on the functions described by the CREAM model.

Identify likely cognitive function failure.
In this step the cognitive function failures that could occur, are determined. This action transforms the cognitive demand profile into a cognitive failure function, and subsequently generates a value for each cognitive function failure.

Assess Common Performance Conditions (CPC).
In this step the conditions under which the performance is expected to take place is characterised. This characterisation is expressed by means of a combined CPC score, resulting in a weighting factor in the HEP calculation.

Determine error probability.
In this step the basic failure probabilities of each cognitive activity is multiplied with the corresponding weighting factor. The overall failure probability of the basic task is subsequently calculated by adding these basic failure probabilities based on full independence between the cognitive activities $(1 - \prod_{i=1}^{n}(1 - P_i))$.

The steps are discussed in more detail in the following four sections. A detailed flow chart of these four steps is presented in figure 49, in order to get an idea of the process and its input and output values. It can be seen from this figure that in total four classification diagrams from the CREAM method are used within the procedure, which are presented in this appendix. The output consists of three parts: a cognitive demand profile, a cognitive function failure and a failure probability. The first two products are intermediate results, while the latter is the end result.

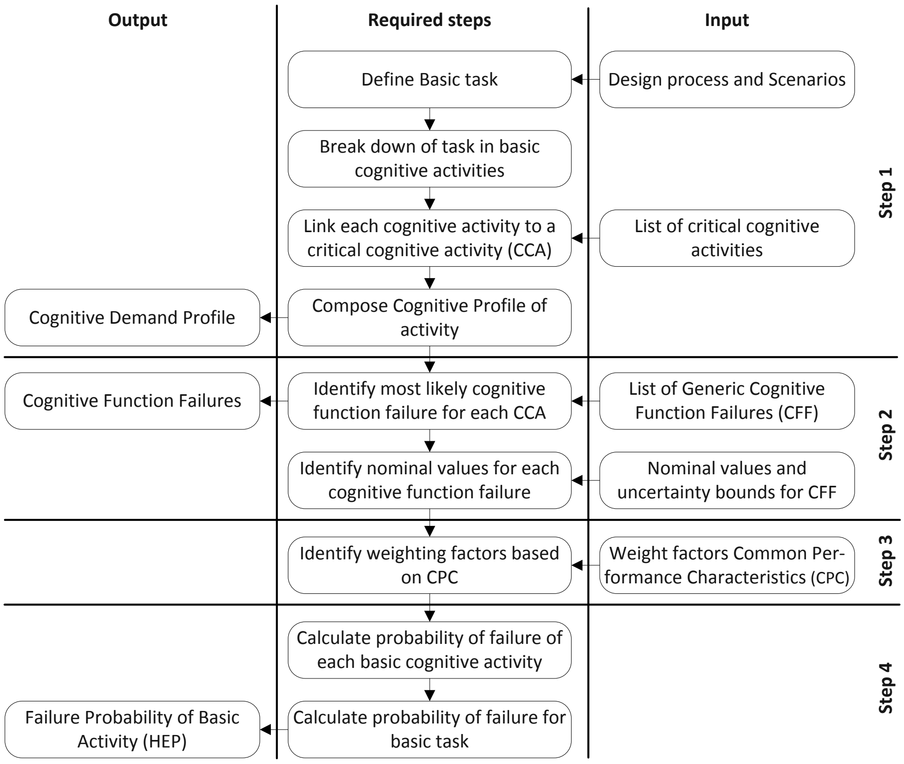


Figure 49: Flow Chart of CREAM quantification method for the Human Error Probability

build a cognitive demand profile

The first step in the CREAM method is to build a cognitive demands profile. The purpose of a cognitive demands profile is to show the specific demands to cognition that are associated with a task segment or a task step. This step is meant to characterise the task steps in terms of the cognitive activities they involve. This is basically an addition to the event sequence description that categorises each task step using a list of characteristic cognitive activities. The list of characteristic cognitive activities is shown in table 21. The purpose of the cognitive profile is to represent the demand characteristics of the task and sub-tasks, and to indicate the kind of failures that should be expected.

Table 21: List of critical cognitive activities. Based on the table presented by Hollnagel (1998) on page 246.

Cognitive activity	General definition
Coordinate	Bring design state and/or control configurations into the specific relation required to carry out a task or task step. Allocate or select resources in preparation for a task/job, etc.
Communicate	Pass on or receive person-to-person information needed for system operation by either verbal, electronic or mechanical means. Communication is an essential part of management.
Compare	Examine the qualities of two or more entities (measurements) with the aim of discovering similarities or differences. The comparison may require calculation.
Diagnose	Recognise or determine the nature or cause of a condition by means of reasoning about signs or symptoms or by the performance of appropriate tests. "diagnose" is more thorough than "identify".
Evaluate	Appraise or assess an actual or hypothetical situation, based on available information without requiring special operations. Related terms are "inspect" and "check".
Identify	Establish the state of a design or sub-design. This may involve specific operations to retrieve information and investigate details. "identify" is more thorough than "evaluate".
Monitor	Keep track of the design process, or follow the development of a set of parameters
Observe	Read specific design information or check specific design indicators.
Plan	Formulate or organise a set of actions by which a goal will be successfully achieved. Plans may be short term or long term.
Record	Write down design parameters, measurements, etc.
Scan	Quick or speedy review of information sources in order to obtain a general idea of the design action.
Verify	Confirm the correctness of a design parameter by inspection or test.

The actual cognitive profile is based on a table of the cognitive functions associated with each of the cognitive activities. This is shown in figure 50. The model underlying this table assumes that there are four basic cognitive functions that have to do with observation, interpretation, planning and execution. Each typical cognitive activity can then be described in terms of which combination of the four cognitive functions it requires. As an example, coordination involves planning as well as execution: the planning is used to specify what is to be done, and the execution is used to carry it out or perform it.

Figure 50: Generic cognitive-activity-by-cognitive-demand matrix. Based on the table presented by Hollnagel (1998) on page 248.

Activity type	COCOM function			
	Observation	Interpretation	Planning	Execution
Co-ordinate			X	X
Communicate				X
Compare		X		
Diagnose		X	X	
Evaluate		X	X	
Execute				X
Identify		X		
Monitor	X	X		
Observe	X			
Plan			X	
Record		X		X
Scan	X			
Verify	X	X		

identify cognitive function failure

The second step of the CREAM method is to identify the likely cognitive function failures. These function failures are presented in figure 51. As shown in this table, the cognitive function failures are defined relative to the four cognitive functions in the associated model. The idea of this is to select one predominant type of failure for the task as a whole.

Once the likely cognitive function failures have been assigned for each task element in the task description, it is possible to assess the probability of failure for each cognitive failure type. Within this step the nominal Human Error Probability for each of the likely cognitive function failures are determined. From a review of existing approaches to PSA/HRA a data base with nominal HEPs and uncertainty bounds for most of the generic cognitive function failures. These values are provided in figure 52.

assess common performance conditions (cpc)

The third step is to assess the effects of Common Performance Conditions on the nominal HEP values. The basic notion in the CREAM is that there is a coupling or dependency between the common performance conditions and the cognitive functions, due to the fact that human cognition and performance take place in, hence are determined by, a context. The simplest way to do that is to define a weighting factor for each Common Performance Condition and its categories, which then is applied to the corresponding HEPs. Appropriate weighting factors for all cognitive function failures are defined in figure 53.

Figure 51: Generic cognitive function failures. (Hollnagel, 1998, page 250)

Cognitive function	Potential cognitive function failure	
Observation error	O1	Observation of wrong object. A response is given to the wrong stimulus or event
	O2	Wrong identification made, due to e.g. a mistaken cue or partial identification
	O3	Observation not made (i.e., omission), overlooking a signal or a measurement
Inter-pretation errors	I1	Faulty diagnosis, either a wrong diagnosis or an incomplete diagnosis
	I2	Decision error, either not making a decision or making a wrong or incomplete decision
	I3	Delayed interpretation, i.e., not made in time
Planning error	P1	Priority error, as in selecting the wrong goal (intention)
	P2	Inadequate plan formulated, when the plan is either incomplete or directly wrong
Execution errors	E1	Execution of wrong type performed, with regard to force, distance speed or direction
	E2	Action performed at wrong time, either too early or to late
	E3	Action on wrong object, (neighbour, similar or unrelated)
	E4	Action performed out of sequence, such as repetitions, jumps, and reversals
	E5	Action missed, not performed (i.e. omission), including the omission of the last actions in a series (''undershoot'')

Figure 52: Nominal values and uncertainty bounds for cognitive function failures (Hollnagel, 1998, page 252)

Cognitive function	Generic failure type	Lower bound (.5)	Basic value	Upper bound (.95)
Observation	O1. Wrong object observed	3.0E-4	1.0E-3	3.0E-3
	O2. Wrong identification	2.0E-2	7.0E-2	1.7E-2
	O3. Observation not made	2.0E-2	7.0E-2	1.7E-2
Inter-pretation errors	I1. Faulty diagnosis	9.0E-2	2.0E-1	6.0E-1
	I2. Decision error	1.0E-3	1.0E-2	1.0E-1
	I3. Delayed interpretation	1.0E-3	1.0E-2	1.0E-1
Planning errors	P1. Priority error	1.0E-3	1.0E-2	1.0E-1
	P2. Inadequate plan	1.0E-3	1.0E-2	1.0E-1
Execution errors	E1. Action of wrong type	1.0E-3	3.0E-3	9.0E-3
	E2. Action at wrong time	1.0E-3	3.0E-3	9.0E-3
	E3. Action on wrong object	5.0E-5	5.0E-4	5.0E-3
	E4. Action out of sequence	1.0E-3	3.0E-3	9.0E-3
	E5. Missed action	2.5E-2	3.0E-2	4.0E-2

Determine error probability

In this step the basic failure probabilities of each cognitive activity is multiplied with the corresponding weighting factor. The overall failure probability of the basic task is subsequently calculated by adding these basic failure probabilities based on full independence between the cognitive activities $(1 - \prod_{i=1}^{n}(1 - P_i))$.

Figure 53: Weighting fators for CPCs (Hollnagel, 1998, page 255)

CPC name	Level	COCOM function			
		OBS	INT	PLAN	EXE
Adequacy of organisation	Very efficient	1.0	1.0	0.8	0.8
	Efficient	1.0	1.0	1.0	1.0
	Inefficient	1.0	1.0	1.2	1.2
	Deficient	1.0	1.0	2.0	2.0
Working conditions	Advantageous	0.8	0.8	1.0	0.8
	Compatible	1.0	1.0	1.0	1.0
	Incompatible	2.0	2.0	1.0	2.0
Adequacy of MMI and operational support	Supportive	0.5	1.0	1.0	0.5
	Adequate	1.0	1.0	1.0	1.0
	Tolerable	1.0	1.0	1.0	1.0
	Inappropriate	5.0	1.0	1.0	5.0
Availability of procedures / plans	Appropriate	0.8	1.0	0.5	0.8
	Acceptable	1.0	1.0	1.0	1.0
	Inappropriate	2.0	1.0	5.0	2.0
Number of simultaneous goals	Fewer than capacity	1.0	1.0	1.0	1.0
	Matching current capacity	1.0	1.0	1.0	1.0
	More than capacity	2.0	2.0	5.0	2.0
Available time	Adequate	0.5	0.5	0.5	0.5
	Temporarily inadequate	1.0	1.0	1.0	1.0
	Continuously inadequate	5.0	5.0	5.0	5.0
Time of day	Day-time (adjusted)	1.0	1.0	1.0	1.0
	Night-time (unadjusted)	1.2	1.2	1.2	1.2
Adequacy of training and preparation	Adequate, high experience	0.8	0.5	0.5	0.8
	Adequate, low experience.	1.0	1.0	1.0	1.0
	Inadequate.	2.0	5.0	5.0	2.0
Crew collaboration quality	Very efficient	0.5	0.5	0.5	0.5
	Efficient	1.0	1.0	1.0	1.0
	Inefficient	1.0	1.0	1.0	1.0
	Deficient	2.0	2.0	2.0	5.0

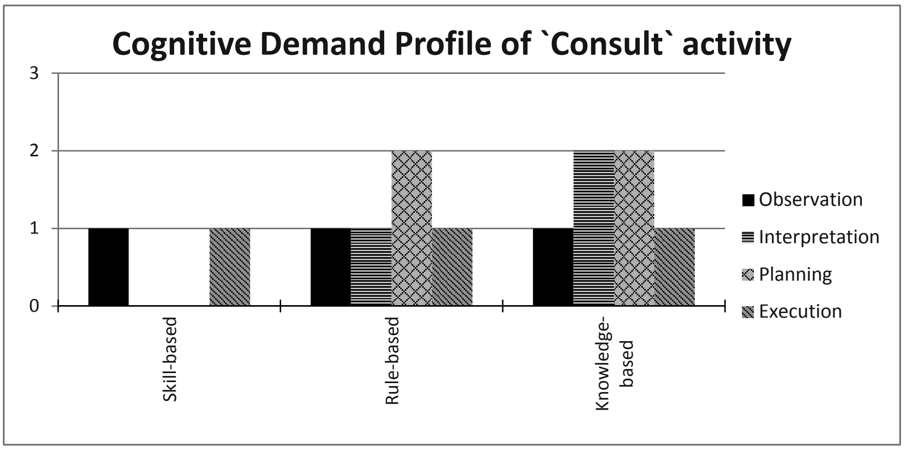
E

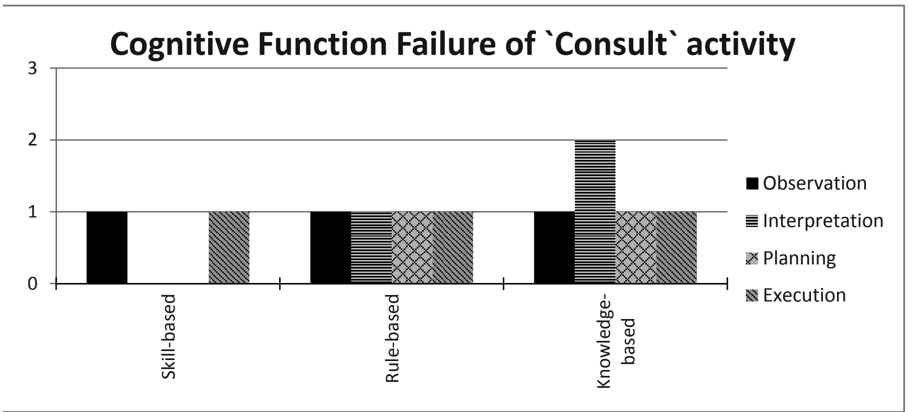
SIMPLIFIED HEP METHOD

Contents

Level	Cognitive activities in the task: `consult`				COCOM[1] function				Credible failure modes													Failure Probabilities		
									Obser-vation			Inter-pretation			Plan-ning		Execution							
	Activity	Task #	Goal	Cognitive activity	Observation	Interpretation	Planning	Execution	O1	O2	O3	I1	I2	I3	P1	P2	E1	E2	E3	E4	E5	Nominal CFP[2]	Weighting factors	Adjusted CFP
Skill-based	Consult	# 1 1	Read text/drawing	Observe	X					X												0,07	0,064	0,0045
Skill-based	Record	# 1 2	Record info/parameters	Execute				X											X			0,0005	0,0512	3E-05
Skill-based	**Failure probability of task execution on Skill-based level**																					**Fully dependent**		**4,48E-03**
																						Independent		**4,51E-03**
Rule-based level	Define activity	# 2 1	Assess task	Plan			X									X						0,01	0,05	0,0005
Rule-based level	Consult	# 2 2	Read text/drawing	Observe	X					X												0,07	0,064	0,0045
Rule-based level		# 2 3	Assess information	Diagnose		X	X					X										0,2	0,1	0,02
Rule-based level	Record	# 2 4	Record info/parameters	Execute				X											X			0,0005	0,0512	3E-05
Rule-based level	**Failure probability of task execution on Rule-based level**																					**Fully dependent**		**2,00E-02**
																						Independent		**2,49E-02**
Knowledge-based level	Define activity	# 3 1	Asses task	Plan			X									X						0,01	0,05	0,0005
Knowledge-based level	Consult	# 3 2	Read text/drawing	Observe	X						X											0,07	0,064	0,0045
Knowledge-based level		# 3 3	Interpret text	Identify		X						X										0,2	0,1	0,02
Knowledge-based level		# 3 4	Deduce required info	Diagnose		X	X					X										0,2	0,1	0,02
Knowledge-based level	Record	# 3 5	Record info/parameters	Execute				X											X			0,0005	0,0512	3E-05
Knowledge-based level	**Failure probability of task execution on Knowledge-based level**																					**Fully dependent**		**2,00E-02**
																						Independent		**4,44E-02**

