Design concrete slabs with openings

A study to the design process and structural behavior of concrete slabs with openings due to non-structural elements

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Master Thesis

door

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Preface

This report is my graduation thesis on the improvement of structural safety of wide floor slabs with embedded non-structural elements. This thesis is to complete the master Construction Management and Engineering (CME) and the master Structural Engineering (SE) at the Delft University of Technology. Besides that this thesis is to complete my masters, it also ends my student lifetime in which I attended both the Bachelor civil engineering as the master CME and SE in Delft.

In this thesis both the structural and managerial aspects of structural safety are covered. In the year making this thesis I have learned a lot about both fields and even more about the connections between both fields. At the beginning of the thesis I never expected that structural engineering and construction management are so intertwined.

I would like to thank KPCV and BAM AE for helping me with this thesis. I also want to thank the construction department of BAM AE in particular for their guidance during my thesis. I also would like to thank my thesis committee for their contributions throughout my study; Marian as being the chair for her knowledge on where to steer the thesis, the feedback and leading the formal meetings; Yuguang as university supervisor for his guidance in the field of structural engineering and the clever solutions for structural problems; Erik-Jan as university supervisor for his support and feedback during our meetings and his guidance towards an integrated thesis; Sander as company supervisor for the feedback and support during the face-to-face meetings and for always directing me to the people I needed for my thesis. Lastly, I want to thank all the interviewees and the task group installations of KPCV for their time and input to my thesis.

Nick Dubbeldam
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Summary

The main issue being discussed is the design of wide floor slabs with embedded non-structural elements, which has become more complex due to the increasing trend of placing more installations inside structural elements making the wide floor slabs a multidisciplinary design. This multidisciplinary design has resulted in various safety issues, including errors in the design due to the absence of a calculation method and a lack of integration between the different disciplines. To address these issues, both the structural problem (lack of calculation method) and managerial problem (insufficient integration) need to be solved simultaneously, as solving one without the other could lead to compliance issues. The research question is:

How could the structural safety of concrete floor slabs with openings due to embedded non-structural elements be improved?

In order to answer this research question, the influence of the non-structural elements on the bearing capacity of the slab is assessed, the integration process is assessed and possible errors are mapped, a solution is formed to resolve the safety error, and this solution is assessed on both its feasibility and effectiveness.

The structural analysis focuses on four parameters of the openings (shape, size, distance from the bottom of the slab, and distance from the support) and their effects are evaluated on three failure modes (bending moment failure, shear failure, and shear plane failure). A workflow of different calculation methods is formed to calculate the bearing capacity of a slab with an opening on these different failure modes. It is found that openings with a larger size lead to lower bearing capacity and that placing openings closer to the compression zone (in vertical direction) or farther from the support (in longitudinal direction) decrease the bearing capacity even more. Openings should not be placed within a distance equal to the height of the slab from the support to avoid brittle failure.

The management analysis assesses the integration process by examining the project process typically used in the building industry (as described by the DNR) and comparing it to a theoretical process. It was found that the DNR process is largely consistent with the theoretical process, but that fragmentation can lead to a phase shift between the structural and mechanical disciplines. Interviews revealed that two events contribute to this phase shift: late commissioning and quality issues in the mechanical design, both of which occur at the interface between the technical design (TO) and pre-construction design (UO) phases. The first event prolongs the TO phase, while the second event prolongs the UO phase. This lack of integration between the structural and mechanical disciplines makes proper design integration difficult.

Using the influence of the openings on the bearing capacity and the disrupting events a solution is proposed in the form of an integrated design strategy. This integrated design strategy contains both design rules and an adjusted design process. The design rules aim to clarify the design process and reduce the number of errors and the phase shift. They include the following: the opening cannot be larger than 40% of the slab height, the opening must be placed on the precasted floor slab, and the opening cannot be placed within a distance equal to the height of the slab from the support. If there is a deviation from these rules, the structural engineers must be notified. The adjusted design process introduces a preparation phase between the technical design (TO) and pre-construction design (UO) phases to detect clashes and make time estimates for the design process.

The feasibility and effectiveness of this integrated design strategy were assessed by an expert panel, who found that it was feasible if applied to all structural elements. In terms of effectiveness, they concluded that the strategy could only mitigate the problems caused by the design shift. They also suggested that other measures, such as changing the DNR-STB and using an integrated project delivery model (PDM), could prevent the events that lead to safety issues.

It was concluded that the integrated design strategy mitigates the events that lead to structural safety issues and therefore the integrated design strategy does improve structural safety. However, there might be better and more effective ways to prevent the events that cause structural safety issues. This can be investigated in further research.

Samenvatting

Het ontwerp van breedplaatvloeren met ingebedde installaties is complexer geworden door het toenemende aantal installaties dat in de vloeren wordt geplaatst. Dit multidisciplinaire ontwerp heeft geleid tot verschillende veiligheidsproblemen, waaronder ontwerpfouten vanwege het ontbreken van een berekeningsmethode en een gebrek aan integratie tussen de verschillende disciplines. Om deze problemen aan te pakken, moet zowel het constructieve probleem (gebrek aan een berekeningsmethode) als het managementprobleem (onvoldoende integratie) integraal worden opgelost, omdat anders de kans ontstaat dat de oplossingen van beide problemen niet samen gebruikt kunnen worden. De onderzoeksvraag is:

Hoe kan de constructieve veiligheid van betonnen breedplaatvloeren met openingen door ingebedde installaties worden verbeterd?

Om deze onderzoeksvraag te beantwoorden, wordt de invloed van de installaties op de draagkracht van de vloer bepaald. Daarbij wordt het integratieproces geëvalueerd en worden eventuele fouten in kaart gebracht. Ten derde wordt een oplossing gevormd om het veiligheidsprobleem op te lossen en wordt deze oplossing geëvalueerd op zowel haalbaarheid als effectiviteit.

Het onderzoek richt zich op vier opening parameters (vorm, grootte, afstand tot de onderkant van de vloer en afstand tot het steunpunt) en evalueert de effecten van de opening op drie faalmechanisme (buigend moment, dwarskracht en afschuiving van het schuifvlak) door middel van rekenmethodes samengevat in een workflow. Over het algemeen leiden grotere openingen tot een lagere draagkracht en leidt het plaatsen van openingen dichter bij de compressiezone of verder van de ondersteuning tot een kleiner effect van de opening of de draagkracht. Openingen moeten niet op een afstand worden geplaatst die gelijk is aan de hoogte van de vloer van het steunpunt om een bros bezwijken te voorkomen.

Het onderzoek heeft het integratieproces geëvalueerd door het projectproces te bekijken dat wordt gebruikt in de bouwindustrie (zoals beschreven door de DNR) en dit te vergelijken met een theoretisch proces. Het bleek dat het DNR-proces grotendeels gelijk is aan het theoretische proces. Fragmentatie in de bouw kan leiden tot een faseverschil tussen de constructie en installatie disciplines. Uit interviews bleek dat twee gebeurtenissen bijdragen aan dit faseverschil: late opdrachtverstrekking en kwaliteitsproblemen in het installatie ontwerp, die beide plaatsvinden tussen de technisch ontwerpfase (TO) en de uitvoeringsgereedheid ontwerpfase (UO). De eerste gebeurtenis verlengt de TO-fase, terwijl de tweede gebeurtenis de UO-fase verlengt. Dit fase verschil tussen de twee disciplines maakt een goede integratie van het ontwerp moeilijk.

Op basis van de invloed van de openingen op de draagkracht en de verstorende gebeur-

tenissen wordt een oplossing voorgesteld in de vorm van een geïntegreerde ontwerpstrategie. Deze geïntegreerde ontwerpstrategie bevat zowel ontwerprichtlijnen als een aangepaste ontwerpproces. De ontwerprichtlijnen hebben als doel het ontwerpproces te verduidelijken en het aantal fouten en faseverschil te verminderen en omvatten het volgende: de opening mag niet groter zijn dan 40% van de vloerhoogte, de opening moet op het prefab vloer worden geplaatst en de opening mag niet binnen een afstand worden geplaatst die gelijk is aan de hoogte van de vloer vanaf het steunpunt. Als er afgeweken wordt van deze regels, moeten de constructeur hiervan op de hoogte worden gesteld. Het aangepaste ontwerpproces introduceert een voorbereidingsfase tussen de TO fase en UO fase om clashes in het ontwerp te detecteren tijdsinschattingen te maken voor het ontwerpproces.

De haalbaarheid en effectiviteit van deze geïntegreerde ontwerpstrategie is geëvalueerd door een expertpanel. Het expertpanel heeft geconcludeerd dat de ontwerpstrategie haalbaar is als deze wordt toegepast op alle constructieve elementen. De ontwerpstrategie is alleen effectief in het verhelpen van de problemen die worden veroorzaakt door het faseverschil. Ze stelden ook voor dat andere maatregelen, zoals het veranderen van de DNR-STB en het gebruik van een geïntegreerd project delivery model (PDM), de gebeurtenissen die leiden tot veiligheidsproblemen kunnen voorkomen.

Het is geconcludeerd dat de geïntegreerde ontwerpstrategie de gebeurtenissen die leiden tot veiligheidsproblemen in de breedplaatvloer vermindert en daarom verbetert de geïntegreerde ontwerpstrategie de constructieve veiligheid. Er kunnen echter betere en effectievere manieren zijn om te voorkomen dat de gebeurtenissen die leiden tot veiligheidsproblemen te voorkomen en zo het constructieve veiligheid meer verbeteren. Dit kan in verdere onderzoeken worden onderzocht.

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List of abbreviations

Critical safety factor	(CSF)
Definitive design	(DO)
Knowledge platform for structural safety	(KPCV)
Mechanical, electrical, and plumbing	(MEP)
Pre-construction design	(UO)
Preliminary design	(VO)
Project Delivery Model	(PDM)
Sketch design	(SO)
Standard task description	(STB)
Strut-and-tie Model	(STM)
Technical design	(TO)
The new rules	(DNR)
Uniform Administrative Conditions	(UAV)

Chapter 1

Introduction

Our standard of living has changed significantly over the years. Instead of opening a window, a ventilation system is required, and due to the invention of the internet, all kinds of cables are required to fulfill our wish to always be connected. This has increased the number of systems we require in our buildings. These newly required systems increase the number of pipes and ducts in buildings. However, all these mechanical, electrical, and plumbing (MEP) systems are considered aesthetically unappealing. Therefore, more systems decrease the aesthetics of the building. To counter this problem the systems are embedded inside the structural elements.

Before all these systems were embedded in the structural elements, projects were relatively simple. The structural part of the designs was solely designed by structural engineers. However, nowadays multiple engineers from different disciplines need to be included to all design a part of the element, thus making the design of the element multidisciplinary. This makes the design process more complex, which can negatively influence the structural safety of the elements (K. C. Terwel, 2014). Analysis from the knowledge platform structural safety (KPCV) shows that most (near) collapses are caused by insufficient integration of the designs from different disciplines, and it mentions specifically the integration between the structural and MEP designs as a cause for design errors (KPCV[1], 2018).

The safety problem described above can happen in the designs of all kinds of structural elements. However, since most MEP systems are directed through the floor into a pipe shaft, the integration problems will mainly occur in the design of floors (Mooi, 2018). The floor system recommended as the best system to embed MEP systems in is the wide floor slab (Dycore[2], n.d.). In designs where this floor system is used, many design errors are detected. This is proven by the fact that the problem is chosen as the building error of the year of the Netherlands three times in the past four years by the Dutch building industry (Redactie bouwwereld, n.d.). In 2017 and 2020 too many elements were placed in the slab, while in 2019 the elements were placed in places where they cannot be placed. The errors are shown in figure 1.1.

The text above sets the scene for this research. Firstly, the background for this research and the link to the current safety issue are provided. Secondly, the research objective and research questions are formulated, and the scope is defined. Thirdly, the research approach explains how to answer the research questions within the scope. Lastly, the structure of the thesis is presented.

(a) Building error of 2017 (Redactie bouwwereld, 2017)

(b) Building error of 2019 (Redactie bouwwereld, 2019)

(c) Building error of 2020 (Redactie bouwwereld, 2020)

Figure 1.1: Building errors of multiple years

1.1 Why is structural safety still not guaranteed?

Structural safety is investigated extensively for years now (C. B. Brown, 1960; Chan et al., 2004; K. C. Terwel, 2014; van Duin, 1992). At first, the literature described structural safety as formulas of ratio and probability and combined these formulas to formulate acceptable risks (C. B. Brown, 1960). This led to the different safety factors we still use to this day ("NEN-EN 1992-1-1+C2:2011 nl", 2011). So structural safety was approached as a field in structural engineering, only influenced by probabilistic factors. Nowadays, the view of the literature on structural safety has changed. The literature describes that in most cases collapses due to design errors are merely a result of managerial errors in the design process (Chan et al., 2004; K. C. Terwel & Jansen, 2015; van Duin, 1992). The studies show that different managerial practices, such as the allocation of responsibilities, have a critical influence on structural safety, and design errors are mostly caused due to errors in these managerial practices (K. C. Terwel, 2014).

Investigations into the cause of a collapse by the industry after every (near) collapse show similar results. Most of these investigations point out that the cause of the collapse is a design error (CUR Bouw & Infra, 2005, 2010). This seems like a pure structural error since the structural engineers make the design and did something wrong. However, the investigators point out that the design errors are merely a cause of other errors rather than errors on their own. Most of the design errors are caused by errors in the managerial field, such as poor communication, a lack of allocation of responsibilities, or poor design change management (CUR Bouw & Infra, 2005). Thus confirming the literature that managerial practices have a big impact on structural safety. The investigators also point out that these managerial factors are not the only factors influencing structural safety. They argue that design errors are not only made due to errors in managerial practices but rather due to a combination of errors in the field of construction management and errors in the field of structural engineering (CUR Bouw & Infra, 2005).

The literature and all the investigations give a high number of recommendations, such as improvements in the allocation of responsibilities, improvements in communication and collaboration, improvements in risk analyses and allocation, improvements in the infrastructure of knowledge, and improvements in the control of designs (CUR Bouw & Infra, 2005, 2011; K. C. Terwel & Mans, 2015). The government and the building industry have tried to implement nearly all the recommendations in the form of registers for engineers and for (near) collapse, structural safety platforms, and the compendium structural safety for a clearer allocation of responsibilities (KPCV[4], 2018; Spekkink, 2011; VROM-inspectie et al., 2006).

Given all these initiatives to improve structural safety, one would expect nearly all buildings to be structurally safe. However, in recent years multiple buildings have party collapsed: the car parking of Eindhoven airport (Onderzoeksraad voor Veiligheid, 2018), the roof of the AZ stadium (Onderzoeksraad voor Veiligheid, 2020), and the stands of the Goffert stadium (Royal Haskoning DHV, 2022). The analyses of these major collapses show that in all three cases the collapse is caused by errors in the field of structural engineering and errors in the field of construction management.

Why is structural safety still not guaranteed? Although the initiatives to improve structural safety undoubtedly have achieved some success, they all solve a problem regarding only one problem, whereas the design errors occur due to a combination of problems (CUR Bouw & Infra, 2005). Currently, structural engineers cannot properly design an element due to a lack of structural knowledge, and a lack of information from other disciplines. The safety issue can only be resolved when both the structural knowledge and information are there to properly design the elements.

This also applies to the case of wide floor slabs with embedded non-structural elements. The design error, in this case, is caused by the absence of a calculation method, which is a knowledge problem, and by the fact that the structural engineers cannot take openings in the element into account due to missing information about the non-structural element caused by insufficient integration between the MEP and structural design. Solving only one problem doesn't suffice to resolve the design error since either the lack of information or the lack of a calculation method will cause the design error. Solving both problems separately also won't solve the problem, because the calculation method should be compatible with the design process. This can only be achieved by solving both problems integrally.

1.2 Research objective, research questions, and scope

Given that there is a safety issue in the design of wide floor slabs with embedded non-structural elements, that this safety issue is caused due to a knowledge problem in the field of structural engineering and an integration problem in the field of construction management, and that the safety issue can only be solved by an integrated solution which solves both the structural and management problem, we come to multiple objectives for this thesis.

The first objective is to investigate the effects of the embedded non-structural elements on the bearing resistance because without this knowledge the slab can't be designed. The second objective is to analyze the design process and find the cause of the integration problem. The aim is to resolve the safety issue in the design of wide floor slabs with embedded non-structural elements by proposing one feasible and effective design strategy for both disciplines. The main research question is:

How could the structural safety of concrete floor slabs with openings due to embedded non-structural elements be improved?

To answer the main research question four sub-questions are used. The sub-questions are:

1. *What is the influence of the embedded non-structural elements on the bearing capacity of a concrete wide floor slab?*
2. *What causes errors in the integration process of the MEP and the structural design?*
3. *What solutions can solve the structural safety issue?*
4. *How feasible and effective is the solution for both disciplines?*

Since this thesis covers an integrated problem that extends over both the structural and managerial disciplines, the topic is very broad. To keep the problem manageable it is wise to limit the scope. The scope limitations have to be set in both fields of disciplines. Firstly, the scope limitations in the field of structural engineering are described, after which the scope limitations in the field of construction management are described.

For the structural discipline, boundary conditions need to be set since the amount of possible structural models is endless. Firstly, it is assumed that the non-structural elements are positioned over the full width of the slab and that the dimensions of the non-structural element are homogeneous over the full width. Secondly, only wide floor slabs with a one-way load transfer capability will be assumed in this thesis, since the wide floor slabs were essentially intended to allow one-way load transfer. This also means that only line supports will be used. It is assumed that only 2 supports are used since this case can be used to isolate the effects of the openings.

Another scope limitation needs to be set for the project delivery model (PDM). There are multiple PDMs that are used in the construction industry. These PDMs influence the design process differently since for each PDM different project members work in different phases on different objectives. In a bouwteam for example the contractors are involved early in the project, while the traditional PDM strictly separates the client and contractors. Wide floor slabs with embedded non-structural elements are mostly used in projects where suspended ceilings are not applicable. These projects are mostly smaller projects, such as house-building projects or apartment complex projects. For these kinds of smaller projects, the traditional PDM is most commonly used (Forbes & Ahmed, 2011). Therefore, In this thesis, it is assumed that a traditional process is used.

1.3 Research approach

This thesis aims to resolve a safety issue caused by problems in both the field of structural engineering and construction management. First of all, a calculation method is needed to be able to calculate the bearing capacity of an element. Secondly, proper design integration is needed to give the engineers information about the non-structural elements so that they can properly apply the calculation method.

To make sure that there is a calculation method before the integration between the designs is analyzed, this thesis makes use of three phases. Phase 1 contains the analysis of the structural problem and the development of the calculation method and phase 2 contains the analysis of the integration problem. These analyses are quite extensive and are therefore split using multiple sub-questions. The research method of these phases is discussed in chapter 1.3.1 and 1.3.2. The third phase contains the development and assessment of the integrated design strategy. The research method of this phase is discussed in chapter 1.3.3.

1.3.1. Phase 1: Analysis of the structural problem

In this phase, the influence of the embedded non-structural elements on the bearing resistance of the wide floor slab is explored. The goal is to investigate the effects of the non-structural elements on the bearing resistance of the wide floor slab and to find out to which extent the non-structural elements can be embedded in the wide floor slab. The main question of this phase is: "What is the influence of the embedded non-structural elements on the bearing capacity of a concrete wide floor slab", which is the first sub-question of this research. In order to answer the main question of this phase, four sub-questions are used:

1.1 *What are the properties and dimensions of the wide floor slabs and the non-structural elements?*

1.2 *How does the common layout of the elements in the slab look like?*

1.3 *What are the failure modes of concrete floor slabs?*

1.4 *How do the elements influence the failure modes?*

The first sub-question of this phase explores the properties of both the non-structural elements and the wide floor slab. These properties are needed to eventually calculate the bearing capacity of the slab and to calculate what the effects of the non-structural elements are.

The second sub-question looks into the possible existence of a common layout for the non-structural elements which can be used in most cases and what this layout looks like. If there is a common layout, only this layout has to be calculated, which reduces the problem.

The third sub-question investigates the current failure modes of wide floor slabs and investigates how the bearing resistance against these failure modes should be calculated. This sets a baseline for the bearing resistance of a full slab.

The fourth sub-question investigates how the failure modes and calculation methods of sub-question three are changed due to the addition of non-structural elements. It is possible that the non-structural elements change the behavior of the slab and therefore will influence the failure modes and bearing capacity. Insight into this change of behavior is needed to map the influence of the non-structural elements on the bearing capacity of the wide floor slab.

Using the four sub-questions the influence of the non-structural elements on the bearing resistance of the wide floor slab can be calculated for each failure mode. The bearing resistance for each failure mode is calculated for different positions of the non-structural element. The position of the non-structural element can change in both the longitudinal and depth direction. The result of this phase will be multiple diagrams for each failure mode showing the change in bearing resistance of each position of non-structural elements compared to the bearing resistance of a full slab.

1.3.2. Phase 2: Analysis of the managerial problems

Errors in management processes are always caused by underlying factors (K. C. Terwel & Jansen, 2015). Since design integration is such a management process, it can be assumed that there are underlying events that cause the integration error. These events should be known before the integration error can be solved. The goal of this phase is to find the events that cause the integration error, to map why these events occur, and to map how these events influence the integration process. The main question of this phase is: "What causes errors in the integration process of the MEP and the structural design?", which is the second sub-question of this research. In order to answer the main question of this phase, multiple sub-questions are used:

2.1 *What is the current design process to design wide floor slabs with embedded non-structural elements and how does this differ from the theory?*

2.2 *What events disrupt the design integration and how do they occur?*

2.3 *How can these events be prevented?*

The first sub-question of this phase investigates the current design process used in the Dutch building industry and investigates if there are differences with the theory. The design process shows when parts of designs are made, by whom, and when integration takes place. Comparing this to the theory can show deficiencies in the current design process and can uncover the events that cause integration errors.

The second sub-question of this phase investigates the events that disrupt the design process that is described in sub-question one. Knowing what these events are, why they occur, and how they occur is of key importance for solving the integration error.

The third sub-question explores ideas to prevent the events from occurring. Preventing the events to happen will solve the integration error since the causes for the error are prevented. Therefore, it is important to map the options to prevent these events from happening.

The end result of this phase will be a set of events that contribute to the integration error. For each event it is described what the event is, why it occurs, and how it occurs. For each event options are listed to prevent the events from occurring.

1.3.3. Phase 3: Development of the design strategy

In the third and final phase of this research, a design strategy is formed and evaluated to answer sub-question 3 and 4 of the research. To answer sub-question 3 a design strategy is formed using the analyses of phases 1 and 2. Firstly, using phase 1 boundary conditions can be described for the design strategy. It describes to which extent the non-structural elements can be embedded in the wide floor slab. Secondly, using these boundary conditions and the events discovered in phase 2, a design process can be proposed which prevents the events from occurring and therefore solves the integration error. The design process should describe starting conditions for a project, and specific tasks that need to be done in project phases while regarding the boundary conditions set in phase 1.

To check if the proposed design strategy would work in practice and thus answering sub-question 4 two checks are performed. The first check regards the feasibility of the design strategy. An expert group will be asked to examine whether it is feasible to use the proposed design strategy in the building industry. The second check regards the effectiveness of the design strategy. The same expert group is asked to examine how effective the proposed design strategy is in solving the integration problem and improving the structural safety of the wide floor slabs with embedded non-structural elements. The feasibility assessment is done first since it can be assumed that the design strategy must be feasible before it can be effective.

The end result of this phase is a feasible and effective design strategy that improves the structural safety of wide floor slabs with embedded non-structural elements. Thus answering the research question.

1.4 Methodology

In order to answer the research questions and different sub-questions, different research methodologies are used. These will be explained per phase.

1.4.1. Phase 1

Phase 1 analyses the effects of the non-structural elements on the bearing capacity of a slab. To do so literature and information of BAM are used. Firstly, information about the material properties is gathered. Information about the wide floor slab is gathered using a wide floor slab design made by a subcontractor of BAM. Information about the material properties of the non-structural elements is gathered from the manufacturers of the non-structural elements. Secondly, information is gathered about the failure modes. Using Eurocode 2 information about the failure modes and bearing capacity of full slabs can be gathered ("NEN-EN 1992-1-1+C2:2011 nl", 2011). Information about slabs with openings is derived from the literature. Using the basic principles of the different failure modes and these derivations from the literature calculation methods for the bearing capacity can be derived.

After the information gathering and derivation of the calculation methods, the effects of the non-structural elements can be calculated. This is done by programming the calculation methods into Python to calculate the effects. The outcome is then validated using FEM.

1.4.2. Phase 2

Phase 2 analyses the events that cause errors in the integration process. Firstly, using the literature and information from the building industry, the difference between the theoretical design process and the design process in practice will be mapped. Secondly, using information from employees of BAM AE and the literature possible disrupting events will be mapped. Using information from the employees of BAM AE possible activities to prevent these events from happening will be listed.

To evaluate that the disrupting events really disrupt the design process semi-structured interviews are used. Different employees from BAM AE will be interviewed and asked if they have experienced the events in their projects and what the effects were. This will show how many events occur and what happens when they occur.

1.4.3. Phase 3

In phase 3 a design strategy is formed. For this design strategy, there is no new information necessary, since the information from phases 1 and 2 is used. However, feasibility and effectiveness are assessed. For these assessments, an expert group is used. Since this thesis is written in collaboration with KPCV, an expert group from KPCV will be asked to assess the design strategy. KPCV is a platform that describes methods to improve structural safety in the building industry (KPCV[4], 2018). Various companies from the construction industry are involved in this platform. An expert group of KPCV will be asked to do the assessment. Since the expert group is formed with employees from different construction companies, it is considered a good representation of the construction industry.

1.5 Thesis outline

Figure 1.2 shows the thesis outline with chapters. The figure links the different phases to the chapters. The thesis starts with the introduction and literature review in chapter

1 and 2. Then phase one starts. In chapter 3 the failure modes are discussed and formulas for the bearing capacity will be adjusted to take the non-structural elements into account (phase sub-question 3). In chapter 4 the slab design and layout are presented (phase sub-question 1 and 2) and in chapter 5 the influence of the non-structural elements on the bearing capacity of the slab will be calculated (phase sub-question 4).

After phase 1 is finished, phase 2 starts. In chapter 6 the project phases used in the construction industry are explained and compared to the theory (phase sub-question 1). Secondly, in chapter 7 the factors that influence the design integration and the factor that most disrupts the integration are discussed (phase sub-question 2). In the same chapter possible solutions to prevent these risks are described (phase sub-question 3).

