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Development and Behaviour of a New Long-Span Composite Floor System

By Florian Bodensiek

A thesis submitted in partial fulfilment of the requirements for the degree of
Doctor of Philosophy

Supervised by Associate Professor John Butterworth
and Associate Professor G. Charles Clifton

University of Auckland
Department of Civil and Environmental Engineering
New Zealand
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ABSTRACT

This thesis describes the concept behind a new long-span composite floor system and the experimental and numerical verification of that concept. Externally, the floor looks like a nine-metre-long double-Tee floor with hanger systems at its supports, but with unusually thin ribs. Internally, a new load-carrying system has been developed, using a centrally placed perforated light-gauge steel sheet. By providing a continuous medium for internal force transfer, the steel sheet offers some significant advantages over conventional shear reinforcement. It is envisaged that the floor unit would be produced as a precast, top-hung element that can be installed with top of concrete level on deforming supports and with no requirement for a topping slab.

Experimental testing has verified the constructability and behaviour of three 4.5-metre-long floor specimens, built in accordance with the proposed concept. Causes of premature failure in the first test were reviewed and the floor concept improved for the following tests, although it is envisaged that further improvements could be made before the concept was commercialised. The tests showed that the steel sheet, which was perforated with holes to improve the connection to the concrete, was able to replace common stirrups and generate full composite action with the surrounding concrete.

A first-principles theoretical model has been developed and implemented in the form of Microsoft Excel spreadsheets, using the “solver” add-in for optimisation purposes. Different stress-strain curves for the concrete’s behaviour under tension have been modelled and investigated, based on the results of small-scale material tests. Incorporating the concrete material model to theoretical models of the experimental floor test specimens have been developed which showed good agreement with the load-deflection curves observed in the large-scale experimental testing, especially in the test in which the failure mode matched that on which the theoretical model was based.

To model the floor behaviour more accurately than the theoretical model could achieve, including more precise modelling of the actual experimental support condition, a numerical model was constructed using Abaqus finite-element software. The results closely supported the outcome of experimental testing and theoretical modelling.
The research demonstrated that the proposed floor type is viable and suggests further topics of research and testing that would be required prior to commercial implementation.
ACKNOWLEDGEMENTS

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This project would not have been possible without the financial contributions made by the Foundation for Research Science & Technology (FRST).

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# NOTATION

**Latin letters**

- $a$: Depth of equivalent rectangular stress block
- $A$: Cross-sectional area
- $A_{\text{comp}}$: Composite area
- $A_s$: Effective shear area
- $A_s$: Shear area or cross-sectional area of reinforcement or area of flexural tension reinforcement
- $A_w$: Gross sectional area of the steel web
- $b$: Width
- $b_{\text{eff}}$: Effective width
- $b_w$: Width of web
- $c$: Distance from extreme compression fibre to neutral axis
- $d$: Effective depth of a cross-section
- $E$: Modulus of elasticity
- $E_c$: Modulus of elasticity of concrete
- $E_s$: Modulus of elasticity of steel
- $EI$: Bending stiffness
- $f_c$: Compressive strength of concrete
- $F_{\text{conc}}$: Force in the concrete
- $F_{\text{machine}}$: Force of the actuator which loads the slab in the experimental test
- $f_n$: Natural frequency
- $F_c$: Compression force in cross-section
- $F_r$: Force in the reinforcement
- $F_{\text{reinf}}$: Force in the reinforcement
- $F_s$: Force in the steel sheet
- $F_{\text{sheet}}$: Force in the steel sheet
- $F_t$: Tension force in cross-section
- $f_{\text{cd}}$: Design value of concrete compression strength
- $f_{ck}$: Characteristic compressive cylinder strength of concrete at 28 days
- $f_{su}$: Average design shear stress in a web
- $f_{vm}$: Maximum design shear stress in a web
- $f_y$: Yield strength of reinforcement
- $f_{yd}$: Design yield strength of reinforcement
- $g$: Standard gravity or line load
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$G_c$</td>
<td>Shear modulus of concrete</td>
</tr>
<tr>
<td>$G_s$</td>
<td>Shear modulus of steel</td>
</tr>
<tr>
<td>$h$</td>
<td>Height</td>
</tr>
<tr>
<td>$I$</td>
<td>Second moment of area</td>
</tr>
<tr>
<td>$I_{\text{comp}}$</td>
<td>Second moment of area for the composite section</td>
</tr>
<tr>
<td>$k_a$</td>
<td>Factor allowing for the influence of aggregate size on shear strength</td>
</tr>
<tr>
<td>$k_d$</td>
<td>Factor allowing for the influence of member depth on shear strength</td>
</tr>
<tr>
<td>$l$</td>
<td>Length</td>
</tr>
<tr>
<td>$m$</td>
<td>Mass</td>
</tr>
<tr>
<td>$M$</td>
<td>Bending-moment</td>
</tr>
<tr>
<td>$M^*$</td>
<td>Design moment at section at the ultimate limit state</td>
</tr>
<tr>
<td>$\bar{M}$</td>
<td>Virtual bending-moment</td>
</tr>
<tr>
<td>$M_n$</td>
<td>Nominal flexural strength of section</td>
</tr>
<tr>
<td>$M_s$</td>
<td>Nominal section moment capacity</td>
</tr>
<tr>
<td>$n$</td>
<td>Ratio of the elastic modulus of steel to the elastic modulus of concrete</td>
</tr>
<tr>
<td>$n_{G_r}$</td>
<td>Ratio of the shear modulus of steel to the shear modulus of concrete</td>
</tr>
<tr>
<td>$p_w$</td>
<td>$= \frac{A_s}{b_1d}$</td>
</tr>
<tr>
<td>$q$</td>
<td>Line load or area load</td>
</tr>
<tr>
<td>$t^*$</td>
<td>Constant steel sheet thickness if the holes in the sheet are smeared over the length</td>
</tr>
<tr>
<td>$u$</td>
<td>Deflection</td>
</tr>
<tr>
<td>$u_{\text{bending}}$</td>
<td>Bending deflection</td>
</tr>
<tr>
<td>$u_{\text{shear}}$</td>
<td>Shear deflection</td>
</tr>
<tr>
<td>$V$</td>
<td>Shear force</td>
</tr>
<tr>
<td>$\bar{V}$</td>
<td>Virtual shear force</td>
</tr>
<tr>
<td>$V^*$</td>
<td>Design shear force at section at the ultimate limit state</td>
</tr>
<tr>
<td>$v_b$</td>
<td>Factor to calculate the shear resisted by concrete</td>
</tr>
<tr>
<td>$v_c$</td>
<td>Shear resisted by concrete</td>
</tr>
<tr>
<td>$V_c$</td>
<td>Nominal shear strength provided by concrete</td>
</tr>
<tr>
<td>$V_n$</td>
<td>Total nominal shear strength of section</td>
</tr>
<tr>
<td>$V_{pl,R}$</td>
<td>Plastic shear resistance</td>
</tr>
<tr>
<td>$V_s$</td>
<td>Nominal shear strength provided by the shear reinforcement</td>
</tr>
<tr>
<td>$V_v$</td>
<td>Nominal shear capacity of the web</td>
</tr>
<tr>
<td>$V_{vm}$</td>
<td>Nominal web shear capacity in the presence of bending moment (interaction)</td>
</tr>
<tr>
<td>$V_{vm}$</td>
<td>Nominal shear capacity of a flat plate with non-uniform shear stress distribution</td>
</tr>
</tbody>
</table>
Nominal shear capacity of a web with uniform shear stress distribution

Nominal shear capacity of a web

Vertical distance to the top surface of the floor

Rectangular distance between $F_{\text{conc}}$ and the neutral axis

Rectangular distance between $F_{\text{reinf}}$ and the neutral axis

Rectangular distance between $F_{\text{sheet}}$ and the neutral axis

Greek letters

$\alpha$  
Coefficient

$\alpha_1$  
Factor for flexural strength calculation

$\beta$  
Factor for flexural strength calculation

$\gamma$  
Shear angle

$\gamma_c$  
Partial factor for concrete

$\gamma_s$  
Partial factor for reinforcing

$\varepsilon$  
Strain

$\varepsilon_b$  
Strain at the bottom surface of the cross-section

$\varepsilon_{\text{reinf}}$  
Strain of reinforcement

$\varepsilon_t$  
Strain at the top surface of the cross-section

$\phi$  
Strength reduction factor

$\kappa$  
Curvature

$\mu_{\text{Eds}}$  
Coefficient for calculating the cross-sectional area of reinforcement for a reinforced concrete beam

$\nu_c$  
Poisson’s ratio for concrete

$\nu_s$  
Poisson’s ratio for steel

$\rho$  
Density

$\sigma$  
Stress

$\tau$  
Shear stress

$\omega$  
Coefficient for calculating the cross-sectional area of reinforcement for a reinforced concrete beam
1 INTRODUCTION

1.1 RESEARCH MOTIVATION

Decades ago, buildings tended to be broadly categorised according to their material of construction, such as masonry, timber, structural steel or reinforced concrete. More recently, the construction industry has tended to use a variety of materials in combination that are both structurally efficient and supportive of architectural innovation. These benefits have increased the number of composite constructions significantly. Newly developed materials, such as high strength and light-weight concrete and geopolymers, or the use of steel and synthetic fibres are promoting this trend.

Against this background, the brief for the research that is described in this thesis has been developed. The objective was established by a consortium of: four industrial companies; two research providers, namely the University of Auckland and the Auckland University of Technology; and the New Zealand Heavy Engineering Research Association (HERA). The consortium was established to develop a range of advanced Composite Structural Assemblies (CSA) for use in the building industry and with potential to export.

This thesis is about the development and analysis process of a new composite floor system, comprising concrete and light-gauge steel acting together to resist bending and shear actions. Generally, the term “composite floor” indicates a floor type based on a formed steel sheet and in-situ concrete supported on a grid of secondary and primary steel beams. In this project “composite” stands for a material combination of thin formed steel sheet and concrete, but used in a different manner to concrete poured onto formed steel deck. The brief for the floor unit was further refined as the result of discussions with industry partners to include the requirement of being precast with no topping layer and being capable of spanning nine metres or more.

The proposed floor concept is new and innovative. From the outside it looks similar to a common double-Tee floor, but inside the floor, a perforated steel sheet replaces
Introduction

conventional stirrups for reinforcement and also for development of flexural strength and anchorage of support forces.

As this research is part of the CSA-project, some thesis objectives might be part of or even identical with some CSA objectives. However, the thesis is focused on some individual aspects only and, therefore, the CSA objectives and thesis objectives are not identical.

Key CSA objectives of any new floor element are:

1. To develop a floor system that uses light-gauge sheet-steel for construction and which should be able to be produced as a precast element.
2. To verify the feasibility of the proposed floor slab with an experimental test on a 9-m long floor specimen.
3. To verify and analyse the experimental test data with theoretical models based on simplified hand calculations or FEM-analysis.
4. To analyse competing floor systems and to show the international export market for the finished product.
5. The floor system should not require any additional concrete topping on site.

However, not all of these CSA objectives are also the objectives of this thesis:

1. Same as CSA objective
2. The maximum span length for experimental tests was limited to 4.5 m by the capabilities of the laboratory. Therefore, the objective was to verify the proposed floor system on test specimens with a length of 4.5 m only. However, only the length is scaled down and not the cross-section. Therefore, the shear and flexural strength of 4.5-m and 9-m long specimens have to be the same.
3. Same as CSA objective
4. This is a CSA project objective which is not part of this thesis.
5. During discussions within the CSA group some ideas were developed on how to achieve this point, such as a levelling support system at the SHS, however, this research is out of the scope of this thesis.

Generally, this thesis aims at developing and proving the concept of a new floor system. On the basis of three experimental tests with a specimen span of 4.5 m the workability of the concept idea was shown and the critical parameters which could lead to premature
failure identified and improved. An analysis based on first principles flexural behaviour has been made and compared with the experimental results. The load-deflection curve has been theoretically deduced with Excel spreadsheets and FEM-analysis and has been compared with the results from experimental testing.

The intended market for the floor system could be office buildings or car parks. However, this thesis does not complete the research on this floor concept necessary to bring it to market readiness. Before a prototype can be built, further research and experimental testing is necessary. For example, this thesis does not investigate the panel to panel connection on the long side of the floor, the levelling support system at the SHS or prove the acceptance for vibration, thermal, fire or acoustic behaviour for different environments. These are part of follow-up investigations by other CSA researchers. Therefore, no target parameters for these topics were given. As this research is focused on the strength and constructability of the floor system, a target live load of 5kN/m² was specified by the CSA partners. Specification of serviceability parameters, such as those relating to vibrations and acoustics, was deferred until research on strength provision had progressed.

1.2 THESIS OUTLINE

Apart from chapters 1, 8 and 9, the Introduction, Conclusions and References respectively, this thesis is organised into the following main chapters:

Chapter 2: FLOOR SYSTEMS OVERVIEW AND LITERATURE REVIEW
This chapter gives a brief overview of all relevant floor systems and orders them into categories. Two groups, the composite floors and the precast floors, are described and analysed in more detail. Current literature is referenced and discussed.

Chapter 3: CONCEPT DEVELOPMENT
The third chapter describes and explains the decisions made when developing the new floor system, named “F1”. It illustrates the development process of the new F1 system. All structural parts of the system are explained, and all dimensions are given in engineering
drawings. Some unique advantages of the new floor system are listed in the last section of chapter 3.

Chapter 4: EXPERIMENTAL TESTS
This chapter describes all the experimental tests that were conducted. These included three large-scale floor specimens, of length of four-and-a-half metres, which were tested to destruction in bending under a central point load. The differences between these tests are explained in the corresponding sections. Before the floor elements were tested under loading, their natural frequencies were measured. Small-scale material tests were conducted for the concrete and for the steel sheet. The end connection was separately tested once, as this part of the floor element contains a key novelty and was not damaged by the bending test.

Chapter 5: THEORETICAL MODELLING
Important results sought in a floor test include the strength and stiffness properties, which can be effectively summarised by a load-deflection curve. This chapter explains the theoretical modelling considerations which are necessary to develop this curve by hand calculations. In order to be more efficient, the Microsoft Excel spreadsheet program was used for automation. The results are discussed and compared with the floor’s behaviour in experimental testing, see especially section 5.2 for this comparison.

Chapter 6: FINITE-ELEMENT ANALYSIS
The finite-element program Abaqus was used to investigate the behaviour of the experimental test specimens. With Abaqus, some details can be modelled with more accuracy and detail than the theoretical model, which had to make some simplifications. However, some aspects are difficult to model in a finite-element program, which is discussed at the beginning of the chapter. The load-deflection curve produced by Abaqus is analysed and compared with the results from the experimental test and from the Excel model.

Chapter 7: DISCUSSION AND SUGGESTIONS FOR FURTHER WORK
This chapter provides a critical discussion of the findings of the research, and gives suggestions for possible improvements and further work. Detailed discussions of chapters 4, 5 and 6 can be found in those chapters.

1.3 CSA

CSA stands for Composite Structural Assemblies. The CSA project is a research project which consist of a partnership between the Heavy Engineering Research Association (HERA), New Zealand industrial companies, such as New Zealand Steel, Fletcher Composite Research, Grayson Engineering and Tandarra Engineering and two academic institutions: the University of Auckland and Auckland University of Technology.

In the funding round from the Foundation for Research Science & Technology (FRST) in 2004/2005, the partnership secured government support for a six-year research programme to develop a range of advanced Composite Structural Assembly products.

The research goal was to develop new composite building products, such as wall or floor elements that use locally produced thin sheet-steel. The initial focus was on the development of a high performance wall unit. While this was being developed, a second line of research was begun to develop an advanced floor unit. The research for the development of the floor unit was conducted by the author and forms the basis of this thesis. Unlike the wall unit, which had numerous researchers working in parallel, the floor unit was entirely the work of the author, assisted by his supervisors.

The CSA project brings the industry partners and research institutions together to work on a common goal of new product development. As a result, the focus is on meeting market needs, cost and constructability, as much as understanding a predicted structural behaviour and developing appropriate design procedures.

Academic institutions, such as universities, have the role of undertaking research projects and analysing problems with academic precision. However, in order to solve industry needs, the combination of industry knowledge and academic research expertise is required.
Introduction

A special challenge in this thesis was to meet the different requirements of the industry partners and the universities involved.

The involvement of industry partners and their need to protect new ideas and intellectual property meant that all project personnel were bound by confidentiality agreements. As a consequence, it was not possible to publish in the technical literature during the time of research. This thesis will also be subject to a two-year embargo on publication. However, due to the fact that HERA and other leading companies and Universities were involved in the project, it was possible to present the ideas and outcomes to them. This internal feedback and advice from the CSA partners helped very much during the development process.
2 FLOOR SYSTEMS OVERVIEW AND LITERATURE REVIEW

2.1 INTRODUCTION

Many different floor systems have been developed. The main differentiating factor between the various floor types is the floor’s spanning capability under working loads, meeting strength and stiffness criteria. Systems can be difficult to compare when their spanning capabilities are very different, however, typically more than one system can be used. Section 2.2 gives a brief overview of the most common floor types, and arranges them into categories.

As this thesis is describing a new floor system using novel combinations of materials, literature directly relevant to this new system does not yet exist. However, the two most important categories relating to the present research are composite floors, which use a steel deck and concrete poured on the construction site (in-situ concrete) and precast floors. The two main types of prestressed precast floors are hollowcore slabs and double-Tee floors.

The floor systems presented are not all competing directly with the new system. They were chosen in this chapter to show the variety of general floor solutions and provide an insight into the development process of the new floor element. Each new floor element is typically not totally new; instead, it is often a combination of different floor types which have been used separately beforehand. During development of the first ideas of this research it was actually hard to find a totally new idea which made sense to investigate further, as by closer analysing and improving the promising new ideas, often one already existing floor type was the outcome.

The four criteria given for the floor development in section 3.2 are not a result of the analysis of the existing floor types in this chapter. They were given by the CSA-project as required criteria. Therefore, this research had to meet these requirements which placed some constraints on the solution developed.
Two of the criteria were that the new floor type should be a precast element and should also involve a steel sheet as a structural component. The new floor element is essentially a combination of two existing floor system concepts, the composite light steel deck/concrete floors and the precast floors. Therefore, the composite floors and the precast floors are reviewed in more detail in section 2.3 and 2.4.

2.2 FLOOR SYSTEMS OVERVIEW

Concrete floors using in-situ concrete can be built as one-way or two-way systems. They can also be built as single or multi-span systems. These floor types can be compared by the span length as shown in Figure 2-3 or by square bay sizes as shown in Figure 2-1.

Precast elements are typically one-way, single span systems only, which makes the span length in one direction the most important parameter for these systems. As the developed floor system in this thesis should also be a one-way, single span precast system, the analysed floor systems in this thesis are best compared by span length rather than bay size.
Figure 2-1: Cost versus bay size and load (Canadian Portland Cement Association, 1989)
Figure 2-2: Floor systems used in Figure 2-1

Figure 2-3 shows the span range for several in-situ floor systems. The types considered are flat plates, flat slabs, ribbed slab and band beams and are illustrated in Figure 2-4.
## 2.2 Floor Systems Overview

![Comparative selection chart for in-situ floor systems (Cement & Concrete Association of Australia, 2003)](image)

<table>
<thead>
<tr>
<th>Floor system</th>
<th>Comparative span (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flat Plate</td>
<td>0 5 10 15 20</td>
</tr>
<tr>
<td>Single span</td>
<td>Reinforced Prestressed</td>
</tr>
<tr>
<td>Multi-span</td>
<td>Reinforced Prestressed</td>
</tr>
<tr>
<td>Flat Slab</td>
<td>0 5 10 15 20</td>
</tr>
<tr>
<td>Multi-span</td>
<td>Reinforced</td>
</tr>
<tr>
<td>Ribbed Slab</td>
<td>0 5 10 15 20</td>
</tr>
<tr>
<td>Single span</td>
<td>Reinforced</td>
</tr>
<tr>
<td>Multi-span</td>
<td>Reinforced</td>
</tr>
<tr>
<td>Band Beam and Slab</td>
<td>0 5 10 15 20</td>
</tr>
<tr>
<td>Single span</td>
<td>Reinforced</td>
</tr>
<tr>
<td>Multi-span</td>
<td>Prestressed</td>
</tr>
<tr>
<td>Band Beams at 8.4 m</td>
<td>0 5 10 15 20</td>
</tr>
<tr>
<td>Single span</td>
<td>Prestressed</td>
</tr>
<tr>
<td>Multi-span</td>
<td>Prestressed</td>
</tr>
</tbody>
</table>

Figure 2-3: Comparative selection chart for in-situ floor systems (Cement & Concrete Association of Australia, 2003)
Similar span tables exist for precast slabs as shown in Figure 2-5. The analysed floor types in this table are: hollowcore planks, permanent formwork, beam and infill, solid slabs, single-Tees and double Tees. A picture of each of these floor types is given in Figure 2-6.

An important parameter for the span length is the thickness or depth of the floor system. The span range changes not only between different systems it also changes significantly between different depths of the same system. Figure 2-5 gives a good overview about the structural depth and span limitations of the various systems.
### 2.2 Floor Systems Overview

#### 2.2.1.2 HOLLOWCORE PLANKS

<table>
<thead>
<tr>
<th>Without topping</th>
<th>Thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>150</td>
</tr>
<tr>
<td></td>
<td>200</td>
</tr>
<tr>
<td></td>
<td>250</td>
</tr>
<tr>
<td></td>
<td>300</td>
</tr>
<tr>
<td></td>
<td>350</td>
</tr>
<tr>
<td></td>
<td>400</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>With topping (60 mm)</th>
<th>Thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>150</td>
</tr>
<tr>
<td></td>
<td>200</td>
</tr>
<tr>
<td></td>
<td>250</td>
</tr>
<tr>
<td></td>
<td>300</td>
</tr>
<tr>
<td></td>
<td>350</td>
</tr>
<tr>
<td></td>
<td>400</td>
</tr>
</tbody>
</table>

#### 2.2.1.3 COMPOSITE FLOORING

1. **Permanent formwork**
   - Total slab thickness (mm): 160, 190, 230, 270

2. **Beam and infill**
   - Beam types: 130R, 150R/130C, 150C, 200C, 250C

#### 2.2.1.4 SOLID SLABS

<table>
<thead>
<tr>
<th>Without topping</th>
<th>Thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>150</td>
</tr>
<tr>
<td></td>
<td>200</td>
</tr>
<tr>
<td></td>
<td>250</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>With topping (60 mm)</th>
<th>Thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>150</td>
</tr>
<tr>
<td></td>
<td>200</td>
</tr>
<tr>
<td></td>
<td>250</td>
</tr>
</tbody>
</table>

#### 2.2.1.5 TEE-BEAMS

<table>
<thead>
<tr>
<th>Single-Tees (60 topping)</th>
<th>Depth (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>500</td>
</tr>
<tr>
<td></td>
<td>300</td>
</tr>
<tr>
<td></td>
<td>1000</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Double-Tees (60 topping)</th>
<th>Depth (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>400</td>
</tr>
<tr>
<td></td>
<td>300</td>
</tr>
<tr>
<td></td>
<td>700</td>
</tr>
</tbody>
</table>

---

Figure 2-5: Comparative selection chart for precast floor systems (Concrete Institute of Australia and National Precast Concrete Association Australia, 2002)
Figure 2-6: Floor systems used in Figure 2-5 (Concrete Institute of Australia, and National Precast Concrete Association Australia, 2002)
Load-span tables for composite floors are normally given by the steel sheet producer. Figure 2-7 gives an example for the ComFlor 210 profile from Corus. The maximum span is dependent on the applied load, the slab depth, the mesh and rebar size and the required fire rating.

![Spar table - Normal weight concrete](image)

Figure 2-7: ComFlor 210 span table (Corus, 2008)

Figure 2-8 shows a general grouping of these existing floors into four categories, which makes it easier to compare the individual systems.

It should be pointed out that the diagram in Figure 2-8 shows apart from ordinary reinforced concrete floors an overview of different floor types, but that the examples shown in each category are only individual choices by the author of this thesis. The aim in selecting the floor type examples was not only focused on presenting the most common floor types, but also to show some interesting and less common floor types such as the BubbleDeck to stimulate the imagination for developing a new floor system. The Cellular-beam (number 6 in Figure 2-8) is only one interesting example for a beam and could be replaced in this category by any other beam, such as a common steel I-beam for example.

Generally, one-way spanning floor slabs are supported on beams. Depending on the beams used and their spacing, they can span longer distances than the floor type itself. This
sometimes makes it difficult to define the main floor system and to compare the span length. With supporting beams every floor system can be used to span 9 m, which is the target length as discussed in section 3.2, as the beam is spanning the 9 m and the floor is spanning, perpendicular to that, the smaller distance between the beams. As the main interest of this chapter is to find new ideas for a new floor system, only different one way spanning floor solutions are analysed and not the supporting beams, which could be in most cases standard I-beams.

Figure 2-8 is organised into two main categories and four subcategories. The two main categories are the floors which have to use in-situ concrete and the floors which are built as precast concrete slabs. The in-situ category itself can be separated into two subcategories: the floor types which use a steel deck and which are commonly named composite floors and the floor types which are built up from any kind of beams with infill between them.

The two subcategories of the precast slabs are the non-prestressed and the prestressed floors. Most precast elements are not totally independent of in-situ concrete, as they use in-situ concrete for their topping.
The pictures in Figure 2-8 are taken from the following sources:
1 ComFlor 51 (Corus, 2008)
2 ComFlor 80, (Corus, 2008)
3 ComFlor 210, (Corus, 2008)
4 (Bauen mit Stahl., 2000)
5 (Concrete Institute of Australia. and National Precast Concrete Association Australia.,
   2002)
6 (Bauen mit Stahl., 2005)
8 (Concrete Institute of Australia. and National Precast Concrete Association Australia.,
   2002)
9 Transfloor (Hanson)
Floor Systems Overview and Literature Review

10 http://www.bubbledeckatlantic.com/content/home
11 Hollowcore (Stahltone) 
12 (Concrete Institute of Australia. and National Precast Concrete Association Australia., 2002)

The products chosen companies for the pictures in Figure 2-8 are examples for each category that have been selected by the author. Many competing products from different companies may exist, differing in trivial ways very often. Even for different countries, the trapezoidal composite floors or hollowcore slabs appear very similar, that there is no benefit from listing the companies’ products separately. It is likely that most products are only slightly different because of patent rights.

Category: In-situ Concrete
Strictly speaking, a composite floor is a floor with a minimum of two different materials. However, both in the construction industry and in this thesis, the term is used to define those floor types which consist of profiled steel sheets, reinforcement and in-situ concrete. The illustrations numbered one to four from Figure 2-8 show examples of this flooring type.

The profile height and shape of the steel sheets depends mainly on the area of application. The re-entrant deck profile, which is commonly referred to as dovetail profile, is shown in illustration one from Figure 2-8. This type is a very effective composite section. During bending, the concrete and steel sheet wedge together so that they form a connected composite section. Due to the small steel section height, this profile can only span three to four metres. Therefore, this construction could not be used for the main span of 9 m. However, in combination with a supporting beam the whole system would be able to span 9 m. In this case the beam would span the 9 m and the dovetail profile would span the shorter distance perpendicular to the beam. But as discussed beforehand, this chapter is looking for a new floor solution that can span 9 m itself and is not analysing the standard supporting beam.

Some profiles with a longer span of up to about six metres are shown in illustrations two to four from Figure 2-8. These are normally trapezoidal profiles with a profile height of
300 mm to 400 mm. Because of patent rights, every company produces its own profile with its own unique shape. However, many products from different companies look very similar. Typically, the trapezoidal steel profile is placed on the top flange of the steel beam. Two exceptions are made for the products in illustrations three and four from Figure 2-8. To reduce the construction height, Corus places its product ComFlor 210 on the bottom flange of the supporting steel beam. In Germany, Hoesch hangs its unit down from the top flange of the steel beam with the same effect. Composite floors are analysed in more detail in section 2.3.