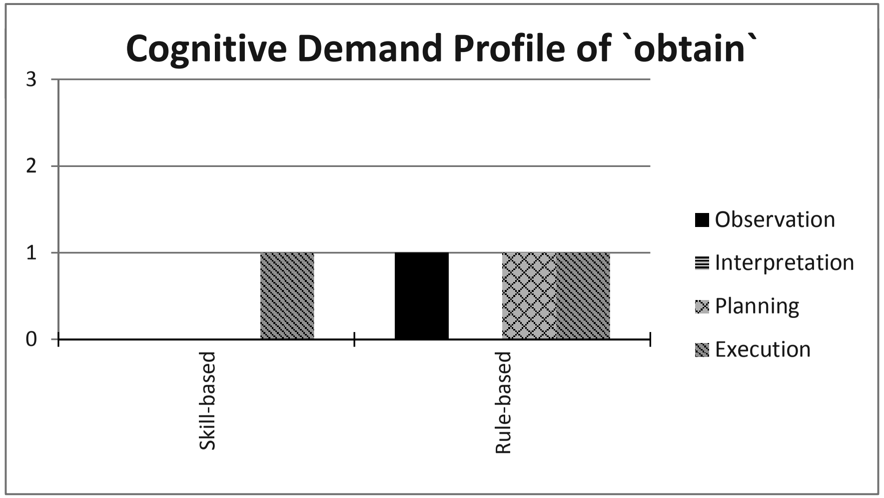
[1] COCOM: Contextual Control Model
[2] CFP: Cognitive Failure Probability

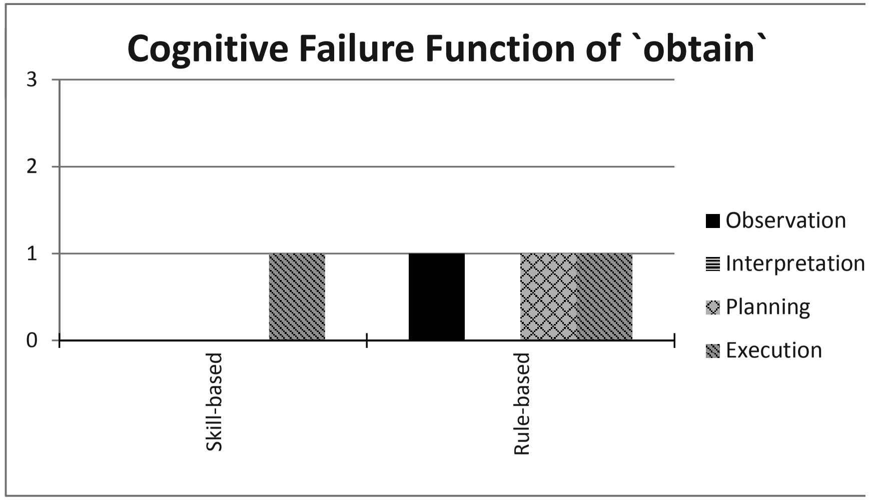

Cognitive Demand Profile of `Consult` activity
3
2
1
0
Skill-based
Rule-based
Knowledge-based
Observation
Interpretation
Planning
Execution


Cognitive Function Failure of `Consult` activity
3
2
1
0
Skill-based
Rule-based
Knowledge-based
Observation
Interpretation
Planning
Execution

Level	Activity	Task #	Goal	Cognitive activity	COCOM[1] function: Observation	Interpretation	Planning	Execution	Credible failure modes: Observation O1	O2	O3	Interpretation I1	I2	I3	Planning P1	P2	Execution E1	E2	E3	E4	E5	Failure Probabilities: Nominal CFP[2]	Weighting factors	Adjusted CFP
Skill-based	Obtain	# 1 1	Obtain information	Execute				X											X			0,0005	0,0512	3E-05
Skill-based	Failure probability of task execution on Skill-based level																					Fully dependent Independent		2,56E-05 2,56E-05
Rule-based level	Define activity	# 2 1	Assess task	Plan			X									X						0,01	0,05	0,0005
Rule-based level	Obtain	# 2 2	Observe information	Observe	X					X												0,07	0,064	0,0045
Rule-based level		# 2 3	Obtain information	Execute				X											X			0,0005	0,0512	3E-05
Rule-based level	Failure probability of task execution on Rule-based level																					Fully dependent Independent		4,48E-03 5,00E-03

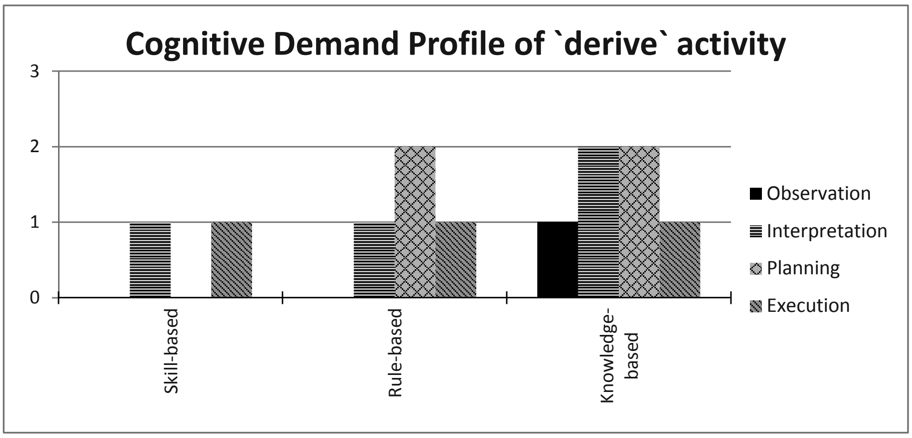
[1] COCOM: Contextual Control Model
[2] CFP: Cognitive Failure Probability





Level	Activity	Task #	Goal	Cognitive activity	COCOM[1] function: Observation	Interpretation	Planning	Execution	Credible failure modes: Observation O1	O2	O3	Interpretation I1	I2	I3	Planning P1	P2	Execution E1	E2	E3	E4	E5	Failure Probabilities: Nominal CFP[2]	Weighting factors	Adjusted CFP
	Cognitive activities in the task: \`derive\`																							
Skill-based	Derive	# 1 1	Derive parameter/info	Identify		X							X									0,01	0,1	0,001
Skill-based	Record	# 1 2	Record parameter/info	Execute				X											X			0,0005	0,0512	3E-05
Skill-based	**Failure probability of task execution on Skill-based level**																					**Fully dependent**		**1,00E-03**
Skill-based																						**Independent**		**1,03E-03**
Rule-based level	Define activity	# 2 1	Assess task	Plan			X									X						0,01	0,05	0,0005
Rule-based level	Derive	# 2 2	Derive parameter/info	Diagnose		X	X						X									0,01	0,1	0,001
Rule-based level	Record	# 2 3	Record parameter/info	Execute				X											X			0,0005	0,0512	3E-05
Rule-based level	**Failure probability of task execution on Rule-based level**																					**Fully dependent**		**1,00E-03**
Rule-based level																						**Independent**		**1,53E-03**
Knowledge-based	Define activity	# 3 1	Assess task	Plan			X									X						0,01	0,05	0,0005
Knowledge-based	Derive	# 3 2	Observe information	Observe	X								X									0,01	0,1	0,001
Knowledge-based		# 3 3	Interpret information	Identify		X						X										0,2	0,1	0,02
Knowledge-based		# 3 4	Derive parameter/info	Diagnose		X	X					X										0,2	0,1	0,02
Knowledge-based	Record	# 3 5	Record parameter/info	Execute				X											X			0,0005	0,0512	3E-05
Knowledge-based	**Failure probability of task execution on Knowledge-based level**																					**Fully dependent**		**2,00E-02**
Knowledge-based																						**Independent**		**4,11E-02**

[1] COCOM: Contextual Control Model
[2] CFP: Cognitive Failure Probability


Cognitive Demand Profile of `derive` activity
3
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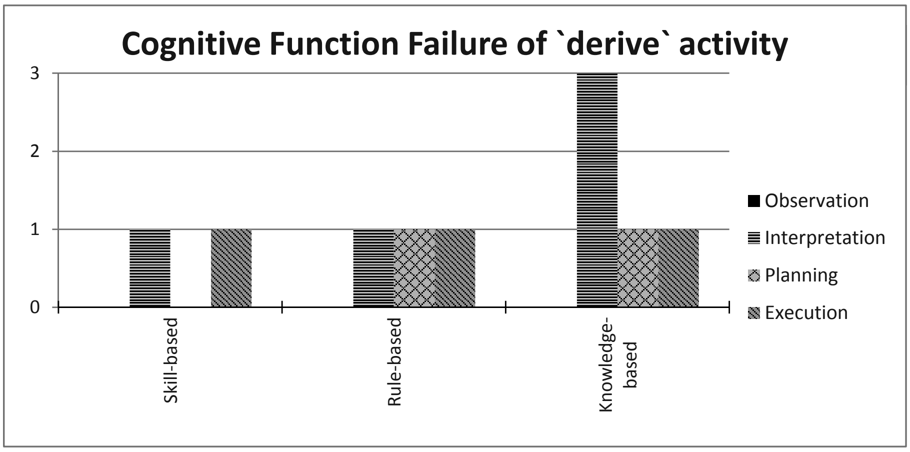
Skill-based
Rule-based
Knowledge-based
Observation
Interpretation
Planning
Execution


Cognitive Function Failure of `derive` activity
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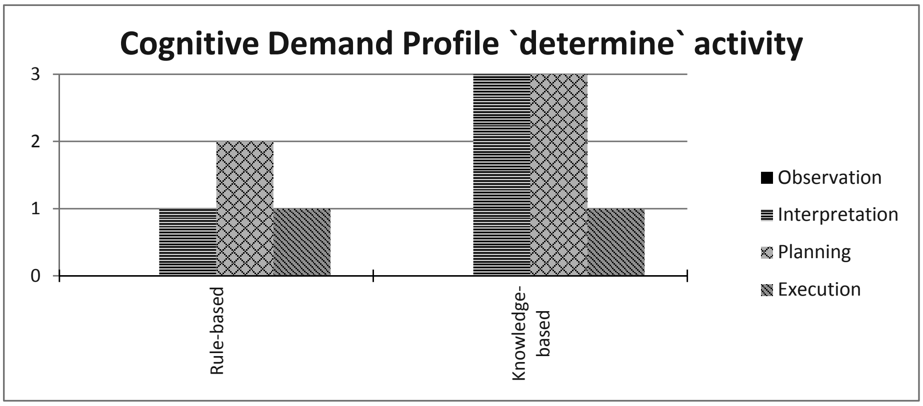
Skill-based
Rule-based
Knowledge-based
Observation
Interpretation
Planning
Execution

Level	Cognitive activities in the task: `determine`				COCOM[1] function				Credible failure modes													Failure Probabilities		
	Activity	Task #	Goal	Cognitive activity	Observation	Interpretation	Planning	Execution	Observation			Interpretation			Planning		Execution					Nominal CFP[2]	Weighting factors	Adjusted CFP
									O1	O2	O3	I1	I2	I3	P1	P2	E1	E2	E3	E4	E5			
Skill-based	Determine	# 1 1	Determine	Identify		X							X									0,01	0,1	0,001
Skill-based	Record	# 1 2	Record decision	Execute				X											X			0,0005	0,0512	3E-05
Failure probability of task execution on Skill-based level																						**Fully dependent**		**1,00E-03**
																						Independent		**1,03E-03**
Rule-based level	Define activity	# 2 1	Assess task	Plan			X									X						0,01	0,05	0,0005
Rule-based level	Determine	# 2 2	Determine	Diagnose		X	X					X										0,2	0,1	0,02
Rule-based level	Record	# 2 3	Record decision	Execute				X											X			0,0005	0,0512	3E-05
Failure probability of task execution on Rule-based level																						**Fully dependent**		**2,00E-02**
																						Independent		**2,05E-02**
Knowledge-based level	Define activity	# 3 1	Assess task	Plan			X									X						0,01	0,05	0,0005
Knowledge-based level		# 3 2	Identify problem	Identify		X						X										0,2	0,1	0,02
Knowledge-based level	Determine	# 3 3	Find higher level analogy	Diagnose		X	X					X										0,2	0,1	0,02
Knowledge-based level		# 3 4	Determine	Diagnose		X	X					X										0,2	0,1	0,02
Knowledge-based level	Record	# 3 5	Record decision	Execute				X											X			0,0005	0,0512	3E-05
Failure probability of task execution on Knowledge-based level																						**Fully dependent**		**2,00E-02**
																						Independent		**5,93E-02**