After both phases 1 and 2 are finished, phase 3 is started using the outcome of the earlier phases. This new design strategy will be proposed in chapter 8. Subsequently, in the same chapter, the design strategy will be validated by the expert panel of KPCV. Lastly, chapter 9 will provide a discussion, conclusion, and recommendation for further research.

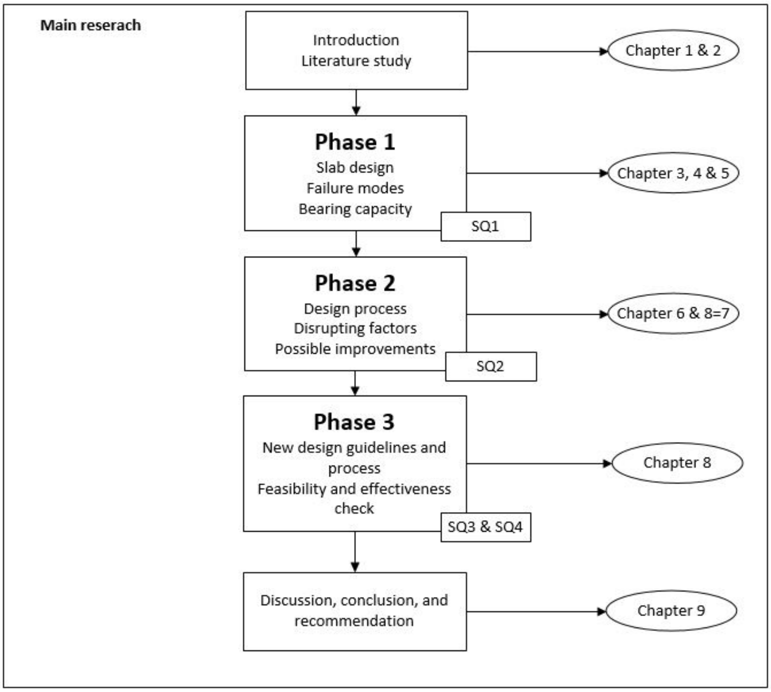


Figure 1.2: Thesis overview

Chapter 2

Literature review

This chapter discusses the literature review which is performed to gain knowledge about the subject and find gaps in the literature. Firstly, the meaning and aspects of structural safety are explained. Secondly, the factors that influence structural safety are explored, after which the initiatives to improve structural safety are shown. Lastly, the knowledge gaps in the literature are explained.

2.1 What is structural safety?

There are a lot of different definitions for structural safety. The Eurocode defines structural safety as the "capacity of a structure to resist all action(s), as well as specified accidental phenomena, it will have to withstand during construction work and anticipated use" (ISO 6707-1:2004 art. 3.7.3.82). This is a purely technical definition focusing on the fact that the strength (or capacity) should always be greater than the maximum load (or actions). This seems logical since a building won't collapse if the strength is greater than the maximum load (Madsen et al., 1986).

In practice, an absolute guarantee that failure will not occur can never be given since we don't have perfect knowledge about strengths and loads, nor do we have perfect analytic techniques and models. There is a possibility that things occur which are so unlikely that we cannot take them into account in any rational analysis (Elms, 2004; Madsen et al., 1986). Therefore, there are a lot of uncertainties and due to these uncertainties, an absolute guarantee that the strength is higher than the load cannot be given. Therefore, the structural safety as described by the Eurocode cannot be reached.

Elms (1999) describes structural safety differently. He described that "a structure is safe if it will not fail under foreseeable demands, and is unlikely to fail under extraordinary demands or circumstances" (Elms, 1999). This means that structural safety can only be reached if the foreseeable demand is covered and the likelihood of extraordinary demands is at an acceptable level. Elms argued that codes, such as the Eurocode, are very well capable of dealing with the foreseeable demand, and that management is needed to coop with the likelihood of extraordinary demands (Elms, 1999).

Terwel (2014) described a similar approach to structural safety, although in different words. He defined structural safety as "the absence of unacceptable risk associated with failure of (part of) a structure" (K. C. Terwel, 2014). he furthermore describes that the Eurocode is capable of dealing with uncertainties in loads and capacities using limit-state design (ISO 6707-1:2004 art. 9.1.18) and that management practices are needed to deal with human errors.

Looking at the definitions for structural safety from both the Eurocode and literature, it can be concluded that structural safety concerns the limitation of the risk of failure to an acceptable level. This can only be achieved by a combination of codes to deal with the foreseeable demand, and management practices to deal with the extraordinary demand. (Elms, 1999, 2004; K. C. Terwel, 2014). This applies to structural safety in general. But how do the codes and the management practices guarantee structural safety for the design of wide floor slabs?

2.2 Codes and calculation methods

In the Netherlands Eurocode 2 is used for the design of concrete elements ("NEN-EN 1992-1-1+C2:2011 nl", 2011). This code describes formulas and design rules to calculate the strengths of concrete elements. These formulas and design rules in this code are gathered from years of research on all kinds of concrete elements. The formulas are generic formulas, which enable the designers to check the strength of structural elements on different failure modes. The code doesn't give specific formulas or exceptions for elements with openings in the cross-section, which implies that generic formulas should be used.

Studies show however that these generic formulas give an inaccurate value for the shear strength. Hanson tested the shear strength of a joist with squared openings in the cross-section (Hanson, 1969). The experiments show that the inaccuracy of the calculated values depends on both the size and the position in the x and y direction of the opening. When these parameters are increased, the difference between the calculated strengths and the actual strengths also increased. Somes and Corley did the same test for joists with circular openings, which gave the same results (Somes & Corley, 1973). When the results of Hanson and Somes and Corley are compared, it is shown that the shape of the opening also affects the shear strength of an element (Hanson, 1969; Somes & Corley, 1973).

Since the design codes didn't provide an accurate calculation method for the design of concrete elements with openings in the cross-section, the literature proposed an improved method. Mansur proposed that not all beams with openings can be calculated in the same way. He proposed that beams with an opening smaller than 40% of the total beam height should be calculated according to the design codes, while beams with an opening bigger than 40% of the total beam height should be calculated as the summation of the shear strength of the part above the opening and the shear strength of the part below the opening (Mansur, 2006). This method gave a conservative value for the shear strength of beams without stirrups. So there is still no accurate calculation method.

Mansur also proposed that the ultimate bending moment is not affected by openings if the openings are positioned in a certain way. When the openings are fully placed in the tension zone of structural elements, the ultimate bending moment won't be affected by the opening. The cracking moment is always negatively affected by an opening. This leads to earlier cracking of the element (Mansur, 1998).

The Eurocode doesn't specify what happens with the bearing capacity of a structural element with an opening in the cross-section in case of other failure modes (Punching shear, bending, and cracking), nor does the literature. This means that generic formulas should be used. So, except for the information the literature describes, there is not much known about the effects of openings in the cross-section of concrete elements.

2.3 Managerial influencing factors

There are a lot of managerial events that can influence structural safety. Not all these factors have the same degree of influence; some will influence structural safety more than others (Ren et al., 2020; K. C. Terwel & Mans, 2015). To understand how management influences structural safety, we first need to know what influences structural safety the most.

As mentioned before, management deals with the extraordinary demand for structural safety. Extraordinary demands are the events that are so unlikely that you could never expect them to happen, such as construction errors (Elms, 1999). These events have been studied for years. After nearly every collapse a study has been performed on the events that lead to the collapse. The earlier studies mostly assessed the technical aspect of designing and tried to find out why the building collapsed. These studies show that most technical failures are caused by extraordinary demands(Frühwald et al., 2007; Inspectorate of Housing & environment (VROM)-Inspectie, 2013).

The studies describe only the human errors, but don't describe why these human errors are made; an error is made in the design, but is this due to technical incompetence, or ignorance, or does it have another reason? Later studies tried to map the different managerial activities that cause these human errors (Baccarini & Collins, 2003; Ren et al., 2020; K. C. Terwel & Mans, 2015; K. Terwel et al., 2013). These studies describe that not all of these activities have the same impact and not all activities lead to human errors. Since some activities are more likely to cause human errors and are therefore key activities which are necessary to achieve a favorable result for structural safety, they are named critical success factors (CSF) (Rockart, 1982).

Terwel (2015) describes that the CSFs that have the most impact on structural safety are the project factors. These are related to the interaction between various project partners and are mostly managerially orientated. These factors are communication and collaboration, control mechanisms, allocation of responsibilities, structural risk management, safety culture, and knowledge infrastructure. In addition to these CSFs, Ren (2020) described that also knowledge and professional competence are factors that highly influence structural safety. So it can be concluded that structural safety is mostly impacted by communication and collaboration, control mechanisms, allocation of responsibilities, structural risk management, safety culture and knowledge infrastructure, knowledge, and professional competence.

2.4 Improvements on structural safety

Using the CSFs (Ren et al., 2020; K. C. Terwel & Jansen, 2015), a lot of initiatives are proposed by the literature and implemented by the construction industry to increase the structural safety in general and to increase the structural safety of wide floor slabs in particular.

The literature proposes measures to improve both the structural part and the managerial part of structural safety. There are endless studies and books describing the theoretical probabilistic and the safety factors that need to be applied to ensure the building is reasonably safe (Elms, 2004; Fraudenthal et al., 1966; Shinozuka, 1983). These studies eventually led to the safety factors that are described in the Eurocode (European Commision, n.d.). This is an improvement on the structural part of structural safety. Terwel (2014) described managerial measures and divided them into three categories: legal, organizational, and behavioral. The improvements vary from more clearness and completeness to the use of BIM and clash detection to a positive attitude towards risk management. All measures have in common that they all improve at least one CSF.

The construction industry and the government have implemented a lot of the proposed solutions from the literature together with solutions from the construction industry itself. One of the solutions is the implementation of the knowledge platform structural safety (KPCV). This platform is implemented to improve structural safety by addressing common problems and proposing solutions to these problems (KPCV[4], 2018). KPCV doesn't address the critical factors separately but rather bundles them into bigger subjects, such as design integration. Each of these subjects is connected to the factors as described by Terwel and Ren (Ren et al., 2020; K. C. Terwel & Jansen, 2015). The platform has integrated a lot of the solutions proposed by the literature into its solutions. For example, KPCV suggests using BIM to improve the design integration and to improve the process of design changes (KPCV[1], 2018; KPCV[2], 2018).

Next to the knowledge platform, several legal and non-legal regulations were implemented to improve the design process by making it more uniform. The European governments implemented the Eurocode to standardize the calculation methods and set boundaries for the required calculations using literature (European Commision, n.d.). The Dutch government implemented UAV to make a clear division of responsibilities between the client and the contracting parties (UAV, 2012). Also, the ISO Norms are implemented to standardize components of all sorts of disciplines (ISO, n.d.).

Next to the different codes and norms the DNR 2011 was implemented to standardize the responsibilities and tasks for each party involved in a project (BNA, 2013). The DNR gives guidelines about the project phases and the activities in the different phases. The Code of Conduct from NEPROM was introduced to enforce engineers to demonstrate good conduct using three core values: care, integrity, and social responsibility (NEPROM, 2021).

To improve the design of wide floor slabs with embedded non-structural elements the guideline for pipes in wide floor slabs were introduced (AB-FAB & Uneto-Vni, 2017). These guidelines ensure that the non-structural elements fit in the slab and that they are not in positions where they cannot be due to structural reasons. The guidelines consist of 12 points describing the placement height of the elements in the slab, the maximum thickness of the elements, the minimum spacing of the elements, and the minimum required thickness of the slab and concrete above the elements. These guidelines however are not supported by literature and are made using practical knowledge from experienced contractors and are mostly meant to give clarity about the placement of non-structural elements. This results in fewer arguments between the designers from the different disciplines (Mooi, 2018).

All solutions proposed by the literature and then taken by the industry are focused on improving one or more critical success factors. Since these critical success factors are managerial factors, the solutions are focused to improve the managerial aspect of structural safety.

2.5 Knowledge gap

Structural safety concerns the limitations of the risks of failure to an acceptable level. This can only be achieved by a combination of codes and management practices (Elms, 1999, 2004; K. C. Terwel, 2014). The codes give calculation methods in the field of structural engineering, while the management practices are subjects in the field of construction management. A lot of codes, regulations, platforms, and other initiatives were implemented over the years(BNA, n.d.; European Commision, n.d.; KPCV[4], 2018; NEPROM, 2021). These initiatives together should ensure structural safety in general.

The code used in the Netherlands for the design of concrete elements is Eurocode 2 (“NEN-EN 1992-1-1+C2:2011 nl”, 2011). The code describes calculation methods for the design of concrete elements. However, it doesn’t give a different calculation method for the design of elements with openings in their cross-section, while studies show that a different calculation method is needed (Hanson, 1969; Mansur & Tan, 1999; Somes & Corley, 1973). Therefore, designers are unable to design wide floor slabs with embedded non-structural elements, which are structurally safe.

The other initiatives are focused on improving structural safety by focusing on multiple CSFs and they are made to improve these CSFs in particular phases of the project (KPCV[4], 2018; Ren et al., 2020; K. C. Terwel & Jansen, 2015). For wide floor slabs, it is unknown what causes the integration error and in which phase of the project this error is made. It can be a lack of communication in an early phase of the project, a deficiency in the project process, or something completely different. This thesis aims to find what error causes the integration error and where this error is made.

Lastly, there is a mix of initiatives. Some improve the structural part of structural safety, such as the codes, while others improve the managerial part of structural safety. However, none of the initiatives improves both parts of structural safety. This means that implementing one initiative is not enough to improve structural safety. Implementing two or more initiatives is also not possible since the initiatives are not always compatible. So to ensure structural safety an integrated approach is needed containing both a calculation method and a managerial approach. This research aims to form such an integrated approach.

Chapter 3

Failure mechanisms

The literature review in chapter 2 has shown that the Eurocode describes calculation methods for the capacity of slabs. However, the literature shows that openings in the cross-section have a big impact on the bearing capacity of a slab and that the calculation methods don't give accurate results for the bearing capacity of slabs with openings in the cross-section. To guarantee the structural safety of those slabs, a new calculation method is required for the calculation of the bearing capacity of slabs with openings in the cross-section. The aim of this chapter is to form a calculation method for the bearing capacity of slabs with openings in the cross-section.

The bearing capacity of a slab with openings depends on different factors. First of all, it depends on the opening itself. The opening can have different shapes and sizes and can be placed on different points in the slab. The literature has shown that all these different factors influence the bearing capacity of a concrete element (Hanson, 1969; Somes & Corley, 1973). Secondly, the bearing capacity of a slab in general depends on the failure mechanism. A slab has multiple failure mechanisms each with its own calculation method for the capacity against this mechanism. These mechanisms should be known before a calculation method can be formed.

This chapter will discuss all the different factors. Firstly, the different parameters for the openings are defined to create a reference on which a calculation method can be formed. Secondly, the calculation methods for bending moment capacity will be explained for a full slab, and a new calculation method will be formed for the case of a slab with an opening in the cross-section. Thirdly, the same thing is done for the different shear capacities. The SLS failure modes are not covered in this thesis since failure in these failure modes doesn't lead to dangerous situations and therefore does not impact structural safety. The punching shear capacity will also not be covered due to upcoming changes in the Eurocode, which makes it physically impossible to place non-structural elements in punching shear regions.

3.1 Openings in the slab

Openings in the cross-section will influence the bearing capacity of a concrete element. However, not all openings influence the capacity to the same degree. The geometry and placement of the opening highly influence the effects of the opening on the bearing capacity. In total four different factors of the opening can be distinguished that influence the effects: the shape, the size, the longitudinal placement, and the placement in height direction (Hanson, 1969; Somes & Corley, 1973). All different factors are explained below.

The geometry factors are the shape and size of the opening. There are two kinds of shapes used for installations: rectangles and circles. Therefore, the openings in the slab will only have either a rectangular shape or a circular shape. The size of the opening also has a big effect on the bearing capacity and the calculation methods. Larger openings reduce the bearing capacity more than smaller openings, and the bearing capacity of slabs with larger openings should be calculated differently than the bearing capacity of slabs with small openings (Mansur, 1998). In this thesis, the size of the opening is defined as the height of the opening and will be presented by the parameter H.

The placement factors are the longitudinal placement and the placement in the height direction. The longitudinal placement of the opening is defined as the distance between the support or point load to the center of the opening and is denoted in this thesis with the parameter X. Studies have shown that the failure mode that occurs depends on this factor (Hanson, 1969; Somes & Corley, 1973). The placement in the height direction is defined as the distance between the bottom of the slab to the center of the opening and is denoted in this thesis with the parameter Y. Studies have shown that this movement in the Y direction has either a negative or positive effect on the bearing capacity (Hanson, 1969; Somes & Corley, 1973). All factors of the openings with their parameters are shown in figure 3.1.

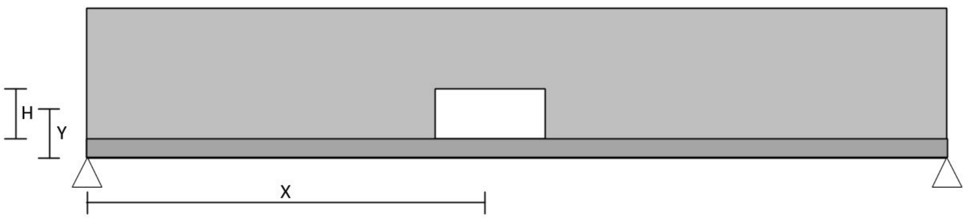


Figure 3.1: Graphical presentation of a simple support slab with openings

In the figure above a simply supported slab with openings is shown. All calculation methods formed in this thesis will be based on this model since the scope of this thesis is limited to a simply supported slab with a one-way carrying capability. A uniform distributed load is assumed as load as described by Eurocode 1 (European Commision, n.d.)

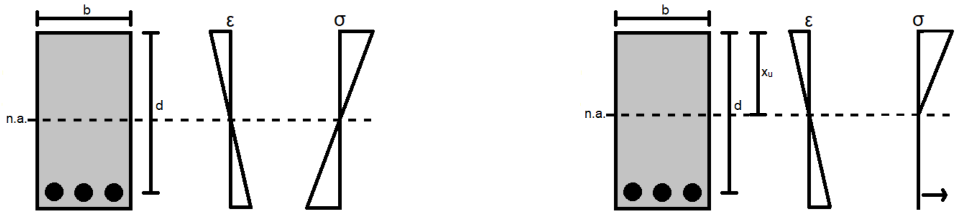
3.2 Bending moment capacity

The bending moment is created by multiple compression and tensile forces. Failure in bending moment is therefore always failure due to tensile or compressive forces. Therefore, Eurocode 2 doesn't give any formulas for the calculation of the bearing capacity ("NEN-EN 1992-1-1+C2:2011 nl", 2011).

The damage process and failure process of reinforced concrete can be described using four characteristic points: cracking moment (M_{cr}), yielding moment (M_y), crushing moment, and ultimate moment (M_u). The cracking moment is the moment at which the concrete in the tension zone starts to crack. After this point, the reinforcement steel starts to bear tension forces. The yielding moment is the moment at which the reinforcement steel starts to yield. This happens at 2.175‰ strain. The elastic moment (M_e) is the moment at which the compression zone of the concrete starts to crush. This happens at 1.75‰ strain. Lastly, the ultimate moment is the moment at which the ultimate concrete crushing strain (3.5‰ strain) is reached in the compression part of the concrete. At this moment the element will fail (Braam & Lagendijk, 2011). The cracking moment capacity can be calculated using a formula, while the other moment capacities are calculated using a workflow.

3.2.1. Moment capacity without openings

Although the cracking moment capacity and the other moment capacities are calculated differently, they use the same assumption; The strain distribution over the height is always linear as shown in figure 3.2. The strain distribution over the height however is different for every moment capacity. Firstly, the formula to calculate the cracking moment capacity will be explained, after which the workflow to calculate the other moment capacities is shown.



(a) Strain and stress at the cracking moment

(b) Strain and stress at the elastic moment

Figure 3.2: Strain and stress at different bending moment points

The cracking moment capacity is calculated using formula 3.1, which is derived using the stress-strain relationship of figure 3.2a (Wijte, 2020).

$$M_{cr} = \frac{f_{ctd} I}{e} \tag{3.1}$$

with:

f_{ctd}	design concrete tensile strength	[MPa]
I	second moment of inertia	[mm^4]
e	height between center of gravity and bottom fibre	[mm]

The calculation of the other moment capacities uses 3 main components: a strain distribution over the height, 3 horizontal forces, and a horizontal force equilibrium. The strain distribution over the height is different for each moment capacity and depends on the known strains, e.g. the 2.175‰ strain in the tensile reinforcement for the yielding moment. The three horizontal forces are the reinforcement tensile forces, the reinforcement compression forces, and the concrete compression force. The formulas for those three forces are shown in equation 3.2. Adding the forces together should always give zero.

$$F_{st} = A_{st} f_{ywd} \tag{3.2a}$$

$$F_c = \int_0^x \epsilon(z)\, dz E_c b \tag{3.2b}$$

$$F_{sc} = A_{sc} \epsilon(z_{sc}) E_s \tag{3.2c}$$

with:

A_{st}	area of tensile reinforcement	[mm^2]
f_{ywd}	yield strength of steel	[MPa]
b	width of the cross-section	[mm]
x	height of compression zone	[mm]
$\epsilon(z)$	strain relation	[-]
E_c	Young's modulus concrete	[MPa]
A_{sc}	area of compression reinforcement	[mm^2]
$\epsilon(z_{sc})$	strain at height of the compression reinforcement	[-]
E_s	Young's modulus steel	[MPa]

The workflow makes use of those three components in an iterative process to calculate the height of the concrete compression zone. This concrete compression zone is needed to calculate the moment capacity. To calculate the concrete compression zone the workflow implements the strain distribution into the force formula. Using the fact that the forces should be in equilibrium, a height for the concrete compression zone can be found. Implementing this height in the forces formulas again and multiplying the forces by their lever arms gives the moment capacity. This workflow is shown in figure 3.3.

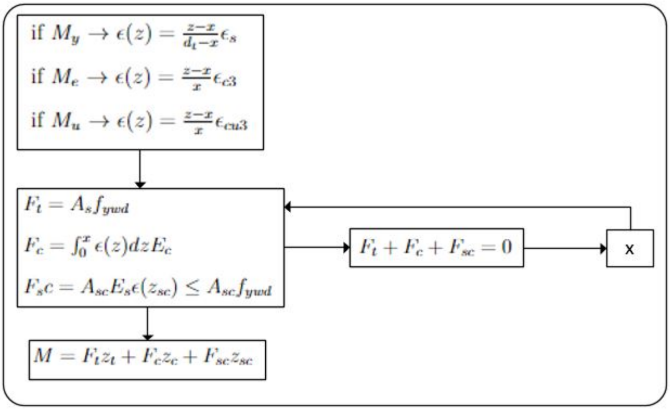


Figure 3.3: Workflow for the calculation of moment capacities

3.2.2. Moment capacity with openings

Openings have only a marginal effect on the moment capacities. According to the literature, the cracking moment capacity is mostly affected and the other moments are only affected if the opening interferes with the concrete compression zone (Mansur & Tan, 1999). Due to this marginal effect, formula 3.1 and the workflow shown in figure 3.3 can still be used. However, some parameters need to be changed.

For the calculation of the cracking moment capacity in the case with openings using formula 3.1, only the second moment of inertia changes. The opening decreases the amount of material in the cross-section, thus decreasing the second moment of inertia. The center of gravity shifts if the opening is not fully placed on the center of gravity. This also influences the value of the second moment of inertia.

3

The workflow shown in figure 3.3 can be fully used in case the opening does not interfere with the concrete compression zone, since the opening only diminishes the amount of concrete and therefore only influence the compression capacity of the concrete. When the opening does interfere with the concrete compression zone, a change in the workflow has to be implemented. Since the opening diminishes the amount of concrete that carries part of the horizontal compression force, the cross-section can carry less compression force. This has to be taken into account. This can be done by changing the formula for the concrete compression force (F_c) from the formula shown in equation 3.2 to the formula shown in equation3.3. By forming these equations it was assumed that strain is linear over the height and that the full cross-section has the same concrete strength.

$$F_c = \int_{Hov}^{x} \epsilon(z)\, dz E_c b \tag{3.3}$$

In this formula the Hov is the distance between the ultimate fiber of the compression zone to the top of the opening. An adjusted workflow for the calculation of the moment capacities in the case that the opening interferes with the concrete compression zone is shown in figure 3.4. An example moment calculation for the case with and without openings is included in Appendix A.

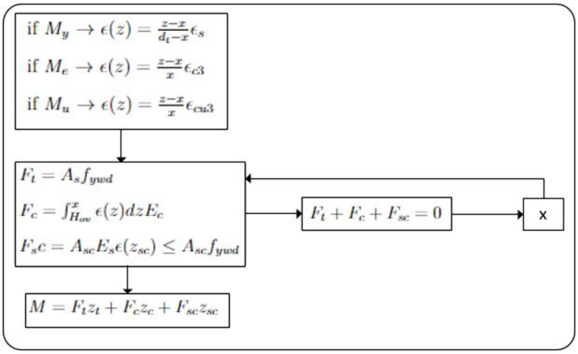


Figure 3.4: Adjusted workflow for the calculation of the moment capacities

3.3 Shear capacity

This thesis covers four different shear failure modes: shear tension failure, shear compression failure, flexural shear failure, and interface failure. interface failure is a failure mode that is unique for composite elements, while the other failure modes can occur in every concrete structure. Therefore it is covered separately in section 3.4. That leaves the other three failure modes for this chapter. Firstly, the failure modes and the resistance against these failure modes are explained for an element without openings, after which the same is done for an element with openings.

3

3.3.1. Shear capacity without openings

Shear tension failure, shear compression failure, and flexural shear failure are not just three separate failure modes. The failure modes are quite similar. Shear tension failure and flexural shear failure are actually the same failure mode, but the latter occurs due to bending cracks while shear tension cracks are purely caused by shear forces (ACI-ASCE Committee 426, 1973; Nawy, 2009; Regan, 1993). Shear tension failure and shear compression failure can occur in the same place on a structural element (Braam & Lagendijk, 2011). Despite the similarities, the resistance against these failure modes is calculated differently. All the failure modes and their resistance are explained below.