Floors with beams and infill include all kinds of beams which span the large distances and infills or other smaller floor slabs between the beams, which fill the gaps. Number five in Figure 2-8 shows a typical example of this category with a span of up to nine metres. If the height of the beam is not too big, the beam and the slab may be considered as a combined floor system. But, if the height of the beam is very large, as for example number six in Figure 2-8, the cellular beam becomes a supporting beam and the actual floor slab is spanning perpendicular to this direction. The inclusion of a flooring system in this category can depend on whether the supporting beams are deemed to be part of the floor system or not. The Speedfloor system shown in Figure 2-8 has a similar construction to the cellular beam and was developed in New Zealand.

**Category: Precast Concrete Slab**

The non-prestressed floors category includes solid and permanent formwork slabs as shown in number eight and nine of Figure 2-8. Both systems do not need a formwork. The solid slab usually only needs a thin concrete topping. The permanent formwork is a similar system to the solid slab, but has a thinner slab thickness and uses much more in-situ concrete on site. In return, the permanent formwork system is cheaper to transport. The picture of the BubbleDeck, number ten of Figure 2-8, shows what different floor systems can look like. It is not widely used, but due to its hollow balls, this system effectively saves weight.

Prestressed floors involve two main floor types: hollowcore slabs and T-beams or double-Tees. Both systems are widely used and normally prestressed. These two flooring systems can span up to 20 metres, along with the cellular beams or other large beams which are
Floor Systems Overview and Literature Review

also able to span this distance. To span distances over 20 metres, a T-beam of around one metre height or more is required. These huge precast elements are used frequently in bridge design. This is outside the scope of the floor product being developed in this project. Precast floors are discussed in more detail in section 2.4.

**Individual floor systems not mentioned in the diagram**

Figure 2-8 sorts the most common floor types on the market into categories. However, not every floor type fits into these categories. Due to this research being mainly focused on steel and concrete products, wooden floors have been excluded. Furthermore, research projects are often a combination of several floor types which have to be analysed individually. One example of such a floor type is the work from Bailey et al. (2006), which uses a trapezoidal steel profile in combination with a prestressed rebar and hollow sections. Another example is the floor concept by Hillman and Murray (1994), which is a combination of two steel decks, as shown in Figure 2-9.

![Figure 2-9: Floor system by Hillman and Murray (1994)](image)

**2.3 COMPOSITE FLOORS**

Metal decks first emerged in the 1950s in America and quickly spread around the world in the 1960s (Nethercot, 2003). It took about two decades before the floor was more
frequently used in construction. In order to optimise the composite connection between the steel sheet and the concrete, the steel profiles have been continuously improved.

There are two profiles which are used widely: the dovetail- and the trapezoidal-formed profiles. The dovetail profiles (Figure 2-10) have a relatively constant height of about 50 mm. The common range of heights for trapezoidal profiles is approximately 50 mm to 240 mm. Figure 2-11 shows the smallest and largest profile available from Corus. The largest profile, the “ComFlor 225”, can span up to 6 metres under typical building load conditions.

Figure 2-10: Dovetail profile ComFlor 51 (Corus, 2008)

Figure 2-11: Trapezoidal profile ComFlor 46 and ComFlor 225 (Corus, 2008)

For most applications, cold-rolled steel sheets are used with a thickness of 0.75 mm to 1.5 mm and with a yield strength from 350 N/mm² and 550 N/mm². The profile can be
formed from galvanised sheet or, if the environment is more aggressive, from precoated sheet.

The steel deck of a composite floor is the building formwork for the wet concrete during the construction process. The steel deck carries the load of the wet concrete to the supports. For longer spans a certain amount of effective floor thickness is necessary. At the same time, the dead load of the floor has to be kept down as much as possible. A floor element with a trapezoidal profile combines these two requirements, similar to a double-Tee floor. The profile is mainly designed for the construction process to resist buckling during the period when the concrete is wet. In order to increase the load-carrying capacity and to improve the connection between the steel sheet and the concrete ribs, embossments are placed into the steel profile. The amount of steel for the bending-moment resistance can be increased by adding rebars at the bottom of the profile.

After the concrete has cured, the steel sheet works together with the concrete as a composite section. In a composite section with only one neutral axis, the effective height of the total floor can be taken into account. However, this is only the case if a full horizontal shear connection between the concrete and the steel sheet is achieved. If the concrete is not connected to the sheet, the two materials work as two separate elements. Each of them has its own neutral axis and its own effective height. This results in a much smaller total bending resistance. Therefore, it is very important for the concrete to work together with the steel deck as a composite unit. In practice, the degree of composite action is usually less than 100%, meaning some slip occurs between the steel sheet and the concrete.

The different behaviour for composite beams, with or without slip, are analysed in a numerical simulation conducted by Poh and Attard (1992). In this paper, the investigated composite floor shown in Figure 2-12a is simplified to obtain the model shown in Figure 2-12c. The concrete in tension is neglected, and the concrete-steel shear interface is simplified to a two-dimensional model. The performance of the different composite beams is summarized in one diagram and is shown in Figure 2-13. This diagram shows the load-deflection curve for complete interaction, which simulates the floor behaviour without slip. The other graphs with a mechanical interlock strength $\tau_m$ of 0 MPa to 0.55 MPa represent floor systems with different amounts of slip. The graph at the very bottom with
\( \tau_m = 0 \text{ MPa} \) simulates a composite floor with no interaction at all. In this case, the steel sheet has to carry the load alone.

![Figure 2-12: Simplified composite floor model (Poh and Attard, 1992)](image)

![Figure 2-13: Load-deflection curve for different amount of slip (Poh and Attard, 1992)](image)

The composite connection is in general a combination of the following:

- **Chemical bond.** A chemical bond or adhesion bond relies on the adhesion of concrete to the steel sheet. This comes about through a chemical process as the
concrete cures. This bond type is relatively weak and brittle. It works only as long as there is no slip at the steel-concrete interface. Once there is slip, the chemical bond is broken and cannot be re-established. Therefore, this bond type is not very reliable and is normally excluded from calculations.

- **Friction forces.** Friction forces act tangentially to the steel sheet with a value proportional to the pressure between the concrete and the steel sheet. Consequently, the friction forces are greatest at the support, where the high shear forces press the two materials together. One exception is the dovetail profile with its re-entrant form. During bending, the dovetail profiles produce a clamping action, which generates some pressure horizontally in addition to vertically. This horizontal pressure also generates friction forces, which help to keep the system connected. In Eurocode 4 (BS EN 1994-1-1, 2004) this clamping action is called “frictional interlock”.

- **Mechanical interlocking.** Mechanical interlocking is a very important type in this category and is defined by the steel sheet shape. Indentations or embossments which are punched into the steel sheet and linearly distributed over the floor length produce a mechanical interlocking system which is able to prevent slip and to carry horizontal shear forces. Furthermore, the indentations or embossments of the profile stabilise the steel sheet during the construction process as the concrete is poured.

- **End anchorage.** With additional end anchorage methods, the shear connection can be improved further. End anchorage devices are placed at each end of the composite floor. Normally, shear studs or rib deformations are used for anchorage.

Figure 2-14 shows some typical forms of interlock systems mentioned in Eurocode 4 with the following allocated numbers.

1. Mechanical interlock, indentations or embossments
2. Frictional interlock, re-entrant form (dovetail)
3. End anchorage by shear studs
4. End anchorage by deformation of the ribs
To design a simply supported composite floor the following three main failure modes have to be analysed:

- **Flexure failure.** Flexure failure occurs when the ultimate resisting bending-moment of the composite cross-section is reached, which usually is at midspan. The bending resistance moment should be determined by the plastic theory (BS EN 1994-1-1, 2004). This implies that the cross-section works as a composite construction with no slip at the interface between the concrete and the steel sheet. On the other hand, if there is slippage, the slab cannot carry the theoretical bending-moment anymore, resulting in horizontal shear failure. Therefore, the theoretical bending-moment can be seen as the limiting value for the best case scenario of no slip.

- **Horizontal shear failure.** Horizontal shear failure is the most common failure mode. For this reason, it is also the most important mode. This failure mode occurs at the interface between the concrete and the steel sheet when the connection between these materials is not strong enough. In order to create a composite section and reach the theoretical bending-moment of the composite floor, the floor slab must avoid horizontal shear failure. Otherwise, with no connection between the materials, the maximum bending-moment cannot exceed the sum of the individual bending-moments from the steel sheet and the concrete, which is a lot less than the maximum bending-moment for the composite section.
Vertical shear failure. Vertical shear failure occurs mainly near the support or at a location with high, concentrated loads and has to be considered in the calculation.

While the flexural and the vertical shear failure are easy to calculate, the horizontal shear failure is not. Therefore, current design methods rely heavily upon full-scale test data. The usual design method is the m-k method, which is the most common testing method used in national standards around the world, for example in Eurocode 4 (BS EN 1994-1-1, 2004).

The m-k method is also known as the shear bond method and is a semi-empirical method that strongly relies on experimental test data. The test is defined as a full-scale composite floor test with two applied line loads at ¼ L and ¾ L, with L being the length of the floor element (Figure 2-15). The results of the test are displayed in a diagram, where the parameter m is the slope and k is the point of intersection with the y-axis (Figure 2-16). The design shear resistance can be calculated by combining these parameters with some other information about floor geometry. The result of this should be higher than the actual maximum design vertical shear force. Six floor tests have to be established for each floor type. Three tests with a preferred long shear span (area A in Figure 2-16) and three with a short shear span (area B in Figure 2-16).

As this method is not based on a theoretical model, new floor tests have to be arranged for each floor assembly with different floor span or depth. This is the biggest disadvantage for this model.
For a long time the m-k method was the only way to analyse a composite deck. The Commentary to the current New Zealand Steel Structures Standard (NZS 3404, 1997)
refers in some parts to the British Standard (BS 5950-4, 1994): “Until a design procedure which utilizes the partial shear connection method of design becomes available for trapezoidal decking profiles, the recommended limit state Standard for design of composite slabs is BS 5950: Part 4” (NZS 3404, 1997, section C13.2.1).

The partial connection method is already used as a standard method in the European standard Eurocode 4 (BS EN 1994-1-1, 2004) as an additional possibility to the m-k method. In Europe, the Eurocodes have replaced all relevant national standards. Useful background information to Eurocode 4 can be found in the Designers’ Guide (Johnson and Anderson, 2004). Bode and Sauerborn explained the current design procedure already in 1992 (Bode and Sauerborn, 1992). During this research, more than 80 full-scale tests were carried out at the University of Kaiserslautern in Germany.

The partial connection method is a new method for the design of composite floors. It is also known as the partial shear connection method or $\tau$-method. This method is based more on a theoretical model than the m-k method and gives a better understanding of the failure mode. This method can reduce the number of full scale tests, which reduces the costs and time required to develop composite floors.

For the partial connection method, a diagram is used to represent different connection situations, from 0% (only the steel caries the load) to 100% connection (full composite action) as shown in Figure 2-17. The same full-scale tests as used in the m-k method are carried out for the partial connection method. The maximum bending-moment of the experimental test is used in Figure 2-17 to measure the degree of shear connection in the test ($\eta_{tot}$). With this value, a new diagram can be drawn showing the design bending resistance at any part of the floor length. Finally, the design bending-moment is also included in the same diagram and should not exceed the design bending resistance at any location.

If only one new floor type is investigated, the partial connection method requires the same number of full-scale tests as the m-k method. When testing multiple similar floor types, the partial connection method requires significantly less tests than the m-k method, because the partial connection method allows modelling of parameter changes using theory. This
means that there are fewer experimental tests when parameters such as the floor span, depth, steel or concrete strength or end anchorage system change.

Figure 2-17: Partial connection method (BS EN 1994-1-1, 2004)

Figure 2-18: Partial connection method with end anchorage (Sauerborn, 1995)
Many researchers are working on the horizontal shear failure mode, because it is the most frequent failure mode. The Virginia Polytechnic Institute and State University has undertaken a significant amount of research on the behaviour of composite slabs in recent years (Abdullah, 2004, Shen, 2001, Traver, 2002, Widjaja, 1997). The behaviour of continuous slabs is analysed by Sauerborn (1995).

One downside of both the m-k method and the partial connection method still exists: the mechanical interlocking and friction behaviour cannot be separately modelled. This issue is of particular importance at the support. There, the vertical shear force is generally higher than at midspan. Models which allow separated consideration of the two connection mechanisms would help to model the failure behaviour more accurately (Bode and Minas, 2000).

Several researchers have worked on improvements of the partial connection method or on a better understanding of the horizontal shear bond in general. Abdullah (2004), Minas (1999), Patrick (1994) and Veljkovic (1996) have developed or analysed in their doctoral theses small-scale tests for evaluating the performance and behaviour of composite slabs. Figure 2-20 shows an example of a slip-block test from Patrick (1994). Other researchers have developed similar push-out or pull-out tests, which are investigated in (Abdullah, 2004). Annex B of Eurocode 4 (BS EN 1994-1-1, 2004) contains a testing arrangement for a standard push test for shear connectors.
Several other papers about the horizontal shear behaviour in composite floors have been published (e.g., Burnet and Oehlers, 2001, Chaklos et al., 2004, Crisinel and Marimom, 2004, Mäkeläinen and Sun, 1999, Tenhovuori and Leskelä, 1998, Thomann and Lebet, 2007). The behaviour is also analysed with finite-element programs in Abdullah and Easterling (2009) and Ferrer et al. (2005).

2.4 PRECAST FLOORS

Precast elements have been used in the construction industry since the 1950s, with ongoing refinement of the floor elements and production process. Today, two main floor types are available which are optimised to span long distances of up to 20 metres. These are the hollowcore slabs and the double-Tees. Both systems need in-situ concrete as topping.

Hollowcore units are prestressed elements which are normally produced on a long-line casting bed of 120 to 170 metres. The elements are saw-cut after being cured to the required length. The common width for these floor units is 1.2 metres or, more rarely, 2.4 metres. Figure 2-21 shows the dimensions of hollowcore units from Stahlton
Engineered Concrete. Stahlton is the largest supplier of precast and prestressed products in New Zealand, and is a division of Fulton Hogan. The picture shows the smallest product, which has a thickness of 150 mm and the largest with 400 mm. Elements with 200 mm and 300 mm thickness are also available. These dimensions are standard types worldwide. The style of holes and details may differ slightly from company to company.

Figure 2-21: Hollowcore units from Stahlton

The collapse of hollowcore floor units in 2001 at the University of Canterbury during an investigation by Matthews has raised serious issues regarding the performance of hollowcore units. The tests were conducted to determine the integrity of hollowcore floors in New Zealand, after several buildings collapsed as a direct result of the failure of hollowcore floor units during the Northridge earthquake in California in 1994 (Matthews et al., 2003).

Matthews’ investigation of a full-scale floor assembly indicated potentially serious gaps between assumed behaviour in the hollowcore calculation and the actual behaviour. In July 2003, the Building Industry Authority, now the Building Controls Group of the Department of Building and Housing, commenced a review of the use of hollow core floor systems in New Zealand (Stannard et al., 2007).
2.4 Precast Floors

Further research was undertaken at the University of Canterbury by Lindsay, Liew and Jensen (Jensen et al., 2006). The failure mechanisms observed in experimental tests are shown in Figure 2-22 (Jensen et al., 2007). The failure modes in this paper are called Loss of Seating (LOS) or Loss of Seating with Delamination (LOSD). Figure 2-23 shows similar failure modes investigated by Matthews and Liew (Jensen et al., 2007).

![Figure 2-22: LOSD and LOS failure modes of hollowcore slabs (Jensen et al., 2007)](image1)

![Figure 2-23: FS and OFS failure modes of hollowcore slabs (Jensen et al., 2007)](image2)

The Department of Building and Housing (2007) provided a summary of the ongoing research work in New Zealand. One of the recommendations given in their report was that owners with concerns about hollowcore floor units should have their buildings reviewed by a structural engineer. The Department supports the document *Seismic Performance of Hollow Core Floor Systems* produced by the Structural Engineering Society of New
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Zealand, the New Zealand Society for Earthquake Engineering and the New Zealand Concrete Society (2009). This document does not resolve all uncertainties with hollowcore floors. However, it does give guidance information and extracts from several research papers presented beforehand. At time of submission of this thesis, the document was only available as a preliminary draft (Department of Building and Housing, version April 2009, http://www.dbh.govt.nz/consulting-on-hollow-core-floor-systems).

Double-Tees are also prestressed units. The thicknesses normally vary between 200 mm and 550 mm at 50-mm intervals and have a constant width of 1.2 metres. Figure 2-24 shows the smallest and largest double-Tee which is available from Stahlton Engineered Concrete.

![Figure 2-24: Double-Tees from Stahlton](image)

Recently, the support detail of double-Tee floors was investigated in an article of the SESOC Journal in April 2009 (Hare et al., 2009). Different systems had been reviewed after questions were raised about the integrity of the support details of double-Tee floors. A major concern was addressed by the authors regarding the use of loop bars or pigtail hangers, which are commonly used in New Zealand.

The support systems can generally be classified into three groups:
2.4 Precast Floors

1. **Web supported.** The floor is directly supported by the base of the web. This is the easiest way to support the floor but has the big disadvantage of a high construction height.

2. **Dapped ends.** A part of the web is cut out as shown in Figure 2-25.

3. **Flange supported.** With a special hanger system, the floor is supported at the flange. This is the system with the lowest possible construction height.

The investigated flange support systems are shown in Figure 2-26 to Figure 2-28.

![Figure 2-25: Dapped end (Hare et al., 2009)](image)

![Figure 2-26: Pigtail hanger (Hare et al., 2009)](image)
The authors of the article came to the conclusion that only the Cazaly hanger shown in Figure 2-27 is considered fully suitable for use in all load situations. Both of the other flange supported hanger systems, the pigtail and the Loov hanger, are not recommended. Specific concern is raised about use of the pigtail hanger. This hanger system was developed in the early 70s and the original design methodology cannot be validated in terms of current code practices. Conventional analysis does not support the claimed load capacity of the pigtail hanger system and the experimental tests conducted have not covered all the loading situations which may be expected in practice (Hare et al., 2009). The deficiency of the support system for flange supported double-Tee units was a significant factor behind the support detail proposed for the new floor product, developed in the next chapter.
3 CONCEPT DEVELOPMENT

3.1 INTRODUCTION

The development process for this new floor system, termed F1, is explained in section 3.2. During this process, four criteria, which are stated in the next section, had to be achieved. Two criteria were to use a steel sheet as part of the structural system and to be able to span a distance of nine metres unpropped. The use of steel sheet was a requirement of the CSA project, and the span of nine metres was recommended by the industry partners as the most common span for precast concrete floor units. The starting floor system in the development process was a composite floor. However, in the end, the F1 unit externally resembled a double-Tee floor.

Section 3.3 introduces the floor concept behind F1. All important parts of the floor system are explained and visualised in drawings. In this section, the dimensions of the floor system are given and the materials used are defined.

Section 3.4 lists some unique advantages of F1.

3.2 DEVELOPMENT OF F1

At the beginning of this research project, the floor unit which is now called F1 and which is explained in detail in the next section did not exist. The challenge was to develop a new floor element which could meet or exceed the performance of traditional systems and which could fulfil the following four criteria:

1. The floor system must use light-gauge steel sheet as part of its structural system.
2. The floor system should be able to span at least nine metres unpropped.
3. The floor element should be produced as a precast element.
4. The system should not require any additional concrete topping on site.
As explained in chapter 2, composite floors are made up of profiled steel sheets and concrete which is poured on site. Due to the fact that the profiled steel sheet has to carry the load of the fresh concrete, the unpropped spans for these systems are limited to about 6 m. In order to span longer distances economically, the new unit should be fabricated as a precast element. This creates the opportunity of producing more complex and more efficient systems than would ever be possible on site.

The starting cross-section was a common trapezoidal composite floor section, as can be seen in Figure 3-1. A high bending-moment resistance together with a high flexural stiffness is generally most important for long floor systems. In order to build a formwork for the concrete, composite floors need an uninterrupted steel deck. That means that the steel deck must underlie the total floor area, without interruptions or holes, in order to contain the wet concrete. After the concrete has cured, some parts of the steel deck are more useful for the composite floor than others. Precast elements have the advantage that the cross-section can be optimised and that steel parts only have to be placed where they are necessary and most efficient, as the steel parts do not have to work as a formwork. Therefore, holes or gaps in the steel sheet are possible, allowing steel sheet to be placed only where needed for structural performance.

In terms of high contribution to flexural strength and stiffness, the steel at the bottom of the trapezoidal profile is most effective. The horizontal steel at the top of a trapezoidal steel sheet is closer to the neutral axis of the composite section and so the contribution to the bending-moment resistance is smaller. Therefore, it is more effective for the development of F1 to take the top horizontal steel parts (Figure 3-1) out and to add them to the more effective bottom area. At this stage the cross-section is only analysed generally for the most effective shape. It has to be clarified later if the area at the bottom is large enough to contain the amount of steel sheet or additional reinforcement that is necessary for the moment capacity.
The sloping steel parts at the side of the web increase the shear resistance. But in the new floor unit, it is not necessary to have the sheet located at the outside. The external location increases the demands on the steel sheet, in order to provide corrosion resistance, and makes it more difficult to develop a good concrete-steel connection. External sheeting is also more vulnerable to fire. Furthermore, it is more economical for the shear resistance to place the sheet over the total height of the cross-section.

With this in mind, the first sketches for an optimised floor system were developed and can be seen in Figure 3-2 and Figure 3-3. The steel sheet is constructed out of two pieces, as is shown in each drawing.

- The bottom steel sheet is important for the bending-moment resistance and looks similar to the bottom steel part in a trapezoidal profile.
- The other part is important for the shear resistance. This part can be created in various ways as long as it runs over the entire height. Two examples are shown in Figure 3-2 and Figure 3-3.
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Having the two steel parts separated has the advantage that the sheets can have different thicknesses. Therefore, the cross-section can be optimised for the bending-moment and the shear resistance individually.

Figure 3-2: Cross-section, type one

Figure 3-3: Cross-section, type two
After analysing the cross-section in more detail, it was observed that the steel sheet at the bottom needed either to have a thickness of several millimetres or to be placed in multiple layers. Considering only the bending-moment resistance, it turns out that rebars can satisfy the requirements in a more economical way than the steel sheet at the bottom could do. Therefore, rebars were chosen to carry the majority of the tension force at the bottom in the floor element F1.

Although cost is a very important parameter, it was not initially given close consideration, because factors such as the cost benefit resulting from not needing to pour an in-situ topping and being able to install floor units that have full flexural and shear strength immediately. The focus was more on performance in flexure and shear (strength and stiffness), constructability, robustness of support and potential fire resistance. After a new floor system is found and the system has proven its workability it can still be checked for cost improvements. In this thesis the structural workability is analysed but the product development process is not finished. Before the floor can be produced as a prototype further requirements will have to be analysed and experimentally tested, as listed in chapter 7 for further work. Therefore it is too early to say if the new floor element will be cost-competitive as this depends on a number of details. However, as the floor system is very similar to existing double-Tee floors the price range should be about in the same area.

The embedded top steel sheet was not replaced by rebars as it has the following advantages over the stirrups, which are used in double-Tees and which were intended to be replaced by this sheet.

1. The sheet is able to carry forces at any location and in any direction in its plane. On the other hand, stirrups can carry loads in a vertical direction only.

2. The strut-and-tie model is only an auxiliary calculation model which helps to understand the load flow and to design the floor. The steel sheet can carry forces in its plane and will develop both direct and shear stress fields to resist the varying proportions of shear force and bending-moment along the length of the floor unit.

3. Continuously distributed forces, as in the steel sheet, are generally better for the concrete cracking behaviour than forces concentrated only at the places where the stirrups are placed.
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4. Usually, closely spaced stirrups are needed close to the support. This can lead to compaction problems as the concrete cannot find its way through the rebars. By contrast, the thickness of a steel sheet can easily be increased to reach a higher shear resistance without any compaction problems.

5. The support is a critical location, where, especially with dapped ends or hanger systems, the force distribution is very complex. Cracks can occur here, if the rebars are not located in the correct direction. By using the steel sheet, which is automatically located in the correct direction and can be readily fixed into the supporting hanger, a strong and stable load path between the unit and the support hanger is achieved, as verified by testing (section 4.5).

6. The main role of the steel sheet is to carry the shear load. However, unlike stirrups, the steel sheet also increases the bending-moment resistance.

Because of these advantages, the top steel sheet is also used in the F1 specimen. Instead of the draft shapes illustrated in Figure 3-2 or Figure 3-3, a plane steel sheet in the middle of the web was chosen for efficiency reasons. In order to enhance the connection from the steel sheet to the concrete, the sheet is perforated with holes as explained in section 3.3.

An alternative cross-section at the support and at midspan was discussed, but not realised. If the cross-section changes over the length as shown in Figure 3-4, it is possible to optimise the floor element for the different requirements at midspan and the support. This is most helpful for uniformly distributed loads, but not for concentrated loads, which may produce high shear forces at locations away from the ends.

A similar effect can be achieved if the sheet height stays constant, but the sheet thickness varies. Steel sheets with variable thicknesses are not commercially available at the moment, as the requirement of complex forming machines makes this option infeasible.
The structural depth or height of the floor and the floor stiffness strongly influence the spanning ability of the floor. In turn the stiffness or the moment of inertia for the composite section is significantly affected by the amount of reinforcing placed in the lower web. This is shown later in section 5.4.1 where the moment of inertia is calculated for two sections, one with two rebars and the other one with four rebars (Figure 5-18 to Figure 5-21). Therefore, to some extent an appropriate height of the floor can be chosen and remain constant while the amount of reinforcing steel is varied to achieve desired stiffness properties.

In Figure 2-1 to Figure 2-7 different floor solutions were shown with their span range and their given height. Figure 2-5 defines a span range for a 460-mm high double-Tee element between 8 and 14 metres. This is based on a prestressed double-Tee. The range for a non-prestressed double-Tee has to be lower. This research project is looking for a non-prestressed solution for a target floor span of 9 m. As the cross section of the new developed floor element looks similar to a double-Tee section (see section 3.3.1) but is not prestressed, the lower limit of the span range is chosen for the floor height to start with. This is around 500 mm for a floor span of 9 m. Prestressed floors are able to span larger distances than concrete floors with conventional rebars, however, prestressed floors require a relatively good concrete quality to accommodate the internal forces from the prestressing tendons. As the concrete strength for light weight concrete is significantly smaller than the
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strength of normal weight concrete and the priority in the CSA-project was to use a light weight concrete, the option of prestressing was not considered in this thesis.

Figure 2-24 shows a web width of 197 mm for a double Tee floor. The width of the web is not too important for the bending stiffness. The width is normally chosen to accommodate the necessary tension and shear reinforcement, maintaining sufficient concrete cover. As no stirrups are used in the new floor system the web width may be reduced to 100 mm. This gives enough space to include the necessary tension reinforcement of four or two rebars with a diameter of 20 mm. However, the 100 mm web thickness is sufficient for structural requirements only, fire resistance has to be checked separately, depending on the desired fire resistance rating. According to (NZS 3101, 2006, Table 4.1), the floor is sufficient for a fire resistance rating of 30 minutes, which requires a minimum width of the web of 80 mm and a minimum axis distance of the longitudinal reinforcement of 25 mm. For a fire resistance rating of 60 minutes, the web thickness has to be increased to 120 mm.