[1] COCOM: Contextual Control Model
[2] CFP: Cognitive Failure Probability


Cognitive Demand Profile `determine` activity
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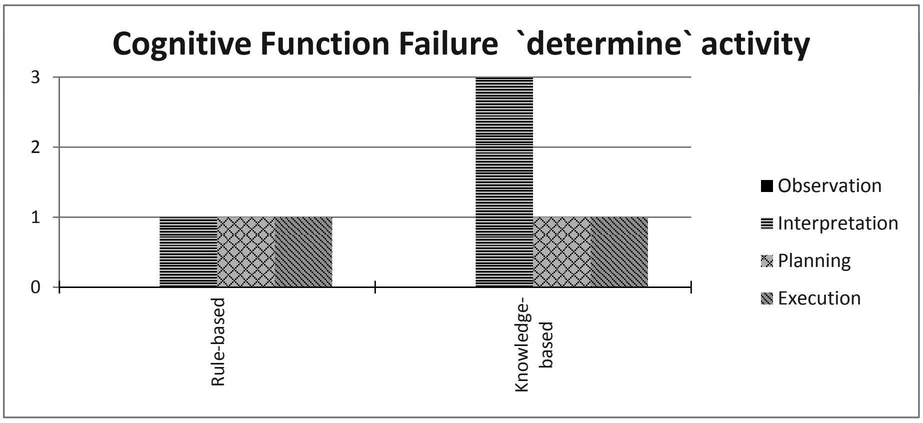
Rule-based
Knowledge-based
Observation
Interpretation
Planning
Execution


Cognitive Function Failure `determine` activity
3
2
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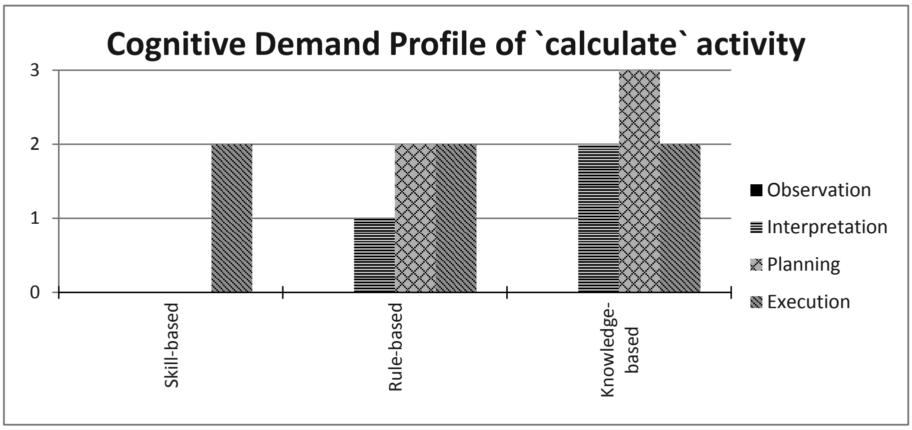
0
Rule-based
Knowledge-based
Observation
Interpretation
Planning
Execution

Level	Cognitive activities in the task: `calculate`				COCOM[1] function				Credible failure modes													Failure Probabilities		
	Activity	Task #	Goal	Cognitive activity	Observation	Interpretation	Planning	Execution	Observation O1	Observation O2	Observation O3	Interpretation I1	Interpretation I2	Interpretation I3	Planning P1	Planning P2	Execution E1	Execution E2	Execution E3	Execution E4	Execution E5	Nominal CFP[2]	Weighting factors	Adjusted CFP
Skill-based	Calculate	# 1 1	Execute calculation	Execute				X											X			0,0005	0,0512	3E-05
Skill-based	Record	# 1 2	Record outcomes	Execute				X											X			0,0005	0,0512	3E-05
Skill-based	**Failure probability of task execution on Skill-based level**																					**Fully dependent**		**2,56E-05**
																						Independent		**5,12E-05**
Rule-based level	Define activity	# 2 1	Assess task	Plan			X									X						0,01	0,05	0,0005
Rule-based level	Calculate	# 2 1	Apply basic rule	Evaluate		X	X						X									0,01	0,1	0,001
		# 2 3	Execute calculation	Execute				X											X			0,0005	0,0512	3E-05
Rule-based level	Record	# 2 4	Record outcomes	Execute				X											X			0,0005	0,0512	3E-05
Rule-based level	**Failure probability of task execution on Rule-based level**																					**Fully dependent**		**1,00E-03**
																						Independent		**1,55E-03**
Knowledge-based level	Define activity	# 3 1	Assess task	Plan			X									X						0,01	0,05	0,0005
Knowledge-based level	Calculate	# 3 2	Diagnose calculation	Diagnose		X	X					X										0,2	0,1	0,02
		# 3 3	Find higher level analogy	Diagnose		X	X					X										0,2	0,1	0,02
		# 3 4	Execute calculation	Execute				X											X			0,0005	0,0512	3E-05
Knowledge-based level	Record	# 3 5	Record outcomes	Execute				X											X			0,0005	0,0512	3E-05
Knowledge-based level	**Failure probability of task execution on Knowledge-based level**																					**Fully dependent**		**2,00E-02**
																						Independent		**4,01E-02**

[1] COCOM: Contextual Control Model
[2] CFP: Cognitive Failure Probability


Cognitive Demand Profile of `calculate` activity
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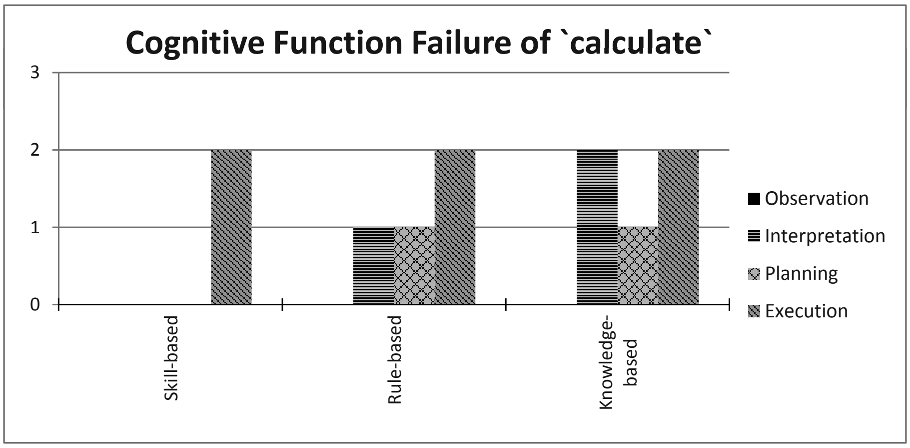
Skill-based
Rule-based
Knowledge-based
Observation
Interpretation
Planning
Execution


Cognitive Function Failure of `calculate`
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Skill-based
Rule-based
Knowledge-based
Observation
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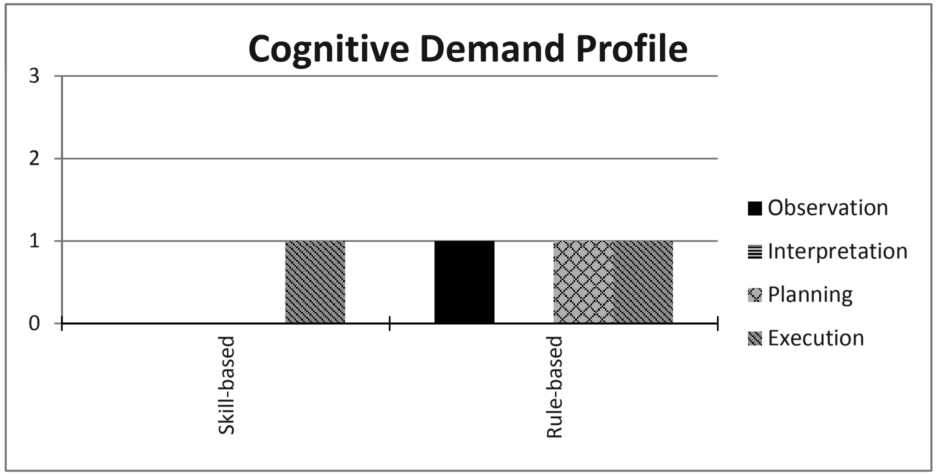
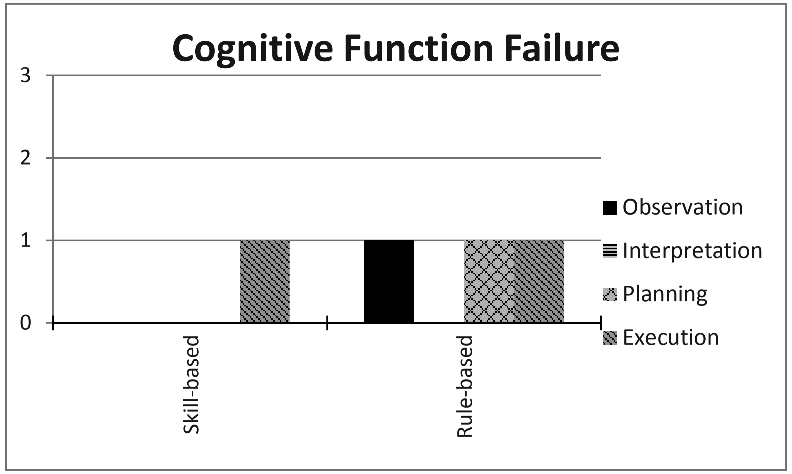
Execution

	Cognitive activities in the task: `Insert`				COCOM[1] function				Credible failure modes													Failure Probabilities		
									Obser-vation			Inter-pretation			Plan-ning		Execution							
	Activity	Task #	Goal	Cognitive activity	Observation	Interpretation	Planning	Execution	O1	O2	O3	I1	I2	I3	P1	P2	E1	E2	E3	E4	E5	Nominal CFP[2]	Weighting factors	Adjusted CFP
Skill-based	Insert	# 1 2	Insert information	Execute				X											X			0,0005	0,0512	3E-05
Skill-based	**Failure probability of task execution on Skill-based level**																**Fully dependent** **Independent**							**1,00E+00** **2,56E-05**
Rule-based level	Define activity	# 2 1	Assess task	Plan			X									X						0,01	0,05	0,0005
Rule-based level	Insert	# 2 2	Observe information	Observe	X					X												0,07	0,064	0,0045
Rule-based level		# 2 3	Insert information	Execute				X											X			0,0005	0,0512	3E-05
Rule-based level	**Failure probability of task execution on Rule-based level**																**Fully dependent** **Independent**							**4,48E-03** **5,00E-03**

[1] COCOM: Contextual Control Model [2] CFP: Cognitive Failure Probability





Level	Activity	Task #	Goal	Cognitive activity	COCOM[1] function: Observation	Interpretation	Planning	Execution	Credible failure modes: Observation O1	O2	O3	Interpretation I1	I2	I3	Planning P1	P2	Execution E1	E2	E3	E4	E5	Failure Probabilities: Nominal CFP[2]	Weighting factors	Adjusted CFP
Cognitive activities in the task: \`communicate\`																								
Skill-based	Communicate	# 1 1	Communicate	Communicat				X													X	0,03	0,0512	0,0015
	Failure probability of task execution on Skill-based level																					Fully dependent		1,54E-03
																						Independent		1,54E-03
Rule-based level	Define activity	# 2 1	Assess task	Plan			X									X						0,01	0,05	0,0005
		# 2 2	Consider intentions	Evaluate		X	X						X											
		# 2 3	Communicate	Communicat				X													X	0,03	0,0512	0,0015
	Failure probability of task execution on Rule-based level																					Fully dependent		1,54E-03
																						Independent		2,04E-03
Knowledge-based level	Define activity	# 3 1	Asses task	Plan			X									X						0,01	0,05	0,0005
		# 3 2	identify goal	Evaluate		X	X						X										0,1	
	Communicate	# 3 3	Interpret discussion	Diagnose		X	X					X										0,2	0,1	0,02
		# 3 4	discussion	Communicat				X													X	0,03	0,0512	0,0015
	Failure probability of task execution on Knowledge-based level																					Fully dependent		2,00E-02
																						Independent		2,20E-02

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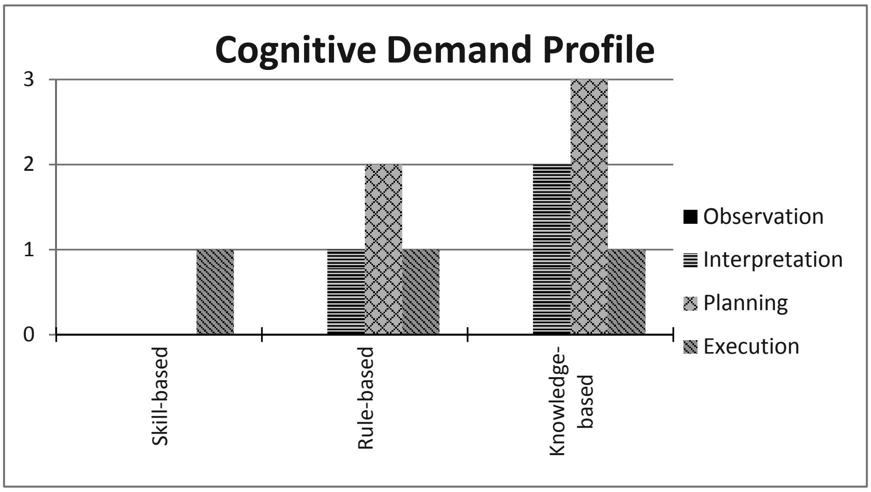
Cognitive Demand Profile
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Skill-based
Rule-based
Knowledge-based
Observation
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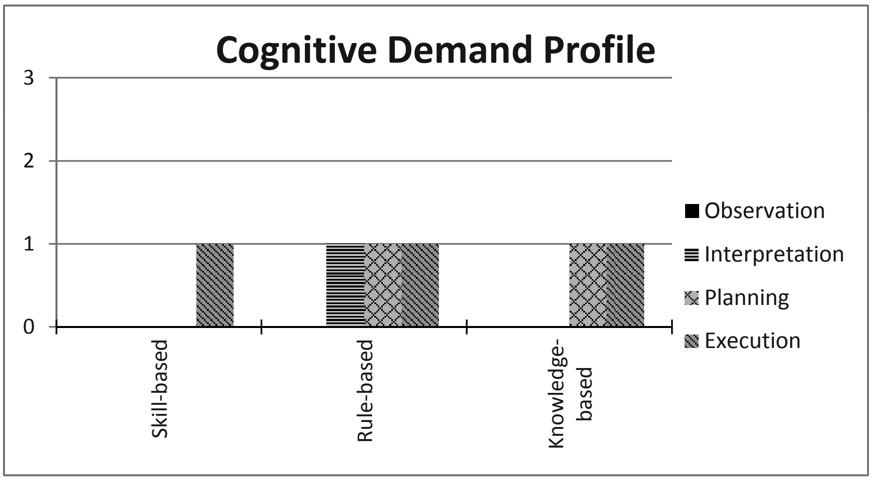
Execution


Cognitive Demand Profile
3
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1
0
Skill-based
Rule-based
Knowledge-based
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Interpretation
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Execution