Flexural shear failure

Flexural shear failure happens gradually i.e. ductile failure. Cracks caused by the bending moment are developed further due to shear forces. This decreases the concrete compression zone and eventually, the concrete will crush (Braam & Lagendijk, 2011). Eurocode 2 describes multiple formulas to calculate the shear bearing capacity. Since shear reinforcement isn't used in wide floor slabs, only the formulas without shear reinforcement are used. The shear resistance formulas for elements without shear reinforcement are summarised in formula 3.4 (NEN EN 1992-1-1, art. 6.2.2, 2011).

$$v_{Rd,c} = [max(C_{Rd,c}k(100\rho_l f_{ck})^{\frac{1}{3}}; 0.035k^{\frac{3}{2}} f_{ck}^{\frac{1}{2}}) + k_1\sigma_{cp}]b_w d \tag{3.4}$$

with:

$C_{Rd,c}$	constant, given by: $\frac{0.18}{\gamma_c}$	[-]
k	constant, given by $1+\sqrt{\frac{200}{d}} \leq 2.0$	[-]
ρ_l	percentage reinforcement, given by: $\frac{A_{sl}}{b_w d}$	[-]
f_{ck}	characteristic cubic concrete strength	[MPa]
k_1	constant with value 0.15	[-]
σ_{cp}	compression stress due to Normal or prestress stress, given by: $\frac{N_{Ed}}{A_c}$	[Mpa]
b_w	width of the slab	[mm]
d	effective height of the slab	[mm]

Shear tension failure

Shear tension shear failure happens suddenly. In parts of the element where the bending moment is lower than the cracking moment resistance, a crack can develop leading to a sudden collapse of the element. Since this happens suddenly, this type of failure is considered brittle (Braam & Lagendijk, 2011). The resistance can be calculated using formula 3.5 (NEN EN 1992-1-1, art. 6.2.2, 2011).

$$V_{Rd,c} = \frac{I b_w}{S} \sqrt{f_{ctd}^2 + \alpha_l \sigma_{cp} f_{ctd}} \tag{3.5}$$

with:

I	Second moment of area	[mm4]
b_w	width of the cross-section	[mm]
S	first moment of the area above the centroidal axis	[mm^3]
f_{ctd}	design tensile strength of concrete	[MPa]
α_l	prestressing factor	[-]
σ_{cp}	concrete compressive stress at centroidal axis due to axial loading	[MPa]

Shear compression failure

Shear compression failure is a brittle way of failure in which a crack penetrates the concrete compression zone which leads to crushing of the concrete. This failure mode occurs near support and is related to a high amount of reinforcement (Braam & Lagendijk, 2011). There are no formulas in Eurocode 2 for calculating the resistance against this failure mode. However, research has already proven that the strut-and-tie model (STM) method can be used to make a good estimation of the bearing resistance of elements having the shear compression failure mode (Collins et al., 2008; Walraven & Lehwalter, 1989; Wight & Parra-Momtesinos, 2003).

The STM method makes use of struts which are in compression and ties which are in tension. The struts and ties transfer the load from the point of load to the support. The points where the struts and ties come together are nodes. These nodes should always be in equilibrium. Figure 3.5 shows a strut-and-tie model.

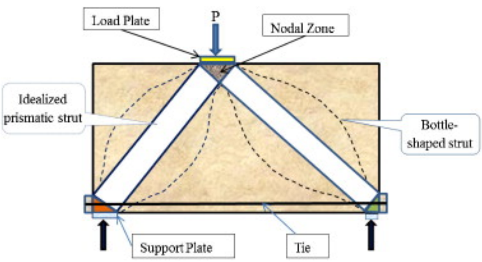


Figure 3.5: Strut-and-tie model components (Shuraim, 2013)

The force a strut can resist depends on the stress a strut can resist, the width of the strut, and the width of the element. The stress a strut can resist can be calculated using the formulas in Eurocode 2 (NEN EN 1992-1-1 art. 6.5.2, 2011). The width of the strut can be calculated using an idealized strut as shown in figure 3.5. The formula for both the width and the stress resistance is shown in formula 3.6.

$$w_s = l_b \sin(\theta) + w_t \cos(\theta) \tag{3.6a}$$

$$\sigma_{Rd,max} = 0.6\nu' f_{cd} \tag{3.6b}$$

with:

3

w_s	Width of the strut	[mm]
$\sigma_{Rd,max}$	maximum stress resistance	[MPa]
ν'	factor, calculated by $1 - \frac{f_{ck}}{250}$	[-]
l_b	length of the support node i.e. the length of the bearing plate	[mm]
w_t	height of the support node calculated by 2(h/d)	[mm]
θ	angle of strut	[°]
f_{cd}	design value of concrete compressive strength	[Mpa]

Only the vertical component of the force will resist the shear forces. Therefore, the vertical component has to be calculated. The vertical component can be calculated using the angle of the strut and geometry. The formula to calculate the shear resistance of a strut is shown in equation 3.7.

$$V_{strut} = \sigma_{Rd,max} w_s b_w \sin(\theta) \tag{3.7}$$

A downside of the STM method is that it only works well for elements loaded with a point load. Floors however are mostly subjected to a uniform distributed load as prescribed by Eurocode 1 (NEN EN 1991-1-1, 2019). To counter this problem the literature proposes to use equivalent loads which roughly represent the uniform distributed load. It is important that these equivalent loads give nearly the same moment and shear distribution as the uniform distributed load (M. D. Brown & Bayrsk, 2007). Using this method the STM method can be used on floors with uniform distributed loads.

3.3.2. Shear capacity with openings

When openings are introduced, the concrete elements start to behave differently. The different failure modes can't be separated so strictly as in the case without the openings. Research shows that the failure mode and shear capacity of an element can change due to openings. The shear capacity of a structural element with an opening depends on the four parameters described in section 3.1 (Hanson, 1969; Somes & Corley, 1973). The effects of the parameters on the bearing capacity are explained below per parameter.

Size of the opening

The bearing capacity is first of all affected by the size of the opening. Mansur and Tan (1999) describe that bigger openings have more effects on the bearing capacity than smaller openings. Thereby, they describe that the calculation method for these effects should be split into two methods; One method for calculating the bearing capacity of an element with small openings and one method for calculating the bearing capacity of an element with large openings.

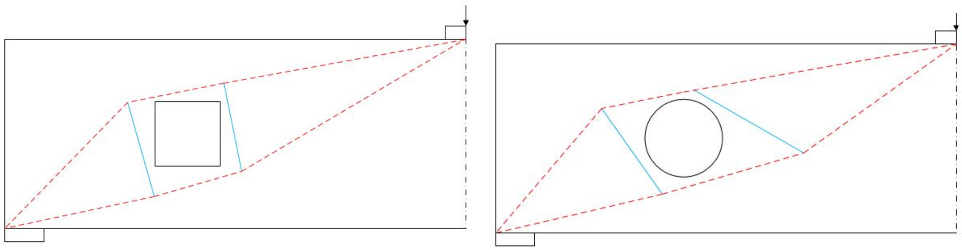
Small openings are defined as openings with a height smaller than 40% of the height of the slab (Mansur & Tan, 1999). Mansur (1998) describes that in the case of a small opening, the effective height of the slab is reduced by the height of the opening. Using this adjusted effective height, the formulas from the codes can be used to calculate the shear bearing capacity. Thus, in the case of small openings formula 3.4 can be used, but the effective height has to be adjusted.

Large openings are defined as openings with a height larger or equal to 40% of the element height (Mansur & Tan, 1999). Mansur (1998) describes that in the case of a large opening, the element cannot be seen as 1 slab when calculating the bearing capacity, but rather as 2 separate slabs; one element below the opening and one above the opening. These elements are named chords in this thesis. Both chords have their own parameter and own shear capacity. Adding the shear capacity from both chords together gives the shear capacity of the slab. Since the chords have their own parameters and are seen as separate elements, the type of element can change for example a chord can be a deep beam. Therefore, formula 3.4 does not always apply. The way of calculating the shear capacity of the chords depends on the position in the X and Y direction of the opening and is explained below.

Positioning of the opening

So when large openings are introduced in the slab, the shear capacity of the chords above and below the opening is calculated separately. Depending on the longitudinal and height placement (X and Y parameter), the calculation method for the chords changes. In total two different calculation methods can be used to calculate the shear capacity of the chords. Firstly, the two calculation methods are explained, after which the conditions to use these methods are covered.

The first calculation method is using formula 3.4. Both chords have their own height and thus their own effective height. Using this effective height and the other properties of the chords, the shear capacity can be calculated. It should be noted that the effective height of the chord which is mostly in the compression of the full element is equal to the concrete cover of the reinforcement.



(a) STM around a rectangular opening

(b) STM model around a circular opening

Figure 3.6: STM model (red = strut, blue = tie)

The second calculation method is using the STM method. Literature shows that when an opening is introduced, a strut above and below the opening can be formed as shown in figure 3.6 (Karimizadeh & Arabzadeh, 2021; Kong & Sharp, 1977). The shear strength of these struts is the shear strength of the chord where they are positioned in. The shear strength of the struts can be calculated using formula 3.7. How to draw these struts around the opening and calculate the angle of these struts is calculated in appendix B.

Now the calculation methods for the chords are known, the conditions in which the methods should be used can be explained. These conditions depend on the X and Y parameter. In total 3 conditions are separated. Firstly, analysis of experimental data shows that the shear strength of both the top and bottom chords should be calculated using the STM method when the opening is placed (partly) within a distance equal to the height of the slab from the support (Hanson, 1969; Somes & Corley, 1973).

Secondly, when the height of a chord is smaller than or equal to the width of the opening, the shear strength of the chord should also be calculated using the STM method. The height of the chord for slabs with rectangular openings is defined as the distance between the bottom\top of the slab to the edge of the opening. For slabs with circular openings, the height is defined as the distance between the bottom\top of the slab to the center of the opening.

Thirdly, when the height of a chord is bigger than the width of the opening, the shear strength of the chord should also be calculated using formula 3.4 and the parameters of the chords. It should be noted that situations can occur where one chord should be calculated using the STM method, while the other should be calculated with formula 3.4.

Decision tree

So the calculation for the shear capacity of a slab with an opening contains a lot of different calculation methods and thresholds to use these methods. Depending on the size of the opening, the shape of the opening, the longitudinal placement of the opening, and the height placement of the opening, different calculation methods should be used. To clarify the calculation method, the method of calculating the shear capacity for different cases of slabs with openings is summarised in a decision tree. This decision tree is shown below.

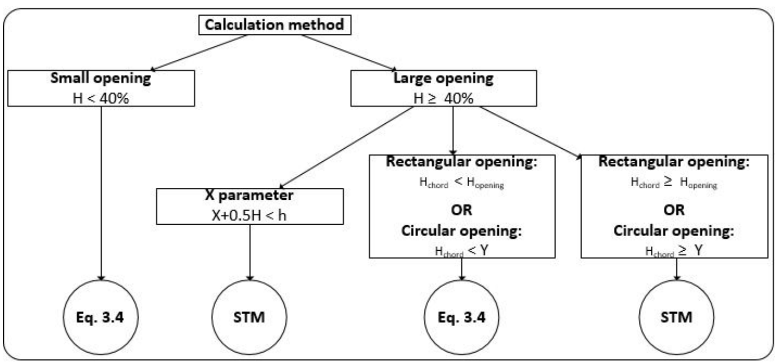


Figure 3.7: Calculation method decision tree

3.4 Interface capacity

A wide floor slab consists of two parts: a precasted plank floor and a cast in-situ top layer. Both layers are cast at different times. The two layers are held together by the cohesion of the interface of the layers. When the shear stress is too high, the two layers can delaminate causing the slab to fail. This failure mode is the shear failure of the interface between the two concrete elements. The resistance against this failure mode can be calculated using formula 3.8 (NEN EN 1992-1-1 art. 6.2.5, 2011).

$$v_{Rdi} = c f_{ctd} + \mu\sigma_n + \rho f_{yd}(\mu \sin\alpha + \cos\alpha) < 0.5\nu f_{cd} \tag{3.8}$$

with:

c	roughness factor	[-]
μ	roughness factor	[-]
σ_n	stress caused by normal forces perpendicular to the shear plane	[Mpa]
ρ	percentage shear plane reinforcement calculated by A_s/A_i	[%]
A_s	Area of reinforcement crossing the shear plane	[mm^2]
A_i	Area of the shear plane	[mm^2]
α	angle of reinforcement on the shear plane	[°]
ν	factor calculated with: $1 - \frac{f_{ck}}{250}$	[-]

In formula 3.8 μ and c are roughness factors that depend on the degree of roughness of the surface of the concrete elements. The surface of the precasted floor plank is roughened in the fabric using a rake. According to NEN-EN 1992-1-1 art. 6.2.5 the roughness factors for raked surfaces are μ = 0.7 and c = 0.4.

So the interface capacity depends on the cohesion between the slabs and the reinforcement holding the slabs together. Since openings don't change the properties of the concrete nor the properties of the interface, it can be concluded that the openings do not change the cohesion between the slabs. It should be noted that the opening decreases the interface area, which reduces the cohesion locally. However, since the shear capacity is calculated as a stress resistance over the full slab, this effect can be neglected.
When the opening is so big that it doesn't fit between the lattice girder, the lattice girder is cut away. This influences the amount of reinforcement that holds the slabs together. A reduced amount of reinforcement does reduce the interface capacity. Therefore, the size of the opening does influence the interface capacity. However, only certain parameters are influenced, and therefore formula 3.8 can be used for both the case with and without the openings.

Chapter 4

Case study and element layout

In the previous chapter, the theory behind the failure mechanisms is explained and formulas are formed to calculate the effects of openings on the bearing capacity of concrete elements. To explore the effects of the openings on a wide floor slab a case study is used. This case study consists of a wide floor slab used in a project made by BAM AE.

As shown in figure 4.1, wide floor slabs consist of different components each having its own properties. These properties together give the properties of the wide floor slab. Understanding the function of all parts is therefore important to understand how the slab works and to correctly calculate the bearing capacity of the slab in the case study. The maximum and minimum dimensions of the non-structural elements are also important since this gives the maximum and minimum opening size. In this chapter, the different components of the wide floor slabs are explained to create a greater understanding of the slab. Secondly, the non-structural elements that are commonly placed in the slabs are explained to establish the maximum and minimum size of the elements. The possibility of a common layout is explored. Lastly, the slab from the case study is shown and the properties of this slab are explained.

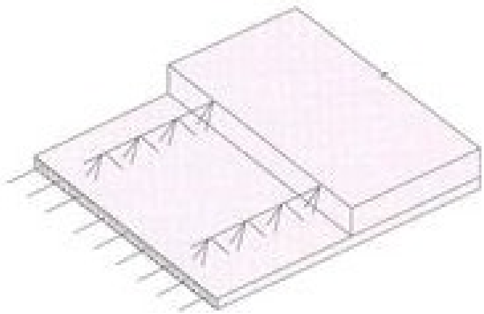


Figure 4.1: Schematic wide floor slab (the concrete society, n.d.)

4.1 Components of the wide floor slab

Figure 4.1 shows the different components of the wide floor slab. The wide floor slab consists of three components: the precasted floor plank, the lattice girder, and the cast in-situ top layer. All these elements have different functions in the building process and have different contributions to the bearing capacity of the slab. All elements are explained below.

4.1.1. Precasted floor plank

The precasted floor plank is a precasted concrete slab element that is used as the bottom part of the formwork and the bottom part of the entire slab. The dimensions of the slab depend on the load and length between the supports. However, the standard width is either 2400 or 3000 mm and the maximum length is 12 meters. The minimum concrete class is C20/25 and the environmental class is at least XC1. The plank is reinforced with a reinforcement grid of B500 reinforcement steel. The diameter and amount of reinforcement depend on the loads acting on the slab. In addition to the reinforcement, the floor can be prestressed. This is done using prestressing cables with class FEP 1670 or FEP 1770 (Dycore[1], n.d.; Havebo, n.d.).

4.1.2. Lattice girder

The lattice girder consists of multiple steel bars: one top bar, multiple diagonal bars, and two bottom bars as shown in figure 4.2. All the bars are connected using welds. The girder is used as a connection between the two concrete layers and as a lifting point to lift the precasted plank in place on the building site. It increases the shear resistance of the shear plane between the concrete elements. The girder is used as a spacer for the top reinforcement in the cast in-situ layer. The height of the girder can vary from 50 mm to 500 mm (Havebo, n.d.; Kiwa, 2017).

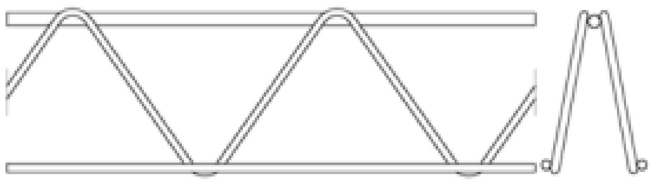


Figure 4.2: Schematic lattice girder (Kiwa, 2017)

The top bar is made from B500 reinforcement steel. In case B500A smooth bars are used, the bars may not be included in the strength calculation, since B500A smooth bars don't satisfy the geometry requirements of the NEN 6008 'Betonstaal' for class B500 reinforcement steel. In all other cases, the bars may be included in the strength calculations. The minimum diameter of the bars is 6 mm (Kiwa, 2017).

The diagonal bars are only made from B500A smooth reinforcement steel. Therefore, they may not be used for strength calculations, since the geometry requirements of NEN 6008 'betonstaal' for class B500 reinforcement steel are not satisfied. However, they can be used in the calculations for the shear resistance in the shear plane (Braam & Lagendijk, 2011). The diameter of the diagonal bars is at least 4 mm (Kiwa, 2017).

The bottom bars are made from B500 reinforcement steel and the bars are ribbed. The bars do fully meet the requirements of the NEN 6008 'Betonstaal' for class B500 reinforcement steel. Therefore, they can be used as reinforcement steel in the bearing capacity calculations. The minimum diameter of the bars is 5 mm (Dycore[1], n.d.; Havebo, n.d.; Kiwa, 2017).

4.1.3. Cast in-situ top layer

The cast in-situ top layer is a concrete layer cast on top of the precasted floor plank to complete the slab. It is cast as one big slab over the whole floor, so over multiple precasted planks. Therefore, the width and length depend on the dimensions of the floor. The minimum concrete class is C20/25 and the minimum environmental class is XC1. The main reinforcement consists of grids made from B500 reinforcement steel. The diameter and amount of reinforcement depend on the load loading the slab. This reinforcement is situated on top of the lattice girders (Dycore[1], n.d.; Havebo, n.d.).

There are also 2 kinds of additional reinforcement: hogging and coupling. Hogging reinforcement is used to resist hogging moments. The reinforcement consists of bars made from B500 reinforcement steel. The reinforcement is positioned on top of the main reinforcement. It is only put in places where there are hogging moments, so only over supports. The diameter and amount of reinforcement depend on the value of the hogging moment.

Coupling reinforcement is used to couple multiple plank floors to each other to enable them to bear forces in two directions. The reinforcement is made from B500 reinforcement steel and situated directly on top of the floor planks. The reinforcement can either be single bars or grids and is only put in places where multiple floor planks come together. The diameter and amount of reinforcement depend on the load it has to bear (Dycore[1], n.d.; Havebo, n.d.).

4.2 Properties and positioning of non-structural elements

There are multiple non-structural elements that can be used embedded in concrete slabs all having their own properties. Despite all the different properties, the elements can be divided into three groups: pipes, bundles, and ducts. All the elements in one group have different shapes; pipes are round, ducts are rectangular, and bundles are a combination of multiple pipes (AB-FAB & Uneto-Vni, 2017). The characteristics and properties of the non-structural element groups are described in chapter 4.2.1. The positioning of the elements is also important. An element in the middle of the beam will have a different effect on the bending moment resistance than an element on the side of the floor. An element on the precasted plank floor will influence other failure mechanisms than the element at the top of the floor and multiple elements in one slab will also influence the failure mechanisms. The positioning of the elements is discussed in chapter 4.2.2.

4.2.1. Non-structural elements

As mentioned before, there are three kinds of non-structural element groups: a pipe, a duct, and a bundle. All elements in one group have their shape in common. However, the properties and characteristics of the elements in one group can differ. To give an overview of all the different elements, the characteristics and material properties of each element group are described below.

The first element group to discuss is pipes. Pipes are categorized by their round shape (AB-FAB & Uneto-Vni, 2017) and are used in a big variety of systems, such as ventilation-, sewage-, drinking water-, electricity, and data systems. However, electricity and data cables are mostly bundled and are therefore considered bundles in the research. Due to this wide variety of usage pipes are made out of a wide variety of materials and in a wide variety of sizes. For example, a ventilation system is mostly made out of steel or PVC and can have pipes with a diameter up to 200 mm (Nijburg-klimaattechniek, n.d.), while a sewage system is mostly made out of PVC or plastic and can have a diameter up to 160 mm. The maximum wall thickness is 12 mm (Wavin, 2022). So pipes are highly adaptive to their usage. Depending on this usage, the diameter of the pipe can vary from 32 mm to 200 mm and the maximum wall thickness is 12 mm. The material of the pipe can vary between PVC, plastics, or steel (Nijburg-klimaattechniek, n.d.; Wavin, 2022).

The second element group which needs to be discussed is the duct. The duct is defined as a rectangular element (AB-FAB & Uneto-Vni, 2017). The ducts are mostly used in the ventilation systems and are therefore mostly one of the biggest elements which are embedded in concrete slabs. The materials used for ducts are plastic of steel (buldit bv, n.d.). The ducts are mostly 80 mm high, and have a wall thickness of 8 mm. They can have a width of up to 250 mm. However, the sizes can be altered if requested (Nijburg-klimaattechniek, n.d.).

The third element group is the bundle. The bundle is described as a bundle of pipes (AB-FAB & Uneto-Vni, 2017). However, in most cases, only small pipes, such as individual electricity and data cables are bundled, since the sewage, ventilation, and drinking water pipes are so big that it is not desirable to bundle these. The data- and electricity cables are bundled in pipes. These pipes are made from PVC and have a maximum diameter of 150 mm (Wavin, 2022). Since these pipes are bundled, the width of a bundle is in theory unlimited. However, the guidelines for pipes in wide floor slabs give restrictions for these bundles: bundles are not allowed to have a bigger width than 250 mm (AB-FAB & Uneto-Vni, 2017).

So there is a wide variety of non-structural elements which are embedded in wide floor slabs. The size of the non-structural elements varies from 5 mm for small cables to 250 mm for ventilation pipes (AB-FAB & Uneto-Vni, 2017; buldit bv, n.d.; Nijburg-klimaattechniek, n.d.; Wavin, 2022). Depending on what fits in the wide floor slab, these elements are placed in the slab. Therefore, in this thesis, it is assumed that the lower boundary for the opening size is 5 mm and the upper boundary is what maximal fits in the wide floor slab.

4.2.2. Positioning of the non-structural elements

Wide floor slabs are promoted as a good option to easily embedded non-structural elements in the floor (Dycore[2], n.d.). This means that embedding elements in floors is common in the construction industry. Some buildings, like houses, have such a similar design that they can even be built in fabrics (Redactie bouwwereld, 2021). Therefore, it can be expected that there is a standard layout for non-structural elements in wide floor slabs for some buildings. But in most projects, the MEP installations are constructed on-site (Snel, n.d.) and not in the same fabric as the slabs. Every client likes to have installations at different points in their houses. Therefore, there is no standard layout for non-structural elements in wide floor slabs.

Despite the absence of standard layouts for non-structural elements in the slabs, there is some guidance on the placement of these elements. This guidance was needed due to the ever-increasing amount of non-structural elements embedded in the wide floor slabs, which resulted in a lot of conflicts between different designers and constructors. Therefore, guidelines were made and combined in the guidelines for pipes in wide floor slabs (AB-FAB & Uneto-Vni, 2017). These guidelines point out 12 rules for the better and safer design of wide floor slabs with embedded elements. A translated version of these guidelines is shown in Appendix C. Although these guidelines are only meant as guidelines, they are considered by the building industry as rules and are nearly always used while designing slabs with embedded elements.

The guidelines however are not verified by research. They are made by the organizations of the industry to guide different designers to a more successful design (Redactie bouwwereld, 2021). Despite the lack of scientific verification, the guidelines can still be used for this research. The guidelines have a simple and easily understandable format. Therefore, it is also good to use this format for the result of this part of the thesis.

4.3 Case study slab

To calculate the influence of the non-structural elements on the wide floor slab a case study is used. The case study makes use of a wide floor slab which is designed by a subcontractor and verified by BAM AE. The wide floor slab is used in a project in the Netherlands and in the original design, there are no non-structural elements embedded in the slab. This particular slab is used because the design is a standard design containing all kinds of reinforcement (Bottom-, top-, coupling-, and hogging-reinforcement). The properties of the slab will be explained using the three separate elements as used in chapter 4.1.

4.3.1. Precasted floor plank

The precasted floor plank is 2400 mm wide, 6720 mm long, and 70 mm thick. The concrete is made for environmental class XC1 and has a strength class of C35/45. The slab is reinforced with a reinforcement grid. The diameter of bars in this grid varies from 6 mm to 16 mm. The area of the main reinforcement has a cross-sectional area of 618 mm^2/m, while the distributional reinforcement has a cross-sectional area of 123.6 mm^2/m. These areas don't include the reinforcement of the lattice girder. The steel is made of class B500A reinforcement steel. The concrete cover under the reinforcement is 25 mm. All properties of the precasted floor plank are shown in figure 4.1

Table 4.1: properties precasted floor plank

Parameter	Value	Unity
Width	2400	mm
Length	6720	mm
Height	70	mm
Bottom concrete cover	25	mm
Cross-sectional area of main reinforcement	618	mm^2/m
Cross-sectional area of distributional reinforcement	123.6	mm2/m
Characteristic cubic concrete strength	45	MPa
Characteristic steel yield strength	500	MPa

4.3.2. Lattice girder

There are four lattice girders in this case study. All girders have the same dimensions and properties, and all are placed on top of the bottom reinforcement in the precasted plank. However, the difference in height between the bottom bars of the lattice girder and the reinforcement grid of the precasted plank is so small that they are assumed to be at the same level. Therefore, the concrete cover under the lattice girder is also 25 mm. The length of the girders is the same as the length of the precasted plank.

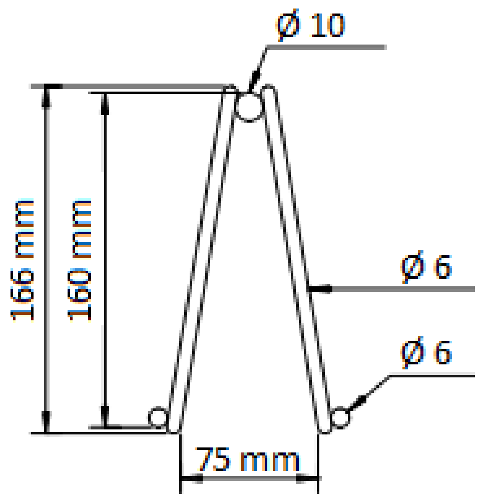


Figure 4.3: Cross-section lattice girder adapted from betonstaal (Betonstaal.nl[1], n.d.)