According to (NZS 3101, 2006, Table 4.3 and 4.4), the slab thickness of 100 mm as used in the third test with its reinforcement axis distance of 30 mm would reach a fire resistance rating of 90 minutes. However, these fire ratings are based on normal weight concrete, for the used light weight concrete, further analysis or fire tests might be necessary.

The rebar amount was chosen to resist a live load of 5 kN/m over a span of 9 m. In section 4.3.4.1 it is shown in a simple hand calculation, without considering the positive influence of the steel sheet that 4 rebars are sufficient. As the first test showed a rebar anchorage problem, the number of rebars for subsequent tests was reduced to two rebars to analyse and solve this anchorage problem first. However, in section 4.3.1 it is shown that the result from the second experimental test, which used only two rebars of 20 mm, is also strong enough to resist a live load of 5 kN/m.

For the steel sheet thickness in this project a standard thickness from New Zealand Steel of 1.6 mm was chosen for all experimental tests. Section 5.2.3 shows the theoretical calculated shear strength reached with this thickness.
The holes or “windows” in the steel sheet were made to increase the bonding between the steel sheet and surrounding concrete. Their shape, size and distribution were chosen as starting parameters to be able to conduct first experimental results. It was intended to optimise these parameters by further test if necessary. However, the steel sheet openings did not seem to be the critical part after the first test as the debonding of the rebars was the main concern, and the holes’ geometry was not changed. The holes’ geometry in the steel sheet had to be cut by hand, which dictated their shape. However, in a floor prototype for commercial production, the holes would be cut or punched by machines such that their final look could be very different, as long as the composite action is guaranteed and that no concrete parts debond from the steel sheet until development of the desired strength.

Another criterion was that the floor units be no heavier than existing precast double-tee units. This required the use of light-weight concrete, as these units include the full depth of slab, whereas double-tees have a topping placed after erection. Depending on the definition used, light weight concrete density can range between 800 and 2000 kg/m$^3$. The CSA-project did not specify a particular target density, however, the intention was to start with a density slightly lower than normal weight concrete and to reduce the density from test to test if the structural behaviour is sufficient. Densities under 1500 kg/ m$^3$ are not considered in this project as the concrete strength of these densities are much lower than normal weight concrete and the risk of vibration problems of the floor is significantly higher.

The floor should also not be much heavier than comparable other floor types. The intention for the new floor element is to use light weight concrete. As most other floor types use normal weight concrete this criteria is fulfilled. The use of light weight concrete was also chosen for its enhanced thermal and acoustic properties.

The common width of a double-Tee floor is 2.4 m as shown in Figure 2-6. The new floor element should not be much heavier than existing double-Tee floors. However, due to the use of light weight concrete the width could be increased in the new system. This has the advantage of less joints in the overall construction and this could save money and time in construction. Therefore, the width was chosen to be 3.0 m, which is the maximum width to handle easily with a truck for transportation.
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The transverse slab dimension between the ribs and the overhang was chosen to approximately equalise the bending moments at midspan and over the support. Therefore, the bending moment at section 1 should be about the same as at section 2 (Figure 3-5). This is fulfilled for the given dimensions as proven in Eq.(3.1) and Eq.(3.2).

Assuming a transverse line load of \( q = 1 \text{kN/m} \) the bending moments are:

\[
M_1 = -\frac{q(0.55 \text{ m})^2}{2} = 0.151 \text{kNm} \quad (3.1)
\]

\[
M_2 = -\frac{q(1.5 \text{ m})^2}{2} + q \times 1.5 \text{ m} \times 0.85 \text{ m} = 0.150 \text{kNm} \quad (3.2)
\]

Figure 3-5: Bending moment at midspan and support
The thickness of 150 mm for the slab was mainly chosen by acoustic requirements rather than by structural requirements. In section 5.2.5 it is shown that the necessary rebar amount for the slab is not critical. The ‘Airborne and Impact Sound’ document prepared by the Department of Building and Housing (2006) suggests for concrete floors a minimum thickness of 150 mm. Therefore, a slab thickness of 150 mm was also used as a starting value in this research. However, the acoustic requirements are dependent on the final use of the floor element. Car parks, for example, have less strict requirements for acoustic behaviour than office buildings. Therefore, the third floor test was conducted with a slab thickness of 100 mm only, even if this might not be suitable for office buildings without any detailed acoustic analysis. However, the proof of the acoustic behaviour was not part of this thesis. Fire resistance would also be affected.

With all the preceding considerations in mind the idea for a new floor system was arrived at and is explained in detail in section 3.3. As the idea of using a steel web sheet inside a double-Tee floor had never been tested or developed before, it was not clear at this stage if the idea had any potential or if the concept was workable.

### 3.3 FLOOR CONCEPT F1

#### 3.3.1 Concept Overview

The integral part of the floor development program comprised three full scale experimental floor tests. Some parameters and dimensions were slightly different between the tests and are explained in section 4.3. All the drawings in this section are based on the dimensions of the third floor test.

F1 was conceived as a precast element in the shape of a double-Tee floor. From the outside, the element looks very similar to existing double-Tee floors with a hanger system. Figure 3-6 shows the entire floor element, which was developed and investigated in this thesis. The originality of this work is hidden in the inside of the floor element as shown in Figure 3-7. This drawing shows a floor element with a totally new load-carrying system.
Figure 3-6: Double-Tee floor
Apart from the concrete, the floor element is made up of the following components, which are highlighted in Figure 3-7.

- 1.6-mm steel sheet with several openings (yellow)
- Two rebars with a diameter of 20 mm at the bottom of each web (red)
- A 700-mm-long square hollow section (SHS) at each end of each web (blue)
- A 6-mm-thick steel plate (green)

In order to connect the steel parts, welding lines are placed at all intersections. These are in detail: the connection between the steel sheet and the SHS and the connection between the steel sheet and the 6-mm steel plate. Furthermore, the 6-mm steel plate is welded to the SHS and to the rebars.

The F1 unit is a three-metre-wide and nine-metre-long specimen with a slab thickness of 100 mm (150 mm for the first two floor tests). The concrete webs are 100 mm wide and
Concept Development

500 mm high in total. The exact dimensions of the floor element are shown in the top, side and section views in Figure 3-8 and Figure 3-9.

![Diagram of top and side views](image)

Figure 3-8: Top and side view
Figure 3-9: Section A-A to section C-C

Figure 3-10 shows the dimensions of the 6-mm steel plate. The plate is needed to transfer the concentrated support load into the steel sheet. Together with the direct connection between the SHS and the steel sheet, this system establishes a very solid support connection.
Concept Development

The rebar is placed through the holes and project beyond the plate by about five millimetres. This makes it easier to weld the rebar to the 6-mm steel plate, and to establish a solid connection. Instead, threaded bars with a nut could also be a solution for the final product.

![Steel Plate Diagram](image)

**Figure 3-10: 6-mm steel plate**

In Figure 3-11 the details of the SHS are shown. The 500-mm cut in the SHS is necessary to connect the steel sheet symmetrically with the SHS in the middle. After the sheet slid into this gap, both parts were welded together. The SHS was designed to provide a convenient and robust top-hanging support. It is planned to combine the SHS with a levelling screw, which would allow the precise alignment of adjacent floor units, ensuring that slab edges were positioned at the same level. This is an important consideration, bearing in mind that no topping concrete was to be used. A levelling screw detail was developed and determined to be practical to make and use on site, however, testing the constructability of this feature would be dependent on the final prototype production and
so was not part of this project. Testing the load carrying performance of the RHS, steel hanger plate and sheet was part of this research (see section 4.5).

For the proper functioning of the steel sheet, a good connection to the concrete is essential. This connection is enhanced by the rectangular holes ("windows") in the sheet. These windows are distributed all over the sheet and placed in horizontal and vertical alignment in order to reach an equally strong connection. The steel sheet and the locations of its cuts are shown in Figure 3-12 and Figure 3-13. The basis for the geometry of the proposed cutting pattern was to start with a pattern which could easily be established with a hand tool. It is very unlikely that a hand tool would be used in the final production process. Instead, machines would be used to perform the cutting or punching process that the cutting pattern may also be changed in the future.

Figure 3-11: Square hollow section (SHS)
Steel sheet:
thickness = 1.6 mm
Grade NZCC4D

The dimensions refer to the midpoint of the "I" cutting element.

Figure 3-12: Steel sheet

Figure 3-13: Steel sheet with cuts
After the cuts in the steel sheet are placed, the rectangular sections between the two parallel cuts are bent by 90 degrees: one half to the left side of the steel sheet and one half to the right. After this procedure, the openings look similar to a window, with one window frame opened to the inside and one opened to the outside. This can be seen in detail in Figure 3-14 and Figure 3-15, which are 3D-models of the steel parts, drawn and rendered with AutoCAD.

Figure 3-14: Steel sheet openings
3.3.2 Materials

The concrete material varies during the three experimental tests from an average saturated density, which is measured accordingly to (NZS 3112, 1986), of 2135 kg/m³ for the first floor test to 1655 kg/m³ for the third floor test. Details of the concrete density and strength are given in section 4.2.1 for each floor test.

In order to make the light-weight concrete, a foam generator from BTL Lightweight Concrete Solutions was used. The generator produces the foam out of compressed air, water and the synthetic foaming agent. From a hose, the produced foam is added to the sand-water-cement mixture in the drum of the concrete truck immediately before the pour. Apart from the sand no other aggregates were used. An image of the produced foam is shown in Figure 3-16.
A fibre dosage of approximately 30 kg/m³ of steel fibres was used to improve the after-crack behaviour of the concrete. The fibres were also expected to help keep the concrete interlocked with the steel sheet. The fibre being used is from Bekaert and is called Dramix RC-65/35-BN. The fibres have hooked ends and are glued together in bundles with water soluble glue as seen in Figure 3-17. As soon as the fibres are added to the concrete mix, they separate and distribute in the mix. A Bekaert representative explained the product to the CSA-team. It was he who recommended the dosage level, about 30 kg/m³, when the performance criteria for this floor unit and anticipated mode of behaviour were explained. This is at the bottom range of normally used dosages before appreciable concrete material improvements are generated. The relatively small amount was also chosen to check if the steel fibres will work with the light weight concrete as most empirical research data was based on normal weight concrete only. If the concrete was observed debonding from the steel web sheet during the first experimental test, it was intended to increase the dosage; this was seen not to be necessary.
Four different types of steel components were used in each floor element. These were rebars, steel sheets, square hollow sections and 6-mm-thick steel plates.

The elastic modulus for all steel parts, which were used for analysis in chapter 5 and 6, was taken to be 205000 MPa.

The rebars were of grade 500 with the material properties given from the test certificates in Appendix 1. The first certificate is from the rebars used in the first experimental test. The average values from the test certificate yields the following simplified stress-strain diagram by assuming linear behaviour between the yielding and ultimate stress point.
The second and third experimental test used rebars from the same heat specified with the second test certificate from Appendix 1. With these values the following stress-strain diagram can be drawn.
The values from the test certificates and, therefore, Figure 3-18 and Figure 3-19 are used for the design according to NZS in section 5.2 and for the FEM analysis in chapter 6. However, as shown, the simplified calculation with the Excel sheets in section 5.6 assumes a constant stress value after yielding. The yielding stress in this section was chosen to be 575 MPa, which is 15% over the base value of 500 MPa for the grade 500 rebar. With this simplification, the calculated yielding point from the Excel sheets was not as accurate as the results from the FEM analysis. However, the intention of the Excel sheets was mainly to help the understanding of the structural behaviour of the floor system; more accurate results can be reached with the FEM analysis in chapter 6.

The steel sheet material is a 1.6-mm-thick cold-rolled NZCC-4D plain ungalvanised steel sheet from New Zealand Steel. For all three experimental floor tests, the material was used from the same steel coil (Figure 3-20). For analysis, the material characteristics shown in Figure 3-21 were used. The steel sheet itself was tested in section 4.2.2 to validate the strength specification.
The 6-mm steel plate is cut out of a rolled steel plate with the grade HA 300 (according AS 1594). The SHS is 6.0 mm thick and consist of the material dualgrade C350/C450 (according AS 1163). Both steel parts are manufactured by New Zealand Steel. The stress-strain diagrams which were used in this thesis for the 6-mm steel plate and the SHS are shown in Figure 3-22 and Figure 3-23.
The maximum values for the material diagrams are used from manufacturer information or from the corresponding Australian/ New Zealand Standards. The two-line material diagrams used are a simplification of the material’s real behaviour, however this simplification allows an easy implementation in the theoretical and finite-element model and differs only slightly from the exact stress-strain curves.
3.4 ADVANTAGES OF F1

Not every floor system is able to span nine metres or more. The two most common floor types with this span width are hollowcore slabs and double-Tee floors. However, the F1 system has still some unique advantages:

- **Weight**: The F1 element uses lightweight concrete. Therefore, it is lighter than conventional floor systems, so saving transportation costs. Also, the total weight of the building will be lighter. This has a positive impact on the design of the supporting system, such as beams, columns, walls and foundations. Additionally, the foam in the concrete results in better thermal and acoustic behaviour compared to conventional concrete products. However, it has to be noted that using lightweight concrete will make vibrations occasioned by walking activities more perceptible compared to cases when normal weight concrete is used.

- **High shear capacity**: Due to the fact that hollowcore slabs do not use stirrups, they are limited to the shear capacity of the concrete. The shear capacity of concrete is hard to control, because it decreases rapidly after the concrete starts cracking. At failure the mode is brittle. For these reasons hollowcore slabs are not recommended for high shear forces: Hollowcore slabs “… are not recommended for highway loadings, in truck docks or similar areas with high shear loads.” (Stresscrete, 2007). However, the F1 unit is designed for high shear forces. The thickness of the steel sheet can easily be increased to support higher shear loads. The failure mode is not brittle.

- **Ability to carry high local shear forces without compromising bending capacity**: In reinforced concrete beams or in double-Tee floors, the load flow is constrained by the way that the shear forces can only be carried at the places where the stirrups or shear reinforcement are located. In a steel sheet, the load path is not constrained. The steel sheet will resist changing bending and shear forces in an optimal manner.

- **Better cracking behaviour in shear**: In reinforced concrete beams or floors with stirrups, the internal forces are concentrated at the stirrups. In terms of the cracking behaviour, it is generally better to carry the forces by stirrups with a small diameter and small intervals as opposed to stirrups with a higher diameter and larger intervals. Therefore, a steel sheet has a very good cracking behaviour, because it
distributes the forces evenly over the floor length, similar to very small stirrups with an infinitely small spacing interval, assuming no slip behaviour between the steel sheet and the concrete.

- **Additional bending-moment capacity**: Normally, the tasks are separated: The rebar cross-section at the bottom of the beam controls the bending-moment and the vertical stirrups are responsible for the shear force. Increasing the number of stirrups does not increase the bending-moment capacity. However, with the F1 element, the steel sheet is able to both carry shear forces and to increase the bending-moment.

- **Robust and direct load path from unit into supporting hanger**: In section 2.4, details of recent problems at the support of hollowcore slabs and at the support of some double-Tee floors were presented. These have raised questions about the integrity of the systems. Both floor systems showed a brittle failure mode in experimental tests. A brittle failure mode for a floor system in a multi-storey building is very critical. If one floor element loses its support and collapses to the floor element one level below, this bottom floor element will most likely also fail, which in turn may lead to a total collapse of the building. The brittle total failure mode, as seen by the hollowcore floors in Figure 2-22 and Figure 2-23, is mostly caused by the fact that no steel crosses the failure plane. The support detail of the F1 unit is very robust. As the steel sheet always crosses the failure plane, the failure mode will never be as brittle as seen by the hollowcore floors. The support detail at double-Tee floors with hanger system is very complex. Figure 3-24 shows a typical end support from Peikko. With its grid of vertical and horizontal rebars, it tries to carry all occurring tension forces. But the force distribution at supports is difficult to predict and may change at different load situations. Cracks often occur close to the supports, because the rebars were placed at wrong positions. This underlines the importance of well-constructed floor ends. Because a steel sheet can carry a load in any direction, the steel sheet in F1 is able to carry this complex load behaviour. For very high shear forces, the thickness of the steel sheet can easily be increased. Double-Tee floors with very high shear forces need a high amount of rebars, which leaves only small gaps for the concrete. This can lead to compaction problems in the concrete. The problem does not exist with a steel sheet.
3.4 Advantages of F1

- **New System:** It needs to be emphasised that this is a totally novel idea. A steel sheet has never been used beforehand in reinforced concrete beams or floors in order to carry shear forces or to improve the behaviour at supports. The floor unit F1 was the outcome of the research undertaken in this thesis. More research is needed to develop the concept to the prototype production stage, however, the author of this thesis believes that it has demonstrated sufficient potential for this.

![Figure 3-24: Corbel support detail (Peikko)](image)

As mentioned in section 3.2, the floor unit F1 is intended to be installed without any additional concrete topping on site. This would save some time and money and would be an advantage over most other floor systems. But it is not listed as one advantage, yet. In order to prove this capability, the connections between the floor elements and a possible levelling system at the supports need further development and testing. This further development does not form part of this thesis.
Concept Development
4 EXPERIMENTAL TESTS

4.1 INTRODUCTION

This chapter describes the experimental testing conducted in the development of the F1 floor. The main objectives of the experiments were to:

- Verify the constructability of the proposed floor concept.
- Demonstrate the viability of the concepts proposed in section 3.3, especially the behaviour of the perforated steel sheet and the end supports.
- Observe failure modes, especially any splitting tendency in the web due to the presence of the steel sheet.
- Capture displacement and strain response data due to the applied vertical loading applied for comparison with theoretical and finite-element analysis models.
- Measure relevant properties of the materials for use in numerical models.
- Measure the natural frequency of the floor for comparison with theoretical calculations.

Three large-scale floor tests (4.5 m long) were carried out to support the development of the F1 floor concept, which has been described in chapter 3. In section 4.2, the component materials used for each of these floor tests are investigated separately. Each time one of the three floor specimens was poured, several concrete test cylinders and beams were taken from the concrete mix for verification. The steel sheet was only tested once, because all sheet material used came from the same steel coil.

Section 4.3 describes the experimental testing and examines the results for the three floor specimens subjected to applied vertical loading. The individual characteristics of each floor element are explained. The results of the tests are critically investigated and compared with each other. Inadequate outcomes are reviewed and improvements made as appropriate for the next test.

After 28 days of concrete curing, and before each floor specimen was tested to ultimate load, a natural frequency test was conducted as described in section 4.4.
Experimental Tests

In order to analyse the support detail of the floor separately, an additional test for the end part was carried out and is described in section 4.5.

4.2 MATERIAL TESTS

4.2.1 Concrete

For each floor test the concrete was tested using the “Concrete Compression Test” and the “Flexural Tensile Strength Test” according to (NZS 3112, 1986). The concrete used was in all three tests a light-weight foam concrete produced with synthetic pre-produced foam. Included in all three tests was a dosage of 30 kg/m^3 Dramix RC-65/35-BN steel fibres from Bekaert (section 3.3.2).

Concrete Compression Test (NZS 3112, 1986)

The actuator used for the compression test was manufactured by Contest Instruments Ltd, type GD10A. A typical concrete cylinder during the test is illustrated in Figure 4-1.

![Figure 4-1: Example of a Concrete Compression Test](image)

At least two cylinders were tested after seven days, to get an indication of the density and the expected compression strength after 28 days. The rest of the cylinders were tested after 28 days. For the second floor test, it was not possible to test the cylinders directly on day
28, instead they were tested on day 32. The calculated strength should therefore have been slightly greater than the strength at 28 days; however, the difference would be minimal and is not considered significant for the purpose of our experiment.

Table 4-1 to Table 4-3 present the density and compression strength of all cylinders for each floor test. For a better and quicker comparison, the average values after 28 days are calculated and printed in blue in the last row of each table.

Table 4-1: Floor test 1, density and concrete compression strength

<table>
<thead>
<tr>
<th>Date Tested</th>
<th>Days</th>
<th>Cylinder No</th>
<th>Saturated Density</th>
<th>Diameter</th>
<th>Height</th>
<th>Area</th>
<th>Breaking Load</th>
<th>Compression Strength</th>
</tr>
</thead>
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<td></td>
<td></td>
<td></td>
<td>Saturated Density</td>
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<td>Weight in Air</td>
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<td>2</td>
<td>Mean</td>
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<td>30/11/2007</td>
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<td>1772.8</td>
<td>3359.8</td>
<td>2117</td>
<td>100.3</td>
<td>100.1</td>
<td>100.1</td>
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<td></td>
<td></td>
<td>2135</td>
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Table 4-2: Floor test 2, density and concrete compression strength

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<th>Date Tested</th>
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<th>Cylinder No</th>
<th>Saturated Density</th>
<th>Diameter</th>
<th>Height</th>
<th>Area</th>
<th>Breaking Load</th>
<th>Compression Strength</th>
</tr>
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<td>Weight in Water</td>
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<td>Density</td>
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<td>100.2</td>
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<td>end</td>
<td>1403</td>
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</tr>
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Experimental Tests

Table 4-3: Floor test 3, density and concrete compression strength

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<th>Date Tested</th>
<th>Days</th>
<th>Cylinder No</th>
<th>Saturated Density</th>
<th>Diameter</th>
<th>Height</th>
<th>Area</th>
<th>Breaking Load</th>
<th>Compression Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Weight in Water g</td>
<td>Weight in Air kg/m³</td>
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<td>Density 2</td>
<td>Density Mean</td>
<td>(mm)</td>
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The displacement for at least two cylinders in each floor test was measured during the compression test as well. A picture of the extensometer is shown in Figure 4-2.

![Figure 4-2: Compression test with recorded displacement](image)

The results are provided in Figure 4-3. The elastic modulus is measured between 0% and 40% of the final compression strength. The stress-strain diagrams for the concrete compression tests are shown up to the peak load, which is around 2000 micro strain (0.2%). This is less than the peak compression strain, but covers the range of compression
strains generated in the concrete during the experimental testing. Showing only the relevant part of the concrete compression stress-strain curve allows the slope and values reached to be more clearly displayed.

![Stress-strain diagram of concrete compression tests](image)

Figure 4-3: Stress-strain diagram of concrete compression tests

To simplify the curves, they are reduced to three straight lines. The first line represents the same elastic modulus as drawn in Figure 4-3. The second line indicates the elastic modulus between 40% and 80% of the final average compression strength, and the third line represents the area between 80% and 100%. Each elastic modulus with its corresponding starting and ending point is specified in Figure 4-4.
Experimental Tests

Figure 4-4: Simplification of stress-strain curves

Flexural Tensile Strength Test (NZS 3112, 1986)

The model of the testing machine used to calculate the flexural tensile strength is a TT-D manufactured by Instron Ltd. A typical example during the test is shown in Figure 4-5. According to (NZS 3112, 1986), the load is applied continuously at a constant rate of increase in extreme fibre stress within the range 1 to 2 MPa/min.
The test results are listed in Table 4-4 to Table 4-6 and, as previously, the average results at 28/32 days are printed in the last line.

Table 4-4: Floor test 1, flexural tensile strength

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<tr>
<th>Date Tested</th>
<th>Days</th>
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<th>Depth</th>
<th>Width</th>
<th>Length</th>
<th>Load</th>
<th>Flexural Tensile Strength</th>
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<td></td>
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<td>14.31</td>
<td>5.5</td>
<td>with diagram</td>
</tr>
</tbody>
</table>

Table 4-5: Floor test 2, flexural tensile strength

<table>
<thead>
<tr>
<th>Date Tested</th>
<th>Days</th>
<th>Beam No</th>
<th>Depth</th>
<th>Width</th>
<th>Length</th>
<th>Load</th>
<th>Flexural Tensile Strength</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>mm</td>
<td>mm</td>
<td>mm</td>
<td>kN</td>
<td></td>
<td></td>
</tr>
<tr>
<td>12/05/2008</td>
<td>32</td>
<td>1 start</td>
<td>101.0</td>
<td>99.5</td>
<td>400</td>
<td>10.80</td>
<td>4.3</td>
<td>with diagram</td>
</tr>
<tr>
<td>12/05/2008</td>
<td>32</td>
<td>2 end</td>
<td>101.3</td>
<td>99.4</td>
<td>400</td>
<td>9.50</td>
<td>3.7</td>
<td>with diagram</td>
</tr>
</tbody>
</table>

4.2 Material Tests
Table 4-6: Floor test 3, flexural tensile strength

<table>
<thead>
<tr>
<th>Date Tested</th>
<th>Days</th>
<th>Beam No</th>
<th>Depth</th>
<th>Width</th>
<th>Length</th>
<th>Load</th>
<th>Flexural Tensile Strength</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>mm</td>
<td>mm</td>
<td>mm</td>
<td>kN</td>
<td>MPa</td>
<td></td>
</tr>
<tr>
<td>13/11/2008</td>
<td>28</td>
<td>1</td>
<td>100.5</td>
<td>101.0</td>
<td>400</td>
<td>6.78</td>
<td>2.7</td>
<td>with diagram</td>
</tr>
<tr>
<td>13/11/2008</td>
<td>28</td>
<td>2</td>
<td>100.7</td>
<td>100.9</td>
<td>400</td>
<td>7.93</td>
<td>3.1</td>
<td>with diagram</td>
</tr>
<tr>
<td>13/11/2008</td>
<td>28</td>
<td>3</td>
<td>100.7</td>
<td>100.8</td>
<td>400</td>
<td>6.89</td>
<td>2.7</td>
<td>with diagram</td>
</tr>
</tbody>
</table>

For all of the 28-day tests, the load-displacement diagram is recorded and shown in Figure 4-6 and Figure 4-7. Caution must be taken when interpreting these results, as the values differ substantially, even for samples with the same concrete compression strength.

Figure 4-6: Floor test 1 and 2, Flexural Tensile Strength Test diagram
4.2 Material Tests

In some tests, e.g. floor test one beam three, the graph trend declines after the crack further down than expected and inclines again in a very short time increment. As the crack happens suddenly causing a big jump in the strength over a very short time, the amplitude of the deflection for the specimen at that point is probably higher than it would otherwise be in a slower cracking process.

The load-displacement curves are transferred to stress-strain curves and displayed in Figure 4-8 to Figure 4-10. For further investigations and theoretical modelling purposes, the graphs are simplified for each floor test to three straight lines.
Experimental Tests

Figure 4-8: Floor test 1, stress-strain diagram simplification

Figure 4-9: Floor test 2, stress-strain diagram simplification

Floortest 1, Beam 1
\[ \varepsilon = 219 \times 10^{-6}, \quad \sigma = 5.72 \text{ MPa} \]
\[ E_1 = 26143 \text{ MPa} \]

Floortest 1, Beam 2
\[ \varepsilon = 241 \times 10^{-6}, \quad \sigma = 0.7 \text{ MPa} \]

Floortest 2, Beam 1
\[ \varepsilon = 242 \times 10^{-6}, \quad \sigma = 4.07 \text{ MPa} \]
\[ E_1 = 16816 \text{ MPa} \]

Floortest 2, Beam 2
\[ \varepsilon = 267 \times 10^{-6}, \quad \sigma = 1.2 \text{ MPa} \]
\[ E_2 = -371.1 \text{ MPa} \]
4.2 Material Tests

4.2.2 Steel Sheet

Although material information obtained from the steel supplier is generally accurate, additional steel tests were carried out on the delivered steel coil. All the steel-sheet material for each floor test was taken from the same coil. Six test pieces (Figure 4-11) were also cut out from that coil for further investigations: three in the direction of the rolled coil (longitudinal) and three perpendicular to it. The dimensions are standardised in accordance to (BS EN 10002-1, 2001).

Figure 4-10: Floor test 3, stress-strain diagram simplification

Figure 4-11: Dimensions of test piece for tensile testing, steel sheet
Experimental Tests

The test machine was an Instron 5567 with the following load cell: Instron 30 kN, autoranging, SN: UK110. The extensometer used was an Instron Advanced Video Extensometer, model 2663-821.