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TASK ANALYSIS

Contents

Human Error Probabilities (HEP)			Properties task			
	Task No.	Task description	Basic task	Operation	Cognitve level: Expert	Cognitve level: Novice
Geo-metry	A0.1.1	Obtain basic drawings	Obtain	SB	SB	SB
	A0.1.2	Communicate requirements	Communicate	KB	RB	KB
	A0.2.1	Derive floor height	Derive	RB	SB	RB
	A0.2.2	Derive raster in x-direction	Derive	RB	SB	RB
	A0.2.3	Derive raster in y-direction	Derive	RB	SB	RB
Material Characteristics (Node A5)	A5.1.1	Obtain concrete requirements	Obtain	SB	SB	SB
	A5.1.2	Communicate concrete req.	Communicate	KB	RB	KB
	A5.1.3	Determine concrete type	Determine	RB	RB	RB
	A5.1.4	Consult NEN-EN 1991	Consult	RB	RB	RB
	A5.1.5	Derive specific mass concrete	Derive	RB	SB	RB
	A5.1.6	Consult NEN-EN 1992	Consult	RB	RB	RB
	A5.1.7	Derive concrete compressive str.	Derive	RB	RB	RB
	A5.1.8	Derive concrete tensile strength	Derive	RB	RB	RB
	A5.1.9	Derive concrete elasticity	Derive	RB	RB	RB
	A5.2.1	Obtain steel requirements	Obtain	SB	SB	SB
	A5.2.2	Communicate steel requirements	Communicate	KB	RB	KB
	A5.2.3	Determine steel type	Determine	RB	RB	RB
	A5.2.4	Consult material specifications	Consult	RB	RB	RB
	A5.2.5	Derive reinforcement strength	Derive	RB	RB	RB
	A5.2.6	Derive steel elasticity	Derive	RB	RB	RB
Determine distributed loads (node A2)	A2.1.0	Obtain beam type	Obtain	SB	SB	SB
	A2.1.1	Consult design rules	Consult	RB	SB	RB
	A2.1.2	Calculate beam height	Calculate	SB	SB	SB
	A2.1.3	Determine beam type	Determine	KB	RB	KB
	A2.1.4	Insert beam length	Insert	SB	SB	SB
	A2.2.1	Calculate width beams	Calculate	SB	SB	SB
	A2.2.2	Consult design rules	Consult	RB	SB	RB
	A2.3.0	Insert specific mass concrete	Insert	SB	SB	SB
	A2.3.1	Calculate weight beams	Calculate	SB	SB	SB
	A2.5.7	Communicate slab height	Communicate	RB	RB	RB
	A2.6.2	Calculate weight slab	Calculate	RB	RB	RB
	A3.1.1	Consult NEN-EN 1991 (table 6.1)	Consult	RB	RB	RB
	A3.1.2	Det. functional use floor field	Determine	KB	KB	KB
	A3.1.3	Read requirements	Obtain	RB	RB	RB
	A3.1.4	Communicate with users/architect	Communicate	RB	RB	RB
	A3.1.5	Consult NEN-EN 1991	Consult	RB	RB	RB
	A3.1.6	Derive imposed load slab	Derive	RB	RB	RB
Calculating longitudinal reinforcement beam (Node B1)	B1.1.1	Derive support length	Derive	RB	SB	RB
	B1.1.3	Consult formula from NEN-EN 1992	Consult	SB	SB	SB
	B1.1.4	Calculate effective beam span	Calculate	SB	SB	SB
	B1.2.2	Update beam height	Insert	SB	SB	SB
	B1.2.4	Update beam width	Insert	SB	SB	SB
	B1.2.5	Calculate beam geometry	Calculate	RB	SB	RB
	B1.2.6	Obtain beam height	Obtain	SB	SB	SB
	B1.2.7	Obtain beam width	Obtain	SB	SB	SB
	B1.3.1	Calculate self-weight beam	Calculate	RB	SB	RB
	B1.3.3	Calculate self-weight on slab	Calculate	RB	RB	RB
	B1.3.4	Calc. Imposed load on slab	Calculate	RB	RB	RB
	B1.3.5	Det. Loading combinations ULS	Determine	KB	KB	KB
	B1.3.6	Consult NEN-EN 1991	Consult	KB	RB	KB

Human Error Probabilities (HEP)			Properties task			
	Task No.	Task description	Basic task	Operation	Cognitve level: Expert	Cognitve level: Novice
Calculating longitudinal reinforcement beam (Node B1)	B1.3.7	Compose loading combinations	Determine	RB	RB	RB
	B1.3.8	Calculate total load	Calculate	RB	SB	RB
	B1.4.1	Calculate maximum field moment	Calculate	RB	RB	RB
	B1.4.2	Calculate max. support moment	Calculate	RB	RB	RB
	B1.4.3	obtain basic formula	Obtain	RB	RB	RB
	B1.4.5	Calculate bottom reinforcement	Calculate	RB	RB	RB
	B1.4.6	Calculate top reinforcement	Calculate	RB	RB	RB
	B1.4.7	Determine Practical reinforcement	Determine	RB	RB	RB
	B1.4.8	Insert steel strength	Insert	SB	SB	SB
	B1.5.1	Consult basic design rules	Consult	RB	RB	RB
	B1.5.2	Derive Cmin	Derive	RB	RB	RB
	B1.5.3	Estimate 1/2·Φmr+Φsr	Determine	RB	RB	RB
	B1.5.4	Calculate d	Calculate	RB	RB	RB
	B1.6.1	Calculate As;max	Calculate	SB	SB	SB
	B1.6.2	Consult design rules As;max	Consult	RB	RB	RB
	B1.6.3	Determine if As<As;max	Determine	SB	SB	SB
	B1.7.1	Insert concrete tensile strength	Insert	SB	SB	SB
	B1.7.2	Insert steel strength	Insert	SB	SB	SB
	B1.7.3	Consult design rules AS;min	Consult	KB	RB	KB
	B1.7.4	Calculate As;min;1	Calculate	RB	SB	RB
	B1.7.5	Consult NEN-EN 1992-1-1	Consult	RB	RB	RB
	B1.7.6	Derive width tension zone	Derive	RB	SB	RB
	B1.7.7	Consult NEN-EN 1992-1-1	Consult	KB	RB	KB
	B1.7.8	Calculate As;min;2	Calculate	SB	SB	SB
	B1.7.9	Decide on As;min	Determine	SB	SB	SB
	B1.7.10	Determine if As>As;min	Determine	SB	SB	SB
	B1.8.5	Decide bottom reinf. layout	Determine	KB	RB	KB
	B1.8.6	Decide top reinforcement layout	Determine	KB	RB	KB
	B1.9.1	Consult defelection control req.	Consult	KB	RB	KB
	B1.9.2	Calculate limiting span/depth ratio	Calculate	RB	RB	RB
	B1.9.3	Check if deflection req. Is satisfied	Determine	RB	RB	RB
Crack width control (node B2)	B2.1.1	Obtain concrete tensile strength	Obtain	SB	SB	SB
	B2.1.5	Calculate x;uncracked	Calculate	KB	RB	KB
	B2.1.6	Consult NEN-EN 1992-1-1	Consult	RB	RB	RB
	B2.1.7	Calculate equivalent height	Derive	RB	RB	RB
	B2.1.8	Calculate concr. area tensile zone	Calculate	RB	SB	RB
	B2.2.1	Consult NEN-EN 1992-1-1	Consult	RB	RB	RB
	B2.2.3	Consult NEN-EN 1992-1-1	Consult	RB	RB	RB
	B2.2.4	Calculate axial force Ned	Calculate	RB	RB	RB
	B2.2.5	Calculate concrete mean stress	Calculate	KB	RB	KB
	B2.2.6	Derive coefficient k1	Derive	RB	SB	RB
	B2.2.7	Derive coefficient h*	Derive	RB	SB	RB
	B2.2.8	Calculate Coefficient Kc	Calculate	RB	SB	RB
	B2.3.1	Consult NEN-EN 1992-1-1	Consult	RB	RB	RB
	B2.3.2	Determine coefficent k	Determine	RB	SB	RB
	B2.3.3	Consult NEN-EN 1992-1-1	Consult	RB	RB	RB
	B2.3.4	Decide on σs = fyk	Determine	SB	SB	SB
	B2.3.5	Obtain steel strength	Obtain	SB	SB	SB
	B2.3.6	Calculate As;min;3	Calculate	RB	RB	RB
	B2.4.1	Consult NEN-EN 1992-1-1	Consult	RB	RB	RB

Human Error Probabilities (HEP)			Properties task			
	Task No.	Task description	Basic task	Operation	Cognitve level Expert	Cognitve level Novice
Crack width control (node B2)	B2.4.2	Consult Basic formulas	Consult	RB	SB	RB
	B2.4.3	Determine neutral axis	Determine	KB	KB	KB
	B2.4.4	Calculate reinforcement stress	Calculate	KB	RB	KB
	B2.5.1	Obtain Steel elasticity	Obtain	SB	SB	SB
	B2.5.2	Obtain concrete elasticity	Obtain	SB	SB	SB
	B2.5.4	Consult NEN-EN 1992-1-1	Consult	RB	SB	RB
	B2.5.5	Derive coefficient kt	Derive	RB	SB	RB
	B2.5.3	Calculate ratio E-moduli	Calculate	SB	SB	SB
	B2.5.6	Consult NEN-EN 1992-1-1	Consult	RB	SB	RB
	B2.5.7	Calculate effective height	Calculate	RB	SB	RB
	B2.5.8	Calculate effective area concrete	Calculate	SB	SB	SB
	B2.5.9	Calculate reinforcement ratio	Calculate	RB	SB	RB
	B2.5.10	Calculate effective strain	Calculate	KB	RB	KB
	B2.6.1	Consult NEN-EN 1992-1-1	Consult	RB	RB	RB
	B2.6.2	Derive k1,k2,k3 or k4	Derive	RB	RB	RB
	B2.6.3	Calculate max. crackspacing	Calculate	RB	RB	RB
	B2.6.4	Calculate crack width	Calculate	RB	RB	RB
	B2.6.5	Consult NEN-EN 1992-1-1	Consult	RB	RB	RB
	B2.6.6	Derive allowed crack width	Derive	RB	RB	RB
	B2.6.7	Check allowable crack width	Determine	RB	RB	RB
Column design (Node B3)	B3.1.1	Calculate maximum normal force	Calculate	RB	RB	RB
	B3.1.2	Calculate maximum Moment	Calculate	RB	RB	RB
	B3.1.0	Communicate structural dimensions	Communicate	RB	SB	RB
	B3.1.3	Derive column width	Derive	RB	RB	RB
	B3.1.4	Determine column depth	Determine	RB	RB	RB
	B3.1.5	Determine concrete cover	Determine	RB	SB	RB
	B3.1.6	Consult basic design rules	Consult	RB	RB	RB
	B3.1.7	Calculate 1st order reinforcement	Calculate	KB	SB	KB
	B3.1.8	Choose reinforcement layout	Determine	KB	RB	KB
	B3.2.1	Consult NEN-EN 1992-1-1	Consult	RB	RB	RB
	B3.2.2	Calculate α-factor	Calculate	RB	RB	RB
	B3.2.3	Calculate reinforcement ratio	Calculate	RB	SB	RB
	B3.2.4	Calculate fictitious elasticity modulus	Calculate	RB	RB	RB
	B3.2.5	Calculate I	Calculate	RB	RB	RB
	B3.2.6	Calculate EI-column	Calculate	RB	RB	RB
	B3.3.1	Consult basic design rules	Consult	RB	RB	RB
	B3.3.2	Calculate buckling force	Calculate	RB	RB	RB
	B3.3.3	Calculate second order moment	Calculate	KB	RB	KB
	B3.3.4	Calculate Concrete compression	Calculate	KB	RB	KB
	B3.3.5	Obtain concrete compr. Strength	Obtain	SB	SB	SB
	B3.3.6	Check allowable concrete compression	Determine	RB	RB	RB
	B3.3.7	Calculate reinforcement stress	Calculate	RB	RB	RB
	B3.3.8	Obtain steel strength	Obtain	SB	SB	SB
	B3.3.9	Check reinforcement stress	Determine	SB	SB	SB
	B3.4.1	Consult NEN-EN 1992-1-1 (9.5.2)	Consult	RB	RB	RB
	B3.4.2	Derive minimum diameter	Derive	RB	SB	RB
	B3.4.3	Check minimum diameter	Determine	SB	SB	SB
	B3.4.4	Calculate minimum reinforcement	Calculate	RB	RB	RB
	B3.4.5	Check minimum reinforcement	Determine	SB	SB	SB