Figure 4.3 shows the cross-section of the lattice girder. The height (distance between the bottom and top bar) of the girder is 160 mm and the width is 70 mm. The total height of the girder is 166 mm (Betonstaal.nl[1], n.d.). The bottom and diagonal bars have a diameter of 6 mm, while the top bar has a diameter of 10 mm. All bars are classed as B500A reinforcement steel. All the properties of the lattice girder are shown in table 4.2

Table 4.2: properties lattice girder

Parameter	Value	Unity
Length	6720	mm
Height	160	mm
width	75	mm
Bottom concrete cover	25	mm
Diameter bottom reinforcement	6	mm
Diameter diagonal reinforcement	6	mm
Diameter top reinforcement	10	mm
Characteristic yield strength	500	MPa

4.3.3. Cast in-situ top layer

The cast in-situ top layer is 180 mm in height. The environmental class of the layer is XC1 and the layer has concrete strength C30/37. The layer contains several kinds of reinforcement. The main reinforcement is placed directly on the lattice girders, and the additional reinforcement is placed on the main reinforcement. The difference in height between the different forms of reinforcement is so small that it is assumed that all the reinforcement is positioned at the same height. The concrete cover on top of the reinforcement is 20 mm.

As mentioned, there are multiple kinds of reinforcement in this layer. The main reinforcement consists of a grid of 7 mm reinforcement bars with a heart-to-heart distance of 150 mm. The bars are made from B500A reinforcement steel. The grid has a cross-sectional area of reinforcement of 257 mm^2/m (Betonstaal.nl[2], n.d.). All bars are made from B500A reinforcement steel.

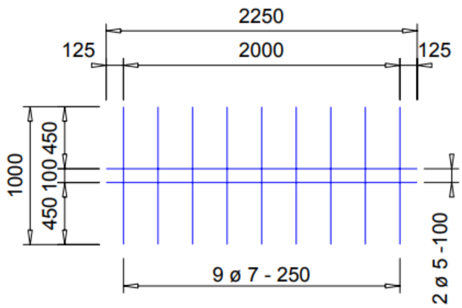


Figure 4.4: Schematic V154 reinforcement grid (Betonstaal.nl[3], n.d.)

The layer also contains additional reinforcement. This reinforcement consists of hogging reinforcement and coupling reinforcement. The coupling reinforcement consists of 10 V154 reinforcement grids. The grids are used to couple multiple precasted floor planks together. They are placed directly on the planks and together they will surround the whole perimeter of one plank coupling it to all the adjacent planks. As shown in figure 4.4, the grids consist of 9 bars with a diameter of 7 mm which couple the planks together, and 2 bars with a diameter of 5 mm which are used as directional reinforcement. The cross-sectional area of the reinforcement is 154 mm^2/m (Betonstaal.nl[3], n.d.). All bars are made from B500A reinforcement steel.

The hogging reinforcement consists of multiple 10 mm bars. Half of them have a length of 3 meters, while the other half has a length of 4 meters. The bars are only positioned in places where there is a hogging moment and they are placed directly on the main reinforcement. The bars have a cross-sectional area of 501 mm^2/m. All bars are made from B500A reinforcement steel. The properties of all the parameters in the cast in-situ top layer are displayed in table 4.3

Table 4.3: properties Cast in-situ top layer

Parameter	Value	Unity
Height	180	mm
Top concrete cover	20	mm
Characteristic cubic concrete strength	37	MPa
Cross-sectional area main reinforcement	257	mm^2/m
Cross-sectional area coupling reinforcement	154	mm^2/m
Cross-sectional area hogging reinforcement	501	mm^2/m
Characteristic cubic concrete strength	37	MPa
Characteristic steel yield strength	500	MPa

Chapter 5

Bearing capacity

In the previous chapter calculation methods to calculate the bearing capacity in different failure modes. A case study was presented to perform the calculations. In this chapter, the results of the bearing capacity calculations for the case study slab with openings are performed. Secondly, the results are validated using a finite element method. Thirdly, the effects of the openings on the bearing capacity of the slab are explored using unity checks and lastly, a conclusion is drawn.

All the results are shown in the same order as chapter 3. So firstly, the results for the bending moment capacity are covered. Secondly, the results for the shear capacity are explained. Thirdly, the results for the shear plane capacity are shown. The results use the different values for the parameters that impact the bearing capacity. The results of the capacities influenced by the different parameters are shown in graphs.

5.1 Bending moment resistance

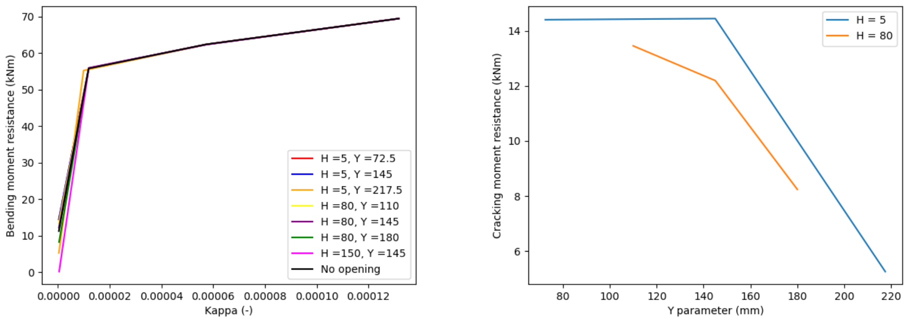
In chapter 3 it has been established that there are 4 characteristic points in the bending moment failure process of a concrete element: the cracking moment, the yielding moment, the elastic moment, and the ultimate moment. It was argued that the values for these characteristic points are influenced by the size of the opening (H parameter), and the distance from the bottom of the slab to the centre of the slab (Y parameter). It was predicted that the opening will influence the crack moment capacity and that the opening will only influence the other moment capacities if the opening interferes with the concrete compression zone.

Since there are four characteristic points on which the bending moment capacity is calculated, and all these values have to be shown for multiple variations of influencing parameters, it is easier to use an M-kappa diagram. In this diagram, the moment capacities are plotted on the vertical axis, while the corner rotation (kappa) is plotted on the x-axis. The bending moment capacities for different variations of the H and Y parameter are shown in figure 5.9. Firstly, the diagram with a changing Y parameter is shown and secondly, the diagram with a changing H parameter is shown. In both graphs, an M-kappa diagram of a slab without openings is shown to enable comparison between a slab with and without an opening. For all calculations, the same assumptions are used as set in chapter 3.

Results
In figure 5.9 the M-kappa graphs are shown of the wide floor slab with an opening on different heights in the slab with different size openings. The calculations are performed

7 times. 3 calculations are done using an H of 5 mm, 3 calculations are done using an H of 80 mm, and 1 calculation is done using an H of 150 mm. These values are chosen to simulate the smallest opening possible, the biggest possible, and to simulate a commonly used size. The Y parameter is chosen to simulate an opening placed on the precasted slab, an opening placed in the middle of the slab, and an opening placed high in the slab. Since the Y parameter presents the distance between the middle of the opening and the bottom of the slab, the value of the Y parameter used depends on the height of the opening. Figure 5.9b shows the cracking moment of a slab with an H of 5 mm and 80 mm with different Y parameters.

Figure 5.1: Bending moment capacity results



(a) M-kappa diagrams with different opening parameters

(b) Cracking moments on different levels of Y

Figure 5.9b shows that both the size of the opening and the distance between the middle of the opening and the bottom of the slab do influence the cracking moment capacity negatively. When the calculated cracking moments of the slab with an opening with an H of 5 mm and an H of 80 mm are compared, it is shown that if the opening is placed more towards the compression zone the reduction of the cracking moment is more than if the opening is placed towards the tension zone. This is also shown in figure b, which zooms in on only the cracking moments

The other moment capacities are not affected by the opening. This is mostly due to the small concrete compression zone, which is in most cases not bigger than the concrete reinforcement cover. Therefore, no opening could interfere with it. However, the data point of a slab with an opening with an H of 5 mm and a Y parameter of 217.5 mm shows that if the opening interferes with the concrete compression zone that this can have a marginal effect on the moment capacity.

Validation

The calculation method for the bending moment capacity is validated using FEM analysis. In this FEM analysis, the different models with different opening parameters are modelled to see if the calculated data is similar to the FEM data. The FEM analysis itself also has to be validated. The validation of the FEM analysis is done by modelling a slab without an opening. The calculation method for this slab is known. Therefore, the analytical calculations represent the real bending moment capacity. If the FEM analysis gives a similar result as the analytical calculation it can be concluded that the FEM analysis is accurate in predicting the capacity of the slab.

Since the slabs are modelled in this thesis as 2D elements on 2 supports, the same is done for the FEM analysis. The finite element program that is used is Diana FEA. The model properties of the FEM models and the results of the analyses are shown in Appendix D. The FEM data is summarized in graphs and shown in figure 5.2. It is shown that the results of the FEM model for a slab without an opening are nearly the same as the calculated values. There is only a slight variation in the data of 2 kN. This might be caused by rounding of values in the calculation. Since the calculated data and the FEM data are nearly equal it can be concluded that the FEM model is valid.

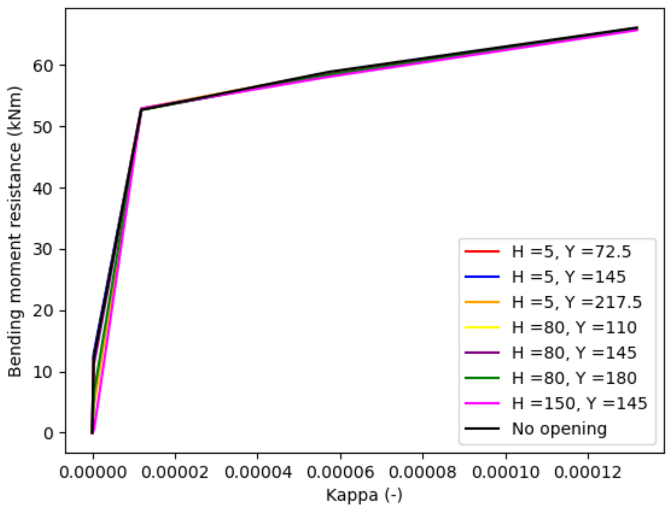


Figure 5.2: Bending moment capacities of the FEM models

In figure 5.2 the FEM data of the models with openings are shown. The values of the calculated bending moment capacity and the values from the data gathered from the FEM analysis show a lot of similarities. There is a slight difference between the values, however, this difference is equal to the difference found between the data of the slab without an opening. So probably this is also due to rounding of values. Since the data shows so many similarities it can be concluded that the calculation method gives a valid estimation of the bending moment capacity for slabs with openings.

5.2 Shear resistance

The shear resistance consists of multiple failure modes. As explained, flexural shear failure, shear tension failure, and shear compression failure have a lot in common and are therefore covered together, while shear plane failure is covered in a separate section. In this section flexural shear failure, shear tension failure, and shear compression failure are covered.

In chapter 3 it is argued that the failure modes can't be strictly separated when openings are introduced in the cross-section. This led to different calculation methods depending on the shape, size (H), longitudinal (X), and height (Y) parameter of the opening. By changing the parameters different calculation methods are used to calculate the shear capacity. To clarify which calculation method should be used in the different conditions a decision tree was introduced. In chapter 3 it was explained that the distributed load should be reformed into an equivalent load to be able to use the STM method. Firstly, the equivalent point load is explained. Secondly, the results of the calculations are discussed. Lastly, the method is validated.

Equivalent point load

According to the Eurocode (European Commision, n.d.) a structural element is loaded with a uniform distributed load. The value for this uniform distributed load is not important for this thesis, since the aim of this thesis is to calculate the bearing capacity of a slab. Figure 5.3 shows the structural model that is used in this thesis.

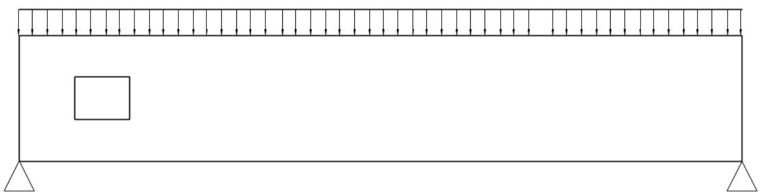


Figure 5.3: Structural model of the slab with uniform distributed load

The uniform distributed load can also be described by equivalent point loads. These point loads should give nearly the same moment and shear distribution as the uniform distributed load (M. D. Brown & Bayrsk, 2007). To do so 2 points are used. One point load is positioned right from the middle of the slab and the other left from the middle of the slab. For this thesis, the distance between the point load and the support is set at 2000 mm. Using two point loads that are 2000 mm from the support approaches the moment and shear distribution. This way the STM method can be used and the shear capacity of a slab loaded with a uniform distributed load can be approached. The new structural model is shown in figure 5.4.

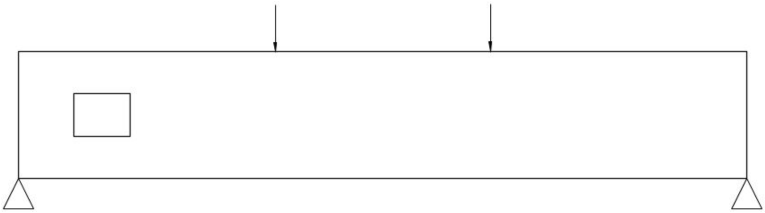
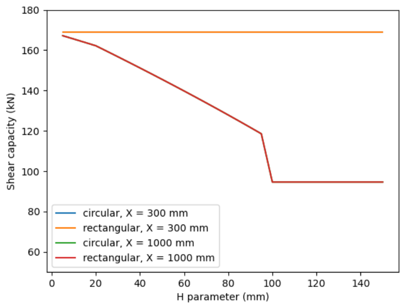


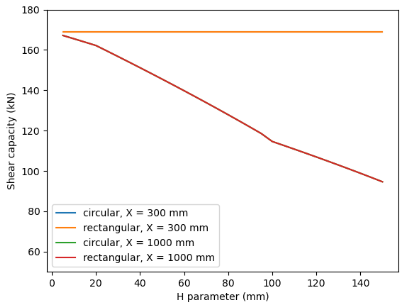
Figure 5.4: Structural model of the slab with uniform distributed load

Results

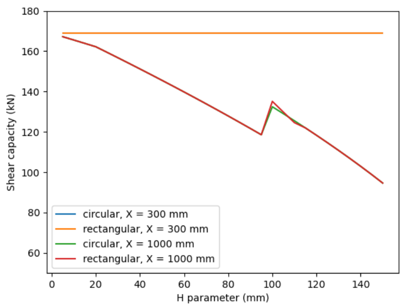
The results of the capacity calculations are shown below. Since there are a lot of parameters that can affect the shear capacity, the results are discussed in parts. Figure 5.5 shows the shear capacity of a slab with different opening sizes for both the circular and rectangular openings. The results are divided into 3 parts: an opening placed on the precasted slab, an opening placed in the middle of the slab, and an opening placed on the top of the slab. For each part, calculations are performed with an X value of 300 mm and 1000 mm for both the circular and rectangular openings. This gives in total 4 graphs per part which are presented in the figures below.



(a) Opening on the precasted slab



(b) Opening in the middle of the slab



(c) Opening in the top of the slab

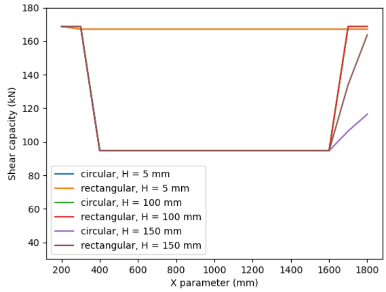
Figure 5.5: Shear resistance with varying opening sizes

5

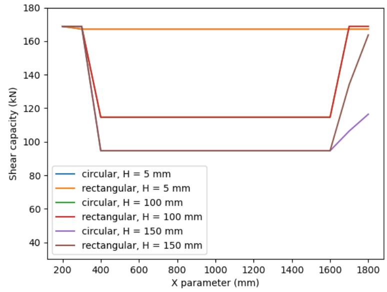
Figure 5.5 shows that an opening placed near the support does not affect the shear capacity no matter the size of the opening. This is mostly caused by the STM method, which can give high values for the shear capacity. In this case, the slab's capacity is limited by the shear capacity of other parts of the slab.

When the opening is placed further from the support, the shear capacity decreases as the opening gets bigger. When the H parameter is smaller than 40% of the height of the slab, the decrease is linear. When the H parameter is bigger than 40% the amount of change depends on the Y placement. If the opening is placed on the precasted slab there is a sudden drop, when placed in the middle the decrease is linear, and when the opening is placed at the top of the shear capacity increases first, after which it steadily increases. The sudden increase and decrease are mostly caused by the boundary conditions.

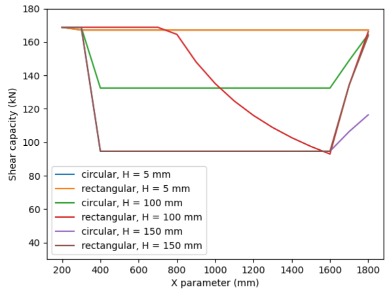
Figure 5.6c shows the shear capacity of a slab with an opening with a varying X parameter. As in figure 5.5, the calculations are split into 3 parts: an opening placed on the precasted slab, an opening placed in the middle of the slab, and an opening placed in the top of the slab. Per part, calculations are performed with an H value of 5 mm, 100 mm, and 150 mm to simulate the smallest, biggest, and mean value of the opening. The calculations are performed with both circular and rectangular openings.



(a) Opening on the precasted slab



(b) Opening in the middle of the slab



(c) Opening in the top of the slab

Figure 5.6: Shear resistance with varying X parameter

Figure 5.6c shows that in the case of a very small H, the place of the opening does not matter since the shear capacity is barely reduced. When H increase, the shear capacity decreases. There is only a decrease if the openings are not within a distance of equal to the height of the slab from the supports. This is also due to the earlier explained results of the STM method.

When the Y parameters increases, the decrease in shear capacity is less. In most cases, there is no difference in shear capacity in slabs with rectangular openings and slabs with circular openings. Only in the case of an opening with an H of 100 mm placed in the top of the slab, does the Y placement and shape of the opening matter. In this case, the rectangular opening can have less shear capacity than the circular opening. This is mostly due to one chord changing from the calculation method.

So overall it doesn't matter if a circular or rectangular opening is used since they give the same results in most cases. As expected the shear capacity decreases if the opening size increases. However, this is only when the opening is not within a distance equal to the height of the slab from the support. In that case, the shear capacity is calculated by the STM method and is way higher than the shear capacity in the slab. Thirdly, it is decreased less when the opening is placed as high as possible in the slab. Lastly, the shear capacity of the slab is higher if the openings are placed close to the support. However, the failure mode changes to brittle failure as explained earlier, and therefore gives more safety issues.

Validation
The calculation methods used above can be validated using the data of the experimental studies of Hanson (1969), Corley, and Somes (1973). Using the material properties described in these studies, shear capacities for beams with different kinds of openings can be calculated using the calculation method.

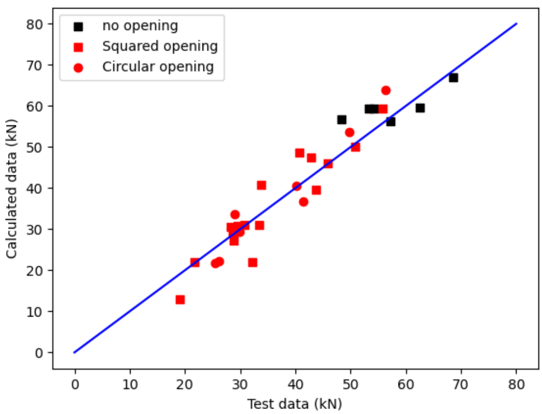


Figure 5.7: tested data versus calculated data (Hanson, 1969; Somes & Corley, 1973)

Figure 5.7 shows the shear capacity from the experimental test on the horizontal axis versus the calculated shear capacity on the vertical axis. If the point is on the diagonal line, the tested shear capacity is the same as the calculated shear capacity. As shown, nearly all points are on or very close to the diagonal line meaning that in nearly all cases the tested data is nearly the same as the calculated data. Due to the deviation that is normal in concrete, it is nearly impossible to do a 100% accurate prediction of the capacity of concrete. Since the calculated values are nearly the same as the tested values, it can be concluded that the method is quite accurate in predicting the shear capacity of a structural element.

5.3 Shear plane resistance

5

The shear plane resistance makes use of the cohesion between the two concrete slabs. This cohesion is formed by the roughness of the slabs and the lattice girder holding the slabs together. Openings don't influence the roughness. However, if they are big enough they will reduce the amount of reinforcement since the lattice girder is cut away to make some for the non-structural element. Therefore, it was concluded in chapter 3 that only the size parameter (H) influences the shear plane resistance.

Figure 5.8 shows the shear plane resistance of the slab with opening with different opening sizes. The opening size is increased by 5 mm for each calculation. Each calculation is performed assuming that the opening is positioned on the precasted slab since the height does not influence the calculations.

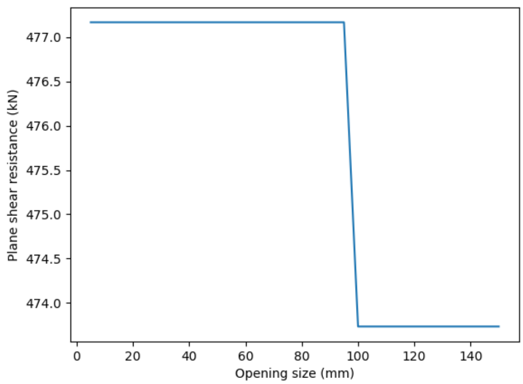


Figure 5.8: Plane shear resistance with varying opening size

Figure 5.8 shows that the interface capacity is reduced when the opening size reaches the same value as the heart-to-heart distance of the lattice girder reinforcement, which is 100 mm. When this is reached, 2 bars from each lattice girder have to be cut away. This reduces the cohesion between the slabs. Validation of this method was not possible, since no experimental data were available and FEM could not simulate this behaviour.

5.4 Unity check

Now the influence on the bearing capacity of the different failure modes is known, the effects of the openings can be normalized using the unity checks to simplify the comparison of the effects. To perform a unity check both the loads and bearing capacities are needed. Firstly, the load on the slab for the case study will be explained, after which the unity checks will be calculated per failure mode.

Loads

The Eurocode distinguishes several loads. For slabs, two different loads apply: variable loads (Q) and permanent loads (G). The variable loads depend on the consequence class. The consequence class for the slab in the case study is CC2, which means that the variable load consists of a distributed load of 4 kN/m (European Commision, n.d.). The permanent loads consist of the weight of the slab itself and other components attached to the floor. The components attached to the slab have a load of 1.6 kN/m according to the slab manufacturer, while the weight of the slab depends on the own weight of the concrete, which is 2500 kg/m^3, and the height of the slab. The permanent load for a slab without an opening is 7.73 kN/m.

To calculate the load on the slab the variable load and permanent load have to be multiplied by safety factors. For permanent loads, this is 1.2 and for variable loads, the safety factor is 1.5, and the value of the total load should always be higher than 1.35 times the permanent load. The total load on a slab without an opening for this case study is 15.3 kN/m

Using this distributed load the maximum bending moment and shear force can be calculated. The maximum bending moment for a simply supported slab loaded by a distributed load can be calculated using $\frac{1}{8}ql^2$, in which q is the total load on the slab and l is the length between the supports. The maximum bending moment will act in the middle of the span between the supports and has a value of 57.5 kNm.

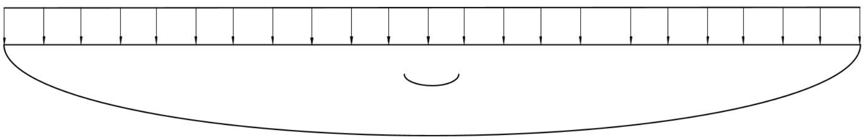


Figure 5.9: Moment line

As explained before, the shear force is calculated using equivalent point loads. These equivalent point loads should apply the same shear force as the distributed load. Since two point loads are used, both point loads should apply half of the shear force applied by the distributed load. Thus, each point load has a value of 52.7 kN. This also means that the maximum shear force is 52.7 kN for a slab without an opening.

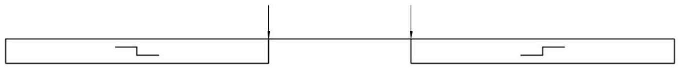


Figure 5.10: Shear force

Openings affect the load loading the slab. Since less concrete is used, the load due to the own weight of the slab is reduced. The reduction is equal to the height of the opening times the own weight and is only a local reduction of the distributed load. The variable load stays the same. The bending moment and shear force are calculated in the same way as explained above.

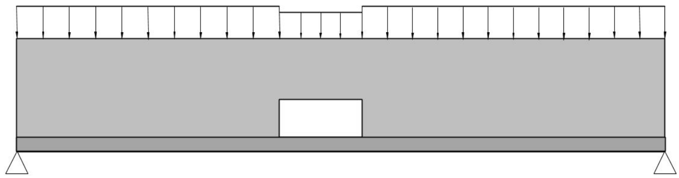


Figure 5.11: Load distribution for a slab with an opening

Bending moment capacity

Now both the bending moment distribution and bending moment capacity are known, a unity check for the bending moment capacity can be performed. The unity check for the bending moment is performed using the most conservative situation, which occurs if the opening is placed in the middle of the span, and using the ultimate moment capacity. Dividing the moment load by the capacity gives the unity check. Since the ultimate bending moment capacity is not dependent on the opening shape and the Y parameter, and the load is only dependent on the H parameter, the unity check is only performed for slabs with a varying H parameter.