The stress-strain curves for the three longitudinal and the three perpendicular specimens are plotted in Figure 4-12. By analysing this diagram, it can be observed that the average final tensile stress for the perpendicular specimens is slightly higher than the average final tensile stress for the longitudinal specimens. This is a common phenomenon due to the cold rolling process of the coil. It is also evident that the strain values differ highly between the tests. The reason for this is that the strain gauges used were not able to measure the total 120 mm parallel length of reduced width at once. When the final failure occurred outside of the strain gauge length, the strain gauges are recording only the region of uniform strain outside the region in which necking and final failure occurs, making the recorded values much lower than the final uniform failure strain. In the conducted tensile tests only the specimen designated L2 broke inside the gauge length, resulting in the low strain data shown in Figure 4-12 for all except L2. For this reason, the strain values must be interpreted with care, especially in the inelastic range. The tensile stress is not affected by this problem and displayed relative good conformity throughout the tests.

![Figure 4-12: Stress-strain diagram, longitudinal and perpendicular specimens](image-url)
The average yield stress for the perpendicular specimens is 460 MPa with an average ultimate tensile stress of 467 MPa. The values normally given as material strength are from longitudinal specimens. The average yield stress for the three longitudinal specimens tested is 409 MPa with an average ultimate tensile stress of 417 MPa. These values are very similar to the data provided initially by the manufacturer, giving a yield stress of 405 MPa and a tensile stress of 410 MPa, tested on a similar coil. The coil tested was produced using the same heat as the delivered coil. As the differences between these tested values and the assumption of the stress-strain diagram in Figure 3-21 are very small, the values from Figure 3-21 are used for the floor analysis.

In Figure 4-13, a typical crack can be observed in the steel specimen after the test.

![Figure 4-13: Example of a specimen after fracture](image)

### 4.3 T-SECTION

#### 4.3.1 Relationship of the Concept Model to the Test Specimens

Composite floors or general floor elements are often tested with two point loadings, one at a quarter and the other one at three-quarters of the floor length. Using this loading scheme
Experimental Tests

with two example loads of 1 kN, a 9-m floor will have the shear force and bending-moment diagram shown in Figure 4-14.

![Real-size floor test](image)

Figure 4-14: Shear force and bending-moment diagram for real-size floor test

As mentioned earlier in section 3.3, the final floor element proposed has a width of 3 m and a length of 9 m. However, this is too large to be accommodated within the test laboratory and required especially the length to be reduced to accommodate the 5 m maximum limitation of the bending test rig. One way of achieving this would be to scale down everything by a factor of one half, so that every single part, such as the steel sheet, the rebar or even the concrete aggregate, is scaled down. This, however, was considered undesirable for the following reason: For a composite floor, the interaction between the steel parts and the concrete is very important, and in a scaled model the slip behaviour is likely to be different to the original slip behaviour.

Also, in order to achieve realistic results, the web height of 500 mm should not be changed. To test a smaller specimen without losing any accuracy, symmetric properties of the floor can be used. As the floor is a double-Tee and therefore symmetric over the length axis and because the floor is carrying the load in one direction only, it is possible to
analyse only one Tee without losing any accuracy. In order to be able to test the floor at the facilities of the University of Auckland, the length of 9 m was also halved to 4.5 m.

Referring to Figure 4-15, for the small-size floor test A, the maximum bending-moment of 1.125 kNm is only half of the bending-moment from Figure 4-14. If, instead of using two point loads, only one load in the middle of the slab is used as demonstrated in the small-size floor test B in Figure 4-15, the maximum shear force and the maximum bending-moment stay the same. The shape of the shear force and bending moment diagram is different for the 9 m and the 4.5 m tests. It should be noted that the likelihood of provoking longitudinal shear failure is reduced due to the longer shear span in floor test B. This would make a difference for composite floors where horizontal shear behaviour is critical. However, the proposed floor type behaves more like a conventional concrete beam where horizontal shear failure is less critical. Therefore, it is considered to be more important that the maximum values of the shear force and bending moment diagram remain constant.

Therefore, all floor tests were conducted with half of the width and half of the length of the original floor dimensions and loaded with one middle transverse line load, as shown in the small-size floor test B.

Figure 4-15: Shear force and bending-moment diagram for small-size floor test
Experimental Tests

Figure 4-16 shows an example of the shear force and bending moment diagram for the maximum load of 186 kN reached in the second experimental test (Figure 4-49). Compared to the real-size floor diagrams it can be seen that the shape of the shear force and bending moment diagram varies in Figure 4-16, but the maximum values stay the same in all three samples shown.

![Shear Force and Bending Moment Diagrams](image)

Figure 4-16: Comparison of shear force and bending moment diagrams

Therefore, it can be concluded that the same floor system which reached 186 kN in a 4.5-m long test, would resist about 20.7 kN/m as a line load for a 9-m long test. As the entire
4.3 T-Section

Floor system consist of two T-beams by a width of 3 m, the corresponding distributed load is given by:

\[
q = \frac{20.7 \text{ kN}}{1.5 \text{ m}} = 13.8 \text{ kN/m}^2
\]  

(4.1)

The equivalency of bending moment and shear force in the two situations given in Figure 4-16 and Eq.(4.1) is correct if the moment due to the dead load of the 9-m floor is the same as the moment due to the dead load from the experimental test. This is not the case, as the length of the floor is different and the floor width for one “Tee” in the experimental test is only 1.2 m compared to 1.5 m in the real-size test. If the dead load in Eq.(4.1) should be considered the total bending moment for the 4.5-m long floor specimen in Figure 4-16 would be:

\[
M_{\text{total}} = 209.3 \text{ kNm} + \frac{4.35 \text{ kN} \times (4.5 \text{ m})^2}{8} = 209.3 \text{ kNm} + 11.0 \text{ kNm} = 220.3 \text{ kNm}
\]  

(4.2)

The value of 4.35 kN/m for the dead load was used from Eq.(4.25) which assumed a width of 1200 mm as used in the experimental test. For a width of 1500 mm and the density of the concrete from Table 4-2 the weight will increase by:

\[
2 \times 0.3 \text{ m} \times 0.15 \text{ m} \times 4.5 \text{ m} \times 1858 \frac{\text{kg}}{\text{m}^3} = 752 \text{ kg}
\]  

(4.3)

Therefore, the total line load will be:

\[
q = 4.35 \frac{\text{ kN}}{\text{ m}} + \frac{752 \text{ kg} \times 9.81 \frac{\text{ m}}{\text{ s}^2}}{4.5 \text{ m}} = 4.35 \frac{\text{ kN}}{\text{ m}} + 1.64 \frac{\text{ kN}}{\text{ m}} = 5.99 \frac{\text{ kN}}{\text{ m}}
\]  

(4.4)

This leads to a total bending moment due to dead load for the 9-m floor to:

\[
M_{\text{dead load, 9m}} = \frac{q l^2}{8} = \frac{5.99 \frac{\text{ kN}}{\text{ m}} \times (9 \text{ m})^2}{8} = 60.6 \text{ kNm}
\]  

(4.5)

This moment has to be subtracted from the total moment in order to calculate the line load in Eq.(4.7) or the linear distributed load in Eq.(4.8).

\[
M_{\text{live load}} = M_{\text{total}} - M_{\text{dead load, 9m}} = 220.3 \text{ kNm} - 60.6 \text{ kNm} = 159.7 \text{ kNm}
\]  

(4.6)

\[
q \left[ \frac{\text{ kN}}{\text{ m}} \right] = \frac{8M_{\text{live load}}}{l^2} = \frac{8 \times 159.7 \text{ kNm}}{(9 \text{ m})^2} = 15.8 \frac{\text{ kN}}{\text{ m}}
\]  

(4.7)
Therefore, a 9-m floor should be able to resist a distributed load of 10.5 kN/m$^2$ additional to its own dead load based on the experimental test results from a 4.5-m floor and constructed with the same material properties used for the second tested floor specimen.

In this research project, the unfactored live load required to be developed was not specified by the CSA group. A value of 5.0 kN/m$^2$ as imposed action was adopted, being a typical upper limit value for commercial floors. The design equation for the flexural strength according to NZS 3101 (2006, Eq. 7-1) is given as:

\[ M^* \leq \phi M_n \]  

where \( M^* \) is the design bending moment, \( M_n \) the nominal flexural strength and \( \phi \) the strength reduction factor. The value of 10.5 kN/m$^2$ from Eq.(4.8) is a load prediction calculated without any safety factors. Therefore, it is an area load which could be reached in an experimental test. The value of 5.0 kN/m$^2$ is often used as a live load for a floor system which is calculated according to the New Zealand Standard. Therefore, load and strength reduction factors have to be included in the calculation. According to (NZS 3101, 2006, 2.3.2.2) the strength reduction factor should be 0.85 for flexure with or without axial tension or compression. According to (NZS 1170: Part 0, 2002) a load factor of 1.2 is used for permanent action (dead load) and a factor of 1.5 is used for imposed action (live load). Therefore, the design bending moment may be calculated to:

\[ \frac{(1.2q_{\text{dead load}} + 1.5q_{\text{live load}})l^2}{8} = \frac{1.2 \times 5.99 \text{kN/m} + 1.5 \left( \frac{5 \text{kN/m}^2 \times 1.5 \text{ m}}{1.5 \text{ m}} \right) \times (9 \text{ m})^2}{8} \]

\[ = 186.7 \text{ kNm} \]

With the total bending moment from Eq.(4.2) the design equation for the flexural strength according to NZS 3101 (Eq. 7-1) yields to:
\[ M^* \leq \phi M_u \]
\[ \Rightarrow 186.7 \text{ kNm} \leq 0.85 \times 220.3 \text{ kNm} \quad (4.11) \]
\[ \Rightarrow 186.7 \text{ kNm} \leq 187.3 \text{ kNm} \]

Eq.(4.11) is just fulfilled. Therefore, the 9-m floor specimen using 2 rebars of 20 mm should be able to resist a live load of 5 kN/m^2.

Further points to note regarding the validity of testing a 4.5 m span with central point load in place of a 9 m span with quarter point loads are as follows:

- Although peak bending moment and shear force are matched, the 4.5 m span reaches peak bending moment at just one section (mid-span) compared with the central 4.5 m of the 9 m span case.
- Strength of the 4.5 m span depends on strength at the central section and is likely to result in a higher standard deviation if repeated tests were carried out (depending on local defects at the critical section).
- A 9 m test subjects half the span to maximum bending moment (apart from dead load effects), although this is combined with maximum shear force only at the loaded points, the remainder having zero shear force. The pre-failure response will depend on the average flexural properties over the central half.
- The longitudinal shear flow is twice that of a 9 m section under UDL, which increases the likelihood of a reinforcement embedment failure within the narrow rib. However, the experimental test more accurately covers the real-life case of applied point loading and ensures high point load performance is determined.

### 4.3.2 General Information

As the floor with the formwork had to be moved through a door from the pouring location to the testing actuator, the width had to be reduced further from 1.5 m to 1.2 m. This, however, should not have affected the maximum load capacity of the floor much.

In Figure 4-17 the cross-section and main dimensions of floor test three can be compared to floor tests one and two.
A few changes were made throughout the three tests. The concrete density was reduced from test to test, to investigate how low the density could be and whether the light-weight concrete would show any problems in the workability and mixing process with the steel fibres.

Using four rebars in floor test one proved to be an overestimate, especially, as the positive influence of the steel sheet is not even considered (Eq.(4.18)). A more detailed calculation according to New Zealand Standards which includes the positive influence of the steel sheet is presented in section 5.2. Consequently, only two rebars were used for floor tests two and three. Eq.(4.11) proved that the midspan load reached in floor test 2, in which only 2 rebars of 20 mm diameter were used, is comparable with the target live load of 5 kN/m².

The slab thickness of 150 mm approximates a common total thickness of floor slabs and was therefore chosen for F1. The theoretical analysis of the floor shows that the 150 mm slab thickness is not necessary in terms of structural issues and was therefore reduced to 100 mm in the third floor test. This thickness reduction is indicated in Figure 4-17.

For reasons given in the discussion section of floor test one (section 4.3.4.4), the rebars were welded at their ends to the 6-mm steel plates from floor test two onwards. In floor test
two, the rebars were also spot-welded to the steel sheet. In order to check if these spot-
welding points are necessary, the third floor test was conducted without them.

Furthermore, some additional minor changes were made, which are mentioned in the
Corresponding sections.

Before the detailed descriptions of the three floor tests are given in sections 4.3.4 to 4.3.6,
section 4.3.3 summarises all general preparations that were the same for all tests.

4.3.3 General Preparations

4.3.3.1 Formwork

The formwork needed to be designed for several uses. Three distinct floor elements were
planned to be produced using the same formwork. Throughout these floor tests, the target
dimensions of the formwork could be changed, e.g. a thicker or thinner web or slab
thickness. Therefore, the formwork needed to be flexible enough to match these changes.
Due to the absence of overhead crane access, the entire formwork needed to be moveable
on rollers or reachable by a pallet truck after the concrete had been poured.

To meet these requirements, the final formwork was built from several separated formwork
elements. The elements were connected with threaded bolts to each other. This way, it was
not only easy to build up the formwork, but it was also very easy to disconnect the
elements from the floor after the concrete had cured. Figure 4-18 illustrates the different
formwork elements, which are numbered from 1 to 7.
Figure 4-18: Cross-section of floor and formwork, separated elements

Figure 4-19 presents the finished, assembled formwork, which could be moved both on rollers and with a pallet truck.

Where the formwork made contact with the concrete, the wood was painted with a waterproof varnish. Just before the concrete was poured into the formwork, oil was applied to the wood surface in order to facilitate the separation of the cured concrete from the formwork.
4.3.3.2 Steel Assembly

The three main steel parts, the steel sheet, the SHS and the rebars were welded together so that they could be placed as one unit into the formwork. Figure 4-20 shows a picture of the welding process.

![Image](image.png)

Figure 4-20: Welding of steel parts

During the bending of the T-section, some tension forces are generated in the flanges perpendicular to the web. These tension forces can be taken by 12-mm rebars at a spacing of 500 mm. Each rebar passed through a top “window” in the steel web. At the ends the rebars were located on small wood spacers as demonstrated in Figure 4-21. The rebars were slightly bent between the steel web and the wood spacer in order to give the rebars and the spacers enough robustness to stay in place during the concrete pouring process.
4.3.3 Pouring Concrete

For each test the concrete was delivered by Firth in a concrete truck directly to the Civil Material Laboratory of the University of Auckland. The formwork was positioned near enough to the entrance so that the concrete could be poured directly from the chute of the concrete truck into the formwork. For a better distribution of the steel fibres, it is preferable to place the steel fibres beforehand into the concrete at the batching plant.

In Figure 4-22 it can be seen how the foam is generated, with the foam machine placed on the trailer, and how it is added with the help of the green hose to the concrete drum on the truck. This only took about a minute. Before the concrete could be poured into the formwork, the drum mixed the concrete for a few more minutes.
Figure 4-22: Foam generator adding foam to the concrete mix

Figure 4-23 shows a view of a typical specimen after the concrete pour was finished and the surface was screeded flat. The red wires coming out of the concrete in the middle of the slab were connected to strain gauges on the steel sheet.

Figure 4-23: Concrete floor straight after pour

In order to avoid shrinkage cracks in the concrete and facilitate curing, the surface had to be kept wet straight after the concrete was poured. As soon as the concrete surface cured enough to carry light pressure, wet cotton-fabric covers were used to cover the entire top
Experimental Tests

surface as seen in Figure 4-24. Every second day, the fabric was wetted to maintain moisture. To avoid rapid evaporation, an additional plastic sheet was used to cover the floor for the next five days as indicated in Figure 4-25.

![Figure 4-24: Wet fabric covering the top surface](image)

All floor specimens were tested 28 days after the concrete was poured.
4.3.3.4 Test Setup

Testing Actuator
All three floor tests were undertaken with the MTS testing actuator in the Civil Material Laboratory at the University of Auckland. The load cell had a capacity of 500 kN (model 661.23B-02).

Strain Gauges/ Instrumentation
The number and placement of the strain gauges varied slightly between the tests. The exact position of each strain gauge can be found under each test description. Four different types of gauges were used for every test to measure the floor behaviour under loading. Electric resistance strain gauges of the following types were attached to the steel and were eventually immersed in fresh concrete:

- **Uniaxial Strain Gauges (Figure 4-26)**
  Company Name: KYOWA  
  Type: KFW-5-120-C1-11L5M2R  
  Strain limit at room temperature: \(28000e^{-6}\)  
  These gauges were used to measure the strain of the steel sheet horizontally at midspan or on the rebar. Figure 4-26 shows some strain gauges glued to the steel sheet and one glued to the rebar. To get a better connection to the rebar, the rebar was locally smoothed with an angle grinder, before the strain gauge was glued onto it.

- **Triaxial Stacked Rosette Strain Gauges (Figure 4-27)**
  Company Name: KYOWA  
  Type: KFG-10-120-D17-11L3M3S  
  Strain limit at room temperature: \(50000e^{-6}\)  
  These gauges were used at the end of the floor to monitor the strain in the steel sheet close to the bearing.
Experimental Tests

Outside the concrete two different gauge types were used as well:

- **Portal Gauges (Figure 4-28 and Figure 4-29)**
  These gauges measured displacement (convertible to surface strain) over gauge lengths, ranging from about 120 mm to 350 mm. They were attached to 4 mm diameter studs, bonded into holes, drilled into the surface of the concrete.

- **Linear Variable Differential Transformer (LVDT; Figure 4-30)**
  These were used to measure displacement at indicated locations.
Figure 4-28: Portal gauges at midspan

Figure 4-29: Portal gauges at end of slab
Experimental Tests

![Figure 4-30: LVDT](image)

**Dimensions**

As mentioned earlier, all floor specimens were 4.5 m long and 1.2 m wide. The web height was consistently 0.5 m and the web width 0.1 m. Only the flange thickness changed from 150 mm for the first two tests to 100 mm for the third test.

The static loading system was a simply supported beam loaded at midspan. The point load from the actuator was transferred over a stiff steel beam, acting as a line load, into the floor as shown in Figure 4-31.
A detail of the support is provided in Figure 4-32. The total floor load was supported entirely by the SHS at each end. Two steel plates, welded perpendicular to a third bottom steel plate, prevented any side movement. The bottom plate was fixed to the ground and could not move in any direction. The SHS was freely placed between the side steel plates so that the SHS was still able to rotate freely (simply supported).
**4.3.3.5 General Test Procedure**

The test procedures for all three tests were similar. Detailed information for each individual test can be found in sections 4.3.4 to 4.3.6.

Before the final load was reached, a small number of load cycles were carried out to analyse the floor behaviour at load relieving and reloading. However, it was still a static test with an increasing load and not a cyclic load test as defined in Eurocode 4 (BS EN 1994-1-1, 2004) for example.

According to Eurocode 4, the cyclic load for testing a composite floor slab should be between $0.2 \times W_t$ and $0.6 \times W_t$ and should be applied for 500 cycles in a time not less than three hours. $W_t$ is the measured failure load conducted from a preliminary static test. Therefore, a static test to failure is necessary first to determine the measured failure load $W_t$, which will then define the cyclic loading range. As the three conducted experimental floor tests were the first floor tests for each situation, they were all conducted as a static test. The measured failure load could be used to define the load range for further cyclic tests which are not part of this thesis. This cyclic test would be necessary for the final prototype product testing.

All observed cracks in the concrete were marked at different load levels with varying colours.

Photographs and written notes were taken during the course of each test.

**4.3.4 Test 1**

**4.3.4.1 Floor Properties**

In order to get some realistic floor dimensions, it was specified for the first floor test that the nine-metre-long floor should be able to resist a service load of $\frac{5.0 \text{ kN}}{m^2}$.

For one T with a width of 1.5 m this would be:
4.3 T-Section

\[ q_{\text{service load}} = 5.0 \frac{kN}{m^2} \times 1.5 \text{ m} = 7.5 \frac{kN}{m} \]  
\hspace{1cm} (4.12)

The dead load was measured in the first experimental test with a width of 1.2 m in Eq. (4.21) to 5.01 kN/m. For a width of 1.5 m the additional slab width of 0.3 m is added.

\[ q_{\text{dead load}} = 5.01 \frac{kN}{m} + 0.3 \text{ m} \times 0.15 \text{ m} \times 2135 \frac{kg}{m^3} \times 9.81 \frac{m}{s^2} = 5.95 \frac{kN}{m} \]  
\hspace{1cm} (4.13)

According to NZS 1170 Part 0 (2002) the total or design load is calculated by multiplying the dead load by a load factor of 1.2 and the live load by 1.5.

\[ q_{\text{total}} = 1.2 \times q_{\text{dead load}} + 1.5 \times q_{\text{service load}} = 1.2 \times 5.95 \frac{kN}{m} + 1.5 \times 7.5 \frac{kN}{m} = 18.39 \frac{kN}{m} \]  
\hspace{1cm} (4.14)

For this load the design bending moment would be:

\[ M^* = \frac{q(l)^2}{8} = \frac{18.39 \frac{kN}{m} (9 \text{ m})^2}{8} = 186.2 \text{ kNm} \]  
\hspace{1cm} (4.15)

For simplification, it is assumed that the distance between the compression force and the tension force is about 0.9 \( d \). The effective depth, \( d \), is taken as 0.44 m. Therefore, the tension force in the cross section may be calculated to:

\[ F_t \approx \frac{M^*}{0.9d} = \frac{186.2 \text{ kNm}}{0.9 \times 0.44 \text{ m}} = 470.2 \text{ kN} \]  
\hspace{1cm} (4.16)

Therefore, the area of flexural tension reinforcement, \( A_s \), has to be:

\[ A_s = \frac{F_t}{f_y} = \frac{470.2 \text{ kN}}{500 \text{ MPa}} = 940.4 \text{ mm}^2 \]  
\hspace{1cm} (4.17)

This amount can be reached for example by four 20-mm rebars with the amount, \( A_s \), of:

\[ A_s = 1257 \frac{\text{mm}^2}{m} \geq 940.4 \text{ mm}^2 \]  
\hspace{1cm} (4.18)

This simple calculation provided an estimate of the required rebar area. A more detailed calculation which includes the positive influence of the steel sheet is presented in section 5.2.
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Therefore, four rebars with a diameter of 20 mm and a total cross-section of 1257 mm$^2$ were chosen for the first test.

The dimensions of the first test can be seen in Figure 4-33.

![Figure 4-33: Floor test 1, dimensions](image)

Figure 4-34 demonstrates the two rebars connected to each side of the steel sheet just before they were placed into the web of the formwork. The rebars were connected with 2-mm-thick wire to the steel sheet with 50 mm spacing, but they were not connected or welded to the 6-mm steel plate at each end.
A concrete density lighter than normal-weight concrete, but not lighter than \( 2000 \text{ kg/m}^3 \), was targeted for the first floor test. If the floor showed good performance under testing, and if the steel fibres did not settle down to the bottom, the density would be decreased further in the following tests. The average concrete density for the first test was \( 2135 \text{ kg/m}^3 \), as listed previously in Table 4-1. The steel fibres amount was 30 kg per cubic metre concrete.

It was planned that the steel fibres should be added to the concrete at the batching plant so that they were well distributed in the mix by the time the concrete truck arrived at the University of Auckland. However, when the truck arrived at the University, no fibres had yet been added. It took some time to find out that the steel fibres were placed behind the seat of the truck, however, the truck driver was not informed of this. As a result the fibres had to be added to the drum right there and then and were mixed for a few minutes. Because of all these circumstances, the start for the planned pour was delayed by about one hour. The concrete was ordered with a slump of 120 mm. Due to the time delay the concrete was at the middle of the pour much stiffer with a slump of 70 mm. At the end of the pour in particular, the consistency was too stiff to fill the formwork properly. Figure 4-35 illustrates the consistency of the concrete at the end of the pour.
As mentioned earlier in section 3.3.2, the steel fibres are glued together in rows when they are added to the concrete. Due to the relatively short mixing time left for this first test, some fibres were still connected in two or three pair formations.

As a result of the poor concrete workability, some surface defects were observed after the formwork was taken apart. Figure 4-36 demonstrates a substantial lack of concrete directly in the middle of the span, and Figure 4-37 illustrates local areas of poor compaction at the end of the slab.
Figure 4-36: Poor compaction in specimen web at midspan, floor test 1

Figure 4-37: Surface evidence of poor compaction in web close to support, floor test 1
The weight of the floor used in the tests may be calculated with (4.19), where the parts in this equation are the mass of the concrete, the steel sheet, the rebars, the 6-mm steel plates and the square hollow sections.

\[
m = 4.5\ m \times (1.2\ m \times 0.15\ m + 0.35\ m \times 0.1\ m + 0.05\ m \times 0.05\ m) \times 2135\ \frac{\text{kg}}{\text{m}^3} \\
+ 0.0016\ m \times 0.44\ m \times 4.4\ m \times 7850\ \frac{\text{kg}}{\text{m}^3} \\
+ 4\ \pi \times (0.01\ m)^2 \times 4.4\ m \times 7850\ \frac{\text{kg}}{\text{m}^3} \\
+ 2\times 0.006\ m \times 0.07\ m \times 0.44\ m \times 7850\ \frac{\text{kg}}{\text{m}^3} \\
+ 12\ \frac{\text{kg}}{\text{m}} \times 1.4\ m \\
= 2089.6\ \text{kg} + 24.3\ \text{kg} + 43.4\ \text{kg} + 2.9\ \text{kg} + 16.8\ \text{kg} \\
= 2177\ \text{kg}
\]

This mass equals a self-weight of:

\[
q = \frac{2177\ \text{kg} \times 9.81\ \frac{\text{m}}{\text{s}^2}}{4.5\ \text{m}} = 4.75\ \frac{\text{kN}}{\text{m}}
\]

(4.20)

The weight on one end of the floor was also measured with a load cell to 1150 kg. Therefore, the total weight of the floor should be 2300 kg, which equals a line load of:

\[
q = \frac{2300\ \text{kg} \times 9.81\ \frac{\text{m}}{\text{s}^2}}{4.5\ \text{m}} = 5.01\ \frac{\text{kN}}{\text{m}}
\]

(4.21)

The measured weight of the floor is about 6% higher than the calculated value. From Eq.(4.19) it can be seen that the concrete density has a big influence on the total weight of the floor. As the concrete in the floor specimen was compacted by a vibrator and the test cylinders were not, it is assumed that the concrete density in the test specimen was slightly higher than measured by the test cylinders. Therefore, the weight measured by the load cell is used for further calculations.
4.3.4.2 Strength Test Procedure

The test was conducted after the concrete had cured for 28 days. The floor was loaded with two cycles to 40 kN and back to zero. Then, the load was increased steadily, until the maximum floor capacity was reached. After the peak load was reached, loading was continued under displacement control. To analyse the behaviour after the peak load, the gauges kept recording, until a significant part of the load-displacement-curve after the peak load was logged. The locations of gauges inside and outside the floor are shown in Figure 4-38.
Figure 4-38: Floor test 1, strain gauge positions
4.3.4.3 Test 1 Results

The load-deflection curve for floor test one is plotted in Figure 4-39. The load is the actuator load applied at midpoint to the floor, and the deflection is the measured vertical displacement in the middle of the slab.

The concrete started cracking at about 70 kN. At 100 kN the load was held constant to permit the marking of established cracks at this stage. 121 kN was the maximum load which the floor carried in this test.