Error Magnitude Properties								First distribution			Second distribution		
	Task Sequence	Task description	Failure prob.	Para-meter	Unit	Mean value	distri-bution	Function	Failure* fraction	Standard dev.	Function	Failure* fraction	Values
Geo-metry	A0.2.1	Derive floor height	0,0015	hi	[m]	3,60	Single	Normal	1,00	0,61			
	A0.2.2	Derive raster in x-direction	0,0015	Lx;i	[m]	6,00	Single	Normal	1,00	0,78			
	A0.2.3	Derive raster in y-direction	0,0015	Ly;i	[m]	7,20	Single	Normal	1,00	0,78			
Material Characteristics (Node A5)	A5.1.1 to A5.1.3	Determine concrete type	0,0425	[-]	[N/mm2]	C25/30	Single	Discrete	1,00				e
	A5.1.4 and A5.1.5	Derive specific mass concrete	0,0264	γ_c	[kN/m3]	25,00	Single	Normal	1,00	0,78			
	A5.1.6 and A5.1.7	Derive concrete compressive str.	0,0264	fck	[N/mm2]	30,00	Single	Normal	1,00	0,78			
	A5.1.6 and A5.1.8	Derive concrete tensile strength	0,0264	fctm	[N/mm2]	2,90	Single	Normal	1,00	0,78			
	A5.1.6 and A5.1.9	Derive concrete elasticity	0,0264	Ec	[N/mm2]	2,8E+04	Single	Normal	1,00	1,48			
	A5.2.1 to A5.2.3	Determine steel type	0,0425	[-]	[N/mm2]	B500B	Single	Discrete	1,00				e
	A5.2.4 and A5.2.5	Derive reinforcement strength	0,0264	fyk	[N/mm2]	435,00	SIngle	Normal	1,00	0,96			
	A5.2.4 and A5.2.6	Derive steel elasticity	0,0264	Es	[N/mm2]	2,1E+05	SIngle	Normal	1,00	1,48			
Determine distributed loads	A2.1.0 to A2.1.4	Calculate beam height	0,0843	hb	[m]	0,50	Comb.	Log-norm	0,70	0,54	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	A2.2.2 and A2.2.1	Calculate width beams	0,0250	bb	[m]	0,275	Comb.	Log-norm	0,70	0,54	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	A2.3.0 and A2.3.1	Calculate weight beams	0,0001	qdb	[kN/m]	3,44	Comb.	Log-norm	0,70	0,67	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	A2.5.7	Communicate slab height	0,0020	hs	[mm]	200	Single	Normal	1,00	0,78			
	A2.6.2	Calculate weight slab	0,0016	qds	[kN/m2]	3,12	Comb.	Log-norm	0,70	0,54	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	A3.1.1 and A3.1.6	Derive imposed load slab	0,1177	qk	[kN/m2]	3,00	Single	Normal	1,00	0,96			
Calculating longitudinal reinforcement beam (Node B1)	B1.1.1	Derive support length	0,0015	a1	[m]	0,10	Single	Normal	1,00	0,96			
	B1.1.3 and B1.1.4	Calculate effective beam span	0,0046	Leff	[m]	7,20	Comb.	Log-norm	0,70	0,54	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B1.2.2	Update beam height	0,0000	hb	[m]	0,50	Single	Normal	1,00	0,61			
	B1.2.4	Update beam width	0,0000	bb	[m]	0,28	Single	Normal	1,00	0,61			
	B1.2.5 to B1.2.7	Calculate beam geometry	0,0016	Ac	[m2]	0,14	Comb.	Log-norm	0,70	0,54	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B1.3.1	Calculate self-weight beam	0,0001	qdb	[kN/m]	3,44	Comb.	Log-norm	0,70	0,54	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B1.3.3	Calculate self-weight on slab	0,0031	qds	[kN/m]	18,72	Comb.	Log-norm	0,70	0,54	Discrete	0,30	1,40 or 0,138 b
	B1.3.4	Calc. Imposed load on slab	0,1192	qks	[kN/m]	18,00	Comb.	Log-norm	0,50	0,54	Discrete	0,50	1,40 or 0,138 b
	B1.3.6 and B1.3.5	Det. Loading combinations ULS	0,1037	$\gamma_{i,1}$	-	1,35	Comb.	Normal	0,50	0,78	Discrete	0,50	0,74 0,88 and 1,11 c
	B1.3.6 and B1.3.5	Det. Loading combinations ULS	0,1037	$\gamma_{i,2}$	-	1,50	Comb.	Normal	0,50	0,78	Discrete	0,50	0,66 0,80 and 0,90 c
	B1.3.7 and B1.3.8	Calculate total load	0,0221	ql	[kN/m]	56,91	Comb.	Lognorm.	0,50	0,67	Discrete	0,50	0,94 0,38 and 0,68 d
	B1.3.7 and B1.3.8	Calculate total dead load	0,0221	qdl	[kN/m]	29,91	Comb.	Lognorm.	0,50	0,67	Discrete	0,50	0,94 0,38 and 0,69

								First distribution			Second distribution		
	Task Sequence	Task description	Failure prob.	Para-meter	Unit	Mean value	distri-bution	Function	Failure* fraction	Standard dev.	Function	Failure* fraction	Values
Calculating longitudinal reinforcement beam (Node B1)	B1.3.7 and B1.3.8	Calculate total variable load	0,0221	qll	[kN/m]	27,00	Comb.	Lognorm.	0,50	0,54	Discrete	0,50	0,94 0,38 and 0,70
	B1.4.1	Calculate maximum field moment	0,0016	Mu,d	[kNm]	264,78	Comb.	Lognorm.	0,70	1,35	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B1.4.2	Calculate max. support moment	0,0016	Mu,s	[kNm]	318,36	Comb.	Lognorm.	0,70	1,35	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B1.4.3 and B1.4.5	Calculate bottom reinforcement	0,0066	As;b	[mm2]	1503	Comb.	Lognorm.	0,70	1,02	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B1.4.3 and B1.4.6	Calculate top reinforcement	0,0066	As;t	[mm2]	1807	Comb.	Lognorm.	0,70	1,02	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B1.4.7	Determine Practical reinforcemen	0,0205	As;pract	[mm2]	628	Single	Lognorm.	1,00	0,67			
	B1.4.7	No. Bars pract. Reinforcement	-	No;Asp	[No.]	2							
	B1.4.7	Bar diameter pract. Reinforcemen	-	d;Asp	[mm]	20							
	B1.5.2	Derive Cmin	0,0015	Cmin	[mm]	30	Single	Normal	1,00	0,78			
	B1.5.3	Estimate 1/2·Φmr+Φsr	0,0205	Φmr+Φsı	[mm]	20	Single	Normal	1,00	0,78			
	B1.5.1 and B1.5.4	Calculate d	0,0265	d	[mm]	450	Comb.	Lognorm.	0,70	0,67	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B1.6.1 and B1.6.2	Calculate As;max	0,0250	As;max	[mm2]	5500	Comb.	Lognorm.	0,70	0,67	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B1.6.3	Determine if As<As;max	0,0010	As	[mm2]	-	Single	Discrete	1,00				e
	B1.7.5 and B1.7.6	Derive width tension zone	0,0264	bt	[mm]	275	Single	Lognorm.	1,00	0,54			
	B1.7.1 to B1.7.4	Calculate As;min;1	0,0460	As;min;1	[mm2]	214,50	Comb.	Lognorm.	0,70	0,67	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B1.7.7 and B1.7.8	Calculate As;min;2	0,0445	As;min;2	[mm2]	160,88	Comb.	Lognorm.	0,70	0,67	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B1.7.9	Decide on As;min	0,0010	As;min	[mm2]	214,50	Single	Discrete	1,00				e
	B1.7.10	Determine if As>As;min	0,0010	As	[mm2]	-	Single	Discrete	1,00				e
	B1.8.5	Decide bottom reinf. layout	0,0593	Asb;prov	[mm2]	1520	Single	Normal	1,00	0,96			
	B1.8.5	No. Bars bottom	-	No;Asb	[No.]	4							
	B1.8.5	Bar diameter bottom	-	d;Asb	[mm]	22							
	B1.8.6	Decide top reinforcement layout	0,0593	Ast;prov	[mm2]	1808	Single	Normal	1,00	0,96			
	B1.8.6	No. Bars top	-	No;Ast	[No.]	4							
	B1.8.6	Bar diameter top	-	d;Ast	[mm]	24							
	B1.9.1 to B1.9.3	Check if deflection req. Is satisfied	0,0665	-	[-]	-	Single	Discrete	1,00				e
Crack width	B2.1.1 and B2.1.5	Calculate x;uncracked	0,0402	x,uncr	[mm]	0,16	Comb.	Lognorm.	0,70	1,02	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B2.1.6 and B2.1.7	Calculate equivalent height	0,0264	h,t,ef	[mm]	0,35	Comb.	Lognorm.	0,70	0,54	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B2.1.6 to B2.1.8	Calculate concr. area tensile zone	0,0681	Act	[mm2]	94875	Comb.	Lognorm.	0,70	0,67	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B2.2.7	Derive coefficient h*	0,0015	h*	[mm]	1000	Single	Normal	1,00	0,96			

	Error Magnitude Properties							First distribution			Second distribution		
	Task Sequence	Task description	Failure prob.	Para-meter	Unit	Mean value	distri-bution	Function	Failure* fraction	Standard dev.	Function	Failure* fraction	Values
Crack width control (node B2)	B2.2.6	Derive coefficient k1	0,0015	k1	[-]	1,50	Single	Normal	1,00	0,96			
	B2.2.4	Calculate axial force Ned	0,0016	Ned	[N]	436600	Comb.	Lognorm.	0,70	0,67	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B2.2.3 and B2.2.5	Calculate concrete mean stress	0,0650	σc	[N/mm2]	3,18	Comb.	Lognorm.	0,70	1,02	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B2.2.1 and B2.2.8	Calculate Coefficient Kc	0,0265	kc	[-]	0,18	Comb.	Lognorm.	0,70	1,02	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B2.3.1 and B2.3.2	Determine coefficent k	0,0454	k	[-]	1,00	Single	Normal	1,00	0,96			
	B2.3.3 to B2.3.5	Decide on σs = fyk	0,0260	σs	[N/mm2]	30,00	Single	Normal	1,00	0,78			
	B2.3.6	Calculate As;min;3	0,0016	As,min,3	[mm2]	116,35	Comb.	Lognorm.	0,70	1,02	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B2.4.2 and B2.4.3	Determine neutral axis	0,0842	x	[mm]	200	Comb.	Lognorm.	0,70	0,67	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B2.4.1 and B2.4.4	Calculate stress bottom reinforcer	0,0650	σsb	[N/mm2]	104,20	Comb.	Lognorm.	0,70	1,02	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B2.4.1 and B2.4.4	Calculate stress top reinforcement	0,0650	σss	[N/mm2]	87,60	Comb.	Lognorm.	0,70	1,02	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B2.5.4 and B2.5.5	Determine coefficient kt	0,0264	kt	[-]	0,60	Single	Normal	1,00	0,96			
	B2.5.6 and B2.5.7	Calculate effective height	0,0265	hc,ef	[mm]	100	Comb.	Lognorm.	0,70	1,02	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B2.5.8	Calculate effective area concrete	0,0001	Ac,eff	[mm2]	27500	Comb.	Lognorm.	0,70	0,67	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B2.4.1 and B2.5.9	Calculate reinforcement ratio bott	0,0265	ρp,eff,b	[-]	0,055	Comb.	Lognorm.	0,70	0,67	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B2.4.1 and B2.5.9	Calculate reinforcement ratio top	0,0265	ρp,eff,s	[-]	0,066	Comb.	Lognorm.	0,70	0,67	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B2.5.1 to B2.5.3	Calculate ratio E-moduli	0,0250	αe	[-]	7,50	Comb.	Lognorm.	0,70	0,67	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B2.4.1 and B2.5.10	Calculate effective strain bottom	0,0650	εsm-εcm)	[-]	2,84E-04	Comb.	Lognorm.	0,70	1,02	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B2.4.1 and B2.5.10	Calculate effective strain btop	0,0650	εsm-εcm)	[-]	2,29E-04	Comb.	Lognorm.	0,70	1,02	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B2.6.1 and B2.6.2	Determine k1	0,0264	k1	[-]	0,80	Single	Normal	1,00	0,96			
	B2.6.1 and B2.6.2	Determine k2	0,0264	k2	[-]	0,50	Single	Normal	1,00	0,96			
	B2.6.1 and B2.6.2	Determine k3	0,0264	k3	[-]	3,40	Single	Normal	1,00	0,96			
	B2.6.1 and B2.6.2	Determine k4	0,0264	k4	[-]	0,43	Single	Normal	1,00	0,96			
	B2.4.1 and B2.6.3	Calculate Sr,max bottom	0,0265	Sr,max,d	[mm]	169,66	Comb.	Lognorm.	0,70	1,02	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B2.4.1 and B2.6.3	Calculate Sr,max top	0,0265	Sr,max,s	[mm]	141,82	Comb.	Lognorm.	0,70	1,02	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B2.6.4	Calculate crack width bottom	0,0016	wk,d	[mm]	0,05	Comb.	Lognorm.	0,70	0,54	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B2.6.4	Calculate crack width top	0,0016	wk,s	[mm]	0,03	Comb.	Lognorm.	0,70	0,54	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B2.6.5 and B2.6.6	Determine allowed crack width	0,0264	wk,all	[mm]	0,30	Comb.	Normal	1,00	0,96	Discrete		e
	B2.6.7	Check allowable crack width	0,0205	-	[-]	Yes							e

Error Magnitude Properties								First distribution			Second distribution		
	Task Sequence	Task description	Failure prob.	Para-meter	Unit	Mean value	distri-bution	Function	Failure* fraction	Standard dev.	Function	Failure* fraction	Values
Column design (Node B3)	B3.1.1	Calculate maximum normal force	0,0016	Nc,1	[kN]	436,60	Comb.	Lognorm.	0,70	1,02	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B3.1.2	Calculate maximum Moment	0,0016	Mc,1	[kNm]	71,47	Comb.	Lognorm.	0,70	1,02	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B3.1.3	Derive column width	0,0015	Bc	[mm]	275	Comb.	Lognorm.	0,70	0,54	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B3.1.4	Determine column depth	0,0205	Hc	[mm]	275	Single	Normal	1,00	0,61			
	B3.1.5	Determine concrete cover	0,0205	Cc	[mm]	30,00	Single	Normal	1,00	0,61			
	B3.1.6 and B3.1.7	Calculate 1st order reinforcement	0,0401	Asc,1	[mm2]	745	Comb.	Lognorm.	0,70	1,02	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B3.1.8	Choose reinforcement layout	0,0593	Asc;prov	[mm2]	756	Single	Normal	1,00	0,96			
		No. Bars top		No;Ast	[No.]	2							
		Bar diameter top		d;Ast	[mm2]	22							
	B3.2.2	Calculate α-factor	0,0016	αn,1	[-]	0,23	Comb.	Lognorm.	0,70	1,02	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B3.2.3	Calculate reinforcement ratio	0,0016	ρc	[-]	0,010	Comb.	Lognorm.	0,70	0,67	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B3.2.4	Calculate fictitious elasticity modu	0,0016	Ef,1	[N/mm2]	1,0E+04	Comb.	Lognorm.	0,70	1,02	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B3.2.5	Calculate I	0,0016	I	[mm4]	4,8E+08	Comb.	Lognorm.	0,70	1,02	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B3.2.6	Calculate EI-column	0,0016	EI,1	[Nmm2]	5,0E+12	Comb.	Lognorm.	0,70	0,54	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B3.3.1 and B3.3.3	Calculate second order moment	0,0401	MEd,1	[kNm]	74,03	Comb.	Lognorm.	0,70	1,02	Discrete	0,30	$10^{(-)3}$ $10^{(-)2}$ and $10^{(-)1}$
	B3.3.4	Calculate Concrete compression	0,0401	fc,1	[N/mm2]	29,20	Single	Discrete	1,00				e
	B3.3.5 and B3.3.6	Check allowable concrete compre	0,0205	-	-	-	Single	Discrete	1,00				e
	B3.3.7 to B3.3.9	Check reinforcement stress	0,0026	fy	[N/mm2]	334,00	Single	Discrete	1,00				e
	B3.4.1 and B3.4.2	Derive minimum diameter	0,0264	ϕmin	[mm]	8,00	Single	Normal	1,00	0,78			
	B3.4.3	Check minimum diameter	0,0010	-	-	-	Single	Discrete	1,00				e
	B3.4.4 and B3.4.5	Check minimum reinforcement	0,0026	As,min,c	[mm2]	100,37	Single	Discrete	1,00				e

* Fraction of the total number of failures in which the distribtion occurred

[a] Selecting beam length in X-direction (5,4) instead of Y-direction.