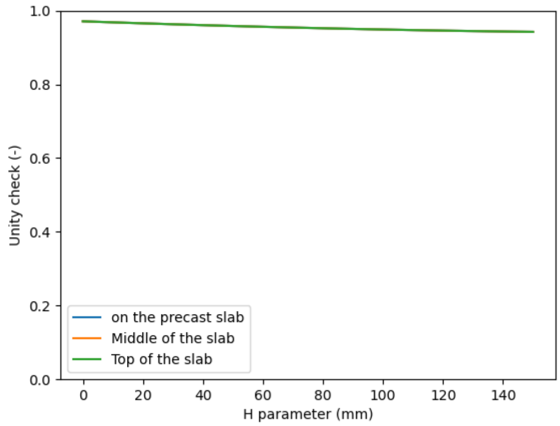
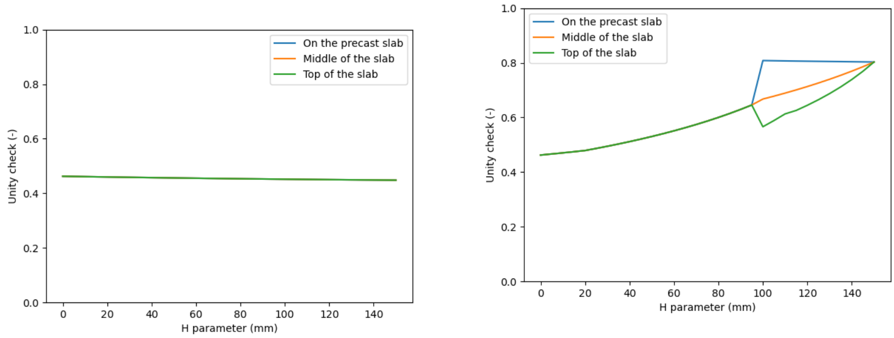


Figure 5.12: Bending moment unity check with different opening sizes

In figure 5.12 the unity checks of slabs with a varying H parameter are shown. It is shown that the unity check decreases when the H increases. This happens because the ultimate moment capacity isn't affected by the H parameter, and the load on the slab decreases because of an increasing H parameter, which decreases the own weight of the slab. The effects however are marginal.

Shear capacity

The unity check for the shear capacity can be calculated by dividing the shear force by the shear capacity. Earlier in this chapter, it was shown that the shear capacity depends on the X and Y placement of the opening and the H parameter of the opening. The shape of the opening doesn't influence the results significantly. Therefore, the unity check is performed on different configurations for the H parameter, X parameter, and Y parameter of the opening. The shape is always rectangular. The results are shown below.

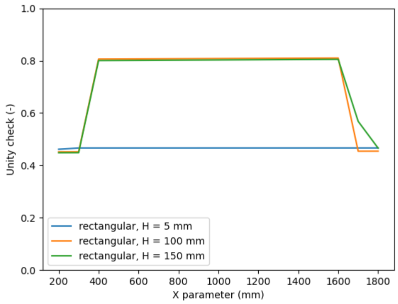


(a) unity check near the left support (X=175 mm)

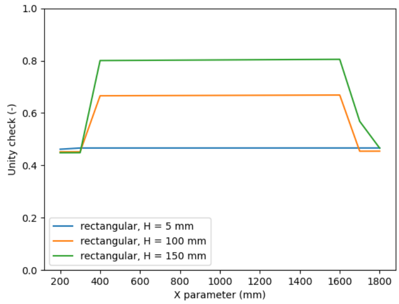
(b) unity check in the middle between the supports (x = 1000 mm)

Figure 5.13: Shear capacity unity check with different opening parameters

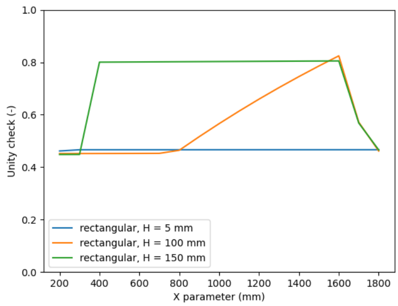
In figure 5.13 the results of the shear unity check are shown for slabs with an opening with a varying H parameter placed at different heights in the beam. The unity checks for an opening placed within a distance equal to the height of the slab from the support are all similar. The H and the Y placement don't influence the results. For an opening placed 1000 mm from the support, the Y and H parameter do influence the results. Till an H equal to 40% of the height of the slab, the Y parameter doesn't affect the results and the unity checks gradually increase when the H increases. After the 40% the Y parameter influences the results and the results of the unity check are less predictable. When the opening is placed near the tension zone the effects are bigger than when the openings are placed near the compression zone.



(a) Opening on the precasted slab



(b) Opening in the middle of the slab



(c) Opening in the top of the slab

Figure 5.14: Unity checks of the shear capacity with varying X parameter

In figure 5.14 the effects of the X placement of the opening with a different H parameter and a different Y placement are shown. It is shown that the influence of the opening increases when the value of the X placement is bigger than a value equal to the height of the slab. It is shown that when the opening is placed more toward the compression zone the influence on the unity check decreases. This is only for the openings with an H bigger than 40% of the slab height. The blue lines in figure 5.14 show that the Y placement doesn't affect the unity check of the slab when the H is smaller than 40% of the slab height.

Interface capacity

The unity check for the interface between the precasted plank and the cast in-situ top part is calculated differently. The shear plane capacity is calculated as stress. So to calculate the unity check, a load in the form of stress is needed. The Eurocode gives a formula to transform the shear force into the stress that loads the interface. This formula is given below.

$$v_{Ed} = \frac{\beta V_{Ed}}{z_i b} \tag{5.1}$$

with:

β	partial factor calculated by $\frac{F_c}{F_{sc}F_c}$	[-]
V_{Ed}	shear force	[N]
z_i	level arm	[mm]
b	slab width	[mm]

The unity check is calculated by dividing the shear stress by the interface capacity. Earlier in this chapter, it was described that the shear force and the interface capacity are only influenced by the H parameter. Therefore, the unity check is only performed for a slab with a varying H.

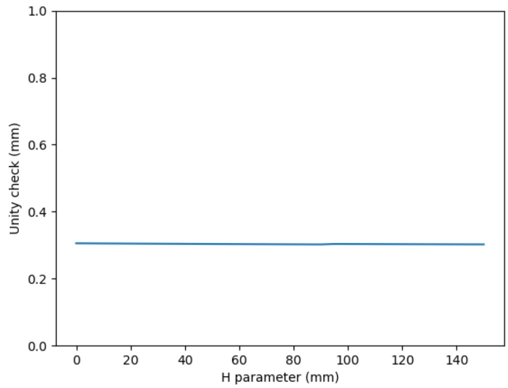


Figure 5.15: interface capacity unity check with different opening sizes

In figure 5.15 the unity check of the shear interface is shown for a slab with varying opening sizes. It is shown that the opening doesn't influence the unity check much.

5.5 Conclusion

The bearing capacities of the slab and the forces loading the slab are calculated for a slab with varying opening parameters. With the bearing capacities and the loads, unity checks are performed. Using the results of the unity checks and the bearing capacity calculations the effects per opening parameters can be mapped and recommendations about the parameters can be formed.

The shape of the opening only influences the shear capacity. The results of the bearing capacity (figure 5.5 and 5.6c) show that the shape of the opening only influences the bearing capacity marginally. Therefore, it doesn't matter which shape is used in terms of structural design and thus there is no clear benefit in using a circular or rectangular shape for the opening. Therefore, no recommendation is needed for the choice of the shape of the opening.

The size of the opening affects all failure modes assessed in this thesis. A bigger opening decreases the cracking moment capacities, while the other moment capacities are not influenced by the opening at all. The unity check for the bending moment is also not influenced by the opening. The shear capacity decrease when the opening increases. This decrease follows one trend till the opening size reaches 40% of the slab height, after this point the decrease is influenced by the other opening parameters and doesn't follow a trend. The interface capacity and unity check are only influenced negatively by the opening if the opening is bigger than the heart-to-heart distance of the reinforcement of the lattice girder. It is recommended to use openings with a size smaller than 40% of the height of the slab since the degree of influence follows a trend and is easily predictable. This predictability will clarify the design process.

The distance between the centre of the opening and the support only influences the shear capacity as shown in figure 5.5, 5.6c, 5.13, and 5.14. When an opening is placed near the support there are no effects on the bearing capacity and unity checks, and when the opening is placed outside an area equal to h from the support the effects depend on the other opening parameters. Research shows that the failure mode changes to shear compression failure if the opening is placed within a distance equal to the height of the slab from the support (Hanson, 1969; Somes & Corley, 1973). Shear compression failure is a brittle failure mode. This reduces the structural safety. Therefore, it is recommended that the opening is placed outside an area equal to h from the support.

Lastly, the distance between the centre of the opening and the bottom of the slab affects the capacity and unity check of each failure mode differently. In the case of the cracking moment, an opening placed towards the compression zone has a negative impact, while the decrease of capacity and unity check in terms of shear are impacted positively. Since there is no clear benefit in placing the opening high or lower in the slab, it is recommended to place the opening on the precasted slab to retain the constructability of the installations. Retaining constructability will lead to fewer construction errors, which improves the structural safety.

Chapter 6

Design process

In chapter 3 a calculation method is proposed to properly calculate the bearing capacity of wide floor slabs with non-structural elements. To properly use this method information about the placement of the non-structural elements is needed from the mechanical design, which requires a good design integration to achieve. However, as concluded in chapter 2, it is unknown where in the project the error is made that leads to the integration error. Understanding the design process is key to finding out where the error is made.

In this chapter, the design process currently used in the construction industry is explored and compared to the ideal theoretical design process to give a greater understanding of the current design process and to explore if there are any errors in the process itself. Firstly, the current design process is explained. Secondly, the theoretical design process is covered. For both the current and theoretical design process emphasis is placed on which phases there are, which tasks are performed and who is responsible. Lastly, both processes are compared and a conclusion is drawn.

6.1 Current design process

Each project in the construction industry is unique; structures are tailored to fit the environment and to accommodate the personal taste of the owner. However, the design process is quite consistent for every project (Sears et al., 2015). Each project follows roughly the same steps to achieve its goals. Depending on the project delivery model (PDM), the tasks and responsibilities are also quite consistent per PDM. In this section, the contractual models that are used for projects and that influence the design process are explained, after which the different project phases and the tasks and responsibilities per phase are shown.

6.1.1. General terms and conditions

In the building industry, different kinds of general terms and conditions can be used depending on the PDM. In projects with a traditional PDM two kinds of contracts are used: the Uniform Administrative Conditions for construction and installation (UAV) and The New Rules (DNR).

The UAV is the standard form for the contract between an owner and a contractor (Chao-Duivis et al., 2018). The traditional project delivery model makes use of the UAV 2012 (UAV, 2012). The UAV is implemented by the government and mandatory to use. It specifies which parties are involved and what the parties are obliged to deliver. The type of payment scheme and the way of solving conflicts is also governed by the UAV (Chao-

Duivis et al., 2018). So it doesn't specify specific tasks or activities which parties have to do.

The DNR is the standard form for contracts between an owner and engineering agencies. Nowadays, the DNR 2011 is used (BNA, 2013). The DNR is made by BNA, which is the branch organization for architects, and origins from the standard forms for engineers (RVOI 2001) and the standard forms for architects (SR 1997) (Chao-Duivis et al., 2018). Since the DNR is made by branch organizations it is not mandatory to use. However, in practice, it is nearly always used for building projects. The DNR describes the agreements on the design work between the owner and engineering agencies and sets a legal basis for the collaborations between the parties (BNA, 2013). Unlike the UAV, the DNR does describe the design process and tasks that have to be done per phase in detail. The DNR describes the paying mechanism, planning, and management of the design process (Chao-Duivis et al., 2018).

6.1.2. Project phases and tasks

The DNR describes the design process with all the tasks that have to be done per phase in the standard task description (STB) (BNA & NLingenieurs, 2014). Figure 6.1 shows a graphical presentation of the nine project phases the DNR-STB distinguishes. These phases are initiation, project definition, sketch design (SO), preliminary design (VO), definitive design (DO), technical design (TO), pre-construction design (UO), construction, and use (BNA, 2013).

6

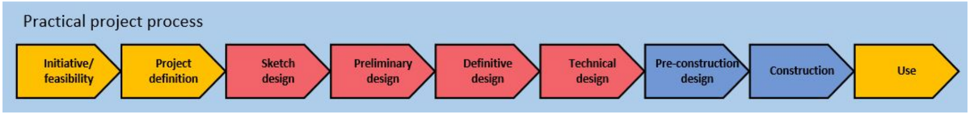


Figure 6.1: Practical project process to the DNR process (BNA, 2013)

In the initiation phase the project is initiated. The demand for the kind of building is investigated and the ambitions of the project owner are mapped. The economical, legal, functional, and technical feasibility are checked. After this is done, the project definition phase is started. In this phase, the ambitions, requirements, wishes, expectations, and conditions of the owner and future users are mapped. This is done till a first price estimation can be made. The owner is responsible for this phase. He will most likely hire a project team to do this work. This project team consists of different architects and advising engineers from different disciplines (BNA, 2011).

After the project definition phase is done the sketch design phase is started. The sketch design phase is not a mandatory phase and should only be used for more complex projects (BNA & NLingenieurs, 2014). In this phase, a global sketch of the building is made to map the main outline of the building. The goal is to make a visual presentation of what the building looks like and how the building will look in its surroundings. It is noted that this phase is not required for each project and that in most projects this phase isn't used. The owner is responsible for this phase. The tasks of this phase are performed by a project team consisting of different architects and advising engineers from different disciplines. The owner is responsible for this phase (BNA, 2011).

After either the sketch design phase or the project definition phase is finished, the preliminary design phase is started. The goal of the preliminary design phase is to give a global impression of the building, its functionality, its facilities, and its surroundings. A first setup of the floor plan is made, and a start is made to the main bearing structure. The owner is still responsible for this phase. A project team consisting of architects, either advising or designing structural engineers, and other advising engineers will perform the tasks of this phase (BNA, 2011)

After the preliminary design phase is finished, the definitive design phase is started. The goal of this phase is to make a detailed design of the building so that a price estimation can be made. This includes the appearance of the building, the bearing structure, the materials used, the capacity of the MEP systems, and fire safety. The owner is still responsible for this phase. A project team consisting of architects, either advising or designing structural engineers, and other advising engineers will perform the tasks of this phase (BNA, 2011)

After the DO is finished, the technical design phase is started. In this phase, all the technical aspects of the building are designed. This means that the main bearing structure is fully calculated, that the MEP systems are designed and that the dimensions of these systems are known, and that the layout of the MEP systems is designed. At the end of this phase, a contractor should be able to make a tender. The owner is still responsible for this phase. The tasks of this phase are performed by a project team consisting of architects and designing engineers from different disciplines will perform the tasks of this phase (BNA, 2011)

After the tender is won a contractor will design the pre-construction design. In this phase, the elements which should be designed by subcontractors are designed by the subcontractors. Various details are calculated and drawn, and the MEP systems are fully calculated for every bolt. At the end of this phase, the contractor should be able to build the building using the designs. In this phase, the contractor is responsible for the materials, and designs of the subcontractors, while the owner is responsible for the design. The tasks of the phase are performed by a project team consisting of architects and designing engineers from different disciplines (BNA, 2011)

In the construction phase, the contractor builds the building. It is important to note that the owner of the project will control or will hire someone to control the progress of the works. The owner should accept parts when they are finished. After the construction is finished, the building can be used. In this phase, the building becomes the responsibility of the owner again (BNA, 2011).

6.2 Theoretical project process

The literature describes an ideal project process formed by best practices, which leads to a successful project if the process and tasks are followed (Fewings, 2012). In the literature the traditional project process is described as a linear process; the current phase has to be completed before the next phase can begin (Sears et al., 2015). The project process can be divided into nine phases: inception, feasibility, project definition, outline proposal, scheme design, detailed design, production design, construction, and project completion (Austen & Neale, 1984; Fewings, 2012; Forbes & Ahmed, 2011). The sequence of these phases is shown in figure 6.2.

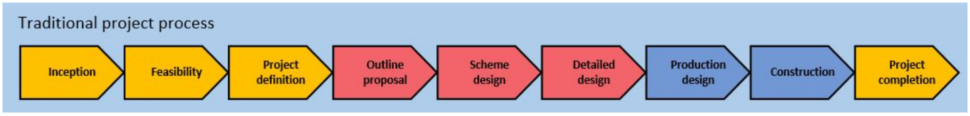


Figure 6.2: Traditional design process

The first phase is the inception phase. In this phase, the owner envisions the project and communicates his business plan to the project team to develop the boundaries of the project (Fewings, 2012; Forbes & Ahmed, 2011). After this phase is done, the feasibility phase starts. In this phase, the economical, legal, and technical feasibility of the project is checked (Forbes & Ahmed, 2011). Also, risks need to be identified and allocated, and the owner needs to make a decision whether to go on with the project or abandon it (Fewings, 2012). The output of this phase is a basic risk assessment, a design concept statement on which a planning is based, and a funding source (Fewings, 2012; Wijnen & Storm, 2018). After this phase, the project definition phase is started. The goal of the project definition is to analyse and describe the ambitions, requirements, wishes, expectations, and conditions of the owner and the future users (Forbes & Ahmed, 2011). The organization structure and the procurement method are selected (Fewings, 2012). The owner is responsible for all three phases. A project team consisting of architects and advising engineers from different disciplines will perform the tasks.

After the project definition phase, the outline proposal phase starts. In this phase, an overall presentation of the building is made (Austen & Neale, 1984; Forbes & Ahmed, 2011). This phase should give a first impression of the architectural presence of the building and of the integration between the structural and MEP components. It should give an impression of the functionalities, facilities, and surroundings. Also, a preliminary structural plan is made (Ko & Chung, 2014). In traditional projects, the owner is responsible for this phase. A project team consisting of architects and advising engineers will perform the tasks.

After the outline proposal phase, the scheme design phase starts. In this phase, the preliminary design is further developed. The architectural design is further detailed, including spatial assignments, flow routes, and a construction-cost estimation (Austen & Neale, 1984; Ko & Chung, 2014). The structural design is also further developed. The loads on the different elements are calculated using the building regulations, the dimensions of the elements are estimated, and the overall strength calculations are performed (Ko & Chung, 2014; Sears et al., 2015). For the installation design, an outline is made for the different MEP installations. The paths and the start and end of the installations should be designed. The owner is still responsible for the phase. In traditional projects, the same project team will do both the outline proposal phase and the scheme design phase (Austen & Neale, 1984).

After the scheme design phase is finished, the detailed design phase is started. The aim of this phase is to make a design on which a contractor can make an accurate cost estimation and make an accurate bid (Forbes & Ahmed, 2011; Sears et al., 2015). The construction designs are further developed. Detailed structural calculations are made and the amount of materials used in the building is calculated. The reinforcement is also designed and calculated. This means that most components are calculated, indicated, and drawn. However, the detailed drawings aren't made yet (Ko & Chung, 2014; Sears et al., 2015). Elements such as windows and doors are selected by the architect. The installations are also designed and drawn. This means that all the installations are given a dimension according to their capacity and that a drawing is made for all the installations. The structural and installation designs are integrated into one design. The phase is the responsibility of the owner, and the tasks are performed by a project team consisting of architects and advising engineers from different disciplines (Forbes & Ahmed, 2011).

When the detailed design phase is finished, contractors will bid on the project and the owner will accept a bid. The selected contractor will construct the building (Sears et al., 2015). The contractor is also responsible for some extra engineering. This is done in the production design phase. The aim of this phase is to develop the design in such a way that the building can be built using the designs. This means that all the details are calculated and that the prefabricated components are ordered and designed by the manufacturer. At the end of this phase, the building can be built (Austen & Neale, 1984). The contractor and owner are both responsible for parts of the phase. The tasks are done by subcontractors and a project team consisting of constructing engineers and architects.

After the design is finished, the construction phase is started. In this phase, the building is constructed (Sears et al., 2015). The construction itself is the responsibility of the contractor. However, the coordination of the design team, supervising the construction work, and keeping track if the building is constructed accordingly to the contract is the responsibility of the owner. After the construction is finished, the project is handed to the owner. The owner is then responsible for the building.

6.3 Comparison

The practical and theoretical project processes have been described. The practical project process is described in the DNR and has 10 phases, which are shown in figure 6.1. The theoretical project process is described by the literature and consists of nine phases. This process is shown in figure 6.2. The DNR and the theoretical project process can be compared to find any discrepancies in the practical project process which can lead to design errors.

A difference between both project processes can immediately be seen in the names of the phases when both processes are compared. Nearly none of the names of the phases of the DNR are the same as the names of the phase of the theoretical project process. This however doesn't mean that the phases are different. In most literature, the names of the different phases differ (Austen & Neale, 1984; Fewings, 2012; Forbes & Ahmed, 2011). It is rather the tasks and aim of the phases which need to be compared to see if there are any differences between the project processes. When the tasks and objectives are compared, a lot of the phases are the same. Table 6.1 shows the theoretical project phases and their practical counterpart.

Table 6.1: Theoretical project phases and their practical counterparts

Theoretical phases	Practical phases
Inception	Initiation
Project definition	Project definition
Feasibility	-
Outline proposal	Preliminary design
-	Sketch design
Scheme design	Definitive design
Detail design	Technical design
Production design	Pre-construction design
Construction	Construction
Project completion	Use

Only 2 phases show differences: initiation/feasibility and sketch design. The theory describes that the initiation and feasibility check are different phases. Both phases have their own aim and objectives, but both phases are performed by the same people. The DNR does describe the initiation phase, but it doesn't describe the feasibility check phase. This feasibility check is an activity within the initiation phase. In the DNR the feasibility check is done by the same people who have done the initiation of the project. Although the DNR doesn't describe the feasibility check as a separate phase, the feasibility is still assessed in the same way, with the same aim, the same objectives, and done by the same people as the theory describes. Therefore, it can be concluded that both the DNR and the theory describe the project process, however in a different form.

The DNR also differs from the theory in the sketch design phase. The DNR describes that there can be a sketch design phase between the project definition phase and the preliminary design phase. The aim of this phase is to make a rough sketch of the building and to show what the building is about to look like. The DNR notes that this phase is not mandatory and that this phase only should be used in bigger and more complex projects (BNA, 2013). The theory doesn't describe a sketch phase and tells us to go directly from the project definition phase toward the preliminary design phase. The theory is mostly a generalization of the real world. Since the sketch phase is a phase that is not used in most general cases, the theory doesn't describe it.

6.4 Conclusion

It can be concluded that the construction projects of utility and residential buildings follow the project process of the DNR. This project process is nearly the same as the theoretical project process. However, there are some small differences. The DNR describes the activities of the initiation and feasibility check in one phase, while the theory separates the activities into two phases. However, they both describe the same activities. Only the name is different. The DNR describes that there is an additional phase on special occasions. However, this phase is only used in very big projects. Residential and utility projects are considered smaller projects and thus this phase does not apply to these kinds of projects. Therefore, this difference is not taken into account.

It can be concluded that the DNR and theory describe the same project process. Since the literature describes an ideal project process that should lead to a successful project and the theoretical project process and DNR project process are similar, it can be assumed that the project process described by the DNR is a process fit for purpose. The different project phases themselves should not lead to errors in the design. So it can be concluded that the integration error is not caused by solely following the project process and thus the integration error is caused by something else.

Chapter 7

Influencing factors

In the previous chapter, it is concluded that the building industry follows the project process of the DNR. It is concluded that the project process of the DNR is nearly the same as the project process of the theory, and that following the design process doesn't in itself cause integration errors.

Since the integration errors are not caused by inconsistencies in the phases of the project process, the integration error is caused by other factors. KPCV describes that integration errors in the design of concrete elements with embedded MEP systems occur due to fragmentation in the construction industry. This fragmentation leads to inconsistencies in the project process. In this chapter, the fragmentation and effects of this fragmentation are elaborated. Secondly, the events that occur due to this fragmentation and lead to integration errors are explained and evaluated. Lastly, a conclusion is drawn on what causes the integration error.

7.1 Fragmentation in the project process

As explained before, the construction industry uses the project process described by the DNR. The DNR describes an ideal project process and describes all the tasks that should be done in each project phase (BNA & NLingenieurs, 2014). It notes that all the disciplines should be in the same phase at the same time and should progress to the next phase together to make sure the project is going smoothly and no errors are made in the design (BNA, 2013). However, in practice working in the same phase at the same time and progressing to the next phase together doesn't always happen (KPCV, 2021).

The construction industry is characterized by fragmentation and so is the project process (Abadi, 2005; KPCV, 2021). All disciplines work on their own design of the current phase and when the owner allows it, the next phase is started (BNA, 2013). This fragmentation doesn't necessarily lead to problems. However, if one of the disciplines is delayed, problems arise.

Since the project process is so fragmented, the disciplines don't wait for each other. So if one discipline is delayed, the other disciplines still progress to the next phase and don't wait for the other discipline to catch up. This causes a phase shift where different disciplines work in different phases at the same time. This phase shift has a big impact on the integration since the disciplines do not finish the designs at the same time. Therefore, they cannot coordinate the designs (KPCV, 2021).

The phase shift is commonly occurring in the design of the structural elements with embedded MEP systems. In these projects, the structural engineering discipline and the mechanical engineering discipline are not always in the same phase. According to KPCV (2021) it is always the mechanical engineering discipline that is running behind regarding the other disciplines. This running behind makes integration of the designs harder, since the designs are finished on different dates. Insufficient design integration leads to design errors. These design errors impact the structural safety. (KPCV, 2021).

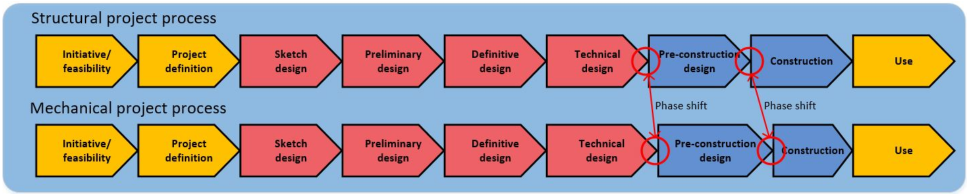


Figure 7.1: Shift in project phases

7.2 Influencing events

So the phase shift makes the design integration harder, which leads to design errors. The phase shift is caused by a combination of fragmentation in the construction industry and the delay of one discipline. Since fragmentation doesn't cause problems if there is no delay, only the delay has to be resolved to improve the integration of the designs and thus the structural safety (Thijhuis & Maas, 1996). Since fragmentation is a phenomenon in the construction industry that cannot be prevented with process steps (Abadi, 2005), it is better to prevent the delay of the discipline.