Small hair cracks were observed and marked during the test only in the web of the T-section. The crack widths were very small, of about 0.1 mm to 0.2 mm during the section of the test up to the peak load. The big crack, seen in Figure 4-40, appeared and increased rapidly after the final load of 121 kN was reached.
Figure 4-40: Crack widening after peak load was reached, floor test 1

Figure 4-41 indicates the failure of the floor about 1m from the support after the actuator reached its maximum deflection. The concrete from the web at this location was removed after the test to inspect the steel sheet. The sheet was cracked over the entire web height as shown in Figure 4-42. This is a similar picture to Figure 4-41, but was photographed from the other side of the floor specimen and after the concrete was removed from the steel sheet.

Figure 4-41: Detail showing debonding of ends of rebars from concrete, floor test 1
4.3.4.4 Test 1 Discussion

The concrete was clearly not compacted properly, as Figure 4-36 and Figure 4-37 indicate. This was mainly caused by the delay in starting the pour. Unfortunately, the full extent of the inadequate compaction did not become obvious until the specimen was tested.

The maximum floor load of 121 kN was less than half of the calculated load capacity of the floor of 246 kN (Table 5-2). This was a direct result of the rebars not being anchored properly in the concrete. The separation of the rebars can be seen in Figure 4-41 after failure. At the point of separation, the rebars and the concrete lose all composite action. At this time, the rebars are no longer able to carry the high tension force, which is necessary for high bending-moment resistance. Therefore, the floor fails at a significantly lower load, before the theoretical bending-moment capacity could be reached.

It was clear that rebar anchorage had to be improved to prevent a recurrence of the behaviour observed in this test.

Apart from this rebar connection problem, the experimental test showed that other aspects of the floor system F1 performed as expected. One of the main issues before the test was the concrete connection to the steel web. As this system had never previously been tested,
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it was not known whether the concrete and the steel sheet would work together as one unit. If the concrete had spalled off the steel web, the entire floor system would have required revision.

After the test, the concrete was removed from the steel sheet at midspan as shown in Figure 4-42. As the floor specimen remained visibly straight until the maximum load was reached (Figure 4-40), it is most likely that the crack in the steel sheet occurred after this maximum load point. Therefore, the crack in the steel sheet has nothing to do with the failure at peak load of the floor. It is only a result of the high curvature supplied to the floor after the maximum load was reached.

As the low density of the concrete did not seem to be a problem, it was decided to reduce the density further in the next experimental test, given that light weight is a desired attribute of this floor system.

4.3.4.5 Test 1 Conclusions

The outcomes of the first experimental test can be concluded as follows:

- The connection of the rebars to the concrete was not satisfactory. As a result, the rebars were able to separate from the floor element. This behaviour led to premature failure of the floor element in an unexpected mode, requiring modification of the connection detail for the next test.
- The concrete workability was poor, resulting in poorly compacted concrete with visible holes at the surface. The mixing time of the concrete in the concrete truck had been too long, and the concrete had begun to cure. This was caused by a communication problem with the concrete supplier and was not expected to recur.
- The main concern that the concrete would separate from the steel sheet under load, proved baseless, and it was demonstrated that the sheet provided a viable means of resisting shear, its primary role.
- The low density of the concrete did not cause any identifiable problems, and it was resolved to reduce the density further in subsequent tests.
4.3.5 Test 2

4.3.5.1 Floor Properties

![Diagram of T-section](image)

Figure 4-43: Floor test 2, dimensions

In section 4.3.4.1 the rebar amount was chosen for the first test as four 20-mm rebars, which is 34% more than required (Eq. (4.18)) and which neglects the positive influence of the steel sheet in the calculation of the bending moment capacity. A more detailed calculation is done in section 5.2.2 which takes into account the flexural contribution of the steel sheet. For two 20-mm rebars only, the bending moment capacity is calculated as 212 kNm (Eq. (5.33)). As developed in section 4.3.1, the target ultimate limit state bending moment capacity for this floor unit with 5 kN/m² imposed load is around 187 kNm (Eq. (4.10)). With a strength reduction factor of 0.85, the design equation for the flexural strength capacity according to NZS 3101 (Eq. 7-1) gives:

\[
M^* \leq \phi M_u
\]

\[
M^* = 187 \text{ kNm}
\]

\[
\phi M_u = 0.85 \times 212 \text{ kNm} = 180 \text{ kNm}
\]
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This is 4 \% under the required value to achieve 5 kN/m^2 specified imposed load. However, this value is sufficiently close to justify the rebar amount to be reduced for the second experimental test from 4 to 2 rebars of 20 mm.

As the peak load reached in the second experimental test is 4 \% higher than calculated by theory (Eq.(5.43)), Eq.(4.22) is just fulfilled by using the results of the second experimental test (Eq.(4.11)).

To improve the failure behaviour from test one, the two rebars were welded with a fillet weld this time to the 6-mm steel plate at each end. Two holes with a diameter of about 22 mm were drilled in the steel plate to allow the 20-mm rebar to go through. The additional 6-mm thick rectangular steel plate, seen in Figure 4-44, was used to provide better welding conditions.

![Figure 4-44: Detail rebars welded to 6mm steel plate, floor test 2](image)

The rebars were connected to the steel sheet with 3-mm-thick wire, similar to the first test. Additionally, the rebars were also tack-welded to the steel sheet every 500 mm as indicated in Figure 4-45. One reason for the tack welds was that the steel assembly used in the first test was very difficult to handle, specifically when placing it into the formwork. The 1.6-mm steel sheet was effectively flat when it was lying horizontally on the ground. However, when it was tried to lift it and to place it vertically into the formwork, it became very
unstable. The sheet buckled and it took considerable effort to keep the steel sheet straight. This was achieved by using small wood pieces as spacers between the formwork of the web and the steel sheet. The wood pieces were left in place during the concrete pour.

After the rebars were tack welded to the steel sheet, the assembly was much easier to handle. It was easy to lift the steel assembly into the formwork and only a few spacers were necessary to keep the steel sheet in place. Another reason for the tack welds is that a good connection between the rebars and the steel sheet helps to prevent disconnection of individual parts of the composite construction during testing. This occurred in the first experimental test where the rebars were not properly connected to the surrounding concrete such that they separated during testing.

At each tack-weld location, a short length of 3-mm thick wire (about 10 mm long) was placed between the steel sheet and the rebar to ensure access for the welding procedure. This small wire was welded with the rebar and steel sheet together and left in place. Therefore, a space of about 3mm between the rebar and the steel sheet was created which would improve the bond of the rebars to the surrounding concrete. The NZS 3101 (2006) requires for a minimum cover distance for rebars in section 3.11.2.2 the maximum nominal aggregate size. As this research project does not use any aggregates apart from sand, this point is not critical. The standard also requires in section 8.3.1 that the clear distance between parallel bars should be equal or greater than the largest of the nominal diameter of the bars. This would be 20 mm in this test and is not fulfilled if the rebars are not considered as bundled bars. However, at this stage it is considered to be more important to achieve a solid steel part which can be lifted easily in the formwork at once and stays stable during the concrete pour. The possibility still exists to improve this connection with steel clips to ensure a larger distance from the steel sheet.
This time, the steel fibres were mixed with the concrete at the batching plant. Therefore, the steel fibres were well distributed in the concrete during the pour as demonstrated in Figure 4-46. The workability of the concrete was also very good so that the concrete filled the formwork with only minor usage of the vibrator. The slump test as conducted in the first test is not applicable to non-plastic or non-cohesive concretes (NZS 3112, 1986). The foamed concrete used for the first test was, because of the time delay, relatively stiff. However, the workability of the foamed concrete for this second test was totally different. After lifting the cone, the concrete was not stiff enough to stay in place to measure the slump-height. Instead, the material was very fluid similar to self-compacting concrete. As the slump is defined as the difference between the highest point of the cone (which is 300 mm) and the highest point of the slumped test specimen (which was a few mm over the ground), the slump would be nearly 300 mm.

A picture of the floor specimen after the formwork was removed can be seen in Figure 4-47.
The weight of the second floor may be calculated with Eq.(4.23):
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\[
m = 4.5 \text{ m} \times (1.2 \text{ m} \times 0.15 \text{ m} + 0.35 \text{ m} \times 0.1 \text{ m} + 0.05 \text{ m} \times 0.05 \text{ m}) 1858 \frac{\text{ kg}}{\text{ m}^3} \\
+ 0.0016 \text{ m} \times 0.44 \text{ m} \times 4.4 \text{ m} \times 7850 \frac{\text{ kg}}{\text{ m}^3} \\
+ 2 \pi (0.01 \text{ m})^2 \times 4.4 \text{ m} \times 7850 \frac{\text{ kg}}{\text{ m}^3} \\
+ 2 \times 0.006 \text{ m} \times 0.07 \text{ m} \times 0.44 \text{ m} \times 7850 \frac{\text{ kg}}{\text{ m}^3} \\
+ 12 \frac{\text{ kg}}{\text{ m}} \times 1.4 \text{ m} \\
= 1818.5 \text{ kg} + 24.3 \text{ kg} + 21.7 \text{ kg} + 2.9 \text{ kg} + 16.8 \text{ kg} \\
= \textbf{1884 kg}
\]

This mass equals a self-weight of:

\[
q = \frac{1884 \text{ kg} \times 9.81 \frac{\text{ m}}{\text{s}^2}}{4.5 \text{ m}} = 4.11 \frac{\text{ kN}}{\text{ m}}
\]

(4.24)

The second floor specimen was also weighed with a load cell, giving a mass of 1994 kg and a self-weight per unit length of:

\[
q = \frac{1994 \text{ kg} \times 9.81 \frac{\text{ m}}{\text{s}^2}}{4.5 \text{ m}} = 4.35 \frac{\text{ kN}}{\text{ m}}
\]

(4.25)

As with the first test, the measured weight from the load cell is used for further calculations.

\subsection*{4.3.5.2 Strength Test Procedure}

A few load cycles were conducted in order to compare the floor behaviour before and after load release. For the first cycle, the floor was loaded up to 30 kN, before the load was released back to 0 kN, as in each of the other cycles. The maximum values in the other load cycles were twice 60 kN and once 100 kN. After this, the load was increased up to the peak load.

The quantity and positions of the recording gauges are similar to test one, and they are all marked in Figure 4-48.
Figure 4-48: Floor test 2, strain gauge positions
4.3.5.3 Test 2 Results

The load-deflection curve of floor test two can be seen in Figure 4-49.

![Figure 4-49: Floor test 2, load-deflection diagram](image)

After several load cycles to 30 kN, 60 kN and 100 kN, the maximum load reached for this floor was 186 kN. At 150 kN, where the cracks marked in red or at the stage where the LVDT under the floor at midspan (number 13) needed a readjustment, the actuator was not only stopped, instead, the actuator slowly decreased the load. This was necessary for a safe working environment under and close to the specimen.

Figure 4-50 shows the strain behaviour at midspan, measured directly on both rebars. The strain increases in the bars linearly with the load of the testing machine. At 171 kN and 2924 με, the graph at rebar two abruptly changes its direction, and the strain is significantly increasing at nearly a constant load level. Rebar 2 yields before the peak load; rebar 1 starts to yield when the peak load is reached. The load-strain history for rebar 1 is shown in Figure 4-51 for the entire test.
Figure 4-50: Strain measured on both rebars at midspan up to the peak load, floor test 2

Figure 4-51: Strain measured at rebar 1 at midspan for the entire test, floor test 2
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The cracks on the concrete surface are marked at three stages in three different colours. Blue is used for cracks at 60 kN, black for cracks at 100 kN and red for cracks at 150 kN. The crack pattern, after the final load of 186 kN was reached, is provided in Figure 4-52. Up to approximately 170 kN, all cracks were very small with width below 0.2 mm.

Figure 4-52: Crack pattern after maximum load reached, floor test 2

Following the peak load being reached, a small horizontal crack established rapidly from the middle to the end at the height of the embedded rebars. As the floor deflection increased, only this crack and a few in the middle of the slab widened further. Figure 4-53 shows the horizontal crack at completion of the test.
Figure 4-53: Horizontal crack at the height of rebars, floor test 2

Figure 4-54 displays the broken floor after test two.

In order to get a better view of the steel sheet after the test, the concrete in the middle of the floor length was removed from the web. As illustrated in Figure 4-55 a nearly vertical crack in the steel sheet could be observed, similar to the crack from the first experimental test.
4.3.5.4 Test 2 Discussion

The main changes from floor test one to floor test two were:

- The concrete density and compression strength changed from 2135 kg/m³ and 34.3 MPa in floor test one to 1858 kg/m³ and 19.6 MPa in floor test two.
- Instead of four rebars, only two rebars with a diameter of 20 mm were used in floor test two.
- In floor test two, the rebars were welded at each end to the 6-mm steel plate. They were also tack-welded every 500 mm to the steel sheet.

Figure 4-56 shows the load-deflection diagram from floor test two in direct comparison with floor test one. The main difference is that the maximum load capacity from floor test two is much higher than that from floor test one. In floor test one, the rebar was not welded to the 6-mm steel plate at the end and also not tack-welded to the steel sheet. Before the expected maximum bending-moment could be reached, the rebar separated from the concrete in the first test, which led to the early failure behaviour. Due to the fact that the rebars were better connected to the concrete in floor test two, the performance was improved in several ways, resulting in a load-deflection curve much closer to expectations. A detailed comparison with the theoretical and finite-element models can be found in section 5.6.5 and section 6.5.
The reduction in the concrete density had little effect on the load-deflection curve. However, the area of the reinforcing steel bar used in each test had a direct quantitative effect on the load-deflection diagram. In floor test one, four rebars of a diameter of 20 mm were used. Therefore, the floor was significantly stiffer in the first test than in the second, where only two 20-mm rebars were used. This can be seen in the different slopes of the graphs in Figure 4-56, indicating a reduction in initial elastic stiffness of about 50% for the second floor test compared to the first.

Another important difference was that the second floor test exhibited very ductile behaviour after achieving its ultimate load. After the maximum load was reached, the load-carrying ability dropped off by about 10% at a displacement ductility factor of about two. Further decreasing of the load-carrying ability led to a constant load plateau. At this plateau, the load resistance was maintained at or above 80% of the ultimate load up to a displacement ductility factor of 6.5. This ductile behaviour with yield plateau could only be reached, if the rebars still made a contribution to the flexural strength after peak load, at which parts of the rebar debonded from the surrounding concrete as shown in Figure 4-53. This was achieved by the connection of the rebars to the steel sheet with tack-welding points and 3-mm wire (Figure 4-45) and by the end anchorage to the 6-mm steel plate. The floor was able to carry the load of about 150 kN over a substantial displacement, before the maximum load decreased to a second plateau of about 45 kN. At this stage, the floor had developed a plastic hinge in the middle of the slab which was clearly visible. The maximum deformation of the actuator was reached, and the floor behaviour could not be recorded any further. At this time, the floor could still carry its own weight.
The concrete showed a very good performance during the entire test. The concrete was very well attached to the steel sheet, with no concrete spalling observed during the test. The cracks in the web concrete were very small and well distributed over the floor length.

The horizontal crack at the height of the rebars, shown in Figure 4-53, resulted from the slip between the rebars and the concrete, which appeared at the ultimate load of the floor. After the bond between rebars and concrete was lost, the final load could not be further increased.

After the test, the concrete at midspan was removed from the steel sheet as shown in Figure 4-55. The vertical crack revealed in the steel sheet occurred with increasing plastic rotation after the maximum load was reached, and the floor was further deformed to the stage shown in Figure 4-54. At peak load, strains in the steel sheet were just above yield, so that the steel sheet should not have been cracked. At this point, the mid-deflection was so small that the floor still looked straight, and the bending of the floor was hardly noticeable.
In sections 5.6.5 and 6.5, the load-deflection curve is analysed in more detail in comparison to the theoretical and analytical model. In order to find the reason for small strain value differences between these models, which occurred especially at a higher load level, all recorded data from the experimental test were reinvestigated. During this process, it was found that the strain in the rebar at each end of the floor specimen did not increase proportionately with the load as expected. The recorded data is shown in Figure 4-57 and Figure 4-58 for the strain gauges located at each end of one rebar, named end A and end B respectively. In the second floor test, only one rebar was prepared with strain gauges at both ends and in the middle. At the other rebar (rebar 2), the strain was only measured at midspan.

Both diagrams show a relatively constant increase of strain proportional to the applied load in the beginning, indicated with the red line. Due to the fact that the floor is a simply supported beam, the bending-moment and the strain in the rebar should theoretically be zero at the end, if load distortion at the support is neglected. And indeed, in the beginning, these strain values were very small as expected. If the load increases further, the slope of the diagram changes significantly at one particular point. This is the point, where the rebar ceases to be connected properly to the surrounding concrete, and slip occurs at the interface between rebar and concrete.
Initially, it was assumed that the recorded strain values at the end of the rebar were of little importance. Because the bending-moment is zero, or it is at least very small at this
location, this part of the floor cannot have much influence on the load-deflection curve. However, the strain was measured on one rebar just at the ends and in the middle. No strain data were available for parts between these locations.

In order to calculate the maximum bending-moment resistance of the floor, only the location where the maximum bending-moment exists is of importance, given that the cross-section details are uniform. This will occur at midspan for the conducted tests. As the strain increased proportionally with the load in both rebars at midspan, it can reasonably be concluded that no slip between rebar and concrete occurred in the vicinity. This will be further confirmed in section 5.6.5 and section 6.5 by the fact that the maximum load from the experimental test is very similar to the calculated maximum loads from the theoretical and finite-element models, neither of which allowed slip between rebars and concrete.

However, for the midspan deflection, all sections in the floor are important and have their influence on the final deflection. If, for example, the rebar is not connected properly at about one quarter of the floor length from the support, the maximum floor load capacity will still be the same, but the deflection will be higher compared to a floor specimen with no slip.

For the second floor test the strain gage results show that no slip occurred at midspan, and very minor slip occurred above a load of approximately 92 kN in the vicinity of the supports, which is the expected location of maximum slip. Closer to midspan, the influence of slip decreases, until an area close to midspan is reached where no slip occurs at all. It can also be assumed that up to a load level of 78 kN, which is the load when the slip started in Figure 4-57, no slip occurred at all in this rebar. This is helpful in that, up to this point, the results from the theoretical model, which assumed no slip, should be similar to the results from the experimental test. It is expected that the slip behaviour of the second rebar is similar to the first rebar, given that they occupy symmetrically identical positions and have the same connection details.

For modelling the theoretical behaviour in chapter 5, the Bernoulli hypothesis, that plane sections remain plane, is used. For this to apply to all components (rebar, concrete and steel sheet) no significant slip between these components should occur. In order to analyse
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the accuracy of this statement, the results from the strain gauges attached to the steel sheet and rebars and the portal gauges attached to the concrete surface at midspan are analysed. Figure 4-59 to Figure 4-65 show the readings from the experimental test for each approximate 10-kN step (using the nearest recorded data point to each step).

The dark blue line (channel 30 to 34) represents the strain readings on the steel sheet at midspan (see Figure 4-48), the light blue line (channel 37 to 39) represents the strain readings for the concrete at midspan and the two black crosses on red background are the readings for the two rebars at midspan. For each load case, the theoretical strain behaviour based on plane strain theory (as used in the Excel sheets and presented in section 5.6) is shown as a red line in the diagram for comparison.
Figure 4-59: Strain measurements for test 2 from 10 to 30 kN
Figure 4-60: Strain measurements for test 2 from 40 to 60 kN
Figure 4-61: Strain measurements for test 2 from 70 to 90 kN
Figure 4-62: Strain measurements for test 2 from 100 to 120 kN

Load = 100.3 kN

Load = 110.4 kN

Load = 120.3 kN
Figure 4-63: Strain measurements for test 2 from 130 to 150 kN
Figure 4-64: Strain measurements for test 2 from 160 to 180 kN
Figure 4-65: Strain measurements for test 2 for 187 kN (peak load)

The diagrams show that the strain in the steel sheet generally matches the strain in the concrete and the rebar and therefore, no major slip between the composite components can be inferred. Further, the results of the theoretical model matches the experimental test data well and the adoption of the Bernoulli hypothesis is justified over most of the loading history.

However, some recorded data, especially at high load levels, does not match the theory. Details are as follows:

1. One rebar strain at the load level of 180 kN is much higher than the other. This can be explained with Figure 4-50. At 171 kN one rebar starts yielding and therefore, the strain gauge results show inelastic strain is concentrated into that region.

2. Channel 34, which is the bottom strain gauge on the steel sheet, has very high strain values at high load levels. They can be explained with reference to Figure 4-66. The steel sheet at this point starts yielding at approximately 106 kN and exhibits no strain hardening, therefore, once yielding starts at a given location, it concentrates into that location rather than spreading away from there, due to strain hardening of the yielding region. This means that the difference to the theory for this channel is increasing for the diagrams with the load of 110 kN and over.
3. Up to 170 kN the concrete data matches the theory reasonably well. However, at 180 kN all three recorded values are higher than the theory or the measured strain from the steel sheet. The measured displacements for all three concrete channels are shown in Figure 4-67 to Figure 4-69 with similar behaviour. Up to a load of 171 kN the displacement or strain increases relatively linearly, after that load, the displacement increases much more rapidly. It is believed that a vertical crack in the concrete between the connection points of the portal gauges (which are 120 mm apart, see Figure 4-48) appearing at 171 kN is the reason. At this load level, bigger vertical concrete cracks were observed at midspan during the experimental test.
Figure 4-67: Load-displacement diagram for channel 37 (concrete), test 2

Figure 4-68: Load-displacement diagram for channel 38 (concrete), test 2
4.3.5.5 Test 2 Conclusions

The results and discussion from the second experimental test can be summarised by the following points:

- The floor in the second test showed a much better performance than the floor in the first test. As the rebars were better connected to the floor specimen, they could not separate from the concrete, as happened in the first test. This was achieved by welding the rebars at each end to the 6-mm steel plate, and by tack-welding the rebars to the steel sheet at 500-mm intervals.

- Due to the better rebar connection, the floor was able to reach a load capacity very close to the prediction of the theoretical model. The behaviour relative to the predictions of the theoretical and finite-element models is investigated further in sections 5.6.5 and 6.5.

- The strain gauge readings for the second test do not indicate any significant horizontal slip before yielding in the midspan region, thus the assumption of full composite action with one neutral axis is supported.
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- The concrete quality and workability was this time in accordance with specifications. The fibres were well distributed in the concrete. The low density of the lightweight concrete did not produce any problems.
- The floor behaviour after failure was ductile. The good connection from the rebars to the concrete and the steel sheet prevented a brittle rebar debonding failure prior to yield, such as occurred in test one.
- When the maximum load was reached, a horizontal crack occurred suddenly at the height of the rebars, resulting in a 10 to 20% decrease in load down to a stable load plateau.
- Sudden increased strain, indicating slip between the rebar and the concrete, was measured at the ends of a rebar. This has no influence on the maximum load, but it has an influence on the deflection. This behaviour is investigated further in sections 5.6.5 and 6.5.

4.3.6 Test 3

4.3.6.1 Floor Properties

![Diagram of a T-section floor test]

Figure 4-70: Floor test 3, dimensions
Experimental Tests

Aside from the concrete density, two other changes were made for floor test 3. The location of the neutral axis in the theoretical and finite-element analysis section showed that the 150-mm slab thickness was not necessary; therefore, the slab thickness was reduced to 100 mm as shown in Figure 4-70.

As a second modification, the rebars were no longer welded to the steel sheet. Instead, they were only welded at each end to the 6-mm steel plate, as this was a great benefit in test two. In order to weld the rebars to the 6-mm steel plate, the plate was widened slightly at the bottom as seen in Figure 4-71. Therefore, the additional steel plate from floor test two was no longer necessary. Figure 4-72 illustrates the finished welded steel assembly.

Figure 4-71: Detail of 6-mm steel plate, floor test 3
Similar to test one and two, the rebars were connected every 500 mm to the steel sheet as indicated in Figure 4-73. The relatively thick wire, used for connecting the rebars with the steel sheet in test one and two, was very hard to bend. Therefore, the thin tie wire manufactured for reinforcement bar assembly was used.

In order to establish a good shear bond, it is required to position the reinforcement slightly off the surface of the steel sheet. This was achieved by using 5 mm wooden spacers. As a result, the rebars were better embedded in the concrete as displayed in Figure 4-73.

Figure 4-72: Finished welded steel assembly, floor test 3
The steel sheet itself tended to be a little bit unstable to handle while nothing held it in position, however, after the rebar and the steel sheet were connected together, the steel assembly proved to be a solid element. This is demonstrated in Figure 4-74, where the steel section could easily be lifted at each end of the SHS sections by the two technicians.

The workability of the concrete in floor test three was not as good as it was in floor test two, causing some compaction problems during the concrete pour and some small holes in
the floor after the formwork was removed. This can be seen in Figure 4-75. The slump was measured at 220 mm. Additionally, a spread test according to NZS 3112 (1986) was conducted which is generally used for high slump concrete. The diameter of the spread test was measured at 430 mm.

![Figure 4-75: Detail of compaction diversity, floor test 3](image)

Although the average concrete density for floor test three was a lot lighter, at \( 1655 \frac{\text{kg}}{\text{m}^3} \), than normal-weight concrete, the steel fibres were well distributed in this particular type of concrete.

For the third floor test the opportunity to measure the weight of the floor by a load cell was missed. Therefore, the weight of the floor has to be taken from the calculation in Eq. (4.26). The parts in that equation are the mass of the concrete, the steel sheet, the rebars, the 6-mm steel plates and the square hollow sections.
Experimental Tests

\[
m = 4.5 \, m \times (1.2 \, m \times 0.1 \, m + 0.35 \, m \times 0.1 \, m + 0.1 \, m \times 0.1 \, m) \times 1655 \, \frac{kg}{m^3}
\]

\[
+ 0.0016 \, m \times 0.44 \, m \times 4.4 \, m \times 7850 \, \frac{kg}{m^3}
\]

\[
+ 2 \times \pi \times (0.01 \, m)^2 \times 4.4 \, m \times 7850 \, \frac{kg}{m^3}
\]

\[
+ 2 \times 0.006 \, m \times 0.07 \, m \times 0.44 \, m \times 7850 \, \frac{kg}{m^3}
\]

\[
+ 12 \, \frac{kg}{m} \times 1.4 \, m
\]

\[
= 1228.8 \, kg + 24.3 \, kg + 21.7 \, kg + 2.9 \, kg + 16.8 \, kg
\]

\[
= 1295 \, kg
\]

This mass equals a self-weight of:

\[
q = \frac{1295 \, kg \times 9.81 \, m \, s^2}{4.5 \, m} = 2.82 \, \frac{kN}{m}
\]

\[ \text{(4.27)} \]

### 4.3.6.2 Strength Test Procedure

The entire test procedure was very similar to test number two. The floor was loaded first in three cycles to 30 kN, 60 kN and 100 kN. Cracks were marked at 60 kN, 100 kN and 150 kN. The strain gauges were placed at similar positions as in test one and two as shown in Figure 4-76.
Figure 4-76: Floor test 3, strain gauge positions
4.3.6.3 Test 3 Results

The load-deflection curve of test number three is plotted in Figure 4-77. In the beginning, the deflection at midspan increased proportional to the load. At about 120 kN, the deflection rate increased. Therefore, the graph changed from approximately a straight line to a curve. After the maximum load at 157 kN was reached, the load capacity of the floor decreased steadily to around 40 kN.

![Figure 4-77: Floor test 3, load-deflection diagram](image)

The crack pattern was similar to test one and two, and all cracks were below 0.2 mm as long as the floor behaviour remained in the elastic range. Figure 4-78 illustrates a crack measured with a crack table card at 0.1 mm.
At the final load stage, the same crack as in floor test two appeared at the height of the rebar from the middle to the end of the floor length. Directly at the point of failure, the crack was initially so small that it was barely seen. As the load increased, the crack also increased and was easily seen as in Figure 4-79.
Experimental Tests

Figure 4-80 and Figure 4-81 illustrate the horizontal crack at the bottom and the main crack in the middle of the slab, after the final load was passed and the mid deflection was increased even further.