[b] Multiplying Load [kN/m] with wrong widths: Y-direction instead of X-direction (7,6) or unity length (1,0)

[c] Selecting wrong safety factor out of 1,0 1,2 1,35 1,5

[d] Overlooking one of the load cases

[e] Error in choice

[f] Choosen concrete strength is C25/30

[g] Equals beam width (hb)

G

LIMIT STATE ANALYSIS

Contents

introduction

This appendix provides in background information on the probabilistic analysis methodology presented in chapter 11. The first part goes into the upper bound analysis. The second part discusses the lower bound analysis and the use of the SCIA calculation program.

upper bound analysis

The upper bound calculations are based on plastic equilibrium equations. In this section the equilibrium equations for the statical determined and undetermined beam will be presented.

Plastic capacity cross section

Within the equilibrium equations used in the upper bound analysis, the plastic moment capacity of a cross-section is required. Within the structure three cross-sectional types are differentiated. Two beam cross section comprising a cross-section at the support location and a cross section at mid-span. The third cross-section is the column cross section. The plastic moment capacity of a cross section is based on the equations given in equation 35. The resulting plastic capacity of a cross section is given in equation 36. Within these formulas the positive effect of reinforcement in the compression zone is not taken into account.

$$\begin{aligned} M &= 0 \Rightarrow Asfyk - XB_b fck \\ V &= 0 \Rightarrow Mp - Asfyk(H_b - c - Z_t) - fckB_b X(Z_t - \frac{1}{2}X) \\ Z_t &= \frac{B_b Ec \frac{1}{2} X^2 + AsEs(H_b - c)}{XB_b Ec + AsEs} \end{aligned} \tag{35}$$

$$Mp = -\frac{\frac{1}{2}Asfyk(-2H_b fckB_b + 2cfckB_b + Asfyk}{fckB_b} \tag{36}$$

Statical determined beam

In order to form a mechanism in the statical determined beam one hinge is required. A logical location is the midpoint of the beam at the location of the maximum field moment. This is depicted in figure 54. The derivation of the equilibrium equation of this mechanism is given in equation 37.

$$\begin{aligned} A &= \frac{1}{4} ql^2 \\ E &= 2Mp \\ Mp &\leqslant \frac{1}{8} ql^2 \end{aligned} \tag{37}$$

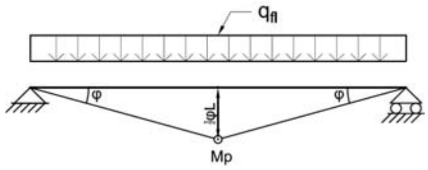


Figure 54: Mechanism statical determined beam

Combination of these formulas results in a formula for the reliability function as a function of the dimensional- and material parameters from the cross section. The basic form of this formula is depicted in figure 55.

$$> \mathrm{UB3} := (\mathrm{Mpb}) \cdot mr = \left(\frac{1}{8} \cdot q1 \cdot l^2\right) \cdot me;$$

$$UB3 := -\frac{1}{2} \frac{Asb\, fyk\, (-2\, hb\, fck\, bb + 2\, c\, fck\, bb + Asb\, fyk)\, mr}{fck\, bb} = 6480000\, q1\, me \qquad \textbf{(5)}$$

Figure 55: Formula reliability function statical determined beam

Statical undetermined beam

In order to form a mechanism in the statical undetermined beam three hinges are required. Two relevant partial mechanisms are depicted in figure 56 and 57. Mechanisms which are not considered are: buckling mechanisms and roof beam mechanisms. The derivation of the equilibrium equations of this mechanisms are given in the equations 38 and 39 respectively.

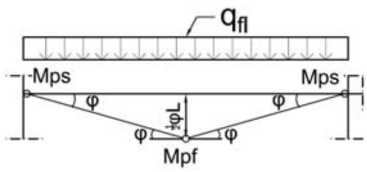


Figure 56: Partial mechanism one of the statical undetermined beam

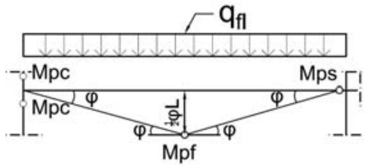


Figure 57: Partial mechanism two of the statical undetermined beam

$$\begin{aligned} A &= 2Mp_S + 2Mp_F \\ E &= \frac{1}{4} q_{FL} l_b^2 \\ Mp_S + Mp_F &\leqslant \frac{1}{8} q_{FL} l_b^2 \end{aligned} \qquad (38)$$

$$
\begin{aligned}
&A = Mp_S + 2Mp_F + 2Mp_C \\
&E = \frac{1}{4} q_{FL} l_b^2 \\
&Mp_S + 2Mp_F + 2Mp_C \leqslant \frac{1}{4} q_{FL} l_b^2
\end{aligned}
\tag{39}
$$

Combination of these formulas results in a formula for the reliability function as a function of the dimensional- and material parameters from the cross section. The basic forms of this formula is presented in figure 58. It should be noted that UB2 is differentiating from the other formula due to the inclusion of normal column force in this formula.

$$> \mathrm{UB1} := (\mathrm{Mpb} + \mathrm{Mpt}) \cdot mr = \left(\frac{1}{8} \cdot q1 \cdot l^2\right) \cdot me;$$

$$UB1 := \left(-\frac{1}{2} \frac{Asb\, fyk\, (-2\, hb\, fck\, bb + 2\, c\, fck\, bb + Asb\, fyk)}{fck\, bb} - \frac{1}{2} \frac{Ast\, fyk\, (-2\, hb\, fck\, bb + 2\, c\, fck\, bb + Ast\, fyk)}{fck\, bb}\right) mr = 6480000\, q1\, me \tag{3}$$

$$> \mathrm{UB2} := (2 \cdot \mathrm{Mpc} + 2 \cdot \mathrm{Mpb} + \mathrm{Mpt}) \cdot mr = \left(\frac{1}{4} \cdot q2 \cdot l^2\right) \cdot me;$$

$$
\begin{aligned}
UB2 := \Bigg(&-\frac{1}{fck\, bc\, (Ec\, N + Ec\, Asc\, fyk + Asc\, Es\, fck)} \big(Asc\, (-2\, fyk\, bc^2\, fck\, Ec\, N \\
&+ 2\, Asc\, fyk^2\, c\, fck\, bc\, Ec + 2\, Asc\, fyk\, c\, fck^2\, bc\, Es + 2\, N\, Es\, c\, fck^2\, bc + 2\, fyk\, c\, fck\, bc\, Ec\, N \\
&- 2\, Asc\, fyk^2\, bc^2\, fck\, Ec - 2\, Asc\, fyk\, bc^2\, fck^2\, Es - 2\, N\, Es\, bc^2\, fck^2 + 2\, N\, Asc\, fyk\, Es\, fck \\
&+ Asc^2\, fyk^3\, Ec + fyk\, Ec\, N^2 + 2\, Asc\, fyk^2\, Ec\, N + N^2\, Es\, fck + Asc^2\, fyk^2\, Es\, fck)\big) \\
&- \frac{Asb\, fyk\, (-2\, hb\, fck\, bb + 2\, c\, fck\, bb + Asb\, fyk)}{fck\, bb} \\
&- \frac{1}{2} \frac{Ast\, fyk\, (-2\, hb\, fck\, bb + 2\, c\, fck\, bb + Ast\, fyk)}{fck\, bb}\Bigg) mr = 12960000\, q2\, me
\end{aligned}
\tag{4}
$$

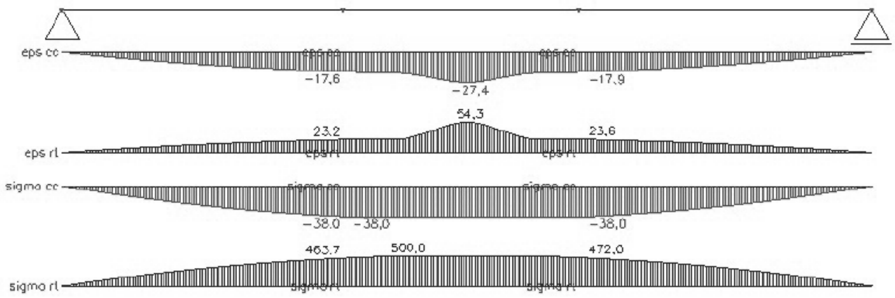
Figure 58: Formulas reliability functions statical undetermined beam

LOWER BOUND ANALYSIS

The lower bound calculations are based on finite element analysis, conducted with the program SCIA engineer. Within the lower bound analysis, the correctness of the applicability of the upper bound equations is checked. If the upper bound and lower bound results coincide, the correct failure mechanism is defined. Within this part the applicability of the upper bound for both beam types is considered.

Statical determined beam

For the lower bound check of the statical determined beam the correctness of formula 37 is checked. For this a single calculation is executed on a predefined point on the failure curve. According to the upper bound equation a structure with a bottom reinforcement of 1808 mm^2 will fail if the height of the beam is smaller then 417 mm. According to the lower bound analysis a plastic hinge will form if the beam height is approximately 420 mm. From this it can be concluded that the lower bound calculation coincides reasonable with the upper bound analysis. For completeness, the stress and strain properties of the beam with a height of 420 mm are presented in figure 59.



eps cc: Max. concrete compressive strain.
eps rt: Max. reinforcement tensional strain.
sigma cc: Max. concrete compressive stress.
sigma rt: Max. reinforcement tensional stress.

Figure 59: Strains and stresses of beam at ultimate capacity.

Statical undetermined beam

For the lower bound check of the statical undetermined beam the correctness of formulas 38 and 39 is checked. Within this analysis the beam width is kept equal to the design value (250 mm), while the beam height is variable in order to coincide with the upper bound results. The material characteristics used within the calculations are showed in figure 60.

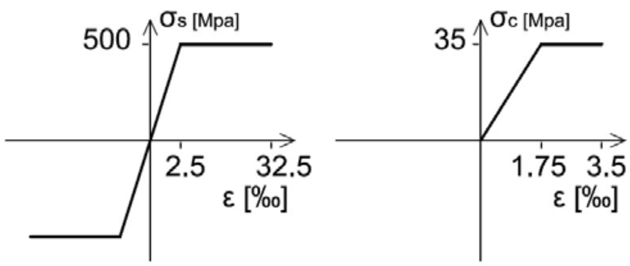


Figure 60: Stress-Strain curves of rienforcement steel (left) and concrete (right) within the lower bound check

Within this section a single case will be considered in more detail. In this case a loading condition of 60 kN/m is applied resulting in a minimum beam height according to the upper bound calculation of 320 mm. Within the upper bound calculation a beam height of 325 mm is considered, as failure occurred in case of a beam height of 320 mm. In the lower bound no failure occurred. With a difference of 5 mm between the lower and upper bound calculation it can be concluded that both calculations coincide, hence the correct upper bound is determined. The stress-strain results of the calculations are shown in the figures 61 to 70.

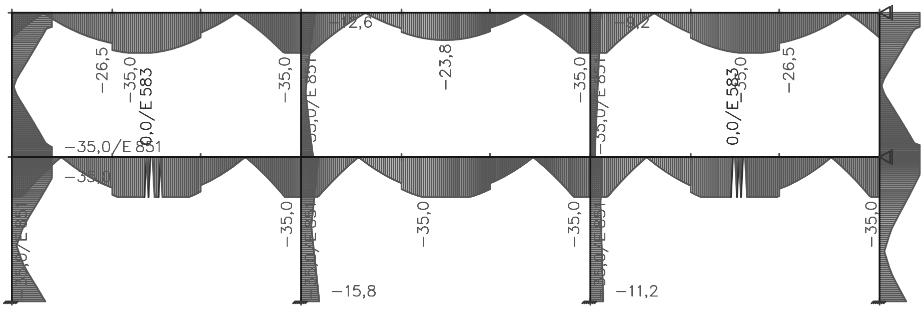


Figure 61: Maximum Concrete compression [N/mm^2] within cross-section of the frame

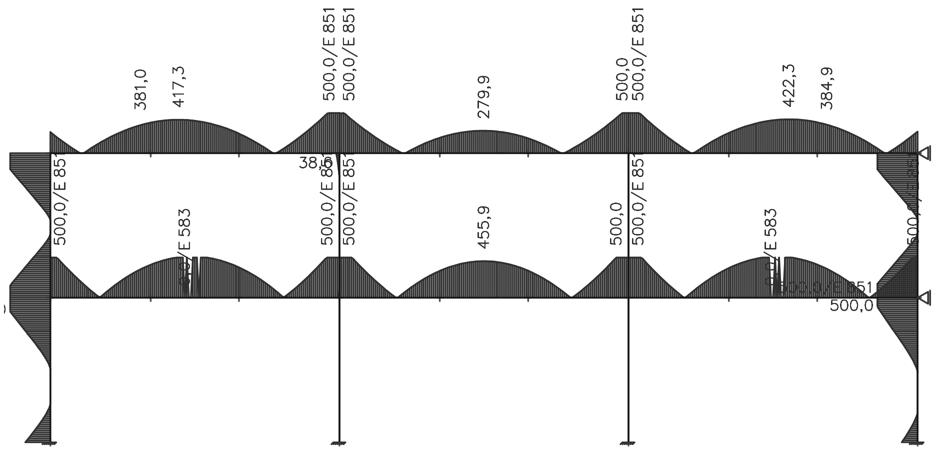


Figure 62: Maximum reinforcement tension [N/mm^2] within cross-section of the frame

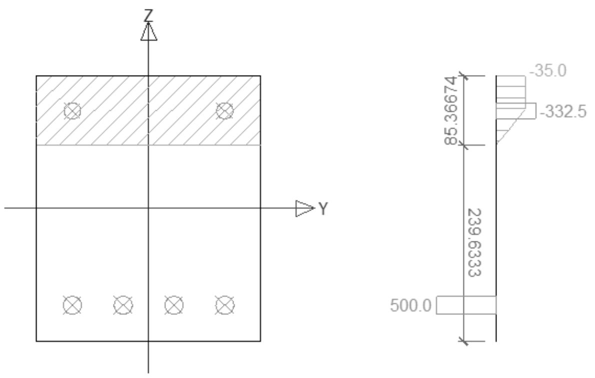


Figure 63: Stresses within the cross-section at mid-span

Within the results two cross sections are considered within the lower left beam: the cross section at beam midspan (figure 63 to 66) and at the right support (figure 67 to 70). The maximum stress in the concrete compression