Before the delay can be prevented, the events that cause the delay should be known. KPCV describes that the delay occurs due to the insufficient commissioning of the assignment to the advising mechanical engineer and due to multiple deficiencies in the design. KPCV also describes that the delay of the mechanical discipline occurs due to late commissioning of the work to the constructing mechanical engineer (KPCV, 2021). In addition to this, personal communication with engineers from BAM AE gives another cause for the delay. Engineers from BAM AE describe that the delay of the mechanical discipline occurs due to late commissioning of the work to the constructing mechanical engineer and due to the structural discipline working ahead of schedule to avoid risks. One engineer stated that "stability calculations for the TO phase are made in the DO-phase to avoid risks in later stages of the project" (W. Diepenhorst, personal communication, 16 June 2022)

These four statements of KPCV and the engineers of BAM AE can be grouped into three events that influence the delay of the mechanical discipline: commissioning, quality, and risk. All three events together cause the phase shift between the mechanical and structural disciplines. The three events are elaborated below.

Commissioning

In the event commissioning the different assignments to the different disciplines are not handed out at the same time. After the technical design phase is finished, constructing engineers are involved in the project. To these engineers work is commissioned. The work commissioning to the mechanical engineers happens at a later date than the work commissioning to the structural engineers. The engineers don't start designing till the work is commissioned. Since mechanical engineers get the work commissioned later, they start designing later than the other disciplines. This creates a delay for the mechanical discipline and this delay shows in a prolonged TO phase as shown in figure 7.1.

Quality

The event quality focuses on the quality of the design which is handed over to the next phase. After the TO phase, constructing engineers will develop the designs from the earlier phase further. In most cases, the designers the constructing engineers get from the previous phase contain errors or information gaps. These information gaps and errors have to be rectified in the pre-construction phase which costs extra time and causes delays.

The errors and information gaps are caused by poor commissioning and material choice (KPCV, 2021). Poor commissioning means that the tasks given to the advising engineers are insufficiently formulated or incomplete. When these engineers start designing, they will make the designs with wrong assumptions or a different goal. This leads to errors. material choice is about the difference in material choice of different engineers. When a constructing engineer chooses to use different materials due to availability, parts of the designs have to be redone. It should be noted that these problems can also happen in the designs of the structural discipline. However, this is a lot less common, since in most cases the structural engineers will do the design from the beginning to the end and only hand over the design to the contractor to build it.

Risk

The third event is Risk. In this event the structural engineers are ahead of schedule, in regard to the DNR, to avoid risks on design changes within the structural design in later stages. This means that the structural engineers will do calculations and design work from later stages earlier in the project. This is done to know in an early stage of the project that the building is constructible. This is needed for two reasons. The first reason is that after the DO phase, the municipality assesses the design on constructability. If you can prove that your building is constructible you may build it in later stages. This also leads to the second reason. Since the engineers have to be sure that the building is constructible and since it is hard to change the size of construction elements after the municipality approves the plans, the engineers will do more calculations and more design work to avoid design changes in later stages. The client will sell the building as fast as possible and therefore he wants the building to be built as fast as possible. Having a nearly complete design in earlier phases will help speed up this process.

7.3 Evaluation

The three influencing events described above are formed using information gathered from KPCV and personal communication with engineers from BAM AE. To find out if the described events and the effects of these events really cause the integration errors, the events are evaluated. The evaluation is done by establishing if professionals recognize the events in their previous and current projects.

The view of the professionals is gathered using semi-structured interviews. the opinion of the professionals should be based on several projects to ensure the events happen in general. To ensure the opinion of the professionals is based on multiple projects, project leaders with at least 10 years of experience are interviewed. Project leaders are responsible for the total design and therefore they have experience with the integration of different designs. Since the project leaders have at least 10 years of experience, they have done several projects and therefore their experiences are based on several projects. The view on the event of the project leaders is asked per event. The transcriptions of the interviews are shown in Appendix E. The evaluation per event and the possible solutions to prevent the events are discussed below.

Commissioning
When asked about the difference in commissioning between the mechanical engineers and the structural engineers, all the engineers say that at the beginning of the project the designing engineers are all contracted at the same time. However, they point out that contracting a constructing mechanical engineer in a later stage of the project sometimes lacks behind. This happens when the project goes from the TO to the UO phase. One project leader states that "In most cases, the TO\UO is already started, while there is no assignment for the mechanical engineer" (Appendix E). When asked why the commissioning is later for the mechanical discipline, all the project leaders give the same reason: because the assignment is not ready and cannot be assigned yet. The delay can be weeks or months and varies per project.

According to the project leaders, the effects of this event are clear. One states that "the consequences can be that the disciplines have to wait for each other to develop the designs. This delays all the disciplines" and another project leader also states that "this delays both disciplines" (Appendix E). Thus, the effects are delay for all the disciplines. Also, since the mechanical discipline is finished later, the structural discipline didn't take the mechanical components into account in their designs. According to all three project managers, this leads to design cases and redesigning. They point out that "sometimes there is no freedom of design left" and that "some installations cannot be placed in the best positions anymore" (Appendix E). This has an impact on the end result of the project.

When asked about a solution to this problem, all three project leaders give a similar answer: "Everyone has to start at the same time in the project" (Appendix E). When asked if it is possible to let one mechanical engineer, either a designing or constructing engineer, make the whole design to avoid problems with contracting in later phases, the project leaders gave negative advice. They pointed out that the designing mechanical engineering companies don't have the capacity to make the design in the later stages of a design. And the other way around, constructing mechanical engineering companies don't have the capacity to make the design in an early phase.

So, the event commissioning does occur in projects. It occurs in the transition from the TO phase to the UO phase and creates a delay for all disciplines. Since the other disciplines don't wait, the mechanical design is finished later causing clashes. Contracting the constructing mechanical engineers at the same time as all the other constructing engineers are the only solution according to the project leaders.

Quality
All three project leaders recognize that mechanical engineers have to do tasks from a previous phase in the current phase. This occurs when the project is in the UO phase. According to the project leaders, there are multiple reasons why this event happens. One project leader states that "sometimes other materials have to be used" and that "sometimes the design is not complete". This is backed up by another project leader who states that "personal preferences" and "clashes" are the reason. Thus, material choice due to personal preferences and errors in the designs is the cause of this event.

The project leaders state that "this event has the same effects as the previous event". There will be delays in the design of the mechanical discipline since extra tasks have to be done. This delay causes a prolonged UO phase of the mechanical discipline as shown in figure 7.1. Due to the delay the structural designers are unable to integrate the designs of the mechanical engineers into the structural designs causing clashes. Due to the clashes, design changes are needed for both the mechanical and structural designs which can lead to integration errors.

When asked for a solution to prevent this event from happening, the project leaders give different solutions. One solution is that "the mechanical engineers have to be more proactive". Instead of waiting till materials are bought, they have to actively design and steer the purchase of materials to what they want. Another option is that "the mechanical engineers should have time to review the design of the previous phase". This way the constructing mechanical engineers will be at the same level as all the other disciplines at the beginning of the next phase. One project leader points out that "Within BAM AE the extra time option is already used in some projects". It is also pointed out that delivering a more complete design in earlier phases isn't possible, since there is no benefit for the designing mechanical engineers. Every mechanical engineer has his own preferences, and delivering a better design in earlier phases wouldn't solve the problem that some mechanical engineers want to use specific materials.

So the project leaders all recognize the quality event and it occurs in projects. It occurs in the transition from the TO to the UO phase and it shows in a prolonged UO phase. It is caused by different material choices, incorrect assumptions, and passive behaviour of the engineers. Solutions are to encourage mechanical engineers to be more active in the design and to give the mechanical engineers extra time to catch up.

Risk
When asked about the event risks and if the structural discipline does more in an earlier phase than the DNR describes, one project leader describes that "structural engineers tend to do more work in earlier phases to make sure that the structural elements suffice", while the other two project leaders state that "we work till the DNR checklist in each phase" and that "you just check the boxes on the DNR forms" (Appendix E). So the project leaders disagree in this state. Since two out of three project leaders do not recognize this event, it is concluded that this event does not occur.

7.4 Conclusion

It can be concluded that the integration error is caused by a phase shift in the project process between the mechanical and structural disciplines. Due to fragmentation, the disciplines don't wait for each other to progress to the next phase. When one discipline is delayed, the other goes to the next phase and a phase shift is created. It is nearly always the mechanical discipline that is lacking behind. The amount of time the mechanical discipline is lacking behind can vary.

The phase shift is caused by two influencing events: commissioning and quality. In the event 'commissioning' the constructing mechanical engineer is contracted later than the other constructing engineers. Since the engineer is later contracted, he will start later and this creates a delay. The event 'quality' focuses on the quality of the design which is handed over to the next phase. In most cases, there are errors in the design from previous phases due to information gaps and poor commissioning. These errors need to be fixed, which cost time and creates delay. Both events occur in the interface between the TO and UO phase. The commissioning event prolongs the TO phase, while the quality event prolongs the UO phase.

Chapter 8

New design strategy

The literature review (chapter 2) resulted in the understanding that structural safety depends on a combination of structural analysis and that in both the field of structural engineering and construction management key factors to ensure structural safety are missing. It was concluded that an integrated approach for structural safety containing both a structural analysis and management practices does not exist. Since structural safety depends on both fields, an integral approach to improve structural safety is needed.

However, before such an integrated approach can be formed, the missing key factors must be found. In the previous chapters, it is shown that a calculation method was missing and that the factors that cause the integration error in the design were unknown. In chapters 3 to 5 a calculation method for the calculation of the bearing capacity of wide floor slabs was formed, and in chapters 6 and 7, the factors that cause the integration error and possible solutions for this error were found.

Now the missing key factors are known for both disciplines, an integrated approach to improve structural safety can be formed. This chapter presents such an approach for wide floor slabs with embedded non-structural elements in the form of an integrated design strategy. Firstly, the integrated design strategy is explained and its contents are shown. Subsequently, the design strategy is evaluated using an interview with an expert panel from KPCV.

8.1 Integrated design strategy

The structural safety issue in the design of wide floor slabs with embedded non-structural elements is caused by integration errors. This integration error is caused by the absence of a calculation method for openings in structural elements, late commissioning, and design quality problems. All factors lead eventually to design errors.

The absence of a calculation method causes a lot of design errors in itself since nobody knows where to put the non-structural elements. Since this isn't clear a lot of design errors arise, which need to be fixed leading to design changes. These design changes lead to delays and design errors.

The late commissioning of the constructing mechanical engineer and quality problems with mechanical designs from previous phases lead to delay in the design process of the mechanical discipline. Due to fragmentation in the construction industry, this can lead to a phase shift between the mechanical discipline and the structural discipline. Due to this design shift, the mechanical designs are later finished than the structural designs which makes design integration hard. This leads to design changes and thus to delay and design errors.

To solve the problems from both disciplines, the integrated design strategy contains two elements: design rules and an adjusted design process. The aim of the design rules is to create a clear overview of where the elements can be placed. By clarifying which element can be placed in the slab and where these elements can be placed, the freedom of solutions is reduced for both the mechanical and structural discipline. This clarifies and simplifies the design process for both disciplines. Engineers are forced to make their designs from the perspective of other disciplines. This leads to more communication and tuning of the designs between the disciplines, which leads to better design integration from the beginning of the project and reduces the phase shift between the disciplines.

The design rules simplify the design process improving the integration between the disciplines and thus reducing the phase shift. However, there are still factors remaining that maintain the phase shift, such as late commissioning and quality errors. These factors also cause part of the design shift and thus the design rules do not fully resolve the phase shift. The adjusted design strategy aims to resolve the remaining phase shift caused by these factors by implementing an adjusted design process. This adjusted design process changes the design process so that the remaining factors don't cause a phase shift. Since design rules resolve the first part of the integration error these are explained first. Secondly, the adjusted design process is covered.

8.1.1. Design rules

As shown in chapter 5, the openings have a significant effect on the bearing capacity and unity check of the slab. The degree of influence of the opening on the bearing capacity and the unity check is described for the four opening parameters. It was found that only 3 parameters affect the unity check. These parameters are the size of the opening, the height of the centre of the opening, and the length between the support or point load to the centre of the opening. Using the effects of these parameters, 3 design rules for design rules are proposed. The design rules are discussed below.

Opening size

The size of the opening affects all failure modes. The degree of influence on the unity check depends on the size of the opening. When the height of the opening is smaller than 40% of the height of the slab, the degree of influence follows a trend. When the height of the opening is bigger than 40% of the height of the slab, the degree of influence is dependent on the other opening parameters and therefore it doesn't follow a trend anymore. Thus, the degree of influence becomes less predictable when the opening is bigger than the 40% threshold.

The goal of the design rules is to clarify the design process and to make designing easier for all disciplines. Predictability improves the clarification of the design process and makes designing easier for all disciplines. Therefore, it is strongly advised to use openings with a size smaller than 40% of the height of the slab.

To achieve this design rule non-structural elements shouldn't be bundled, since bundling them will increase the size of the opening. Also, it is better to use ducts instead of ventilation pipes, since in most cases the ducts are not higher than 40% of the slab height. When it is not possible to use non-structural elements with a height smaller than 40% of the slab height, the structural engineer should be notified to prevent design errors.

Height between the centre of the opening and bottom of the slab
The distance between the centre of the opening and the bottom of the slab affects the bearing capacities and unity checks differently for each failure mode. When the opening is placed more towards the compression zone, the bending moment capacity is compromised more, while the shear capacity is compromised less in comparison to an opening placed more towards the tension zone. And the other way around, when the opening is placed more towards the tension zone the shear capacity is compromised more and the bending moment capacity is compromised less. Therefore, there is no clear benefit in terms of capacities and unity checks for the placement of the opening in height direction.

Since there is no clear advantage of placing the opening on any height in the slab, the decision for the opening is made using the constructability reason. When the opening is placed high in the slab, the non-structural element has to be suspended in the air while making the slab. This is not practical in terms of constructability. Taking into account the constructability and the effects on the failure modes, it is advised to place the opening on the prefabricated slab. This has some negative consequences for some failure modes. However, these negative effects are not so big and can be taken into account, while suspending an element in the air can give different construction errors. These construction errors affect the structural safety negatively and cannot be taken into account. Therefore, it is better to have more reliable constructability.

Length between the centre of the opening and support
The distance between the centre of the opening and the support only influences the shear capacity and unity check. When the opening is placed near a support there are no effects on the bearing capacity, and when the opening is placed outside an area equal to h from the support the effects depend on the other opening parameters. It seems that placing the opening near a support is better for the strength of the slab. However, research shows that when openings are placed within a distance equal to h from the support, the shear failure mode can change to shear compression failure (Hanson, 1969; Somes & Corley, 1973). This is a brittle failure mode. Brittleness decreases the structural safety.

Since the failure mode changes to a brittle failure mode when the opening is placed within a distance equal to h from the support, it is recommended to place the opening at least a distance equal to the height of the slab from the support. Although this decrease the shear capacity of the slab, it prevents brittle behaviour of the slab. Figure 8.1 shows where an element can be placed. No elements should be placed in the areas which are marked red.

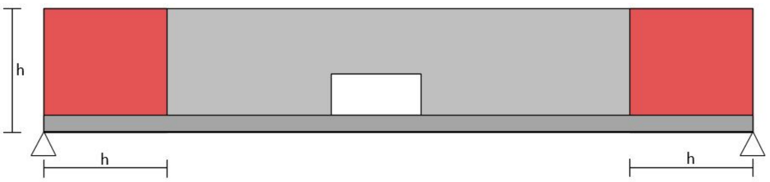


Figure 8.1: Longitudinal placement of the opening

8.1.2. Adjusted design process

The design rules described above clarify where and which non-structural elements can be placed in the slab. As a result, the mechanical design and structural design will most likely be better integrated, since the design boundaries are clarified and the engineers are forced to make their design from the perspective of the other discipline. This leads to more communication and integration of the designs, which reduces the phase shift between the disciplines due to less redesigning and better communication

The design rules only resolve one factor that causes the design shift, namely wrong assumptions. However, chapter 7 shows that there are other factors besides the wrong assumptions that lead to parts of the design shift. As shown in figure 7.1 late commissioning leads to a prolonged TO phase, while material choice and passive behaviour lead to a prolonged UO phase. Since the design rules only solve one factor and the other factors also cause a phase shift, it is concluded that the design rules only resolve a part of the design shift. To resolve the remaining phase shift, a solution is needed for the other factors. This solution is given in the form of an adjusted design process.

The adjusted design process focuses on resolving the remaining influencing factors to resolve the remaining phase shift. The remaining influencing factors are late commissioning of the work to the constructing mechanical engineer and quality errors in the design of the mechanical engineers due to other material choices and passive behaviour of the mechanical engineers.

An additional preparation phase should be added between the TO phase and the UO phase. Such a preparation phase is already used in some projects of BAM AE and has proven to improve communication and clash detection. This phase should give time to all disciplines to review the work and designs of previous phases. By reviewing the work and designs of the previous phases, potential clashes can be detected early in the phase and a time estimation can be made on the amount of time needed to adjust the designs. Because the amount of time needed to adjust the design is known, every discipline can take this additional time into account preventing unexpected delays. Early clash detection enables the engineers to prevent design errors by redesigning their design early on in the phase. The new phase is called the preparation phase. The old and adjusted design processes are shown in the figure below.

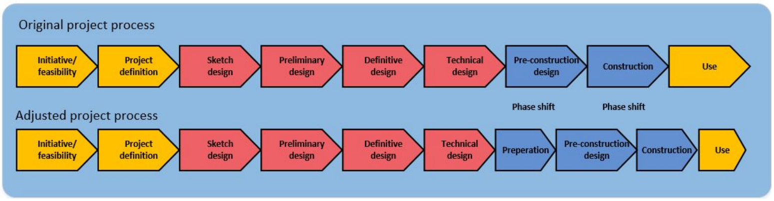


Figure 8.2: Original design process and adjusted design process

It is important that all disciplines only use the preparation phase for the review of designs and work from the previous phases. If one discipline uses this phase to already further develop its design, a new phase shift is created since one discipline is ahead of the other disciplines. Therefore, the only tasks in the preparation phase should be a review of the previous designs, clash detection, and making a time estimation of the time needed to develop the design to UO standards. The end result of the phase should be a list of possible clashes and a time estimation for the development of the design.

It should be noted that the extra project phase does not prevent the influencing events to occur, but rather mitigates the effects of these influencing events. The interviewees point out that events as different material choices can always happen due to the availability of materials and personal preferences. Therefore, the events could not be prevented and thus mitigation is the best option.

8.2 Evaluation

To assess if the proposed integrated design strategy actually resolves the structural safety issue in the design of wide floor slabs with embedded non-structural elements, an expert panel from KPCV is asked to assess the design strategy on feasibility and effectiveness. The expert panel consists of 8 engineers with different backgrounds with multiple years of experience in the building industry. The integrated design strategy is presented to the experts using a presentation, after which they gave their reflection on the integrated design strategy. The full reflection is shown in appendix F.

The expert panel concludes that implementing design rules is feasible. However, they point out that implementing design rules in terms of strict rules which always need to be followed is not a good idea. They rather see the rules as guidelines to allow some freedom of design. They think that guidelines like the guidelines for installations in wide floor slabs can help to clarify the design process.

The adjusted design process is not feasible if implemented solely for the design of wide floor slabs. However, if implemented to improve the structural safety for all elements, it is feasible. The expert panel argues that a preparation phase late in the project only mitigates the effects of the phase shift. However, it does not prevent the phase shift to occur. They think to fully solve the integration problem the phase shift has to be prevented.

The expert panel points out that the integration problem can only be prevented by a combination of measures. Firstly, the commissioned work should request an equal level of detail from all disciplines. To achieve this the DNR-STB has to be reformulated. Currently, the DNR-STB requests a lower level of detail from the mechanical designs than from the structural designs. This causes the mechanical discipline to lack behind at the end of the project. However, when all disciplines have the same level of detail, non will lack behind thus resolving the phase shift.

Secondly, the expert panel argues that earlier involvement of mechanical engineers can prevent a phase shift. When the constructing mechanical engineers are included earlier in the project, they can start earlier with designing the design in detail. This prevents clashes early on in the project and makes sure that the mechanical design has the same detail as the other designs. This prevents the phase shift from occurring.

Involving the constructing mechanical engineers earlier in the project requires the contractor to be earlier involved in the project since the contractor hires the mechanical engineer. The expert panel points out that the mechanical engineer should be included before the DO. This requires the contractor to be contracted before the DO. To achieve this, a different PDM is needed. This thesis is based on the traditional PDM in which the contractor is involved after the TO phase. However, to involve the contractor before the DO phase an integrated PDM should be used. The design and build PDM allows contractors to be contracted before the VO phase. As shown in figure 8.3 the contractor becomes responsible for the whole design process in the design and build project process, while in the traditional design process the contractor is only responsible for the UO phase.

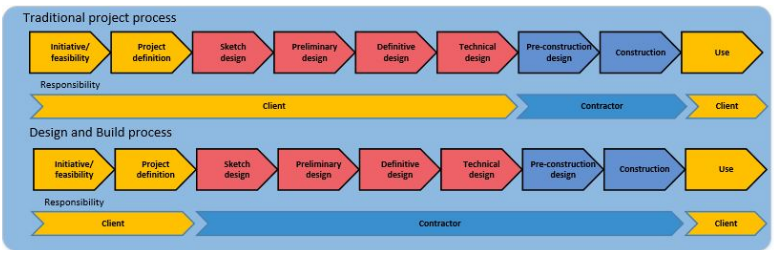


Figure 8.3: Traditional and Design and Build project processes with responsibilities per phase based on (KPCV, 2022)

8.3 Conclusion

A new integrated design strategy is needed to improve the structural safety of wide floor slabs with embedded non-structural elements. The structural safety issue is caused by the absence of a calculation method and a phase shift between the mechanical and structural disciplines. The phase shift is caused by late commissioning of the mechanical engineers and quality errors in the mechanical design. To solve the safety issue both problems need to be resolved

The integrated design strategy contains both design rules and an adjusted design process. The design rules clarify where the non-structural elements can be placed. This makes the design process easier and forces engineers to make their designs from the perspective of other disciplines. This lead to more communication and more integration between the designs which leads to fewer design errors and a reduced phase shift. The remaining part of the phase shift is caused by other quality errors, such as material choice, and late commissioning of the mechanical engineers. To mitigate the effects of these events, an additional project phase is placed between the TO and UO. This phase is named the preparation phase and functions as a review phase. In this phase, clashes can be detected and the time to develop the design further is estimated to clarify the project process for all disciplines and resolve the phase shift.

The expert panel from KPCV argues that the new design strategy is feasible if its application is broadened. Instead of only being focused on wide floor slabs it should allow the review of all structural elements. They argue that the strategy only mitigates the effects of the phase shift, but does not solve it. To solve it an adjustment to the DNR-STB has to be made and an integrated PDM should be used to involve the contractor and thus the constructing mechanical engineer earlier. Using the design and build PDM this can be achieved.

Chapter 9

Discussion, conclusion, and recommendation

In this concluding chapter, the outcome of the thesis is summarized and recommendations for further research and implementation of the outcome of the thesis are proposed. Throughout the research, assumptions are done and methods have been used that need to be discussed. Firstly, these assumptions and methods are discussed. Secondly, a conclusion is drawn for this thesis and lastly, recommendations for implementing the outcome in practice and future research are given.

9.1 Discussion

In this section, the validity of assumptions and methods is discussed. Firstly, the structural part is discussed. Secondly, the management part is discussed, and lastly, the design strategy is discussed.

Structural model

The structural model that is used in this thesis is a simply supported slab with an opening through the full depth of the slab. This led to the slab having only a one-way load distribution. Redistribution of the loads was not taken into account. Based on these assumptions the bearing capacity was calculated.

In practice, slabs are made so that they can redistribute the force loading it so different sections of the slab bear more forces if one section is not able to bear the forces loading it. This can increase the amount of force a slab can carry (Lantsoght et al., 2015). A different structural model can give a different load pattern. This can influence which part of the slab is in tension and compression. Thirdly, in most cases, non-structural elements are not placed straight through a slab but are placed diagonally through the slab. This can reduce the effect the non-structural elements have on the spread of the load over the element.

However, both the structural model, redistribution of forces, and the inclination of the non-structural element do not significantly increase or decrease the local bearing capacity of the slab. Therefore, the calculated bearing capacities are not influenced significantly due to the structural model. Thus, the bearing capacities calculated in this thesis can give a first estimation of what the slab is able to carry.

Case study

To map the influences of openings a case study was used from BAM AE. This case study is a real slab used in a building and therefore the slab was modelled to that building. This means that material properties were used to resist the demand of that particular building. Material properties highly influence the strength of a slab. Therefore, the results might be influenced by the material properties and the conclusion of this thesis might only apply to the particular case study. However, using a unity check the influences of these properties and loads can be prevented. The unity checks divide the strength by the load. The outcome is a unitless number that is not dependent on specific cases. Thus, using this unity check the general effects of the opening on slabs can be established no matter the material properties and loads.

Validation of the structural methods
The structural calculation methods are validated by either data from experimental research from the literature or a FEM analysis. The data from the literature was limited. Therefore, to validate the calculation method for the shear capacity, experimental data was used from a simply supported beam. The simply supported beam used in these experiments is similar to the slab used in this thesis. Both the slab and the beam are simply supported and both only contain longitudinal reinforcement. In addition to this, it is assumed in this thesis that the slab only has a one-way load distribution, which resembles the load distribution of a beam. Since the structural model of both the experimental beam and the slab from this thesis are similar, the shear resistance of both can be calculated in the same way (Sherwood et al., 2006). Therefore, the calculation method for slabs with openings can be applied to beams with openings, and thus the experimental data can be used to validate the calculation method. This is also shown in the fact that the calculation method can accurately calculate the shear capacity of the experimental beams used in the literature as shown in chapter 5.2.

The bending moment calculation is validated using a FEM analysis with Diana. For the FEM analysis, a simplified model is used. No lattice girders are modelled, and the slab is modelled as one full slab. This is different from the real-world situation, but not so much different from the calculation method. The calculation method is based on the calculation method used for the design of slabs used in the construction industry. This method neglects the lattice girder due to its material properties. In this method, it is also assumed that the slab is a full slab. Therefore, the validation using the simplified FEM model is accurate regarding the calculation method.

Project delivery model
The scope of this thesis is limited to the traditional PDM. This PDM strictly separates the tasks and responsibilities of the client and the contractor (Zaliha et al., 2018). The results and improvements proposed in this thesis are based on the traditional PDM. The traditional PDM has multiple shortcomings. One of these shortcomings is the occurrence of quality errors in the design the contractor gets to develop (Nasrun et al., 2014). This shortcoming is also described by the interviewees who describe that "mechanical engineers have to redesign parts of the design from the previous phase" due to "personal preferences" and "clashes" (Appendix E).