![Figure 4-80: Horizontal and middle crack, floor test 3](image)

The concrete was removed from the steel section at one end to investigate the behaviour of the rebar at the welded end. As it can be seen in Figure 4-82, the rebar was not
disconnected from the 6-mm steel plate, instead, the tension forces in the rebar pulled the steel plate inwards and crushed the concrete and steel plate in that area.

Figure 4-82: Detail of locally crushed steel sheet at end of rebar, floor test 3

4.3.6.4 Test 3 Discussion

Before discussing the results from floor test three, the main changes to floor test two are repeated below.

- The concrete density and compression strength was reduced from 1858 kg/m³ and 19.6 MPa in floor test two to 1655 kg/m³ and 13.7 MPa in floor test three.
- The slab thickness was reduced from 150 mm to 100 mm.
- The rebars were welded at each end to the 6-mm steel plate as in floor test two, but the rebars were not tack-welded to the steel sheet.

Figure 4-83 shows the load-deflection diagram from floor test 3 together with the results from floor tests one and two. Three features are obvious:

1. The deflections from floor test three are higher than those from floor test two. In particular from a load of around 120 kN the graph from test three departs from linearity more rapidly than the graph from test two.
2. The maximum load in floor test three (157 kN) is significantly smaller than the maximum load from floor test two (186 kN).
Experimental Tests

3. The behaviour after passing the peak load is not ductile as in floor test two. Instead, the load decreases rapidly after the maximum load was reached. The load shedding is about 70% of the floor strength, before the floor stabilises at a load plateau of about 30 kN.

![Figure 4-83: Floor test 3 load-deflection diagram compared with floor test 1 and 2](image)

Due to the fact that the concrete compression strength and the slab thickness do not have much influence on floor strength and stiffness, the reasons for the differences in behaviour compared with floor test two can be found in the slip behaviour of the rebar.

Figure 4-84 to Figure 4-87 show the recorded strain data from the two rebars. Similar to floor test two, the proportional part at low load levels is marked with a red line. Figure 4-85 to Figure 4-87 show the rebar behaviour at the ends of the bars. In each diagram, the rebar starts slipping at a certain load level, indicated by the rapid increase in strain. This is the same behaviour as discussed in section 4.3.5.4 for floor test two.

The major difference to floor test two can be seen in Figure 4-84. The strain at midspan in rebar one increased proportionally with the load, until the load level of 123 kN was reached. From this point onwards, the strain stayed relatively constant during the time...
when the load was increased. The short blue line parallel to the red line in the range of 130 kN to 150 kN is the result of removing the load at this time for marking the cracks and then reapplying it.

![Figure 4-84: Strain measured on both rebars at midspan, floor test 3](image)

The rapid change in strain at 123 kN can be interpreted as the time when the slip in rebar one reached the region at midspan. It is a similar effect to the slip described in section 4.3.5.4 for the end parts of the rebar. The difference is that this time the strain values do not increase compared to the behaviour before the slip starts, instead, they decrease. This can be explained as follows: when the region of slip reaches midspan, the effective anchorage of the rebar ceases, which results in less tension and therefore, less strain in the rebar. At the end of the rebars, instead, the tension in the rebar should be close to zero for the entire test if the embedment does not fail. At the time slip occurred, the tension in the rebar was increasing, because the rebar was still welded to the 6-mm steel plate. The increasing tension was, of course, accompanied by increasing strain.
Figure 4-85: Recordings from strain gauge on channel 36 (end A rebar 1), floor test 3

Figure 4-86: Recordings from strain gauge on channel 15 (end B rebar 1), floor test 3
As a result of the slip extending to midspan, the theoretical bending-moment capacity could not be reached. This is the reason why the maximum load for floor test three is less than the maximum load for floor test two as shown in Figure 4-83.

The slip extending to midspan also resulted in a higher deflection in floor test three when compared to floor test two. In particular, after 123 kN, when the slip area reached midspan, the deflection was increasing significantly as seen in Figure 4-83.

The very ductile behaviour in floor test two clearly required a good connection between rebar and steel sheet. In floor test three, the rebar was only connected with small gauge-tie-wire to the steel sheet as shown in Figure 4-73. This was adequate during construction to connect the steel parts as a single, stiff, stable unit. However, during the test, the wire connection proved too weak to keep the rebar connected to the steel sheet. After the horizontal shear failure occurred at the maximum load, as shown in Figure 4-79, the rebars debonded from the concrete. Therefore, the tension forces from the rebars were missing, leading to a large decrease in the bending-moment capacity and the load capacity of the floor.
In floor test two, the horizontal shear failure occurred in a similar way as shown in Figure 4-53. However, the tack-welding of the rebars to the steel sheet in this test resulted in a much better connection than was achieved with the wire only. It is considered that the rebars were still able to carry loads after the horizontal shear failure occurred, because they were connected to the steel sheet. The connection directly to the concrete might be destroyed, but the spot-welded points to the steel sheet were still intact. As the steel sheet was connected to the concrete, the rebars were indirectly also still connected to the concrete. The additional anchorage provided by welding the rebar ends to the 6-mm steel plate also contributed to the strength retention after peak load was passed.

Therefore, the system in floor test two could carry a very high load of about 150 kN for a long period, even after the maximum load was reached. After the capacity of the spot-welded points was reached, the load level in floor test two decreased to about 40 kN, a similar load level to that of floor test three at the same stage.

The connection between a rebar and the concrete is generally better when using a high, rather than low, density concrete. Therefore, the concrete type used in floor test two certainly helped to achieve a better connection compared to test three. As the concrete type and the tack-welding points were both changed between test two and test three, it is not totally clear which had the greater influence on the different results. Probably, it is a combination of both. If the concrete compression strength is too weak, the concrete at the member ends, where the ends of the rebars are anchored, becomes prone to crushing, leading to a reduction in strength. Therefore, it is recommended not to use a weaker concrete type than that used in the second floor test, until the suitability of the concrete type is verified by further testing.

Similar to the strain analysis at midspan for floor test two (Figure 4-59 to Figure 4-65), the following diagrams (Figure 4-88 to Figure 4-93) show the recorded strains of the rebars, steel sheet and concrete compared to the theory in 10-kN steps for the third test.
4.3 T-Section

Figure 4-88: Strain measurements for test 3 from 10 to 30 kN
Figure 4-89: Strain measurements for test 3 from 40 to 60 kN
Figure 4-90: Strain measurements for test 3 from 70 to 90 kN
Figure 4-91: Strain measurements for test 3 from 100 to 120 kN
4.3 T-Section

Figure 4-92: Strain measurements for test 3 from 130 to 150 kN
In each graph the dark blue line represents the strain readings from the steel sheet, the light blue line the concrete strain and the two black crosses on red backgrounds the strain in the rebar at midspan. The red line shows the strain distribution in the cross-section according to conventional theory as implemented in the Excel sheets.

The readings of channel 17 look suspect in the shown diagrams. If the readings were correct, it would show that the Bernoulli hypothesis does not hold even for the steel sheet alone and would indicate a horizontal crack in the steel sheet even before the floor is loaded (or at a very low load level of 9.2 kN). This is very unlikely. It is more likely that channel 17 has failed and is not measuring the correct strain on the steel sheet. Therefore, this channel is not investigated further and is taken out of the analysis, leaving the steel sheet strain to be interpolated between channels 21 and 19 by the dashed line in Figure 4-88 to Figure 4-93. The diagrams appear to indicate that the readings for channel 17 are zero for the entire test. However, this is not true, they are only very low as Figure 4-94, drawn to a different scale, shows. A possible explanation is that while compacting the concrete with a rod, the rod may have accidently hit and de-bonded the strain gauge, channel 17, from the steel sheet. If this has happened, channel 17 would not have recorded the strain in the steel sheet, instead, it may have recorded the strain in the concrete,
assuming it bonded to the wet concrete as it cured. This theory is supported by the fact that the values of channel 17 match very well with the strain readings in the concrete (light blue line in Figure 4-88 to Figure 4-93).

![Figure 4-94: Load-strain diagram for channel 17 (steel sheet), test 3](image)

The biggest change in the strain gauge measurements compared to test 2 is that the light blue line, the concrete strain, does not match closely with the steel sheet strain, as it was the case in the second experimental test. Whereas the steel sheet and rebar strains agree well up to a load of 110 kN with the predicted theory line, the concrete readings do not. The readings indicate that straight from the beginning of the test the cross section is not working as a fully connected composite section without slip. The Bernoulli hypothesis does clearly not hold over the entire composite section. It is apparent that components of the cross-section are bending more or less independently, rather than in a combined, composite mode as intended. The strain readings show that there is incompatibility between the strains in the steel sheet and the surrounding concrete. The steel sheet, considered in isolation, has a strain distribution that is close to linear up to a load of about 120 kN, indicating that it is bending in accordance with engineering beam theory and in close agreement with the theoretical model. The weak, lightweight concrete was unable to
Experimental Tests

provide connectivity and strain compatibility between the different components and developed a compression region in the concrete web, extending to within 200 mm of the beam bottom from the start of the loading (Figure 4-88).

The beam action initially consists of tensile forces developed in the lower part of the sheet steel web and the two rebars, in equilibrium with compressive forces in the upper part of the web sheet and in the concrete from the top of the beam down to 200 mm above the beam bottom. In Figure 4-91 it can be seen that the strain in the bottom of the web sheet (channel 19) begins to decrease, eventually going into compression at loads above 135 kN (Figure 4-92). The sudden change in behaviour can be seen more clearly in Figure 4-95.

Possible reasons for the observed behaviour are that the steel sheet cracked at a load of 108 kN (Figure 4-95), with the crack initiating at the web opening closest to the channel 19 strain gauge (Figure 4-96). The strain at channel 19 was 1217 µε when the crack appeared to start, compared with a yield strain of 2000 µε. The stress raisers at the corners of the web opening would have created conditions suitable for crack propagation. Such a crack would explain the drop-off in strain shown in Figure 4-95.

The increasing difference between the web sheet strain and the concrete strain are indicative for the debonding and slip occurring between the steel and concrete. Slip was also measured in one rebar, starting at 123 kN (Figure 4-84). Debonding and slip of the steel (rebars and lower web sheet) and degradation of the concrete would result in the rebars relying increasingly on the 6 mm steel brackets at each end for anchorage. The return path of the compressive forces generated at the end anchorages would no longer be able channelled through the concrete and would instead find its way through the lower web sheet (stabilised to remain in plane by the surrounding concrete), resulting in the observed compression strain at channel 19.
Figure 4-95: Load-strain diagram for channel 19 (steel sheet), test 3

Figure 4-96: Location of channel 19 at midspan
Experimental Tests

In parallel, when channel 19 is changing from tension to compression, the strain-to-load-ratio is increasing at 133 kN for channel 21 (Figure 4-97 and Figure 4-88 to Figure 4-93). This can be explained as a resulting action from the behaviour mentioned before. If the bottom part of the steel sheet is not in tension anymore, cracked or de-bonded from the concrete, the entire cross-section is less effective and the remaining smaller steel sheet at the top has to carry higher strains.

![Figure 4-97: Load-strain diagram for channel 21 (steel sheet), test 3](image)

4.3.6.5 Test 3 Conclusions

The following conclusions can be made from the third experimental floor test:

- The maximum applied load is smaller in test three than in test two. The reason for this is the measured slip between rebar and concrete at midspan. In test two, no slip at midspan occurred, and the theoretical maximum value was reached.
- Because of the slip at midspan, the deflection begins to increase significantly at load levels higher than about 120 kN, as seen in Figure 4-83. The recorded slip at midspan with 123 kN, seen in Figure 4-84, matches this finding.
• The Bernoulli hypothesis is not fulfilled for the third test. The concrete chosen for the third test is apparently too weak to keep the cross-section together as one composite section. Therefore, two separated neutral axes (one for the concrete and one for the steel sheet) were recorded during the test. This is reducing the stiffness and the strength of the floor.

• Slip was also measured at the ends of the rebars, similar to test number two. This slip behaviour also contributes to the deflection increase.

• The behaviour after the peak load was much less ductile. Instead, the load decreased quite rapidly after the maximum load was reached. It is considered that the inadequate connection between the rebars and steel sheet (provided by tack-welds in test two) was the main reason for this behaviour. At the point of maximum load, the concrete cracked rapidly at the height of the rebars in the same way as in the second experimental test. As a result, the connection between rebar and concrete was destroyed. But in the second test, the rebars were still tack-welded to the steel sheet. As the steel sheet was very well connected to the concrete at all times, the rebars were also still indirectly connected to the floor and held in their intended positions in the cross-section. Therefore, the rebars were able to carry tension forces, even after the floor failed and the rebars were disconnected from much of the surrounding concrete.

In the third floor test, this connection was missing. After connection failure, the rebars were no longer able to carry the tension forces, leading to an overall brittle failure mode.

• The connection to the rebar is generally better in a high density concrete than in a low density concrete. Because the concrete density and the tack-welding points were changed at the same time, it is not totally clear how big the influence of each factor, concrete density and strength, is on the resulting behaviour of test three.

• The smaller slab thickness of 100 mm instead of 150 mm did not seem to have any adverse effects on performance.

4.4 NATURAL FREQUENCY

General Preparations
Experimental Tests

Before each floor element was tested to the point of failure, the natural frequency was analysed. The floor was stimulated in vibration by a sledge hammer, which was dropped from a height of about 500 mm above the centre of the floor. To protect the concrete surface, the hammer did not hit the surface directly, but instead, the hammer hit a 40 mm thick piece of wood, which was placed on the floor surface. The generated vibration was measured using two accelerometers, LSBC-.5g from Jewell Instruments. One was placed in the middle of the floor and the other at one quarter of the floor length.

A typical test setup can be seen in Figure 4-98.

![Figure 4-98: Natural Frequency test, test setup](image)

The results of the tests for floor one to three are provided in Figure 4-99 to Figure 4-101. The natural frequencies can be read off directly from the diagrams. For the first floor, the diagram shows 26 waves with a time interval of one second, which gives a natural frequency of 26 Hz. Floor test two shows a natural frequency of 19.5 Hz, and floor test three is vibrating at a frequency of 22 Hz.
4.4 Natural Frequency

Figure 4-99: Floor test 1, free vibration response to impulse (vertical axis proportional to acceleration)

Figure 4-100: Floor test 2, free vibration response to impulse (vertical axis proportional to acceleration)
At the completion of the test of the second floor specimen, the slab was broken in the middle. Due to the horizontal crack at the bottom as shown in Figure 4-53, one half of the slab was destroyed. However, the other half appeared as it did before the test. The hair cracks in the web of the floor were, of course, still present, but they had closed again after unloading, and no bigger concrete pieces were chipped off.

One of the big advantages of this floor type is the simple and strong end connection. As the floor slab had failed as intended in the middle, one half remained intact, and it was decided to test the end connection separately on the same specimen.

The idea was to clamp and hold the floor together at about half a metre from the end, as a jack pushed the SHS from the bottom upwards. A sketch of the end section is shown in Figure 4-102. Therefore, the static system was a cantilever, which was vertically pushed upwards at the end. Figure 4-103 demonstrates the setup before the test.
The load-displacement diagram in Figure 4-104 plots the up pushing load from the jack against the displacement of the jack or the SHS. It shows that the behaviour after failure is relatively symmetric to the loading curve, which results in a wide outstretched plateau at the maximum load level.
During loading, only a few new cracks were established. There was, more or less, only one crack beginning at the bottom corner of the web, inclining at an angle of about 45 degrees. This crack was about 0.1 mm or 0.2 mm wide before the final load was reached and widened up as seen in Figure 4-105. The other red and blue marked cracks in the photo were from the previous floor test.
Even after the test, the top surface was not damaged at all, apart from a few hair cracks as demonstrated in Figure 4-106.

![Figure 4-106: Topview after the test](image)

The final load of the total slab for floor test two was 186 kN. This proved that the end-connection was capable of carrying at least 93 kN.

When the end-connection was tested separately, as shown in Figure 4-104, it proved that the end connection alone is significantly stronger. The final load capacity at the support for this test was, at 239 kN, much higher than the maximum load the support had to resist beforehand during the entire floor test, at 93 kN.
5 THEORETICAL MODELLING

5.1 INTRODUCTION

Objectives of this chapter are:

- To describe the assumptions and theory used to analyse a floor element, consisting of a composite assembly of fibre-reinforced concrete, conventional steel reinforcing bars and thin-gauge cold-formed steel sheet.
- To apply the theoretical model to the specific cases of the three experimental floor tests described in chapter 4 and to compare the model predictions with the observed results.
- Of special interest, to examine the load-deflection curve at midspan as measured in the experimental tests.

The approach taken to the development of a theoretical model was to make a small number of generally well-tested assumptions and then, apply them very carefully and precisely to the analysed floor units. The resulting model should then be simple and easily understood, and preferably capable of use in a largely manual calculation mode. Due to variable floor behaviour under different load levels and complex modelling effects such as the concrete cracking, on individual section properties, it became necessary to employ computational assistance in the form of an Excel spreadsheet. Initially used just to check hand calculations, the Excel spreadsheet’s facilities were gradually exploited to enable more precise calculations and optimisations to be made. Eventually, the spreadsheet was able to calculate the entire load-deflection curve. The use of Excel had the following benefits:

- Although a hand calculation was theoretically possible, for complex situations and for optimising processes it becomes too time-consuming. Excel is able to organise the theory from the hand calculation more effectively.
- Excel spreadsheets are essentially transparent, enabling users to view or to change the data or calculation process, unlike finite-element programs, which operate more like “black boxes”. All equations in the Excel program are programmed into cells of the worksheet. By clicking on the cells, it is easy to review or to change the
Theoretical Modelling

equations. A general understanding of Excel spreadsheets is necessary, and for different floor assumptions, manual adjustments of Excel cells may be required.

- Excel is very fast for the relatively simple model developed here. As the equations for each cell are generally simple, the results can be observed immediately. This is very helpful for optimisation processes, where a range of different input values have to be investigated. When using a finite-element program, it is invariably more time consuming to obtain results.

Section 5.2 shows the calculation of the floors according to the New Zealand Standards and compares the results with the experimental tests.

In section 5.3, an example to determine the bending-moment capacity is given in terms of a hand calculation. The neutral axis determined by the Excel program is recalculated by hand.

The composite section properties are analysed in section 5.4. This section describes how the bending stiffness, $EI$, or the moment of inertia, $I$, can be calculated for a composite section. The calculation method for the shear area, $A_s$, is given, which is used for the calculation of shear deformation.

The calculation of the flexural and shear deflections using the unit load method is explained in section 5.5.

Section 5.6 describes the Excel program developed on the basis of the theory presented in the previous sections.

Section 5.7 calculates the natural frequency by theoretical equations and compares the results with the measured values in the experimental tests.
5.2 Design According to New Zealand Standards

5.2 DESIGN ACCORDING TO NEW ZEALAND STANDARDS

5.2.1 Introduction

The New Zealand Standards are applicable for specific types of structures, e.g. NZS 3101 for concrete structures or NZS 3404 for steel constructions. The proposed composite floor construction does not fit entirely in any one of New Zealand’s Standards. However, in the following sections it is attempted to design the proposed floor concept according to the New Zealand Standard by combining several standards and by adjusting some equations to the proposed concept.

For the flexural design in section 5.2.2 NZS 3101 (2006) is used as the basic standard. The equations are adjusted accordingly to consider the influence of the steel sheet on the bending moment capacity.

The resistance to shear is generated both by the concrete and the steel sheet. NZS 3101 determines shear capacity in a reinforced concrete member from a concrete contribution and a shear stirrup contribution. The concrete contribution is based on experimental testing of reinforced concrete with longitudinal and transverse (stirrup) reinforcement. For this floor product, the assumption is made that the steel will provide sufficient support to the concrete to allow the NZS 3101 provisions for concrete shear capacity to be used. The shear contribution of the steel sheet is determined using the rectangular flat plate provisions of NZS 3404.

It has to be noted that the standards are generally applicable only if some boundary conditions are maintained. Outside this range the standard may not reflect the analysed behaviour very accurately. For example, in NZS 3101 the concrete compressive strength should be between 25 MPa and 100 MPa (NZS 3101, 2006, 5.2.1) and the density should be in the range of 1800 kg/m³ to 2800 kg/m³ (NZS 3101, 2006, 5.2.2) which is only fulfilled for the first experimental test. The equations for the shear design in NZS 3101 are based on a mixture of theory and testing, incorporating empirical values gained from tests. As some construction details or material properties from the proposed floor concept may...
vary from the experimental testing used for the equation development in the standard, the results gained from the calculation according to this standard for the last two experimental tests can be used as a guideline only.

In order to compare the results with the experimental tests, all strength reduction factors used in the standard are set to 1.0. Actual material strengths are used instead of nominal strengths. For example the yield strength for the rebars obtained from the test certificate is used instead of the nominal value from the standard. However, to keep the equations as close as possible to the standards the subscript “n” is retained in the written calculations, although this refers to the actual capacity based on measured material properties.

Firstly, in section 5.2.2 and 5.2.3 the theory and equations being used are explained first. After that a specific calculation based on the parameters from the second floor test is shown as this test was the most important test which showed the close match to the expected floor behaviour. At the end of each section the corresponding analyses of the first and third tests are shown for comparison.

### 5.2.2 Flexural Strength

The flexural strength is calculated according to NZS 3101. Where necessary the equations were adjusted to allow the contribution of the steel sheet to the flexural strength.

The flexural strength requirement according to NZS 3101 (2006, Eq.7-1) is given as follows:

$$M^* \leq \phi M_n$$  \hspace{1cm} (5.1)

Where $M^*$ is the design moment at the section at ultimate limit state, $\phi$ is the strength reduction factor and $M_n$ is the nominal flexural strength of the section.

In order to compare the calculated results with the results from the experiment, the strength reduction factor and all other safety factors are set to 1.0.
According to 7.4.2.7 of NZS 3101 an equivalent rectangular concrete stress distribution is used as shown in Figure 5-1 which is based on a maximum strain value of 0.003 for concrete in compression. The tensile strength of concrete is neglected as recommended in 7.4.2.5 (NZS 3101, 2006). The stress-strain relationship for the reinforcement should be bilinear (NZS 3101, 2006, 7.4.2.4) with $E_s = 200000$ MPa (NZS 3101, 2006, 5.3.4) and a constant stress value of $f_y$ after yielding.

![Figure 5-1: Equivalent rectangular concrete stress distribution and resulting forces](image)

As shown in Figure 5-1, $a$ is the depth of the equivalent rectangular stress block which may be taken as:

$$a = \beta c$$  \hspace{1cm} (5.2)

$\beta$ is a factor defined in 7.4.2.7 (NZS 3101, 2006) and which shall be taken as 0.85 for a concrete compression strength $f'_c$ up to 30 MPa, which is applicable for all three tests.

$c$ is the distance from the extreme compression fibre to the neutral axis and $b$ is the width of the specimen in compression. If the effective width is smaller than the specimen width, the effective width has to be used instead. According to 9.3.1.2 (NZS 3101, 2006)
the effective width \( b_{\text{eff}} \) shall be equal to or less than the width of the web plus one-quarter of the span length of the beam:

\[
b_{\text{eff}} \leq 100 \, \text{mm} + \frac{4500 \, \text{mm}}{4} = 1225 \, \text{mm} \geq 1200 \, \text{mm}
\] (5.3)

This requirement is fulfilled, so that the total specimen width of \( b = 1200 \, \text{mm} \) may be used in design.

Furthermore, the standard requires that the effective compressive overhanging slab width on each side of the web shall not exceed eight times the minimum slab thickness or the total depth of the beam. Assuming that the overhanging slab width is measured up to the web fillet, this requirement is given in Eq. (5.4) for the first and second experimental floor test with a slab thickness of 150 mm. The requirement for the third floor test with a slab thickness of 100 mm is given in Eq. (5.5). The dimensions from the three floor tests are given in Figure 4-33, Figure 4-43 and Figure 4-70.

\[
500 \, \text{mm} \leq \min \left\{ \frac{8 \times 150 \, \text{mm}}{500 \, \text{mm}} \right\}
\] (5.4)

\[
450 \, \text{mm} \leq \min \left\{ \frac{8 \times 100 \, \text{mm}}{500 \, \text{mm}} \right\}
\] (5.5)

Both equations are also fulfilled, so that the total specimen width of \( b = 1200 \, \text{mm} \) may be used in all three floor designs and does not have to be reduced.

\( F_c \) in Figure 5-1 is the compression force and may be calculated as follows:

\[
F_c = \alpha_i f_c' ab
\] (5.6)

where \( \alpha_i \) is a factor defined in 7.4.2.7 (NZS 3101, 2006) and which shall be taken as 0.85 for a concrete compression strength \( f_c' \) up to 55 MPa.

\( F_t \) is the tension force resulting from the rebar and steel sheet forces:

\[
F_t = F_r + F_{s1} + F_{s2} + F_{s3}
\] (5.7)

with:

\[
F_r = f_y A_y
\] (5.8)
5.2 Design According to New Zealand Standards

\[ F_{s1} = f_{y, sheet} A_{sheet,1} \]  \hspace{1cm} (5.9)
\[ F_{s2} = f_{y, sheet} A_{sheet,2} \]  \hspace{1cm} (5.10)
\[ F_{s3} = f_{y, sheet} A_{sheet,3} \]  \hspace{1cm} (5.11)

where \( f_y \) is the yield strength of the reinforcement or steel sheet and \( A_s \) is the area of
flexural tension reinforcement or part of the steel sheet which is interrupted by the two
holes (Figure 5-1).

For equilibrium considerations, the compression force \( F_c \) and the tension force \( F_t \) have to
be the same:

\[ F_c = F_t \]  \hspace{1cm} (5.12)

Eq.(5.6) and Eq.(5.8) substituted in Eq. (5.12) gives the depth of the equivalent rectangular
stress block \( a \).

\[ \alpha_i f_y a b = f_y A_s \]
\[ \Rightarrow a = \frac{\alpha_i f_y b}{f_y} \]  \hspace{1cm} (5.13)

With the value \( a \) known, the distance \( c \) from the extreme compression fibre to the neutral
axis may be obtained from Eq.(5.2). If the neutral axis is known and all tension and
compression forces have been calculated, the nominal flexural moment \( M_n \) can be
calculated.

With the known location of the neutral axis the strain value at the height of the rebar can
also be deduced and checked that it exceeds yield strain. If this is verified, the assumption
that the reinforcement has yielded is true and the use of the yield stress of the
reinforcement \( f_y \) in the calculation is valid.

**Test 2**

The parameters used for the second test are as follows:

\[ b = 1200 \text{ mm} \]
\[ f_y = 532 \text{ MPa} \]
Theoretical Modelling

\[ f_{y,\text{sheet}} = 410 \text{ MPa} \]

\[ A_s = 2\pi (10 \text{ mm})^2 = 628.3 \text{ mm}^2 \]

\[ f'_c = 19.6 \text{ MPa} \]

The yielding strength of \( f_y = 532 \text{ MPa} \) is taken from Figure 5-2 which is based on the test certificate given in the appendix.