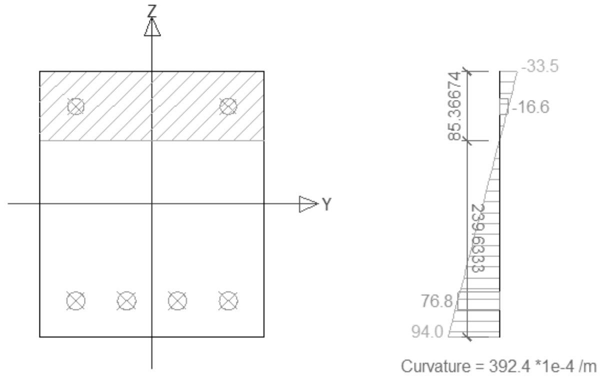


Figure 64: Strains within the cross-section at mid-span

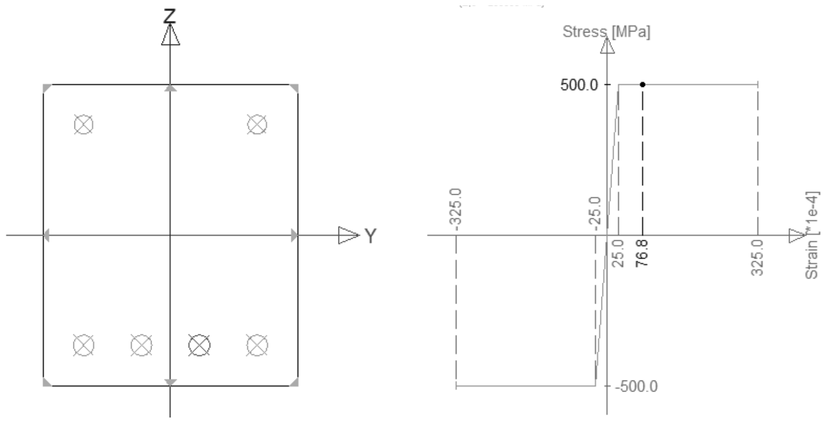


Figure 65: Reinforcement Stress - strain relation within tensile reinforcement of the cross-section at mid-span

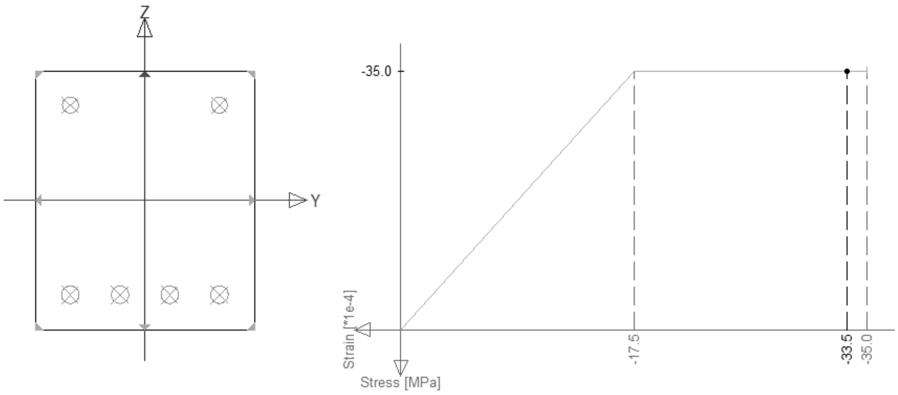


Figure 66: Concrete Stress - strain relation within concrete compressive zone of the cross-section at mid-span

zone in both cross sections still has some extra capacity. However the maximum concrete strain is almost reached, from this it can be concluded that the maximum capacity of the concrete is reached. The stress in the tensile reinforcement equals 500 N/mm^2. Furthermore the reinforcement strain has not reached his limits in both cross sections. This entails that the reinforcement capacity is not reached yet. It can be concluded from this analysis that the maximum cross sectional capacity is almost reached, as concrete

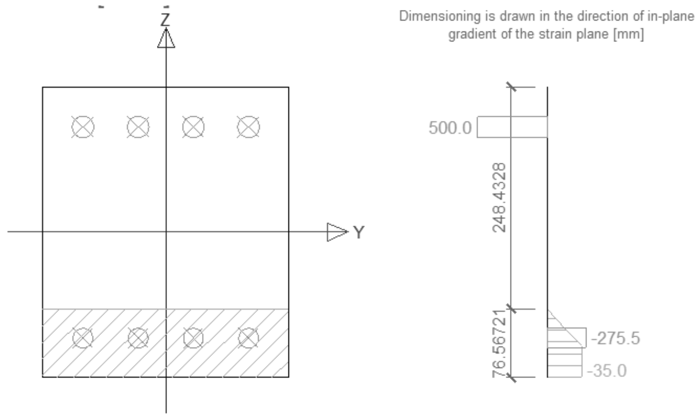


Figure 67: Stresses within the cross-section at the support

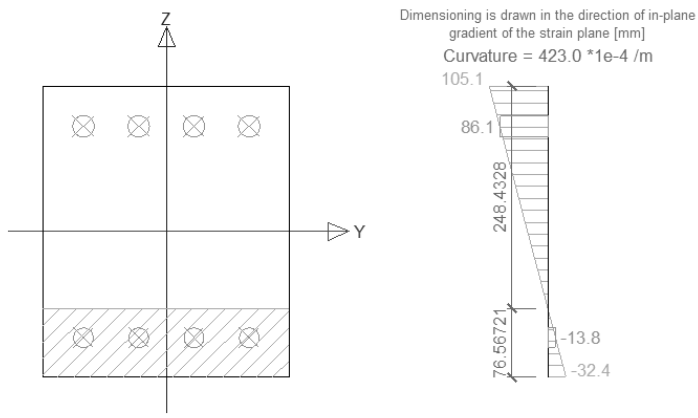


Figure 68: Strains within the cross-section at the support

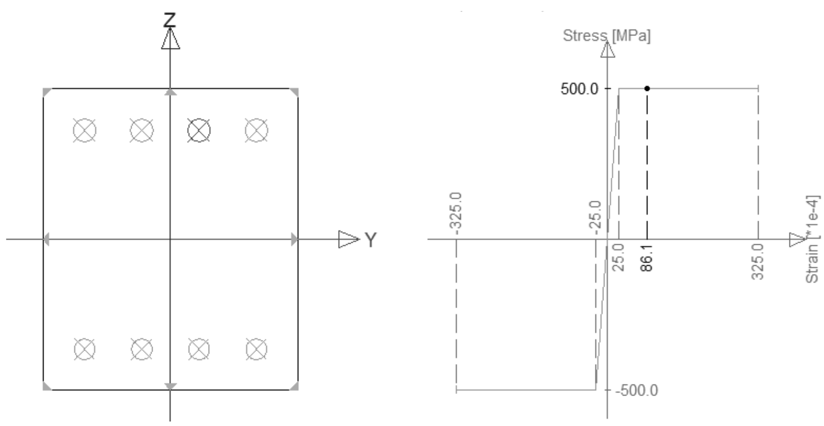


Figure 69: Reinforcement Stress - strain relation within tensile reinforcement of the cross-section at the support

crushing is almost occurring. Further increase in the loading conditions will result in a mechanism, hence SCIA cannot run this analysis.

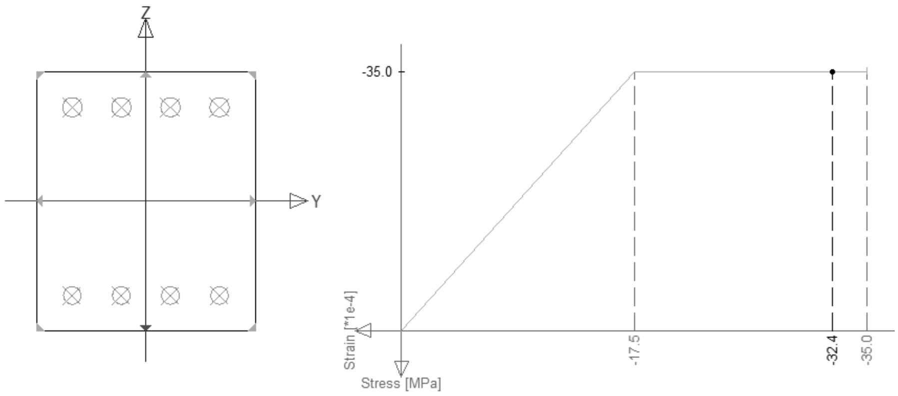


Figure 70: Concrete Stress - strain relation within concrete compressive zone of the cross-section at the support

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SIMULATION SCRIPT

Contents

introduction

Within this appendix the use of the Matlab program for the Monte Carlo and the FORM analysis is set-apart. This appendix contains three parts. Firstly the predefined bounds within the self-checking processes are presented and secondly visual checks of the reliability of the FORM-analysis are presented. Finally some brief elements of the used Matlab script are given.

self-checking boundaries

Within the Monte-Carlo simulation, self-control is based on the notion that a designer uses his previous experience as a reference for assessing the correctness of the results. Within this process, the output of a series of micro-tasks is compared with the correct output of the series of micro-tasks. If the output is within predefined limits, the output is deemed correct and the design process continues. If the output is not within these predefined limits, reconsidering of the series of micro-tasks is performed. In order to perform this comparison, the correct output is multiplied with a numerical value to represent the upper- and lower bound limits. These numerical values are different for experienced and inexperienced designers. Furthermore, the values are also based on overview of the situation. For instance the beam height is quite easy to overview as anyone has some idea about a correct beam height. Opposite to this are parameters which are harder to estimate on advance. The values are presented in table 22.

Table 22: Numerical multiplication values of the self-checking limits

Parameter	Lower bound		Upper bound	
	Experienced	Inexperienced	Experienced	Inexperienced
Hb	0,6	0,5	1,6	1,7
Bb	0,55	0,45	2,18	2,36
gl	0,45	0,36	2,63	2,82
Md/Ms	0,46	0,38	3,46	3,61
Ast	0,24	0,20	4,24	4,39
Asb	0,23	0,20	4,24	4,39
Asprac	0,32	0,29	2,94	3,08
Asmax	0,5	0,5	-	-
Asmin	0,4	0,4	2	2
AsbProv	0,33	0,26	4,96	5,25
AstProv	0,28	0,22	4,17	4,24
Bc	0,55	0,45	2,18	2,36
Mc	0,46	0,38	3,46	3,61
Asc	0,24	0,20	4,24	4,39
AscProv	0,33	0,26	4,96	5,25

VISUAL CHECK FORM ANALYSIS

In order to check the reliability of the FORM-analysis, two visual checks are performed: convergence of the reliability index and suitability of the Rackwitz transformation. Both checks are presented beneath.

One basic aspect of a FORM analysis is that the parameters should converge to a constant value after a sufficient number of iterations. Within the FORM analysis a maximum of 10 iterations is applied. The suitability of this is checked by means of a visual inspection of the convergence within the iterative procedure of β_z. This is shown in figure 71. It can be concluded that the method converges after 6 to 8 iterations, which entails that 10 iterations is sufficient.

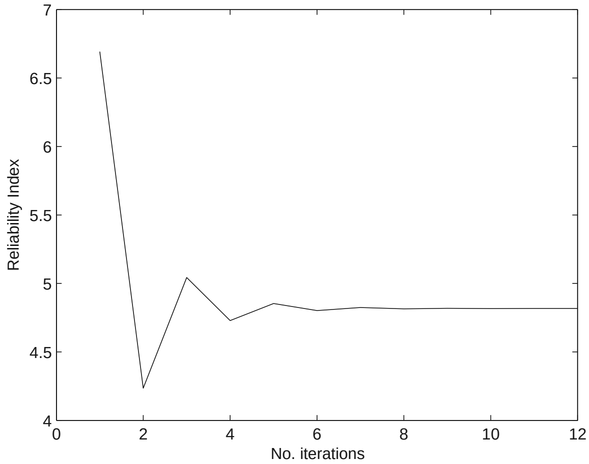
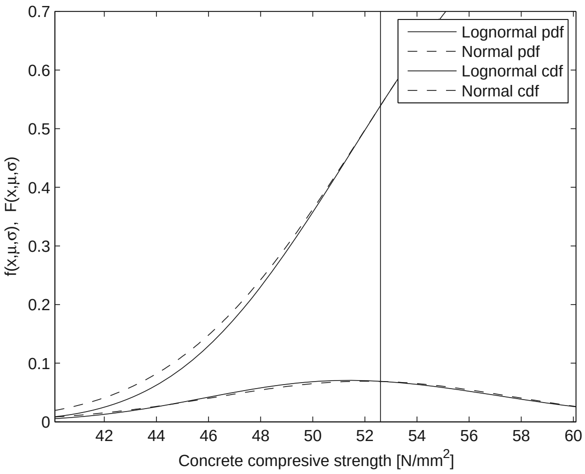


Figure 71: Convergence of the reliability index

In order to fit the non-normally distributed base variables, a transformation proposed by Rackwitz & Fiessler (1977) is used. A visual representation of the transformation is shown in figure 72 for the concrete strength. It can be seen from this figure that the cumulative distribution function and the probability density function are coinciding nicely in the design point (vertical line).



Vertical line = X-value in design point

Figure 72: Transformation of a log-normal distribution to a normal distribution

MATLAB SCRIPT

Within this part the used Matlab script is presented. The following elements of the script are presented: basic micro-task function file, self-checking and micro-task sequences, superior control, Monte Carlo Loading conditions, Monte Carlo Probability analysis and FORM analysis.