Other PDMs, such as the integrated PDM, don't have this shortcoming since the contractor is also responsible for the design phase (Forbes & Ahmed, 2011). Thus, by using another PDM than the traditional PDM, as proposed by the expert panel, the events and problems with the design integration can change. Therefore, the outcome of this thesis is influenced by the traditional PDM and different PDMs can lead to different results in the thesis.

Expert evaluation
The evaluation by experts is done using interviews. In this case, the interviewer was the researcher who has limited experience with interviewing. The interviewees are all working in the construction industry, and all had multiple years of experience in this industry. Interviews can be biased in the case that the interviewer asks suggestive questions (Gorden, 1998). Using a semi-structured interview technique, leading questions are formulated upfront to steer the interviews so that unintended leading questions that suggest an answer can be avoided.

Another aspect of the expert evaluation that needs to be discussed is that all the interviews are held within the BAM AE. Only three interviews were held to evaluate the influencing events. The fact that all three interviewees are employees of BAM AE can influence the results since it is likely that they have a lot of the same experiences due to the way BAM works. However, the interviewees were selected to have a lot of experience and all interviewees worked on multiple different projects with a lot of different subcontractors.

The above-described situation does not apply to the expert panel from KPCV since that expert panel consists of eight experts from different companies. So they for sure have different experiences.

Implementation of the results
This research deals with a problem in the Dutch construction industry. Therefore, only practices used in the Dutch construction industry are used, like the DNR and the Eurocode. The results are evaluated by Dutch engineers. Therefore, the improvements described in this study are limited to the Dutch construction industry. Cultural backgrounds, norms, and values might have influenced the improvements and therefore the improvements might not have much value in foreign construction sectors.

9.2 Conclusion

In this section, the conclusion is presented. The conclusion is presented by answering the research questions. The main research question was:

How could the structural safety of concrete floor slabs with openings due to embedded non-structural elements be improved?

In order to answer this research question, multiple sub-questions were used. The answers to these sub-questions are presented below. After that, the research question is answered.

Sub-question 1
SQ1: *What is the influence of the embedded non-structural elements on the bearing capacity of a concrete wide floor slab?*
It can be concluded that the influences of the embedded non-structural elements on the

bearing capacity of a concrete wide floor slab depend on multiple parameters and the failure mechanisms themselves.

In total 4 different parameters of the opening that influence the bearing capacity are distinguished from the literature. These parameters are the shape of the opening, the size of the opening (H parameter), the distance between the centre of the opening and the support (X parameter), and the distance between the centre of the opening and the bottom of the slab (Y parameter). All parameters are shown in figure 9.1.

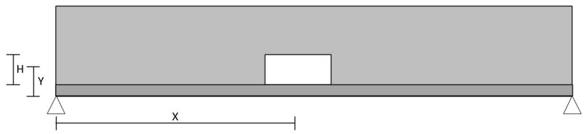


Figure 9.1: Graphical presentation of a simple support slab with openings

Since there was no calculation method, a calculation method was formed based on the literature and the four established parameters. In total 3 calculation methods were formed for three different failure modes. These failure modes are bending moment failure, shear failure, and interface failure. The term shear failure is a collective name for shear compression failure, shear tension failure, and flexural shear failure. For all three failure modes, the influence of the openings is mapped separately.

The bending moment capacity is only influenced by the size of the parameter and the distance between the centre of the opening and the bottom of the slab. It can be concluded that when the opening is increased, the bearing moment capacity decreases. In the case of the Y parameter is it concluded that when the opening is placed more towards the compression zone, the effects of the openings on the bearing capacity are more. Thus, when an opening is placed more toward the compression zone, the decrease in bearing capacity is more than when an opening is placed toward the tension zone.

9

The shear capacity is influenced by all 4 opening parameters. First of all, it can be concluded that if an opening is placed within a distance equal to the effective height, the opening does not influence the bearing capacity. However, the failure mode changes to a brittle failure mode. When an opening is placed outside this area, the opening size and height depend on the amount of influence. Smaller openings have decreased the bearing capacity less than big openings. The influence of openings that are placed more toward the compression zone is less than when the opening is placed more toward the tension zone. The shape of the opening only influences the boundary conditions of the calculation methods.

Lastly, the interface capacity is only influenced by the size of the opening. If the width of the opening is bigger than the heart-to-hear distance of the lattice girder reinforcement, parts of the lattice girder reinforcement have to be cut away. This reduces the interface capacity. Thus, the bigger the opening the less the interface capacity.

Sub-question 2
SQ2: *What causes errors in the integration process of the MEP and the structural design?*
The errors in the integration process of the MEP and the structural design are caused by a phase shift between the structural and mechanical disciplines. The different disciplines don't wait for each other to progress to the next phase due to fragmentation in the construction industry. When one discipline has a delay and can't progress to the next phase, the other disciplines just progress to the next phase. Due to this phase shift, the different designs are finished at different times in the project. This makes integration hard and causes a lot of design changes, which can lead to design errors.

In most projects, the mechanical discipline is lacking behind on the other disciplines. The amount of time that the mechanical discipline is lacking behind varies per project. In most cases, the lacking behind starts at the TO phase and goes on in the UO phase.
The mechanical engineers start lacking behind due to two influencing events. The first event is late commissioning. After the TO phase, the contractor takes over the design and constructing engineers are contracted and work is commissioned to them. When the work is later commissioned to constructing mechanical engineers than to the other engineers, the design of the mechanical discipline is delayed since the engineer starts his work at a later time. This delay creates a phase shift. As mentioned this happens in the transition between the TO and UO phase. As long as there is no mechanical engineer the project cannot progress to the UO phase and therefore this influencing event prolongs the TO phase.

The second influencing event is quality. After the TO phase, the constructing engineers will further develop the design from the earlier phase. In most cases, the mechanical design from the previous phase contains errors or information gaps. These errors and information gaps have to be rectified, which costs time and delays the design. The errors and information gaps are caused by poor commissioning of the design and different material choices from different engineers. The errors are mostly discovered in the UO phase and rectified in this phase. This prolongs the UO phase.

Sub-question 3
SQ3: *What solution can solve the structural safety issue?* The solution that is formed to solve the structural safety issue is an integrated design process. Since the problem is caused by the fact that it is unknown where to put the non-structural openings and a phase shift in the design, a solution is needed which solves both. The integrated design strategy contains both design rules to make it clear where to put the non-structural elements in the slab and an adjusted process to resolve the phase shift. Both are explained below

The aim of the design rules is to create a clear overview of where the elements can be placed. By clarifying which elements can be placed in the slab and where these elements can be placed, the freedom of solutions is reduced for both the mechanical and structural disciplines. This simplifies the design process. Engineers are forced to make their designs with respect to the other disciplines. This leads to more communication and tuning of the design, which leads to better integration from the beginning of the project. This makes the design easier and reduces the phase shift since fewer design changes have to be made. Therefore, the design rules are not only solving the problem with placement but also reducing the phase shift.

The design rules consist of three rules. Firstly, the size of the opening should be kept lower than 40% of the height of the slab to prevent big changes. If the opening can't be smaller than 40% of the height of the slab, then the structural engineer should be notified. Secondly, the openings should be placed on the precasted slab to preserve the constructability. Constructability is important since poor constructability can lead to building errors which reduces structural safety. Therefore, retaining constructability is important. Thirdly, the opening should be placed at least a distance equal to the height of the slab from the support. This is to prevent brittle failure of the slab.

The aim of the adjusted design process is to resolve the remaining phase shift caused by late commissioning and the quality errors, such as material choice. The design shift is caused by events that occur in the interface between the TO and UO phase. To mitigate the effects of the influencing factors an additional preparation phase is added between the TO and UO phase as shown in figure 9.2. The aim of this phase is to review the designs from the previous phase to detect clashes and estimate the time needed to further develop the design to UO level. This ensures that all disciplines start with the same information in the UO phase. Since the clashes are detected and the time needed is estimated, unexpected delays are prevented.

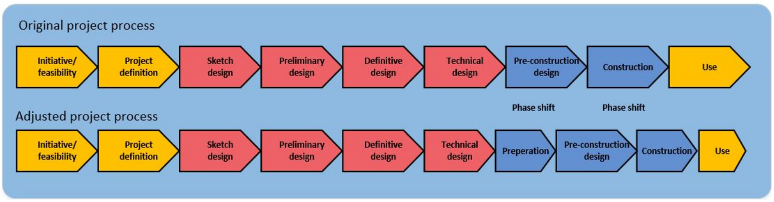


Figure 9.2: Original design process and adjusted design process

It is important that all disciplines only use the preparation phase for clash detection and time estimation. If one of the disciplines uses the phase to further develop the design a new phase shift is created since one discipline is ahead of the others.

9

Sub-question 4

SQ4: *How feasible and effective is the solution for both disciplines?*

The feasibility and effectiveness of the solution proposed in the third sub-question are assessed by an expert group of KPCV. The expert group concluded that the new design strategy is feasible if it is implemented broader than only for wide floor slabs. They conclude that an extra design phase late in the project process only mitigates the problems. It doesn't solve it. To solve the problems other methods are needed.

The expert group argues that the safety issue can only be resolved by a combination of measures. Firstly, the commissioned work should request an equal level of detail. This ensures that all disciplines work on the same level of detail and makes integration easier. This measure requires a change in the DNR-STB since this STB now requests a lower level of detail from the mechanical discipline than from the structural discipline.

Secondly, the expert group argues that the constructing mechanical engineer should be included earlier in the project. This prevents clashes early in the project. This measure requires the contractor to be involved earlier in the project since the contractor hires a mechanical engineer. To involve the contractor earlier in the project a different PDM is needed. Using an integrated PDM the contractor can be involved earlier in the project.

Research question

RQ: How could the structural safety of concrete floor slabs with openings due to embedded non-structural elements be improved?

It can be concluded that the structural safety of concrete floor slabs with openings due to embedded non-structural elements can be improved by implementing an integrated design strategy. This integrated design strategy contains both design rules and an adjusted design process. The design rules consist of 3 rules for the placement of the opening; The opening cannot be bigger than 40% of the slab height, the opening must be placed on the precasted floor slab, and the opening cannot be placed within a distance equal to the height from the support. When there is a deviation from these rules the structural engineers has to be notified.

The adjusted design process contains an additional project phase. This phase is the preparation phase and the aim of this phase is to review the design of the previous phase. Activities in this phase are clash detection and time estimation, and the end result of this phase is a list of possible clashes and an estimated time needed to develop the design to the UO level. The new design process is shown in figure 9.2.

The integrated design strategy however only mitigates the problem. To fully solve the safety issue another approach is needed. The expert group of KPCV proposes two measures. Firstly, the level of detail should be equal for all the disciplines. The DNR-STB requires a different level of detail from the structural engineers than from the mechanical engineers. To ensure this level of detail is equal for all disciplines, the DNR-STB has to be changed. Secondly, the experts argue that the constructing mechanical engineer should be involved earlier. This requires a change in PDM. An integrated PDM should be used to achieve this. Thus, it is argued that the safety issue can only be prevented if the DNR-STB is changed and an integrated PDM is used even for small projects.

So, the new design strategy can mitigate the problems leading to the safety issue and therefore it improves structural safety. However, to prevent the problems from occurring and therefore to prevent safety issues from occurring other solutions might work better.

9.3 Recommendation

This section provides recommendations for the research. Firstly, recommendations for the use of the results of this thesis in practice are provided, after which recommendations for future research are done.

Using the results in practice
The proposed design strategy is formed with the idea to use it in practice. Therefore, the design strategy can be used in the construction industry. However, there are some important comments which are needed to use the design strategy properly

It might look like the new design strategy consists of two parts: a structural part and a managerial part. However, both parts shouldn't be seen separately. As explained in the thesis, both parts can't be separated because both parts only solve a part of the problem. The design rules clarify the design process and reduce the phase shift, but it doesn't resolve the phase shift. And the adjusted project process only reduces the phase shift on some points, but without the design rules, errors will still arise in the design. Therefore, the new design strategy only works if it is fully implemented and if all parts of the strategy are implemented simultaneously.

As mentioned, a new design strategy is proposed regarding the strict separation of responsibilities from the traditional PDM. Therefore, when the traditional PDM is used this design strategy can be used. However, the evaluation of the design strategy gives a different view on the improvements in structural safety. According to the experts from KPCV, it is better to involve the contractor early in the project. This is not possible using the traditional PDM, but is possible using an integrated PDM. Therefore, it might be beneficial to use an integrated PDM, such as design and build, for small projects in the building industry. However, the differences between the different PDMs are extensively explained in the literature and using a different PDM can improve some points of the process, but can also lead to higher costs, quality issues in the design, or other hazards (Forbes & Ahmed, 2011; Zaliha et al., 2018). Whether or not using a different PDM than the traditional PDM for small projects is feasible and effective should be investigated.

Future research
This thesis resulted in a new design strategy containing both design rules and an adjusted design process. However, the thesis also provided a lot of new research opportunities. Firstly, the calculation method is now validated using data from years ago and a FEM model. It is already explained that these methods make use of simplified cases, which approach the real capacity of the wide floor slab. The redistribution of forces was not taken into account. Therefore, the real bearing capacity of a wide floor slab can be higher than the one calculated in this thesis. The only way of knowing this is by testing. Therefore, it is recommended that wide floor slabs with openings are tested to assess the real capacity.

Secondly, this thesis only assesses a wide floor slab with one circular or rectangular opening. However, there are many more options; There can be more openings in the slab, the openings can have different forms, or the openings can only go halfway through the slab. There are multiple different structural systems, such as a slab on three supports. For all these options there is no calculation method. Research can be performed on the bearing capacity of the slab with these options.

Thirdly, not all failure modes are addressed in this thesis. Therefore, the bearing capacity of the slab in these failure modes is not known yet. Therefore, research can be performed on these failure modes. The failure modes that are not researched yet are: punching shear failure, deflection, and crack width.

Lastly, during the evaluation of the design strategy, it was pointed out by the expert group that the integration problem is only solved if the DNR changes and if the contractor is involved earlier in the project. Research can be performed as to what parts of the DNR have to be changed and how they must be changed. Involving the contractor earlier in the project requires a change in PDM from the traditional PDM to an integrated PDM. Since there are a lot of integrated PDMs, research can be done on how much earlier the builder should be involved in the project and therefore which integrated PDM can best be used. Also, research can be done on the feasibility and effectiveness of using an integrated PDM. This can be researched by assessing housing projects where an integrated PDM is used.

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Appendix A

Example calculation for the bending moment

In this appendix:

- Yielding moment calculation of a slab without opening,
- Yielding moment calculation of a slab with opening

Yielding moment calculation of a slab without opening

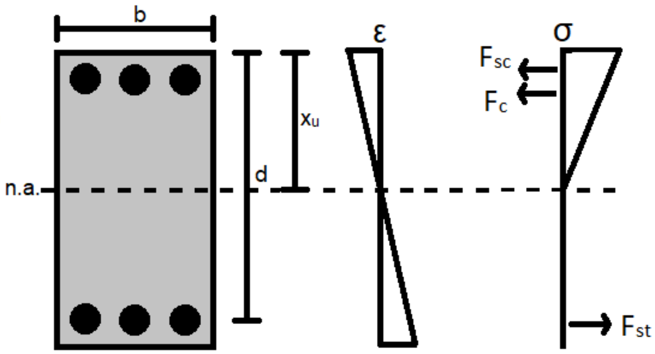

Figure A.1: Cross-section with stress-strain relation

Step 1: Forming the strain formula over the height.

The formula used for the strain distribution of the yielding moment is:

$\epsilon(z) = \frac{z - x_y}{d_t - x_y} \epsilon_s$
with:

z	position of the strain	[mm]
x_y	height of the concrete compressive zone	[mm]
d_t	distance between the bottom reinforcement and top of the cross-section	[mm]
ϵ_s	yield strain of steel	[-]

In the case of the yielding moment capacity the ϵ_s is 2.175‰ and the effective height is 220 mm. This gives the following strain formula:

$\epsilon(z) = \frac{z - x_y}{220 - x_y} 0.002175$

Step 2: Filling the strain formula in into the horizontal force formulas

The horizontal force formulas are shown below:

$F_{st} = A_{st} f_{ywd}$
$F_c = \int_0^x \epsilon(z)\, dz E_c b$
$F_{sc} = A_{sc}\epsilon(z_{sc})E_s$ with:

A_{st}	area of tensile reinforcement	[mm^2]
f_{ywd}	yield strength of steel	[MPa]
b	width of the cross-section	[mm]
x_y	compression zone of the yield moment	[mm]
$\epsilon(0)$	strain at z = 0	[-]
E_c	Young's modulus concrete	[MPa]
A_{sc}	area of compression reinforcement	[mm^2]
$\epsilon(z_{sc})$	strain at heigh of the compression reinforcement	[-]
E_s	Young's modulus steel	[MPa]

When the strain formula is filled in this formula we get the following three equations:

$F_{st} = A_{st} f_{ywd} = 618 * 435 = 268830N$
$F_c = \frac{1}{2} b x_y \epsilon(0) E_c = 35887.5 \frac{-x_y^2}{220-x_y}$
$F_{sc} = A_{sc}\epsilon(z_{sc})E_s = 117384 \frac{25-x_y}{220-x_y}$

Step 3: Using the force equilibrium to calculate height of the concrete compression zone (x$_y$

Next, the force equilibrium shown below can be used to calculate $x_[y]$. Using the fact that the sum of all the forces should give 0, a value for x can be calculated by looking for the equilibrium in an iterative process. This starts with an arbitrary value for x. This value is adjusted till there is equilibrium.

$F_{st} + F_{sc} + F_c = 0$

Using this iterative method, a value of 36.58 mm for the concrete compression zone.

Step 4: Calculating the yielding moment capacity

Filling in this x in the force equations gives value for the forces:

$F_{st} = 268830$
$F_C = -261808.61$
$F_{sc} = -7021.39$

Multiplying the forces by the lever arm between the force and the top of the cross-section and adding these values together gives the moment capacity.

$M_y = 268830 * 220 - 261808.61 * 12.19 - 7021.39 * 25 = 55.8kNm$

Yielding moment calculation of a slab with opening
For this calculation, the size opening is set at 80 mm and the Y parameter is set on 180 mm.

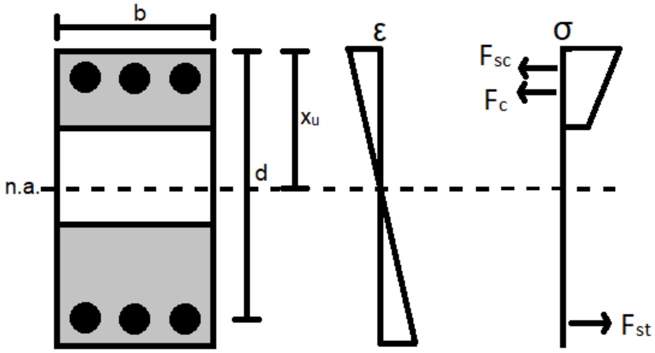

Figure A.2: Cross-section with stress-strain relation

Step 1: Forming the strain formula over the height.

The formula used for the strain distribution of the yielding moment is:

$\epsilon(z) = \frac{z-x_y}{d_t-x_y}\epsilon_s$
In the case of the yielding moment capacity the ϵ_s is 2.175‰ and the effective height is 220 mm. This gives the following strain formula:

$\epsilon(z) = \frac{z-x_y}{220-x_y}0.002175$

Step 2: Filling the strain formula in into the horizontal force formulas

The horizontal force formulas are shown below. Dov is the overlap between the opening and the concrete compression zone calculated as $Dov = Y + 0.5H - (h - x_y)$

$F_{st} = A_{st} f_{ywd}$
$F_c = (\int_0^x \epsilon(z)\, dz - \int_{Dov}^x \epsilon(z)\, dz) E_c b$
$F_{sc} = A_{sc}\epsilon(z_{sc})E_s$ When the strain formula is filled in this formula we get the following three equations:

$F_{st} = A_{st} f_{ywd} = 618 * 435 = 268830N$
$F_c = \frac{1}{2}bx_y\epsilon(0)E_c - \frac{1}{2}bx_y\epsilon(Dov)E_c = 35887.5(\frac{-x_y^2}{220-x_y} - \frac{-Dov^2}{220-x_y}$

$F_{sc} = A_{sc}\epsilon(z_{sc})E_s = 117384\frac{25-x_y}{220-x_y}$

Step 3: Using the force equilibrium to calculate height of the concrete compression zone (x$_y$

Next, the force equilibrium shown below can be used to calculate x$_[y]$. Using the fact that the sum of all the forces should give 0, a value for x can be calculated by looking for the equilibrium in an iterative process. This starts with an arbitrary value for x. This value is adjusted till there is equilibrium.

$F_{st} + F_{sc} + F_c = 0$

Using this iterative method, a value of 37.19 mm for the concrete compression zone.

Step 4: Calculating the yielding moment capacity

Filling in this x in the force equations gives value for the forces:

$F_{st} = 268830$
$F_C = -261375$
$F_{sc} = -7455$

Multiplying the forces by the lever arm between the force and the top of the cross-section and adding these values together gives the moment capacity.

$M_y = 268830 * 220 - 261375 * 12.4 - 7455.39 * 25 = 55.8kNm$

Appendix B

Strut-and-Tie model

In this appendix:

- The Strut-and-Tie model of a slab with a rectangular opening
- The Strut-and-Tie model of a slab with a circular opening

The Strut-and-Tie model of a slab with a rectangular opening

The strut and tie model around a rectangular opening can be drawn in 4 steps. These four steps are explained below. It is also explained how the angles of the struts that are used for the shear capacity calculations can be calculated.

Step 1

In the first step, two lines are drawn from the corner points of the bearing/loading plates to the opposite corner points of the opening. These lines are not struts or ties, but are only there to help draw the strut and tie model. However, the angles between the line and the bearing/load plates becomes the angle of the strut used for calculating the shear capacity. Therefore, the angels of these lines are important for the calculation.

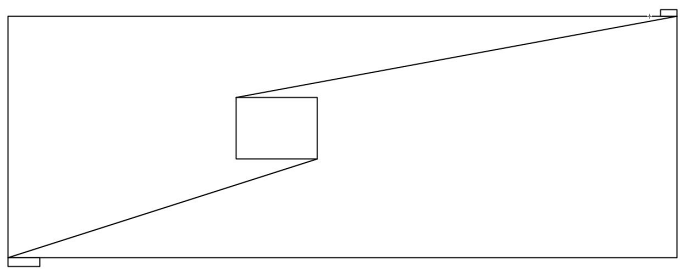

Figure B.1: Step 1

Step 2

In step two, the two struts are drawn parallel to the lines drawn in step 1. The struts are drawn a half-time the strut width away from the opening and the lines from step 1.

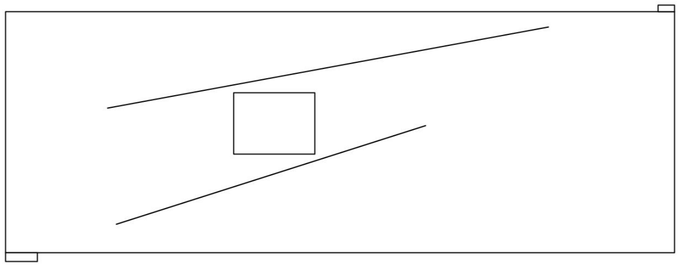

Figure B.2: Step 2

Step 3

In step three, 2 ties are drawn. One tie is drawn perpendicular from the bottom strut to the top strut. This tie is drawn on the opposite site of the opening looking from the origin of the strut. The same thing is done for the second tie, however this tie is drawn perpendicular to the top strut on the opposite site of the opening looking from the top struts' origin. Both ties are drawn at least a half-time the tie width away from the opening and connect the bottom and top strut.

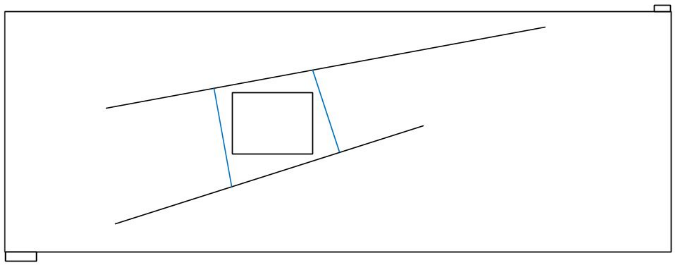

Figure B.3: Step 3

Step 4

In step 4 the bottom and top strut are connected to the bearing and load plate using 4 struts. These struts are drawn from the corner point of the bearing/load plate to the intersection of the strut and ties. Thus making equilibrium. Red lines symbolize the strut, while blue lines symbolize the ties.

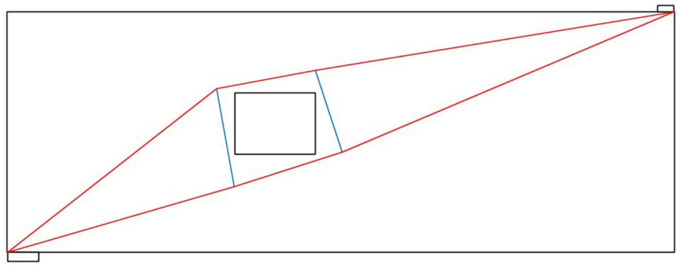

Figure B.4: Step 4

The Strut-and-Tie model of a slab with a circular opening

The strut and tie model around a circular opening can be drawn in 4 steps. These four steps are explained below. It is also explained how the angles of the struts that are used for the shear capacity calculations can be calculated. All steps are similar to drawing struts around rectangular openings. However, step 1 is a bit different.

Step 1

In the first step, two lines are drawn from the corner points of the bearing/loading plates to the tangent point of the circle closest to the horizontal axis of the plate. These lines are not struts or ties, but are only there to help draw the strut and tie model. However, the angles between the line and the bearing/load plates becomes the angle of the strut used for calculating the shear capacity. Therefore, the angels of these lines are important for the calculation.

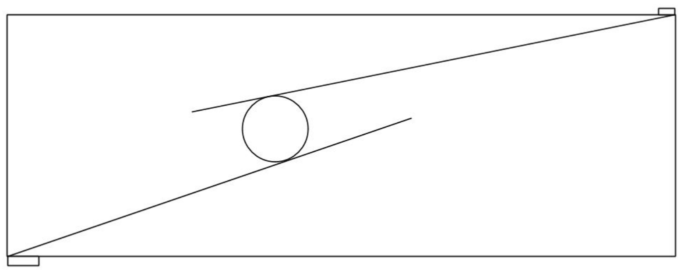

Figure B.5: Step 1

Step 2

In step two, the two struts are drawn parallel to the lines drawn in step 1. The struts are drawn a half-time the strut width away from the opening and the lines from step 1.