![Figure 5-2: Stress-strain distribution of the rebars used in the second and third test according to test certificate (Appendix 1 REBAR Test Certificates)](image)

The tension forces for the second test may be calculated as:

\[ F_t = F_r + F_{s1} + F_{s2} + F_{s3} \]  

\[ F_r = f_y A_s = 532 \text{ MPa} \times 628.3 \text{ mm}^2 = 334.3 \text{ kN} \]  

\[ F_{s1} = f_{y,\text{sheet}} A_{\text{sheet,1}} = 410 \text{ MPa} \times 55 \text{ mm} \times 1.6 \text{ mm} = 36.1 \text{ kN} \]  

\[ F_{s2} = f_{y,\text{sheet}} A_{\text{sheet,2}} = 410 \text{ MPa} \times 245 \text{ mm} \times 1.6 \text{ mm} = 160.7 \text{ kN} \]  

\[ F_{s3} = f_{y,\text{sheet}} A_{\text{sheet,3}} = 410 \text{ MPa} \times 15 \text{ mm} \times 1.6 \text{ mm} = 9.84 \text{ kN} \]
5.2 Design According to New Zealand Standards

\[ F_i = 334.3 \text{kN} + 36.1 \text{kN} + 160.7 \text{kN} + 9.84 \text{kN} = 540.9 \text{kN} \quad (5.19) \]

\[ F_c = F_i \]
\[ \Rightarrow \alpha_i f_c' ab = F_i \quad (5.20) \]
\[ \Rightarrow a = \frac{F_i}{\alpha_i f_c' b} = \frac{540.9 \text{kN}}{0.85 \times 19.6 \text{MPa} \times 1200 \text{mm}} = 27.06 \text{mm} \]

Check if reinforcement has yielded at this stage:

\[ c = \frac{a}{\beta_1} = \frac{27.06 \text{mm}}{0.85} = 31.83 \text{mm} \quad (5.21) \]

\[ \varepsilon_r = \frac{0.003 \times (460 \text{mm} - c)}{c} = \frac{0.003 \times (460 \text{mm} - 31.83 \text{mm})}{31.83 \text{mm}} = 0.040 \quad (5.22) \]

The reinforcement starts yielding at:

\[ \varepsilon_{r, yield} = \frac{f_y}{E_s} = \frac{500 \text{MPa}}{200000 \text{MPa}} = 0.0025 \quad (5.23) \]

which is lower than the calculated strain in Eq.(5.22). Therefore, the use of the lower characteristic yield strength \( f_y \) in the calculation is valid.

This may be accurate enough if a bi-linear stress strain curve is used as recommended in section 7.4.2.4 of NZS 3101 where the stress in the rebar after yielding should be considered as a constant value \( f_y \).

However, the test certificate (Appendix 1 REBAR Test Certificates) of the rebar used shows that the stress is not constant after yielding. The test certificate specifies only two points of the stress-strain curve, between these points linear behaviour is assumed (Figure 5-2).

Eq.(5.22) indicated a strain value of 4.0\% in the rebar. At this strain Figure 5-3 shows a slightly higher value of 568 MPa, in comparison to the value of 532 MPa used in the calculation beforehand.
Therefore, the calculation is repeated with a stress value in the rebar of 568 MPa.

\[
F_t = F_r + F_{s1} + F_{s2} + F_{s3}
\]

\[
F_r = f_y A_s = 568 \text{ MPa} \times 628.3 \text{ mm}^2 = 356.9 \text{ kN}
\]

\[
F_{s1} = f_{y,\text{sheet}} A_{\text{sheet,1}} = 410 \text{ MPa} \times 55 \text{ mm} \times 1.6 \text{ mm} = 36.1 \text{ kN}
\]

\[
F_{s2} = f_{y,\text{sheet}} A_{\text{sheet,2}} = 410 \text{ MPa} \times 245 \text{ mm} \times 1.6 \text{ mm} = 160.7 \text{ kN}
\]

\[
F_{s3} = f_{y,\text{sheet}} A_{\text{sheet,3}} = 410 \text{ MPa} \times 15 \text{ mm} \times 1.6 \text{ mm} = 9.84 \text{ kN}
\]

\[
F_t = 356.9 \text{ kN} + 36.1 \text{ kN} + 160.7 \text{ kN} + 9.84 \text{ kN} = 563.5 \text{ kN}
\]

\[
F_c = F_t
\]

\[
\Rightarrow \alpha \beta F_c b = F_t
\]

\[
\Rightarrow a = \frac{F_t}{\alpha \beta F_c b} = \frac{563.5 \text{ kN}}{0.85 \times 19.6 \text{ MPa} \times 1200 \text{ mm}} = 28.19 \text{ mm}
\]

\[
\Rightarrow c = \frac{a}{\beta} = \frac{28.19 \text{ mm}}{0.85} = 33.16 \text{ mm}
\]
5.2 Design According to New Zealand Standards

\[ \varepsilon_r = \frac{0.003 \times (460 \text{ mm} - c)}{c} = \frac{0.003 \times (460 \text{ mm} - 33.16 \text{ mm})}{33.16 \text{ mm}} = 0.039 \] (5.32)

The strain value of 0.039 is close enough to the assumed strain of 0.040, so that the stress in the rebar will not change significantly (see Figure 5-3). Therefore, no further iteration process is necessary and the continuing calculation is done with the a-value from Eq.(5.30).

In order to justify the assumption that the steel sheet has already yielded, the strain in the steel sheet has to be checked also. Figure 5-4 shows the strain distribution in the cross section as calculated at midspan. The blue lines are the parts of the steel sheet interrupted by the 50 and 75 mm hole. The strain distribution is based on the strain calculated in the rebar (Eq.(5.32)) and on the distance of the neutral axis from the top concrete surface (Eq.(5.31)).

Figure 5-4: Strain distribution in the cross section
The red line in Figure 5-4 indicates the strain of 0.002 at which the steel sheet starts yielding as shown in Figure 3-21. From Figure 5-4 it can be seen that the two bottom steel parts have yielded. The small top steel part has not yielded. In Eq.(5.28) it was assumed that the steel sheet has yielded, which is not true for the 15mm top part. However, the same calculation was done neglecting the top steel part \( F_s \), which is on the conservative side, and which showed a difference for all three floor tests for the nominal bending moment, \( M_n \), of less than 0.1%. As the real solution lies between these two results, the assumption that the entire steel sheet has yielded is acceptable.

The ultimate bending moment of the cross section may be calculated as follows (see Figure 5-1 and for dimensions Figure 5-18):

\[
M_n = F_t \left( 460 \text{ mm} - \frac{a}{2} \right) + F_{s1} \left( 442.5 \text{ mm} - \frac{a}{2} \right) + F_{s2} \left( 242.5 \text{ mm} - \frac{a}{2} \right) + F_{s3} \left( 37.5 \text{ mm} - \frac{a}{2} \right) \\
= 356.9 \text{kN} \left( 460 \text{ mm} - \frac{28.19 \text{ mm}}{2} \right) + 36.1 \text{kN} \left( 442.5 \text{ mm} - \frac{28.19 \text{ mm}}{2} \right) + 9.84 \text{kN} \left( 37.5 \text{ mm} - \frac{28.19 \text{ mm}}{2} \right) \\
= 211.5 \text{kNm}
\]

For comparison the same calculation is made for an ordinary Tee-section built with stirrups and no steel sheet:

\[
F_t = f_r = f_y A_s = 568 \text{ MPa} \times 628.3 \text{ mm}^2 = 356.9 \text{kN} \quad (5.34)
\]

\[
a = \frac{F_t}{\alpha f_c b} = \frac{356.9 \text{ kN}}{0.85 \times 19.6 \text{ MPa} \times 1200 \text{ mm}} = 17.85 \text{ mm} \quad (5.35)
\]

\[
z = 460 \text{ mm} - \frac{a}{2} = 460 \text{ mm} - \frac{17.85 \text{ mm}}{2} = 451.1 \text{ mm} \quad (5.36)
\]

\[
M_n = F_t z = 356.9 \text{kN} \times 451.1 \text{ mm} = 161.0 \text{kNm} \quad (5.37)
\]

\[
\frac{M_{n, \text{with steel sheet}}}{M_{n, \text{without steel sheet}}} = \frac{211.5 \text{kNm}}{161.0 \text{kNm}} = 1.31 \quad (5.38)
\]
5.2 Design According to New Zealand Standards

Therefore, the steel sheet increases the flexural strength by 31% compared to a comparable Tee-section built with ordinary stirrups.

The calculated bending moment of 211.5 kNm relates to a force at midspan of:

\[ F = \frac{4M}{l} = \frac{4 \times 211.5 \text{ kNm}}{4.5 \text{ m}} = 188.0 \text{ kN} \]  

(5.39)

In order to compare this force with the measured actuator force in the experimental test, the dead load of the floor has to be subtracted (0 kN in the experimental test relates to the floor loaded with its own dead load):

\[ M_{\text{line load}} = \frac{ql^2}{8} = \frac{Fl}{4} = M_{\text{point load}} \]

\[ \Rightarrow F = \frac{ql}{2} \]

(5.40)

With the line load \( q \) from Eq.(4.25) this gives:

\[ F_{\text{dead load}} = \frac{ql}{2} = \frac{4.35 \text{ kN} \times 4.5 \text{ m}}{2} = 9.79 \text{ kN} \]  

(5.41)

Therefore, the comparable load for the experimental floor test according to NZS 3101 would be:

\[ F = 188.0 \text{ kN} - 9.79 \text{ kN} = 178.2 \text{ kN} \]  

(5.42)

The calculated failure load is 4% lower than measured in the experiment:

\[ \frac{F_{\text{NZS 3101}}}{F_{\text{experiment}}} = \frac{178.2 \text{ kN}}{186 \text{ kN}} = 0.96 \]  

(5.43)

**Test 1**

The same procedure used for the calculation of the second test is used for the first test with the following parameters:

\[ b = 1200 \text{ mm} \]

\[ f_y = 523 \text{ MPa} \]

\[ f_{y,\text{sheet}} = 410 \text{ MPa} \]

\[ A_y = 4\pi(10 \text{ mm})^2 = 1257 \text{ mm}^2 \]

\[ f'_c = 34.3 \text{ MPa} \]
Theoretical Modelling

The stress-strain curve for the rebars used in the first test according to the test certificate is shown in Figure 5-5.

![Stress-strain distribution of rebars used in the first test according to test certificate](image)

Figure 5-5: Stress-strain distribution of the rebars used in the first test according to test certificate

(Appendix 1  REBAR Test Certificates)

The tension forces may be calculated as follows:

\[ F_t = F_r + F_{s1} + F_{s2} + F_{s3} \]  \hspace{1cm} (5.44)

\[ F_r = f_y A_s = 523 \text{ MPa} \times 1257 \text{ mm}^2 = 657.2 \text{ kN} \]  \hspace{1cm} (5.45)

\[ F_{s1} = f_{y,\text{sheet}} A_{\text{sheet,1}} = 410 \text{ MPa} \times 55 \text{ mm} \times 1.6 \text{ mm} = 36.1 \text{ kN} \]  \hspace{1cm} (5.46)

\[ F_{s2} = f_{y,\text{sheet}} A_{\text{sheet,2}} = 410 \text{ MPa} \times 245 \text{ mm} \times 1.6 \text{ mm} = 160.7 \text{ kN} \]  \hspace{1cm} (5.47)

\[ F_{s3} = f_{y,\text{sheet}} A_{\text{sheet,3}} = 410 \text{ MPa} \times 15 \text{ mm} \times 1.6 \text{ mm} = 9.84 \text{ kN} \]  \hspace{1cm} (5.48)

\[ F_t = 657.2 \text{ kN} + 36.1 \text{ kN} + 160.7 \text{ kN} + 9.84 \text{ kN} = 863.8 \text{ kN} \]  \hspace{1cm} (5.49)

\[ F_c = F_t \]

\[ \Rightarrow \alpha_f f'_c ab = F_t \]  \hspace{1cm} (5.50)

\[ \Rightarrow a = \frac{F_t}{\alpha_f f'_c b} = \frac{863.8 \text{ kN}}{0.85 \times 34.3 \text{ MPa} \times 1200 \text{ mm}} = 24.69 \text{ mm} \]
Check if reinforcement has yielded at this stage:

\[ c = \frac{a}{\beta_1} = \frac{24.69 \text{ mm}}{0.85} = 29.05 \text{ mm} \]  \hspace{1cm} (5.51)

\[ \varepsilon_r = \frac{0.003 \times (440 \text{ mm} - c)}{c} = \frac{0.003 \times (440 \text{ mm} - 29.05 \text{ mm})}{29.05 \text{ mm}} = 0.042 \geq 0.0025 \]  \hspace{1cm} (5.52)

\[ F_t = F_r + F_{s1} + F_{s2} + F_{s3} \]  \hspace{1cm} (5.53)

\[ F_r = f_y A_s = 560 \text{ MPa} \times 1257 \text{ mm}^2 = 703.9 \text{ kN} \]  \hspace{1cm} (5.54)

\[ F_{s1} = f_{y,sheet} A_{sheet,1} = 410 \text{ MPa} \times 55 \text{ mm} \times 1.6 \text{ mm} = 36.1 \text{ kN} \]  \hspace{1cm} (5.55)

\[ F_{s2} = f_{y,sheet} A_{sheet,2} = 410 \text{ MPa} \times 245 \text{ mm} \times 1.6 \text{ mm} = 160.7 \text{ kN} \]  \hspace{1cm} (5.56)

\[ F_{s3} = f_{y,sheet} A_{sheet,3} = 410 \text{ MPa} \times 15 \text{ mm} \times 1.6 \text{ mm} = 9.84 \text{ kN} \]  \hspace{1cm} (5.57)

\[ F_t = 703.9 \text{ kN} + 36.1 \text{ kN} + 160.7 \text{ kN} + 9.84 \text{ kN} = 910.5 \text{ kN} \]  \hspace{1cm} (5.58)
Theoretical Modelling

\[ F_c = F_i \]
\[ \Rightarrow \alpha \frac{f_c}{f} ab = F_i \]
\[ \Rightarrow a = \frac{F_i}{\alpha f_c b} = \frac{910.5 \text{ kN}}{0.85 \times 34.3 \text{ MPa} \times 1200 \text{ mm}} = 26.03 \text{ mm} \]
\[ \Rightarrow c = \frac{a}{\beta} = \frac{26.03 \text{ mm}}{0.85} = 30.62 \text{ mm} \]
\[ \varepsilon_r = \frac{0.003 \times (440 \text{ mm} - c)}{c} = \frac{0.003 \times (440 \text{ mm} - 30.62 \text{ mm})}{30.62 \text{ mm}} = 0.040 \]

The nominal bending moment of the cross section may be calculated as follows (see Figure 5-1 and for dimensions Figure 5-16):

\[ M_n = F_i \left(440 \text{ mm} - \frac{a}{2}\right) + F_{s1} \left(442.5 \text{ mm} - \frac{a}{2}\right) + F_{s2} \left(242.5 \text{ mm} - \frac{a}{2}\right) \]
\[ + F_{s3} \left(37.5 \text{ mm} - \frac{a}{2}\right) \]
\[ = 703.9 \text{ kN} \left(440 \text{ mm} - \frac{26.03 \text{ mm}}{2}\right) + 36.1 \text{ kN} \left(442.5 \text{ mm} - \frac{26.03 \text{ mm}}{2}\right) \]
\[ + 160.7 \text{ kN} \left(242.5 \text{ mm} - \frac{26.03 \text{ mm}}{2}\right) + 9.84 \text{ kN} \left(37.5 \text{ mm} - \frac{26.03 \text{ mm}}{2}\right) \]
\[ = 353.2 \text{ kNm} \]

For comparison the same calculation is done for an ordinary Tee-section built with stirrups and no steel sheet:

\[ F_i = F_r = f_y A_s = 560 \text{ MPa} \times 1257 \text{ mm}^2 = 703.9 \text{ kN} \]
\[ a = \frac{F_i}{\alpha f_c b} = \frac{703.9 \text{ kN}}{0.85 \times 34.3 \text{ MPa} \times 1200 \text{ mm}} = 20.12 \text{ mm} \]
\[ z = 460 \text{ mm} - \frac{a}{2} = 440 \text{ mm} - \frac{20.12 \text{ mm}}{2} = 429.9 \text{ mm} \]
\[ M_n = F_i z = 703.9 \text{ kN} \times 429.9 \text{ mm} = 302.6 \text{ kNm} \]
\[ \frac{M_{n, \text{with steel sheet}}}{M_{n, \text{without steel sheet}}} = \frac{353.2 \text{ kNm}}{302.6 \text{ kNm}} = 1.17 \]
Therefore, the steel sheet is able to increase the flexural strength by 17% compared to a comparable Tee-section built with ordinary stirrups.

The calculated bending moment of 353.2 kNm relates to a force at midspan of:

\[ F = \frac{4M}{l} = \frac{4 \times 353.2 \text{ kNm}}{4.5 \text{ m}} = 314.0 \text{ kN} \]  

(5.68)

With the line load \( q \) from Eq.(4.21) the dead load which has to be subtracted from Eq.(5.68) gives:

\[ F_{\text{dead load}} = \frac{ql}{2} = \frac{5.01 \text{ kN} \times 4.5 \text{ m}}{2} = 11.27 \text{ kN} \]  

(5.69)

Therefore, the comparable load to the experimental floor test after NZS 3101 would be:

\[ F = 314.0 \text{ kN} - 11.27 \text{ kN} = 302.7 \text{ kN} \]  

(5.70)

This is 2.5 times the value measured in the experiment:

\[ \frac{F_{\text{NZS 3101}}}{F_{\text{experiment}}} = \frac{302.7 \text{ kN}}{121 \text{ kN}} = 2.50 \]  

(5.71)

**Test 3**

The parameters for the third test are:

\[ b = 1200 \text{ mm} \]

\[ f_y = 532 \text{ MPa} \]

\[ f_{y,\text{sheet}} = 410 \text{ MPa} \]

\[ A = 4\pi (10 \text{ mm})^2 = 628.3 \text{ mm}^2 \]

\[ f' = 13.7 \text{ MPa} \]

The third test used rebars from the same test certificate as test two (Figure 5-2). Therefore, the tension forces may be calculated as follows:

\[ F_i = F_r + F_{s1} + F_{s2} + F_{s3} \]  

(5.72)

\[ F_r = f_y A = 532 \text{ MPa} \times 628.3 \text{ mm}^2 = 334.3 \text{ kN} \]  

(5.73)

\[ F_{s1} = f_{y,\text{sheet}} A_{\text{sheet,3}} = 410 \text{ MPa} \times 70 \text{ mm} \times 1.6 \text{ mm} = 45.9 \text{ kN} \]  

(5.74)
Theoretical Modelling

\[ F_{s2} = f_{y,\text{sheet}} A_{\text{sheet,2}} = 410 \text{ MPa} \times 245 \text{ mm} \times 1.6 \text{ mm} = 160.7 \text{ kN} \]  \hspace{1cm} (5.75)

\[ F_{s3} = f_{y,\text{sheet}} A_{\text{sheet,3}} = 410 \text{ MPa} \times 15 \text{ mm} \times 1.6 \text{ mm} = 9.84 \text{ kN} \]  \hspace{1cm} (5.76)

\[ F_t = 334.3 \text{ kN} + 45.9 \text{ kN} + 160.7 \text{ kN} + 9.84 \text{ kN} = 550.7 \text{ kN} \]  \hspace{1cm} (5.77)

\[ F_c = F_t \]

\[ \Rightarrow \alpha_f f'_c ab = F_t \]

\[ \Rightarrow a = \frac{F_t}{\alpha_f f'_c b} = \frac{550.7 \text{ kN}}{0.85 \times 13.7 \text{ MPa} \times 1200 \text{ mm}} = 39.41 \text{ mm} \]  \hspace{1cm} (5.78)

Check if reinforcement has yielded at this stage:

\[ c = \frac{a}{\beta_i} = \frac{39.41 \text{ mm}}{0.85} = 46.37 \text{ mm} \]  \hspace{1cm} (5.79)

\[ \varepsilon_r = \frac{0.003 \times (460 \text{ mm} - c)}{c} = \frac{0.003 \times (460 \text{ mm} - 46.37 \text{ mm})}{46.37 \text{ mm}} = 0.027 \geq 0.0025 \]  \hspace{1cm} (5.80)

Figure 5-7: Stress-strain distribution according to Figure 5-2 with an additional pair of values
5.2 Design According to New Zealand Standards

\[ F_i = F_r + F_{s1} + F_{s2} + F_{s3} \]  
\[ F_r = f_y A_s = 556 \text{ MPa} \times 628.3 \text{ mm}^2 = 349.3 \text{ kN} \]  
\[ F_{s1} = f_{y,\text{sheet}} A_{\text{sheet,1}} = 410 \text{ MPa} \times 70 \text{ mm} \times 1.6 \text{ mm} = 45.9 \text{ kN} \]  
\[ F_{s2} = f_{y,\text{sheet}} A_{\text{sheet,2}} = 410 \text{ MPa} \times 245 \text{ mm} \times 1.6 \text{ mm} = 160.7 \text{ kN} \]  
\[ F_{s3} = f_{y,\text{sheet}} A_{\text{sheet,3}} = 410 \text{ MPa} \times 15 \text{ mm} \times 1.6 \text{ mm} = 9.84 \text{ kN} \]  
\[ F_i = 349.3 \text{ kN} + 45.9 \text{ kN} + 160.7 \text{ kN} + 9.84 \text{ kN} = 565.7 \text{ kN} \]  
\[ F_c = F_i \]  
\[ a = \frac{F_i}{\alpha f_c b} = \frac{565.7 \text{ kN}}{0.85 \times 13.7 \text{ MPa} \times 1200 \text{ mm}} = 40.49 \text{ mm} \]  
\[ c = \frac{a}{\beta_1} = \frac{40.49 \text{ mm}}{0.85} = 47.63 \text{ mm} \]  
\[ \varepsilon_r = \frac{0.003 \times (460 \text{ mm} - c)}{c} = \frac{0.003 \times (460 \text{ mm} - 47.63 \text{ mm})}{47.63 \text{ mm}} = 0.026 \]  

The nominal bending moment of the cross section may be calculated as follows (see Figure 5-1 and Figure 5-20 for dimensions):

\[ M_n = F_r \left( 460 \text{ mm} - \frac{a}{2} \right) + F_{s1} \left( 450 \text{ mm} - \frac{a}{2} \right) + F_{s2} \left( 242.5 \text{ mm} - \frac{a}{2} \right) \]  
\[ + F_{s3} \left( 37.5 \text{ mm} - \frac{a}{2} \right) \]  
\[ = 349.3 \text{ kN} \left( 460 \text{ mm} - \frac{40.49 \text{ mm}}{2} \right) + 45.9 \text{ kN} \left( 450 \text{ mm} - \frac{40.49 \text{ mm}}{2} \right) \]  
\[ + 160.7 \text{ kN} \left( 242.5 \text{ mm} - \frac{40.49 \text{ mm}}{2} \right) + 9.84 \text{ kN} \left( 37.5 \text{ mm} - \frac{40.49 \text{ mm}}{2} \right) \]  
\[ = 209.2 \text{ kNm} \]  

For comparison the same calculation is done for an ordinary Tee-section built with stirrups and no steel sheet:

\[ F_i = F_r = f_y A_s = 556 \text{ MPa} \times 628.3 \text{ mm}^2 = 349.3 \text{ kN} \]  
\[ a = \frac{F_i}{\alpha f_c b} = \frac{349.3 \text{ kN}}{0.85 \times 13.7 \text{ MPa} \times 1200 \text{ mm}} = 25.00 \text{ mm} \]
Theoretical Modelling

\[ z = 460 \text{ mm} - \frac{a}{2} = 460 \text{ mm} - \frac{25.00 \text{ mm}}{2} = 447.5 \text{ mm} \quad (5.93) \]

\[ M_n = F_n z = 349.3 \text{ kN} \times 447.5 \text{ mm} = 156.3 \text{ kNm} \quad (5.94) \]

\[ \frac{M_{n, \text{with steel sheet}}}{M_{n, \text{without steel sheet}}} = \frac{209.2 \text{ kNm}}{156.3 \text{ kNm}} = 1.34 \quad (5.95) \]

Therefore, the steel sheet is able to increase the flexural strength by 34% compared to a comparable Tee-section built with ordinary stirrups.

The calculated bending moment of 209.2 kNm relates to a force at midspan of:

\[ F = \frac{4M}{l} = \frac{4 \times 209.2 \text{ kNm}}{4.5 \text{ m}} = 186.0 \text{ kN} \quad (5.96) \]

With the line load \( q \) from Eq. (4.27) the dead load which has to be subtracted from Eq.(5.96) gives:

\[ F_{\text{dead load}} = \frac{ql}{2} = \frac{2.82 \text{ kN} \times 4.5 \text{ m}}{2} = 6.35 \text{ kN} \quad (5.97) \]

Therefore, the comparable load to the experimental floor test after NZS 3101 would be:

\[ F = 186.0 \text{ kN} - 6.35 \text{ kN} = 179.7 \text{ kN} \quad (5.98) \]

This is 14% higher than the value measured in the experiment:

\[ \frac{F_{\text{NZS 3101}}}{F_{\text{experiment}}} = \frac{179.7 \text{ kN}}{157 \text{ kN}} = 1.14 \quad (5.99) \]

5.2.3 Shear Strength

According to NZS 3101 (2006, Eq.7-4) the design of cross-sections of members shall be based on:

\[ V^* \leq \phi V \quad (5.100) \]
5.2 Design According to New Zealand Standards

\( \nu^* \) is the design shear force at the section at ultimate limit state and \( \phi \) is the strength reduction factor. The strength reduction factors are set to 1.0 as in the flexural strength calculation.

\( V_n \) is the total nominal shear strength of the section in N, defined of a concrete and a steel part (NZS 3101, 2006, page 7-4):

\[
V_n = V_c + V_s
\]

(5.101)

where \( V_c \) is the nominal shear strength provided by the concrete in N and \( V_s \) is the nominal shear strength provided by the shear reinforcement in N.

\( V_c \) can be calculated from Eq.(5.102):

\[
V_c = \nu_c A_{cy}
\]

(5.102)

where \( A_{cy} \) is the effective shear area, the area used to calculate the shear stress in mm\(^2\), and \( \nu_c \) is the shear resisted by the concrete in MPa.

\( \nu_c \) is given by:

\[
\nu_c = k_a k_d \nu_b
\]

(5.103)

\( k_a \) is a factor allowing for the influence of aggregate size on shear strength and shall be taken as 0.85 if the maximum aggregate size is less than 10 mm (NZS 3101, 2006, page 9-6) which is true for all three tests.

\( k_d \) is a factor allowing for the influence of the member depth on the shear strength:

\[
k_d = \left( \frac{400}{d} \right)^{0.25}
\]

(5.104)

where \( d \) is the distance from the extreme compression fibre to the centroid of the longitudinal tension reinforcement, measured in mm.
Theoretical Modelling

$v_b$ is given in Eq.(5.105). The factor of 0.85 in front of $\sqrt{f_c}$ should be used for “sand-lightweight” concrete where the average splitting tensile strength is not specified (NZS 3101, 2006, page 9-7) which is applicable for all three tests.

$$v_b = \min \left\{ \frac{(0.07 + 10p_w)(0.85\sqrt{f'_c})}{0.2(0.85\sqrt{f'_c})} \right\} \text{ but } v_b \geq 0.08(0.85\sqrt{f'_c}) \tag{5.105}$$

where $f'_c$ is the specified compressive strength of concrete in MPa and $p_w$ is given in Eq.(5.106).

$$p_w = \frac{A_s}{b_wd} \tag{5.106}$$

$A_s$ is the area of flexural tension reinforcement in mm$^2$ and $b_w$ is the width of the web in mm.

$V_s$ in Eq.(5.101) is the nominal shear strength provided by the shear reinforcement and is reflecting the steel part in the shear calculation. NZS 3101 generally requires stirrups for beams if the shear resistance of the concrete alone is not sufficient. Therefore, the equations for $V_s$ are only valid for stirrups. However, to incorporate the shear resistance of the steel sheet in the calculation according to NZS 3101, the shear resistance of the steel sheet is calculated according to the Steel Structures Standard (NZS 3404, 1997).