MICRO-TASK FUNCTION FILES

```
% B1.5.2: Estimate Cmin

function[Cminn] = Cminn(ndata)
[r] = find(ndata==2152);
FPCmin=ndata(r,3);
CMIN=ndata(r,2);
EMCmin=random('Normal',1,ndata(r,4));
RNCmin=rand(1);
if   RNCmin<FPCmin;
     Cminn=EMCmin*CMIN;
else Cminn=CMIN;
end
```

```
% B1.7.4: Calculate As;min;1

function[Output] = Asmin1n(ndata,fctm,fyk,Bt,d)
[r] = find(ndata==2174);
FP=ndata(r,3);
Parameter= 0.26*(fctm/fyk)*Bt*d;

EM=random('Lognormal',0,ndata(r,4));

FP=FP*0.70;
FP1=FP*0.105;
FP2=FP*0.015;
FP3=FP*0.03;

NP=1-FP;
DFP = randp([FP FP3 FP2 FP1 FP1 FP2 FP3 NP],1,1);

Output(DFP==1)=EM*Parameter;
Output(DFP==2)=10^-3*Parameter;
Output(DFP==3)=10^-2*Parameter;
Output(DFP==4)=10^-1*Parameter;
Output(DFP==5)=10^1*Parameter;
Output(DFP==6)=10^2*Parameter;
Output(DFP==7)=10^3*Parameter;
Output(DFP==8)=Parameter;
```

```
% B1.8.6: Decide top reinforcement layout

function[Asc_Prov,D_Asc,No_Asc] = Ascprovn(ndata,Astab,Asc)

[r] = find(ndata==2317);
FP=ndata(r,3);

FP1=FP/2;
NP=1-FP;
DFP = randp([NP FP1 FP1],1,1);

C1=(Astab(:)-Asc);
C1(C1 < 0) = [1000];
[~,ind1] = min(C1);
[m1,n1] = ind2sub(size(Astab),ind1);

Asc_Prov(DFP==1) = Astab(m1,n1);
D_Asc(DFP==1) = Astab(m1,1);
No_Asc(DFP==1) = Astab(1,n1);

C2=(Astab(:)-(Asc-(0.1*Asc)));
C2(C2 < 0) = [1000];
[~,ind2] = min(C2);
[m2,n2] = ind2sub(size(Astab),ind2);

Asc_Prov(DFP==2) = Astab(m2,n2);
D_Asc(DFP==2) = Astab(m2,1);
No_Asc(DFP==2) = Astab(1,n2);

C3=(Astab(:)-(Asc+(0.1*Asc)));
C3(C3 < 0) = [1000];
[~,ind3] = min(C3);
[m3,n3] = ind2sub(size(Astab),ind3);

Asc_Prov(DFP==3) = Astab(m3,n3);
D_Asc(DFP==3) = Astab(m3,1);
No_Asc(DFP==3) = Astab(1,n3);
```

```
NoSIM=100000;

for i=1:NoSIM

% Task 0: Determine material parameters
fykd(i)=fykn(ndata);
fckd(i)=fckn(ndata);
fctm(i)=fctmn(ndata);

% Task A: Calculate Reinforcement
% Step A1: Calculate beam height (with self-control)
count=0;
while 0.6*Hbcorrect > Hb(i) || Hb(i)>1.6*Hbcorrect;
%while 0.5*Hbcorrect > Hb(i) || Hb(i)>1.7*Hbcorrect;
a1(i)=a1n(ndata);
Ly(i)=Lyn(ndata);
Leff(i)=Leffn(ndata,Ly(i),a1(i));
Hb(i)=Hbn(ndata,Leff(i));
count=count+1;
if count>2;
    break
end
end

% Step A3: Calculate Distributed load/moment force(with self-control)

count=0;
while (5/11)*qlcorrect > ql(i) || ql(i) > (29/11)*qlcorrect ||
(6/13)*Mdcorrect > Md(i) || Md(i) > (45/13)*Mdcorrect || (6/13)*Mscorrect >
Ms(i) || Ms(i) > (45/13)*Mscorrect;
%while (4/11)*qlcorrect > ql(i) || ql(i) > (31/11)*qlcorrect ||
(5/13)*Mdcorrect > Md(i) || Md(i) > (47/13)*Mdcorrect || (5/13)*Mscorrect >
Ms(i) || Ms(i) > (47/13)*Mscorrect;
Yc(i)=Ycn(ndata);
qdb(i)=qdbn(ndata,Ac(i),Yc(i));
qds(i)=qdsn(ndata);
qks(i)=qksn(ndata);
Yi1(i)=Yi1n(ndata);
Yi2(i)=Yi2n(ndata);
qdl(i)=qdln(ndata,Yi1(i),qdb(i),qds(i));
qll(i)=qlln(ndata,Yi2(i),qks(i));
ql(i)=qdl(i)+qll(i);
Md(i)=Mdn(ndata,qdl(i),qll(i),Leff(i));
Ms(i)=Msn(ndata,qdl(i),qll(i),Leff(i));
count=count+1;
if count>2;
    break
end
end
```

```
for i=1:NoSIM
count=0;
% Task S: Checking by different person
while 0.6*Hbcorrect > HbS(i) || HbS(i)>1.6*Hbcorrect || (6/11)*Bbcorrect >
BbS(i) || BbS(i)>0.8*HbS(i) || BbS(i)>(24/11)*Bbcorrect || (6/13)*Mdcorrect >
MdS(i) || MdS(i) > (45/13)*Mdcorrect || (6/13)*Mscorrect > MsS(i) || MsS(i) >
(45/13)*Mscorrect || (500/1808)*Ast_Provcorrect > Ast_ProvS(i) || Ast_ProvS(i)
> (7541/1808)*Ast_Provcorrect || (500/1520)*Asb_Provcorrect > Asb_ProvS(i) ||
Asb_ProvS(i) > (7541/1520)*Asb_Provcorrect;
% Task 0: Determine material parameters
    fykS(i)=fykn(ndataS);
    fckS(i)=fckn(ndataS);
    fctmS(i)=fctmn(ndataS);
    YcS(i)=Ycn(ndataS);
% Step A1: Calculate beam height
    HbS(i)=Hbn(ndataS,Leff(i));
%Step A2: Calculate concrete area
    BbS(i)=Bbn(ndataS,HbS(i));
% Step A3: Calculate Distributed load / Moment force
    qdbS(i)=qdbn(ndataS,Ac(i),YcS(i));
    qdsS(i)=qdsn(ndataS);
    qksS(i)=qksn(ndataS);
    qdlS(i)=qdln(ndataS,Yi1S(i),qdbS(i),qdsS(i));
    qllS(i)=qlln(ndataS,Yi2S(i),qksS(i));
    qlS(i)=qdlS(i)+qllS(i);
    MdS(i)=Mdn(ndataS,qdlS(i),qllS(i),Leff(i));
    MsS(i)=Msn(ndataS,qdlS(i),qllS(i),Leff(i));
% step A5: Calculate Applied reinforcement
    dS(i)=dn(ndataS,HbS(i),Cmin(i),mr_sr(i));
    AstS(i)=Astn(ndataS,MsS(i),fykS(i),d(i));
    AsbS(i)=Asbn(ndataS,MdS(i),fykS(i),d(i));
    [AspracS(i),D_AspS(i),No_AspS(i)]=Aspracn(ndataS);
% Step D11: Choose Top/bottom reinforcement
    [Ast_ProvS(i),D_AstS(i),No_AstS(i)] = Astprovn(ndataS,Astab,AstS(i));
    AstS(i)=Ast_ProvS(i);
    [Asb_ProvS(i),D_AsbS(i),No_AsbS(i)] = Asbprovn(ndataS,Astab,AsbS(i));
    AsbS(i)=Asb_ProvS(i);
count=count+1;
if count>3;
    break
end

if Hb(i) < 0.7*HbS(i) || Hb(i) > 1.35*HbS(i) || Bb(i) < 0.7*BbS(i) || Bb(i) >
1.35*BbS(i) || Ast(i) < 0.7*AstS(i) || Ast(i) > 1.35*AstS(i) || Asb(i) <
0.7*AsbS(i) || Asb(i) > 1.35*AsbS(i);
Ast(i) = [0];
end

if Ast(i) > 0;
    No(i)=1;
else No(i)=0;
end
```

```
% model uncertainties
mumr=1;
sigmamr=0.05;
mr(i)=random('Normal',mumr,sigmamr);
mume=1;
sigmame=0.1;
me(i)=random('Normal',mume,sigmame);

% Permanent load beam
murhobeam=25;
sigmarhobeam=1;
murhocolumn=25;
sigmarhocolumn=1;
R = [1.000 0.7
     0.7 1.000];
s = [1;1];
V = s*s';
SIGMA = V.*R;
MU=[25,25];
R = mvnrnd(MU,SIGMA);
rhoB(i)=R(1,1);
rhoC(i)=R(1,2);
qbeam(i)=rhoB(i)*Hb(i)*Bb(i)*10^-6;

% Permanent load slab floors

murhoslab=25;
sigmarhoslab=1;
rhoslab(i)=random('Normal',murhoslab,sigmarhoslab);
muHslabMC=160;
sigmaHslab=1.12;
Hslab(i)=random('Normal',muHslabMC,sigmaHslab);
Lxreal(i)=Lxrealn(ndata);
qslab(i)=Lxreal(i)*rhoslab(i)*Hslab(i)*10^-3;

% Concrete strengths
mufck = fckd(i);
sigmafck = 0.123;
fcktr(i)=random('Lognormal',mufck,sigmafck);

mualphafck = 0.85;
sigmaalphafck = 0.085;
fck(i)=random('Normal',mualphafck,sigmaalphafck)*fcktr(i);

% Steel strengths
mufyk = 560;
sigmafyk = 30;
fyk(i)=random('Normal',mufyk,sigmafyk);
```

```
% Imposed load slab floor
for z = 1:10;
shapeqlong=.5;
scaleqlong=.637;
qlongtr(z)=random('Gamma',shapeqlong,scaleqlong);
end

for x = 1:5;
qlong(x)=qlongtr(1);
end
for x = 6:10;
qlong(x)=qlongtr(2);
end
for x = 11:15;
qlong(x)=qlongtr(3);
end
for x = 16:20;
qlong(x)=qlongtr(4);
end
for x = 21:25;
qlong(x)=qlongtr(5);
end
for x = 26:30;
qlong(x)=qlongtr(6);
end
for x = 31:35;
qlong(x)=qlongtr(7);
end
for x = 36:40;
qlong(x)=qlongtr(8);
end
for x = 41:45;
qlong(x)=qlongtr(9);
end
for x = 46:50;
qlong(x)=qlongtr(10);
end

for x=1:50;
muqshort=.2;
sigmashort=.32;
qshort(x)=random('Gamma',muqshort,sigmashort);
qimposedtr(x)=qlong(x)+qshort(x);
qimposed(i)=max(qimposedtr)*Lxreal(i);
end

% Transfer forces
qload(i)=qimposed(i)+qbeam(i)+qslab(i);
```

```
for i=1:NoSIMtr(ii)

% Failure Case 1; three hinges in beam element (UB1)

C1(i) = -6480000*(qload(i));
C2(i) = -(1/2)*((-
2*Asb(i)*Hb(i)*fck(i)*Bb(i)+2*Asb(i)*Cmin(i)*fck(i)*Bb(i)+2*Asprac(i)*Cmin(i)*
fck(i)*Bb(i)+fyk(i)*Asb(i)^2-
2*Asprac(i)*fyk(i)*Asb(i)+Asprac(i)^2*fyk(i))*fyk(i))/(fck(i)*Bb(i));
C3(i) = -(1/2)*((-
2*Ast(i)*Hb(i)*fck(i)*Bb(i)+2*Ast(i)*Cmin(i)*fck(i)*Bb(i)+2*Asb(i)*Cmin(i)*fck
(i)*Bb(i)+fyk(i)*Ast(i)^2-
2*Asb(i)*fyk(i)*Ast(i)+Asb(i)^2*fyk(i))*fyk(i))/(fck(i)*Bb(i));
FM1(i)=C1(i)*me(i)+(C2(i)+C3(i))*mr(i);

C4(i) = -12960000*qload(i);
C5(i) = (-
120*Asc(i)^2*fyk(i)*Cmin(i)*fck(i)^2*Bc(i)+60*Asc(i)^2*fyk(i)*Bc(i)^2*fck(i)^2
-
2680000*Asc(i)*fyk(i)*Cmin(i)*fck(i)*Bc(i)+1340000*Asc(i)*fyk(i)*Bc(i)^2*fck(i
)-
3366750000000*Asc(i)*fck(i)+10050000*Asc(i)*Bc(i)^2*fck(i)^2)/(fck(i)*Bc(i)*(6
70000+30*Asc(i)*fck(i)));
C6(i) = -(((-
2*Asb(i)*Hb(i)*fck(i)*Bb(i)+2*Asb(i)*Cmin(i)*fck(i)*Bb(i)+2*Asprac(i)*Cmin(i)*
fck(i)*Bb(i)+fyk(i)*Asb(i)^2-
2*Asprac(i)*fyk(i)*Asb(i)+Asprac(i)*fyk(i)*Asb(i)+Asprac(i)^2*fyk(i))*fyk(i))/
(fck(i)*Bb(i)));
C7(i) = -(1/2)*(((-
2*Ast(i)*Hb(i)*fck(i)*Bb(i)+2*Ast(i)*Cmin(i)*fck(i)*Bb(i)+2*Asb(i)*Cmin(i)*fck
(i)*Bb(i)+fyk(i)*Ast(i)^2-
2*Asb(i)*fyk(i)*Ast(i)+fyk(i)*Asb(i)^2)*fyk(i))/(fck(i)*Bb(i)));

FM2(i)=C4(i)*me(i)+(C5(i)+C6(i)+C7(i))*mr(i);

if FM1(i)<0 || FM2(i)<0;
    b(i)=1;
else b(i)=0;
end

bt(ii)=sum(b);
pfMC(ii)=sum(b)/NoSIMtr(ii)
BetaMC(ii)=sqrt(2)*erfinv((2*pfMC(ii)-1))

end
```

```
% model uncertainties
%X5: Model factor uncertainty beam/column
X5=mumr;
muX5=X5;
sigmaX5=sigmamr;

% Permanent load beam
%X7: rhobeam
X7=murhobeam;
sigmaX7=sigmarhobeam;
%X8: rhocolumn
X8=murhocolumn;
sigmaX8=sigmarhobeam;

% Lognormal Transformation concrete strength (X13)
fxX13(j)=lognpdf(X13,muX13tr,sigmaX13tr);
FxX13(j)=logncdf(X13,muX13tr,sigmaX13tr);
PhiInvX13(j)=sqrt(2)*erfinv((2*FxX13(j)-1));
fxnormalX13(j)=normpdf(PhiInvX13(j));
sigmaX13= fxnormalX13(j)/fxX13(j);
sigmaX13t(j)=sigmaX13

dX1(j)= X5*(Asb(i)*X14 + Ast(i)*X14) - (162*X2*X6*X7)/25;
dX2(j)= X5*((X14*(Asb(i)^2*X14 + Asprac(i)^2*X14 -
2*Asb(i)*Asprac(i)*X14 - 2*Asb(i)*X1*X13*X2 + 2*Asb(i)*X13*X2*X3 +
2*Asprac(i)*X13*X2*X3))/(2*X13*X2^2) + (X14*(Asb(i)^2*X14 +
Ast(i)^2*X14 - 2*Asb(i)*Ast(i)*X14 - 2*Ast(i)*X1*X13*X2;
2*Asprac(i)*X13*X3))/(2*X13*X2) - (X14*(2*Asb(i)*X13*X3 -
2*Ast(i)*X1*X13 + 2*Ast(i)*X13*X3))/(2*X13*X2)) - (162*X1*X6*X7)/25;

SX1(j)=(dX1(j)*sigmaX1);
SXk1(j)=(SX1(j))^2;

SigmaZ(j)=(SXk1(j)+SXk2(j)+SXk3(j)+SXk5(j)+SXk6(j)+SXk7(j)+SXk9(j)+SXk1
0(j)+SXk11(j)+SXk12(j)+SXk13(j)+SXk14(j))^0.5;
MX1(j)=dX1(j)*(muX1-X1);

SumMX(j)=MX1(j)+MX1(j)+MX2(j)+MX3(j)+MX5(j)+MX6(j)+MX7(j)+MX9(j)+MX10(j
)+MX11(j)+MX12(j)+MX13(j)+MX14(j);

alpha1(j)=(SX1(j)/SigmaZ(j));
alphak1(j)=(SX1(j)/SigmaZ(j))^2;

X1=muX1+alpha1(j)*BetaZtr(j)*sigmaX1;

BetaZ(i)=BetaZtr(NoFORM);
pfZ(i)=(1/2)*(1+erf(-BetaZ(i)/sqrt(2)));
pfZZ1(i)=pfZ(i)*50;

if pfZZ1(i)>1;
    pfZZ1(i)=1;
end
```