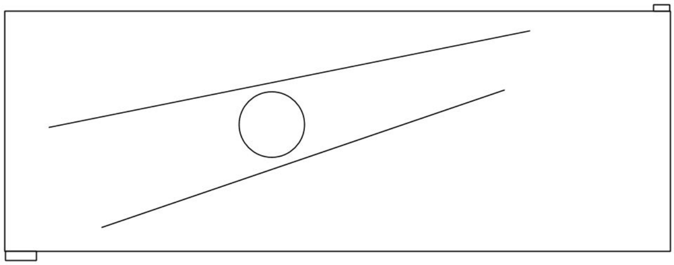

Figure B.6: Step 2

Step 3

In step three, 2 ties are drawn. One tie is drawn perpendicular from the bottom strut to the top strut. This tie is drawn on the opposite site of the opening looking from the origin of the strut. The same thing is done for the second tie, however this tie is drawn perpendicular to the top strut on the opposite site of the opening looking from the top struts' origin. Both ties are drawn at least a half-time the tie width away from the opening and connect the bottom and top strut.

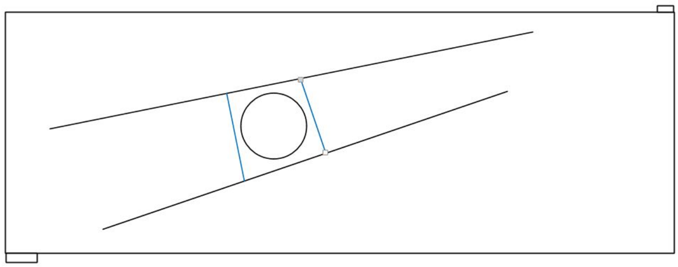

Figure B.7: Step 3

Step 4

In step 4 the bottom and top strut are connected to the bearing and load plate using 4 struts. These struts are drawn from the corner point of the bearing/load plate to the intersection of the strut and ties. Thus making equilibrium. Red lines symbolize the strut, while blue lines symbolize the ties.

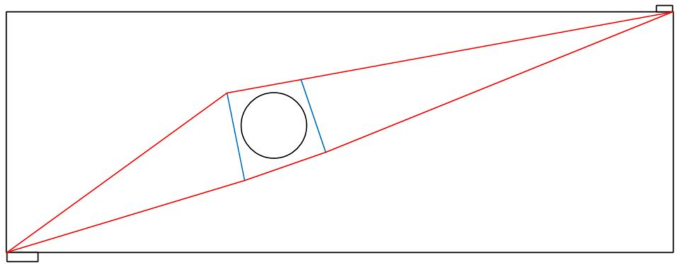

Figure B.8: Step 4

Appendix C

Guidelines pipes in wide floor slabs

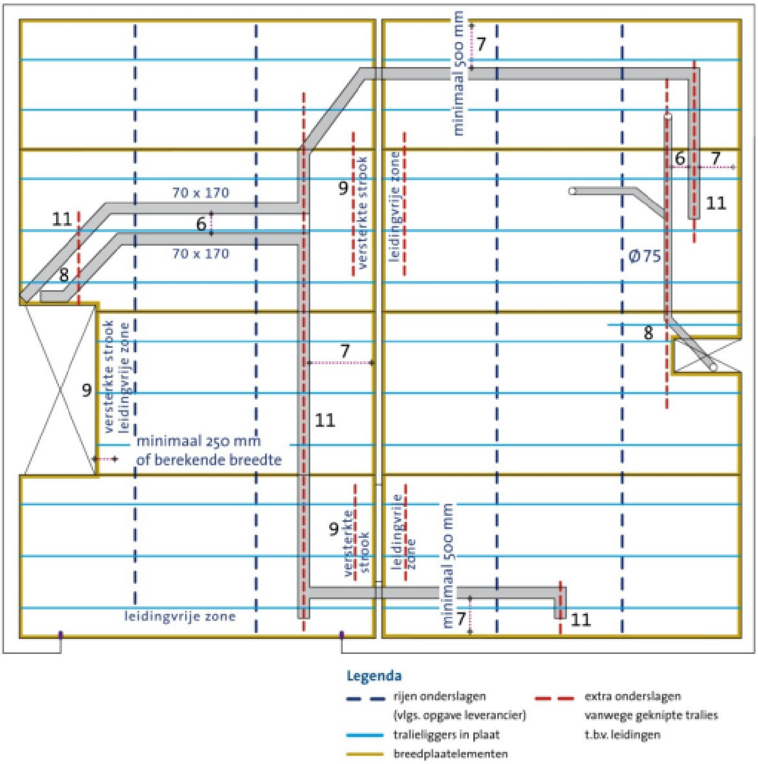

Figure C.1: Example floor plan

1. The maximum thickness of an element = Thickness floor - slab thickness - 60 mm.

2. On top of the elements a minimum of 60 mm concrete is required.

3. If a crossing of elements cannot be prevented, then the element parallel to the lattice girder should be placed in the top part of the floor with and the element perpendicular to the girders should be placed in the lower part of the floor.

4. Elements with a size over 50 by 50 mm should be noted by the designer for assessment.

5. The maximum element width is 250 mm.

6. The distance between elements should be at least the width of the thickest element.
7. The distance between an element and a support should be 1/10 of the length of the slab with a minimum of 500 mm.
8. Elements should be directed away at angle from shafts and stairwells.
9. Elements should not be put in reinforced strips.
10. Electric components should be placed directly on the slab.
11. When girders are cut, the guidelines should be followed.
12. For additional advice you should contact the supplier.

Appendix D

Results of FEM analysis for the validation of the bending moment

This appendix provides the information of the FEM models used to validate the bending moment capacities calculated in chapter 5.1. Besides the basic configurations of the models, the data gathered to validate the calculated data is also shown in this appendix.

The properties of the concrete element used in the FEM models are listed in table D.1.

Table D.1: Material properties for the concrete elements in the models

Parameter	Value
Elastic Modulus	33000 MPa
Poison's ratio	0.2
Tensile strength	1.35 MPa
Fracture energy	0.1 N/mm
Crack band width model	rots
plasticity model	Total strain based crack model
crack orientation	Rotating crack
reduction	no reduction model

In the models, the total strain based crack model was used to simulate the cracking behaviour of the concrete. The properties of table D.1 are used in all models. The only thing that changes in the models are the size and height of the opening. The properties of the

Table D.2: Material properties for the reinforcement bars in the models

Parameter	Value
Elastic Modulus	200000 MPa
Yield strength	435 MPa
Ultimate strength	470 MPa
Plasticity model	Von Mises plasticity
	Total strain-yield stress

reinforcement steel used in the elements for the FEM models are listed in table D.3. The properties apply for both the tension and compression reinforcement. The only parameter that is different between the two kinds of reinforcement is the area, which is 618 mm^2 for the tension reinforcement and 257 mm^2 for the compression reinforcement.

In the table below the cracking moment, yielding moment, elastic moment, and ultimate moment gathered from the analyses are shown per analysis. The moments are calculated on different load steps in the FEM analysis. The points are chosen using different parameters. For the cracking moment the load step was used on which the stress in the bottom of the element reached the cracking capacity of the concrete. For the yielding moment the load step was used on which the stress in the tensile reinforcement reached the yielding stress of the steel. For the elastic moment the load step was used on which the strain in the top of the concrete model reached 1.75‰. Lastly, for the ultimate moment the load step was used on which the strain in the top reached 3.5‰.

In total 7 FEM analyses are performed. One for each configuration of the opening size and the height of the opening. Another FEM analysis is performed with a model of a beam without an opening to see how accurate the FEM model is regarding the calculated data. All the data is shown below.

Table D.3: Moment capacities from the FEM analysis

Model no.	Configuration	M_{cr}	M_y	M_e	M_u
1	No opening	12.2 kN	52.6 kN	58.5 kN	66.1 kN
2	H = 5 mm, Y = 72.5 mm	12.2kN	52.8 kN	58.4 kN	66.0 kN
3	H = 5 mm, Y = 145 mm	12.8kN	52.8 kN	58.4 kN	66.0 kN
4	H = 5 mm, Y = 217.5 mm	4.1kN	52.1kN	58.4 kN	66.0 kN
5	H = 80 mm, Y = 110 mm	11.8kN	52.7kN	58.3 kN	65.9 kN
6	H = 80 mm, Y = 145 mm	11kN	52.7kN	58.3 kN	65.9 kN
7	H = 80 mm, Y = 180 mm	6.5kN	52.9kN	58.3 kN	65.9 kN
8	H = 150 mm, Y = 145 mm	0.5kN	52.9kN	58.1 kN	65.7 kN

Appendix E

Interviews project leaders

In this appendix:

- Overview evaluation interviews;
- Interview Project leader 1;
- Interview Project leader 2;
- Interview Project leader 3;

Overview evaluation interviews
In this thesis, 3 evaluation interviews are held. The goal of the interviews is to validate the previously identified factors that influence the design integration between the mechanical and structural design. The interviews are about the experience the interviewees have with design integration between the mechanical and structural design. Because the interview is focused on validation, a semi-structured interview is used. This allows the interview to be structured along the factors. This gives room to ask follow-up questions on specific answers and to obtain more context on these answers.

The interviewees need experience with integrating designs from different disciplines. Therefore, it was decided to interview project leaders who have at least 10 years of experience as project leader. Project leaders are responsible for the overall design and therefore also have experience with the integration of different designs. With 10 years of experience, the project leaders have experienced multiple projects and their experiences are therefore based on multiple projects. A total of 3 project leaders will be interviewed.

The interviews were conducted face-to-face or via teams and were recorded for later transcription. During the transcription of the interviews, (company) names are anonymized so that the interviewees can speak freely without fear of putting other companies in a bad light. Prior to the interview, the interviewee was asked for permission to record the conversation.

In the thesis, 3 factors have been identified that influence the integration process. These 3 factors are:

1. The client gives the design assignment for a design to the mechanical engineers later than to the structural engineer (work commissioning).
2. The mechanical design from a previous phase does not meet standards (quality).
3. The structural engineers are ahead of the design process to avoid risks (risk).

These 3 factors are the design of the interviews. Questions will be asked on each subject. Before the questions are asked, the subjects will be introduced. After the questions have been asked on each subject, some general questions will be asked. The questions are shown below

Work commissioning

1. Have you worked on projects where the assignment was given to the mechanical engineer later than to the structural engineer?
2. How much later was the assignment given?
3. Why do you think the client gave the assignment to the mechanical engineer later than to the structural engineer?
4. What were the consequences for the mechanical engineer and what were the consequences for the structural engineer?
5. What was the effect on the design process?
6. What were the consequences for the design integration of the designs?
7. What were the consequences for the project?
8. How can it be prevented that the mechanical engineer receive their assignment later than the structural engineer?

Quality

9. Have you worked on projects where the quality of the mechanical design from the previous phase was not good enough to start the new phase?
10. What do you think was the reason the quality of the mechanical design from the previous phase was not good enough?
11. What impact did this have on the progress of the mechanical design?
12. What was ultimately done to solve this problem?
13. What was the effect on the design process?
14. What were the consequences for the design integration of the designs?
15. What were the consequences for the project?
16. How do you think the quality problem in the mechanical design can be prevented?

Risk

17. Have you worked on projects where the progress of the structural design was ahead of the DNR and the mechanical design was in line with the DNR?

18. How far did the design process run ahead of the DNR?
19. Why did the design process run ahead of the DNR?
20. What was the effect on the design process?
21. What were the consequences for the design integration of the designs?
22. What were the consequences for the project?
23. How can it be ensured that the structural design and the mechanical design run in line with the design process?

General

24. Are there any other factors besides the discussed factors that negatively influence the design process?

Interview Project leader 1

Data:	26-09-2022
Interviewer:	Nick Dubbeldam
Function interviewee:	Project leader

Question(Q): Have you worked on projects where the assignment was given to the mechanical engineer later than to the structural engineer?

Answer(A): Yes, I have experienced this a lot in projects. There is however a small nuance. The structural engineer is often already involved in the SO/VO phase of the project. Later in the project, when the contractor is involved, the constructing mechanical engineer is involved. At this point, the trouble starts. In most cases, the TO/UO phase is already started, while there is no assignment for the mechanical engineer. This delays the mechanical engineers.

Q: How much later was the assignment given?

A: That depends on the project. In some projects, the commissioning is going quite fast, while in other projects this takes weeks or even a month.

Q: Why do you think the client gave the assignment to the mechanical engineer later than to the structural engineer?

A: Answered in the first question.

Q: What were the consequences for the mechanical engineer and what were the consequences for the structural engineer?

A: The openings that are needed in the structural elements are communicated too late. In some cases, the UO structural design is nearly finished when the mechanical engineers share this information. This leads to additional iteration steps in the design for both the structural and mechanical engineers since the installations have to be placed in the building. This delays both disciplines.

Q: What was the effect on the design process?

A: Answered in the previous question

Q: What were the consequences for the design integration of the designs?

A: Consequences can be that there are clashes in the design which should have been fixed earlier. Sometimes some installations don't fit anymore. This leads to redesign.

Q: What were the consequences of the project?

A: When the mechanical design isn't finished in time, the whole project stagnates. Also, additional iteration steps cost more money.

Q: How can it be prevented that the mechanical engineer receive their assignment later than the structural engineer?

A: The only way to prevent this is to make sure the mechanical engineer is involved from the beginning of the TO/UO phase. It is even better to have the constructing mechanical engineer even earlier in the project. But this is not always an option.

Q: Have you worked on projects where the quality of the mechanical design from the previous phase was not good enough to start the new phase?

A: The designer in the earlier phases will only make the design till the minimal request of the client. So the quality is what is requested by the client.

Q: What do you think was the reason the quality of the mechanical design from the previous phase was not good enough?

A: Answered in the previous question.

Q: What impact did this have on the progress of the mechanical design?

A: It can have a big impact. If there is a design that only satisfies the minimal requirements and there is an error in this design, there this a chance that a big part of the design has to be redesigned. This costs a lot of time

Q: What was ultimately done to solve this problem?

A: When this happens, the only thing that an engineer can do is just redesigning. However, there are also a lot of cases in which the redesign was done quickly, and the consequences were minimal.

Q: What was the effect on the design process?

A: For the structural design process there are no consequences. However, somewhere in the project, you have to integrate the designs and detect the clashes. If the mechanical design isn't finished in time due to delay, there is a big chance that clashes are not found or found quite late. This can lead to redesign for all disciplines. This delays all the disciplines. It can also lead to aesthetic changes. For example, if a pipe doesn't fit in the element anymore and it has to be made next to the element.

Q: What were the consequences for the design integration of the designs?

A: answered in the previous question.

Q: What were the consequences for the project?

A: The consequences are the same as for the previous factor.

Q: How do you think the quality problem in the mechanical design can be prevented?

A: The mechanical engineers should have time to review the designs of the previous phase. He can use this time to find possible clashes, and he can fix those clashes before the next phase starts. This reduces the number of problems. Increasing the requirements for the DO design is not an option, since there is no benefit for the engineers of the previous phases. Within BAM AE the extra time option is already used in some projects.

Q: Have you worked on projects where the progress of the structural design was ahead of the DNR and the mechanical design was in line with the DNR?

A: Yes. The DNR describes the minimal requirements. Structural engineers tend to do more work in earlier phases to make sure that the structural element suffice. You have to prove to the municipality that the elements suffice, otherwise you can't get a permit. You don't have to make the full calculation for this permit. However, if you already started with a part of this calculation, it is easier to finish it immediately. This reduces the number of risks and design time.

Q: How far did the design process run ahead of the DNR?

A: In most cases, only 1 phase.

Q: Why did the design process run ahead of the DNR?

A: Answered in question 1.

Q: What was the effect on the design process?

A: Exact the same effects as for the previous factors.

Q: What were the consequences for the design integration of the designs?

A: Exact the same effects as for the previous factors.

Q: What were the consequences for the project?

A: Exact the same effects as for the previous factors.

Q: How can it be ensured that the construction design and the installation design run in line with the design process?

A: The best option is to involve everyone from the start of the project. When they work together from the start everyone is on the same page and in most cases, nobody is working ahead.
Q: Are there any other factors besides the discussed factors that negatively influence the design process?

A: Human behaviour. If people don't want to work together, they will frustrate the design process.

Interview Project leader 2

Data:	10-10-2022
Interviewer:	Nick Dubbeldam
Function interviewee:	Project leader

Question(Q): Have you worked on projects where the assignment was given to the mechanical engineer later than to the structural engineer?

Answer(A): Yes, I have experienced this in projects.

Q: How much later was the assignment given?

A: That depends on the project, the parties involved in the project, and the people involved in the project. However, the best situation is that everyone begins at the same time in the project. When everyone starts at the same time, the designs are made at the same time and everything is constructed at the same time.

Q: Why do you think the client gave the assignment to the mechanical engineer later than to the structural engineer?

A: In most cases, the structural discipline has to work fast, since the concrete is the first thing that has to be built. We cannot wait for the mechanical discipline. Also, the design process is different. Structural engineers design a building from the top down, while mechanical engineers design the other way around. So when the structural engineers finish designing the top of the building, the mechanical engineers didn't even start on that part of the building. This causes clashes.

Q: What were the consequences for the mechanical engineer and what were the consequences for the structural engineer?

A: The consequences can be that the disciplines have to wait for each other to develop the design. This delays all the disciplines.

Q: What was the effect on the design process?

A: It can happen that a structural element is already finished. If the mechanical engineers communicate the openings in the elements too late, then the element has to be redesigned. Late changes to the mechanical design can also influence the structural design. In some cases, the design has to start all the way from the beginning.

Q: What were the consequences for the design integration of the designs?

A: answered in the previous question.

Q: What were the consequences of the project?

A: Answered in the previous question.

Q: How can it be prevented that the mechanical engineer receive their assignment later than the structural engineer?

A: Everyone has to start at the same time in the project and everyone has to be able to start designing at the same time. If one starts later, clashes can arise.

Q: Have you worked on projects where the quality of the mechanical design from the previous phase was not good enough to start the new phase?

A: Yes, in most cases the mechanical engineers have to redesign parts of the design from previous phases.

Q: What do you think was the reason the quality of the mechanical design from the previous phase was not good enough?

A: There are plenty of reasons. Sometimes other materials have to be used since other materials were already bought. Sometimes the design is just not complete and in other projects, the designs are not developed enough.

Q: What impact did this have on the progress of the mechanical design?

A: Parts of the design have to be redesigned. This can cause delay, but mostly causes additional work. This leads to clashes with the structural design. When these clashes have to be solved even more redesigning is necessary.

Q: What was ultimately done to solve this problem?

A: The only option is just redesigning. If the problem is already caused, the only thing you can do is fix it.

Q: What was the effect on the design process?

A: It has the same effects as the previous factors.

Q: What were the consequences for the design integration of the designs?

A: It has the same consequences as the previous factors.

Q: What were the consequences for the project?

A: It has the same consequences as the previous factors.

Q: How do you think the quality problem in the mechanical design can be prevented?

A: The mechanical engineers have to be more proactive. They shouldn't wait for the materials to be bought or for the other disciplines to design. When they wait too long the other disciplines get a head start, which complicates the design.

Q: Have you worked on projects where the progress of the structural design was ahead of the DNR and the mechanical design was in line with the DNR?

A: No, I have never experienced this. You just check the boxes on the DNR forms. In some cases, some extra work is done. However, this depends on the project. This can be the case if the subcontractor is not able to design certain elements. However, this is not working ahead, but just additional work.

Q: How far did the design process run ahead of the DNR?

A: not relevant

Q: Why did the design process run ahead of the DNR?

A: not relevant

Q: What was the effect on the design process?

A: not relevant

Q: What were the consequences for the design integration of the designs?

A: not relevant

Q: What were the consequences for the project?

A: not relevant

Q: How can it be ensured that the construction design and the installation design run in line with the design process?

A: not relevant

Q: Are there any other factors besides the discussed factors that negatively influence the design process?

A: no

Interview Project leader 3

Data:	11-10-2022
Interviewer:	Nick Dubbeldam
Function interviewee:	Project leader

Question(Q): Have you worked on projects where the assignment was given to the mechanical engineer later than to the structural engineer?

Answer(A): Yes, in some cases the constructing mechanical engineer is involved late in the project.

Q: How much later was the assignment given?

A: That depends on the project. In some projects, the contracting mechanical engineers are so late involved that no freedom of design was left. It also depends on the level of detail in earlier designs. Sometimes it is also the preference of the project leader and sometimes the constructing engineer is not needed yet.

Q: Why do you think the client gave the assignment to the mechanical engineer later than to the structural engineer?

A: explained in the answer above.

Q: What were the consequences for the mechanical engineer and what were the consequences for the structural engineer?

A: Sometimes there is no freedom of design left. Due to the late involvement, the designs are finished later. This causes more design changes for all disciplines.

Q: What was the effect on the design process?

A: Design changes have an impact on the process. However, when the mechanical design is done later, there is less freedom of design, e.g. some installations cannot be placed on the best position anymore. This might increase the price for the installations. The bottom line is that it is always bad for the end result.

Q: What were the consequences for the design integration of the designs?

A: As explained, the integration leads to more clashes. In some elements, there is no room anymore for installations. The installations have to be placed somewhere else.

Q: What were the consequences of the project?

A: There are no major consequences for the project. The project will always go on.

Q: How can it be prevented that the mechanical engineer receive their assignment later than the structural engineer?

A: Making a better overall model, will lead to better and faster contracting. A better model makes clash detection easier. This reduces the number of problems. Secondly, the constructing engineers can be contracted easier.

Q: Have you worked on projects where the quality of the mechanical design from the previous phase was not good enough to start the new phase?

A: Yes, In some cases the constructing mechanical engineers have another perspective on the design. It can happen that the personal preferences of the engineer leads to design changes. In some projects, clashes are only detected after the detail engineering. This can also be marked as a quality aspect.

Q: What do you think was the reason the quality of the mechanical design from the previous phase was not good enough?

A: The quality of the design is not necessarily low. However, the designing mechanical engineers cannot design in enough detail. Therefore, the constructing mechanical engineers have to do most of the design and do the detailed design.

Q: What impact did this have on the progress of the mechanical design?

A: In most cases, this has a small impact on the progress. The mechanical engineers have to do more work, which makes the schedule tighter. Real problems show up when in the detail engineering new information is found which requires design changes. In most cases, there are always solutions for these problems. However, the problems disrupt the process. The problems require redesigning. That cost time.

Q: What was ultimately done to solve this problem?

A: The solution to the problem depends on the problem itself. In some cases, an opening is drilled instead of cast in the concrete, and in other cases, an installation is placed in a different place. As mentioned before, there is always a solution. The best solution is better engineering upfront.

Q: What was the effect on the design process?

A: The problems frustrate the process. However, this doesn't mean that the building cannot be built.

Q: What were the consequences for the design integration of the designs?

A: A lot of errors can occur while integrating the designs if some designs are finished quite late. When these errors are noticed too late, the problems become harder to solve. Therefore, if a design is finished late and therefore the integration is late, there can be big consequences for the design. In some cases, floor plans have to be adjusted, which causes a lot of other design changes.

Q: What were the consequences for the project?

A: The consequences described in the previous answer are not only consequences for the mechanical discipline, but rather for all disciplines. So this problem can have big consequences for the whole project.

Q: How do you think the quality problem in the mechanical design can be prevented?

A: The best case is to involve the constructing engineer earlier in the project. However, due to building costs and other practical issues, this is not possible. A good solution is to get a modeller in between the designing and constructing engineers. This modeller can already find some clashes, which can then already be solved.

Q: Have you worked on projects where the progress of the structural design was ahead of the DNR and the mechanical design was in line with the DNR?

A: We don't do more than the DNR describes. We work till the DNR checklist in each phase. In some cases, some details are designed earlier. However, these activities are so small that they are barely noticeable. The DNR gives a good project process, which can always be followed.

Q: How far did the design process run ahead of the DNR?

A: not relevant

Q: Why did the design process run ahead of the DNR?

A: not relevant

Q: What was the effect on the design process?

A: not relevant

Q: What were the consequences for the design integration of the designs?

A: not relevant

Q: What were the consequences for the project?

A: not relevant

Q: How can it be ensured that the construction design and the installation design run in line with the design process?

A: not relevant

Q: Are there any other factors besides the discussed factors that negatively influence the design process?

A: Changes in plans are always possible. This can require a lot of redesigning. Buildability can also change the designs, and overall design changes can always have big consequences.

Appendix F

Interview Expert panel

To evaluate the integrated design strategy an expert group is asked to discuss the feasibility and effectiveness of the strategy. The expert group consists of 8 experts with different backgrounds. However, all experts developing methods to improve structural safety in a task group of KPCV.

The summary of the integrated design strategy was first sent to the experts, after which a presentation was given to clarify any ambiguities. After this presentation, the experts were asked to give their view on the feasibility and effectiveness of the solutions. Their views on these matters are stated below. Since all experts had a similar view, their view is summarized per question.

How feasible is the solution?

The expert group argues that implementing design rules and an additional design phase only to improve the design of wide floor slabs is not feasible. A wide floor slab is a relatively unimportant element of the building. Therefore, adding a phase just to improve the safety of this element is not feasible. However, if the method improves other structural elements as while, it can be implemented since the effort put into the phase pays off.

How effective is the solution?

The expert group argues that the problem is in the difference in the level of detail of the design of the mechanical discipline and the structural discipline. The designs of the mechanical discipline are low in detail till late in the project. When the detail is added clashes are detected. Since the clashes are detected so late in the project, solving them is hard. Using the fact that more detail is needed earlier in the project, the expert group argues that an additional design phase is a good idea. However, placing it so late in the project can only mitigate the effects of the problems and cannot solve the problems. To solve the problems, the preparation phase must be placed between the VO and DO phase.

The expert group also argues that there are better solutions to solve the problem. Since the problem occurs due to a different level of detail for the different disciplines, the expert group argues that the required level of detail of the mechanical designs should be improved. To do so the DNR-STB should be improved. Secondly, the constructing mechanical engineer should be involved earlier in the project. When he is involved earlier in the project, the level of detail improves. Since the constructing mechanical engineer is contracted by the contractor, the contractor should be involved earlier in the project

Lastly, the expert group argues that there must be a change in mindset. The mindset should work towards more collaboration and communication. When everyone in the project works with this mindset from the beginning of the project a lot of problems can be prevented.