In NZS 3404 the shear capacity $V_{sn}$ of a flat plate with non-uniform shear stress distribution may be calculated with Eq.(5.107) (NZS 3404, 1997, chapter 5.11.3). This value $V_{sn}$ is then used in Eq.(5.101) as the shear strength of the steel sheet ($V_s$).

$$V_s = V_{sn} = \frac{2V_{sn}}{0.9 + \left( \frac{f'_{sv}}{f'_{sw}} \right)} \leq V_{sw} \tag{5.107}$$

where $V_{sw}$ is the nominal shear capacity of the flat plate calculated assuming a uniform shear stress distribution and where $f'_{sv}$ and $f'_{sw}$ are the maximum and average design shear stresses in the web determined by a rational elastic analysis.
5.2 Design According to New Zealand Standards

$V_{uw}$ is given by chapter 5.11.2.2 (NZS 3404, 1997) for a stocky web, on the basis that the steel sheet is unable to buckle as it is imbedded in the concrete web.

$$V_{uw} = V_w = 0.6 f_y A_w$$  \hspace{1cm} (5.108)

where $f_y$ is the yield stress of the steel sheet and $A_w$ is the gross sectional area of the steel web.

**Test 2**

With the corresponding data from the second test the concrete contribution $V_c$ is determined as follows:

$$p_w = \frac{A_w}{b_w d} = \frac{628.3 \text{ mm}^2}{100 \text{ mm} \times 460 \text{ mm}} = 0.0137$$  \hspace{1cm} (5.109)

$$v_b = \min \left\{ (0.07 + 10 p_w) \left( 0.85 \sqrt{f_c} \right), 0.2 \left( 0.85 \sqrt{f_c} \right) \right\}$$

$$= \min \left\{ (0.07 + 10 \times 0.0137) \left( 0.85 \sqrt{19.6} \right), 0.2 \left( 0.85 \sqrt{19.6} \right) \right\}$$  \hspace{1cm} (5.110)

$$= \min \left\{ 0.78, 0.75 \right\} = 0.75$$

$$k_d = \left( \frac{400}{d} \right)^{0.25} = \left( \frac{400}{460} \right)^{0.25} = 0.97$$  \hspace{1cm} (5.111)

$$v_c = k_d k_w v_b = 0.97 \times 0.85 \times 0.75 = 0.62 \text{ MPa}$$  \hspace{1cm} (5.112)

$$V_c = v_c A_{sy} = 0.62 \times (100 \times 460) = 28.5 \text{ kN}$$  \hspace{1cm} (5.113)

The shear area for concrete strength is taken as the depth of the section times the width of the web. This is in accordance with NZS 3101. The gross sectional area of the steel web $A_w$ is calculated at midspan under consideration of the two holes in the steel sheet as shown in Figure 5-18:

$$A_w = (15 \text{ mm} + 245 \text{ mm} + 55 \text{ mm}) \times 1.6 \text{ mm} = 504.0 \text{ mm}^2$$  \hspace{1cm} (5.114)

$$V_{uw} = V_w = 0.6 f_y A_w = 0.6 \times 410 \text{ MPa} \times 504.0 \text{ mm}^2 = 124.0 \text{ kN}$$  \hspace{1cm} (5.115)
For a non-uniform shear stress distribution, NZS 3404 requires determination of the average shear stress, $f^*_v$, and maximum shear stress, $f^*_{vm}$, for input into Eq.(5.107). These change according to the internal actions which are a function of the applied loads, however, it is conservative to use the values applicable for a rectangular flat plate in shear, namely $f^*_{vm} / f^*_v = 1.5$.

$$V_s = V_{vm} = \frac{2V_{vm}}{0.9 + \left(\frac{f^*_{vm}}{f^*_v}\right)} \leq V_{uw}$$

$$= \frac{2 \times 124 \text{ kN}}{0.9 + \left(\frac{1.5}{1.0}\right)} \leq 124 \text{ kN}$$

$$= 103.3 \text{ kN} \leq 124 \text{ kN} \quad (5.116)$$

Therefore, the nominal shear strength provided by the steel sheet may be taken as:

$$V_s = 103.3 \text{ kN} \quad (5.117)$$

With $V_c$ from Eq.(5.113) it follows:

$$V_n = V_c + V_s = 28.5 \text{ kN} + 103.3 \text{ kN} = 131.8 \text{ kN} \quad (5.118)$$

The maximum measured actuator compression force at midspan in the second experimental test was 186 kN and hence the maximum shear force was 93 kN. Similar to the bending moment calculation, the dead load of the floor has to be subtracted from Eq.(5.118) in order to compare the results from the experimental test with the results from the calculation.

With the corresponding load at midspan for the dead load $F_{\text{dead load}}$ from Eq.(5.41) the total nominal shear strength of the section is:

$$V_n = 131.8 \text{ kN} - \frac{F_{\text{dead load}}}{2} = 131.8 \text{ kN} - \frac{9.79 \text{ kN}}{2} = 126.9 \text{ kN} \quad (5.119)$$

This results in a theoretical maximum actuator force needed to cause shear failure of:
5.2 Design According to New Zealand Standards

\[ F = 2 \cdot V_n = 2 \cdot 126.9 \text{kN} = 253.8 \text{kN} \quad (5.120) \]

which is 42% higher than the maximum calculated force of 178.2 kN (Eq.(5.42)) for the flexural resistance. Therefore, the floor should fail in bending before the shear capacity is reached.

If the ratio of applied shear to calculated capacity is higher than 60% and that for bending moment is higher than 75%, the interaction between shear and bending moment has to be considered according to Eq.(5.121) or to Figure 5-8, both from NZS 3404.

![Figure 5-8: Shear and bending moment interaction (NZS 3404, 1997, Figure C5.12.2)](image)

\[ V_v \text{ and } M_v \text{ are the nominal shear and the nominal section moment capacity. } V^* \text{ and } M^* \text{ are the design shear force and the design moment at the section at the ultimate limit state.} \]

\[ V'_{im} = V_v \left[ 2.2 - \left( \frac{1.6M^*}{\phi M_v} \right) \right] \quad (5.121) \]
Theoretical Modelling

\( V_{vm} \) is the nominal web shear capacity in the presence of bending moment and \( \phi \) is the strength reduction factor which is set to 1.0 as in the previous calculation.

The shear and bending interaction from NZS 3404 is applied to the steel sheet alone. Therefore, \( V_v \) in Eq.(5.121) is the nominal shear capacity of the steel sheet only, which is 103.3 kN taken from Eq.(5.116). With a 100% bending moment utilisation which means that \( M^* = M \), Eq.(5.121) becomes:

\[
\begin{align*}
V_{vm} &= V_v \left[ 2.2 - \left( \frac{1.6M^*}{\phi M_s} \right) \right] \\
&= 103.3 \text{ kN} \left[ 0.6 \right] \\
&= 62.0 \text{ kN}
\end{align*}
\]

Due to the 100% bending moment utilisation the allowable steel sheet shear force is reduced to 60% of its capacity.

Therefore, the total nominal shear strength \( V_n \) from Eq.(5.118) becomes:

\[
V_n = V_c + V_v = 28.5 \text{ kN} + 62.0 \text{ kN} = 90.5 \text{ kN}
\]

This gives a maximum midspan force of:

\[
F = 2V_n = 2 \times 90.5 \text{ kN} = 181.0 \text{ kN}
\]

If the dead load from Eq.(5.41) is subtracted this yields a maximum actuator load of:

\[
F = 181.0 \text{ kN} - 9.79 \text{ kN} = 171.2 \text{ kN}
\]

which is lower than the calculated maximum midspan force of 178.2 kN in Eq.(5.42) when the bending moment capacity is reached. Therefore the maximum applicable actuator force at the middle of the floor due to bending moment and shear interaction should be 171.2 kN.

However, with this reduced force the bending moment capacity is not fully utilised anymore. Therefore, the shear capacity does not have to be reduced by the maximum 40% and can be increased. The required force must lie between 171.2 kN (Eq.(5.125)) and
178.2 kN (Eq.(5.42)). This force can be calculated iteratively or by writing Eq.(5.121) in terms of the unknown force $F$.

Thus $V_m$ in Eq.(5.121) is replaced by $V^*$ which is given by:

$$V^* = \frac{F + F_{\text{dead load}}}{2} - V_c$$  \hspace{1cm} (5.126)

and $M^*$ is replaced by:

$$M^* = \frac{Fl}{4} + \frac{ql^2}{8}$$  \hspace{1cm} (5.127)

Substituting in Eq.(5.121) gives:

$$\frac{F + F_{\text{dead load}}}{2} - V_c = V_v + 2.2 - \left( \frac{1.6F}{4} + \frac{ql^2}{8} \right)$$

and solving for the unknown $F$:

$$\Rightarrow 0.5F = 2.2V_v - \frac{1.6V_v}{M_s} \left( \frac{Fl}{4} + \frac{ql^2}{8} \right) - \frac{F_{\text{dead load}}}{2} + V_c$$

$$\Rightarrow 0.5F + \frac{1.6V_v Fl}{4M_s} = 2.2V_v - \frac{1.6V_v ql^2}{8M_s} - \frac{F_{\text{dead load}}}{2} + V_c$$  \hspace{1cm} (5.129)

$$\Rightarrow F = \frac{2.2V_v - \frac{1.6V_v ql^2}{8M_s} - \frac{F_{\text{dead load}}}{2} + V_c}{0.5 + \frac{1.6V_v l}{4M_s}}$$

If all known values are substituted in Eq.(5.129), the unknown force $F$ may be calculated to:

$$F = \frac{2.2 \times 103.3 \text{kN} - \frac{1.6 \times 103.3 \text{kN} \times 4.35 \text{m}^2 \times (4.5 \text{m})^2}{8 \times 211.5 \text{kNm}} - \frac{9.79 \text{kN}}{2} + 28.5 \text{kN}}{0.5 + \frac{1.6 \times 103.3 \text{kN} \times 4.5 \text{m}}{4 \times 211.5 \text{kNm}}}$$

$$= 175.7 \text{kN}$$
Theoretical Modelling

This is the maximum theoretical actuator force which can be applied to the floor before failure occurs under consideration of shear and bending moment interaction. This force is 5.5% lower than the measured actuator force in the experimental test of 186 kN.

The position of the maximum value in the shear and bending moment interaction diagram is shown in Figure 5-9.

![Figure 5-9: Shear and bending moment interaction floor test 2](image)

Test 1

The same procedure used to calculate the shear strength of the second test can be adopted for the first test:

\[
p_w = \frac{A_t}{b_w d} = \frac{1257 \text{ mm}^2}{100 \text{ mm} \times 440 \text{ mm}} = 0.0286
\]

\[(5.131)\]
5.2 Design According to New Zealand Standards

\[
v_b = \min \left\{ \left( 0.07 + 10 p_w \right) \left( 0.85 \sqrt{f_y} \right), \frac{0.2}{0.85} \left( 0.85 \sqrt{f_y} \right) \right\} = \min \left\{ \left( 0.07 + 10 \times 0.0286 \right) \left( 0.85 \sqrt{34.3} \right), \frac{0.2}{0.85} \left( 0.85 \sqrt{34.3} \right) \right\} = \min \left\{ 1.77, 1.00 \right\} = 1.00 \tag{5.132}\]

\[
k_d = \left( \frac{400}{d} \right)^{0.25} = \left( \frac{400}{440} \right)^{0.25} = 0.98 \tag{5.133}\]

\[
v_c = k_d k_a v_b = 0.98 \times 0.85 \times 1.00 = 0.83 \text{MPa} \tag{5.134}\]

\[
V_c = v_c A_w = 0.83 \times (100 \times 440) = 36.5 \text{kN} \tag{5.135}\]

The gross sectional area of the steel web \( A_w \) is calculated at midspan taking into account the two holes in the steel sheet as shown in Figure 5-16:

\[
A_w = (15 \text{ mm} + 245 \text{ mm} + 55 \text{ mm}) \times 1.6 \text{ mm} = 504.0 \text{ mm}^2 \tag{5.136}\]

\[
V_{uw} = V_w = 0.6 f_y A_w = 0.6 \times 410 \text{ MPa} \times 504.0 \text{ mm}^2 = 124.0 \text{kN} \tag{5.137}\]

\[
\Rightarrow V_s = V_{sn} = \frac{2V_{uw}}{0.9 + \left( \frac{f_{uy}}{f_{uy}} \right)} \leq V_{uw} = \frac{2 \times 124 \text{ kN}}{0.9 + \left( \frac{1.5}{1.0} \right)} \leq 124 \text{ kN} \tag{5.138}\]

\[
= 103.3 \text{kN} \leq 124 \text{kN} \]

Therefore, the nominal shear strength provided by the steel sheet may be taken as:

\[
V_s = 103.3 \text{kN} \tag{5.139}\]

With \( V_c \) from Eq.(5.113) it follows:

\[
V_n = V_c + V_s = 36.5 \text{kN} + 103.3 \text{kN} = 139.8 \text{kN} \tag{5.140}\]

With the corresponding load at midspan for the dead load \( F_{\text{dead load}} \) from Eq.(5.69) the total nominal shear strength of the section after subtracting the dead load is:
This results in a theoretical maximum actuator force needed to cause shear failure of:

\[ F = 2 \cdot V_n = 2 \cdot 134.2 \text{kN} = 268.3 \text{kN} \] (5.142)

which is 11% lower than the maximum calculated force of 302.7 kN (Eq.(5.70)) for the flexural resistance but which is also 2.2-times higher than the maximum force of 121 kN recorded in the first experimental test.

With Eq.(5.129) the maximum force at midspan considering shear and bending moment interaction for the first test may be calculated as follows:

\[
F = 2.2V_v - \frac{1.6V_q l^2}{8M_s} - \frac{F_{\text{dead load}}}{2} + V_c \\
= 2.2 \cdot 103.3 \text{kN} - \frac{1.6 \cdot 103.3 \text{kN} \cdot 5.01 \frac{\text{kN}}{\text{m}} \cdot (4.5 \text{m})^2}{8 \cdot 353.2 \text{ kNm}} - \frac{11.27 \text{kN}}{2} + 36.5 \text{kN} \\
= 245.7 \text{kN} \] (5.143)

The position of the maximum value on the shear and bending moment interaction diagram is shown in Figure 5-10.
5.2 Design According to New Zealand Standards

Test 3
With the corresponding values for the third test the same calculation gives:

$$p_w = \frac{A_s}{b_w d} = \frac{628.3 \text{ mm}^2}{100 \text{ mm} \times 460 \text{ mm}} = 0.0137$$

$$v_b = \min \left\{ \left(0.07 + 10 p_w\right) \left(0.85 \sqrt{f_c}\right) \right\}$$

$$= \min \left\{ \left(0.07 + 10 \times 0.0137\right) \left(0.85 \sqrt{13.7}\right) \right\}$$

$$= \min \left\{ 0.65 \right\} = 0.63$$

$$k_d = \left(\frac{400}{d}\right)^{0.25} = \left(\frac{400}{460}\right)^{0.25} = 0.97$$

$$v_c = k_d k_a v_b = 0.97 \times 0.85 \times 0.63 = 0.52 \text{ MPa}$$

$$V_c = v_c A_v = 0.52 \times (100 \times 460) = 23.8 \text{ kN}$$
The gross sectional area of the steel web \( A_w \) is calculated at midspan taking into account the two holes in the steel sheet as shown in Figure 5-20:

\[
A_w = (15 \text{ mm} + 245 \text{ mm} + 70 \text{ mm}) \times 1.6 \text{ mm} = 528.0 \text{ mm}^2
\]  \hspace{1cm} (5.149)

\[
V_{vu} = V_w = 0.6 f_y A_w = 0.6 \times 410 \text{ MPa} \times 528.0 \text{ mm}^2 = 129.9 \text{ kN}
\]  \hspace{1cm} (5.150)

\[
\Rightarrow V_s = V_{vu} = \frac{2V_{vu}}{0.9 + \left( \frac{f_{mu}}{f_{vu}} \right)} \leq V_{vu}
\]

\[
= \frac{2 \times 129.9 \text{ kN}}{0.9 + \left( \frac{1.5}{1.0} \right)} \leq 124 \text{ kN}
\]  \hspace{1cm} (5.151)

\[
= 108.2 \text{ kN} \leq 124 \text{ kN}
\]

Therefore, the nominal shear strength provided by the steel sheet may be taken as:

\[
V_s = 108.2 \text{ kN}
\]  \hspace{1cm} (5.152)

With \( V_c \) from Eq.(5.113) it follows:

\[
V_n = V_c + V_s = 23.8 \text{ kN} + 108.2 \text{ kN} = 132.0 \text{ kN}
\]  \hspace{1cm} (5.153)

With the corresponding load at midspan for the dead load \( F_{\text{dead load}} \) from Eq.(5.97) the total nominal shear strength of the section after subtracting the dead load is:

\[
V_n = 132.0 \text{ kN} - \frac{F_{\text{dead load}}}{2} = 132.0 \text{ kN} - \frac{6.35 \text{ kN}}{2} = 128.8 \text{ kN}
\]  \hspace{1cm} (5.154)

This results in a theoretical maximum actuator force of:

\[
F = 2V_n = 2 \times 128.8 \text{ kN} = 257.7 \text{ kN}
\]  \hspace{1cm} (5.155)

which is 43% higher than the maximum calculated force of 179.7 kN (Eq.(5.98)) for the flexural resistance and 64% higher than the maximum recorded force in the experimental test of 157 kN.

With Eq.(5.129) the maximum force at midspan considering shear and bending moment interaction for the third test may be calculated as follows:
5.2 Design According to New Zealand Standards

\[
F = 2.2V_v - \frac{1.6V_q l^2}{8M_s} - \frac{F_{\text{dead load}} + V_c}{2} \div 0.5 + \frac{1.6V_q l}{4M_s}
\]

\[
= 2.2 \times 108.2 \text{kN} - \frac{1.6 \times 108.2 \text{kN} \times 2.82 \text{kN} \times (4.5 \text{m})^2}{8 \times 209.2 \text{kNm}} - \frac{6.35 \text{kN}}{2} + 23.8 \text{kN}
\]

\[= 176.6 \text{kN} \quad (5.156)\]

The position of the maximum value on the shear and bending moment interaction diagram is shown in Figure 5-11.

![Figure 5-11: Shear and bending moment interaction floor test 3](image)

The shear and bending moment interaction has to be considered if the maximum shear and maximum bending moment act at the same place or close to each other. In Figure 4-16 it is shown that this is the case for a single load at midspan as used in the experimental tests. However, the 9-m floor in reality will more likely be loaded by a line or area load in which the maximum shear force and the maximum bending moment are not at the same place.
Theoretical Modelling

(Figure 4-16). In this case, a shear and bending moment interaction calculation would probably not be necessary.

### 5.2.4 Summary of Flexural and Shear Strength Results

The flexural and shear strength results from section 5.2.2 and 5.2.3 are summarised in Table 5-1.

<table>
<thead>
<tr>
<th></th>
<th>Test 1</th>
<th>Test 2</th>
<th>Test 3</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Flexural Strength</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1: with steel sheet</td>
<td>353.2 kN Eq.(5.62)</td>
<td>211.5 kN Eq.(5.33)</td>
<td>209.2 kN Eq.(5.90)</td>
</tr>
<tr>
<td>2: without steel sheet</td>
<td>302.6 kN Eq.(5.66)</td>
<td>161.0 kN Eq.(5.37)</td>
<td>156.3 kN Eq.(5.94)</td>
</tr>
<tr>
<td>( \frac{1}{2} = )</td>
<td>1.17</td>
<td>1.31</td>
<td>1.34</td>
</tr>
<tr>
<td><strong>Shear Strength</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Concrete</td>
<td>36.5 kN Eq.(5.135)</td>
<td>28.5 kN Eq.(5.113)</td>
<td>23.8 kN Eq.(5.148)</td>
</tr>
<tr>
<td>Steel Sheet</td>
<td>103.3 kN Eq.(5.139)</td>
<td>103.3 kN Eq.(5.117)</td>
<td>108.2 kN Eq.(5.152)</td>
</tr>
<tr>
<td>Total</td>
<td>139.8 kN Eq.(5.140)</td>
<td>131.8 kN Eq.(5.118)</td>
<td>132.0 kN Eq.(5.153)</td>
</tr>
</tbody>
</table>

Due to the use of four rebars in test 1 instead of two, the flexural strength in test 1 is much higher than the flexural strength for tests 2 or 3. The shear strength provided by the concrete is decreasing from test 1 to test 3 as a result of the decreasing concrete strength. The steel sheet shear strength is very similar in all three tests. In the last test it is a little
higher as the steel sheet height was slightly increased (compare Figure 5-20 with Figure 5-18).

The results for the maximum applicable actuator force at midspan calculated in section 5.2.2 and 5.2.3 are summarised in Table 5-2 and are compared to the values measured in the experimental tests.

<table>
<thead>
<tr>
<th></th>
<th>Test 1</th>
<th>Test 2</th>
<th>Test 3</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Calculated Flexural</strong></td>
<td>302.7 kN</td>
<td>178.2 kN</td>
<td>179.7 kN</td>
</tr>
<tr>
<td><strong>Strength Alone</strong></td>
<td>Eq.(5.70)</td>
<td>Eq.(5.42)</td>
<td>Eq.(5.98)</td>
</tr>
<tr>
<td><strong>Calculated Shear</strong></td>
<td>268.3 kN</td>
<td>253.8 kN</td>
<td>257.7 kN</td>
</tr>
<tr>
<td><strong>Strength Alone</strong></td>
<td>Eq.(5.142)</td>
<td>Eq.(5.120)</td>
<td>Eq.(5.155)</td>
</tr>
<tr>
<td><strong>Calculated Combined</strong></td>
<td>245.7 kN</td>
<td>175.7 kN</td>
<td>176.6 kN</td>
</tr>
<tr>
<td><strong>Flexural/Shear</strong></td>
<td>Eq.(5.143)</td>
<td>Eq.(5.130)</td>
<td>Eq.(5.156)</td>
</tr>
<tr>
<td><strong>Experiment</strong></td>
<td>121 kN</td>
<td>186 kN</td>
<td>157 kN</td>
</tr>
<tr>
<td></td>
<td>Figure 4-39</td>
<td>Figure 4-49</td>
<td>Figure 4-77</td>
</tr>
<tr>
<td><strong>Calculated/Experiment</strong></td>
<td>0.49</td>
<td>1.06</td>
<td>0.89</td>
</tr>
</tbody>
</table>

It can be seen that in all three tests the interaction equation of shear and bending is crucial. Therefore, the failure mode is not a well-defined single failure mode in bending or shear. Instead, the shear and the bending moment capacity interact, so that at the failure load both capacities have reached their limits at the same time.

The calculated moment/shear capacities are made on the basis that the reinforcing bars and bottom of the steel sheet reach tensile yield under positive moment action. This condition
was met only in test number two, for which the ratio of experimental/ calculated applied load is 6%. This shows that the calculation method is robust and incorporates the key aspects of the floor system’s behaviour. For test number one and three, embedment failure of the rebar developed prior to yielding, limiting the experimental capacity to less than the calculated capacity. This was especially severe in test number one.

5.2.5 Slab Reinforcement

The slab spans between the two webs and cantilevers at each side of the double-Tee section, with reinforcement generally needed to resist the sagging and hogging moments. As the slab span is perpendicular to the floor span of the T-section, the design of the slab can be separated from the design of the T-section for vertical loads.

During the experimental testing described in chapter 4, the loading was aimed at testing the strength and stiffness of the T-beam along its main span direction. The concrete slab between the webs of the double-Tee specimen was not loaded or tested apart from carrying its self-weight. The rebar amount chosen for the experimental test was 12-mm rebars at a spacing of 500 mm (section 4.3.3.2). This gives a value per metre of:

\[ A_s = 226 \frac{mm^2}{m} \]  

(5.157)

This amount was chosen to maintain the structural integrity of the slab during fabrication and testing of the T-section.

The final slab would need to resist the chosen service load of 5 kN/m² and has to fulfil the required equations for the minimum amount of reinforcement according to NZS 3101.

As detailed on the following pages, 10-mm rebars at every 200 mm are required for the top and bottom reinforcement in the slab by a concrete cover of 25 mm which gives for each layer:

\[ A_s = 393 \frac{mm^2}{m} \]  

(5.158)
Additional holes are necessary in the 1.6 mm steel sheet to run the bottom and top layer through it. If the concrete cover for the top layer would be reduced to 20 mm, no additional holes would be needed, as the concrete cover to the 1.6 mm steel sheet is 30 mm and, therefore, the rebars could be placed directly on top of the sheet. However, the holes have an advantage in positioning the bars in pre-defined locations and holding them during concrete placement.

According to NZS 3101 (9.3.8.2.1) the minimum reinforcement shall be greater than:

\[
A_s = \max \left( \frac{\sqrt{f'_c}}{4f_y} b_w d, \frac{1.4 b_w d}{f_y} \right)
\]  

(5.159)

The positive contribution of the steel fibres is ignored.

For the second floor test, the specified compressive strength of concrete is: \( f'_c = 19.6 \text{ MPa} \),
the lower characteristic yield strength of longitudinal reinforcement is: \( f_y = 500 \text{ MPa} \),
the width of web/slab for 1 m is: \( b_w = 1000 \text{ mm} \),
the distance from extreme compression fibre to centroid of longitudinal tension reinforcement is: \( d = 120 \text{ mm} \),
and, therefore, the minimum reinforcement is:

\[
A_s = \max \left\{ \frac{\sqrt{19.6}}{4 \cdot 500} \cdot 1000 \cdot 120 = 266 \text{ mm}^2, \frac{1.4 \cdot 1000 \cdot 120}{500} = 336 \text{ mm}^2 \right\}
\]  

(5.160)

For the 10-mm rebars at every 200 mm chosen, this requirement is fulfilled:

\[
A_s = 393 \frac{\text{mm}^2}{\text{m}} \geq 336 \frac{\text{mm}^2}{\text{m}}
\]  

(5.161)

The slab has also to be designed for the dead and service load which is shown for the floor specimen from the second test with a concrete density of 1858 kg/m³ (Table 4-2) and a slab thickness of 0.15m. Therefore, the mass for a width of one metre will be:
Theoretical Modelling

\[ m = 0.15 \text{ m} \times 1.0 \text{ m} \times 1858 \frac{\text{kg}}{\text{m}^3} = 279 \frac{\text{kg}}{\text{m}} \]  

(5.162)

and the line load due to dead load will be:

\[ q_{\text{dead load}} = 279 \frac{\text{kg}}{\text{m}} \times 9.81 \frac{\text{m}}{\text{s}^2} = 2.73 \frac{\text{kN}}{\text{m}} \]  

(5.163)

A common service load of 5 kN/m^2 is used in this example, which yields for the 1-m width:

\[ q_{\text{service load}} = 5.0 \frac{\text{kN}}{\text{m}^2} \times 1.0 \text{ m} = 5.0 \frac{\text{kN}}{\text{m}} \]  

(5.164)

With load factors of 1.2 for the dead load and 1.5 for the live load, the total factored load is:

\[ q_{\text{total}} = 1.2 \times q_{\text{dead load}} + 1.5 \times q_{\text{service load}} = 1.2 \times 2.73 \frac{\text{kN}}{\text{m}} + 1.5 \times 5.0 \frac{\text{kN}}{\text{m}} = 10.78 \frac{\text{kN}}{\text{m}} \]  

(5.165)

The bottom reinforcement is calculated for the sagging moment from Eq. (3.2).

\[ M = -q \frac{(1.5 \text{ m})^2}{2} + q * 1.5 \text{ m} * 0.85 \text{ m} \]

\[ = -\frac{10.78 \frac{\text{kN}}{\text{m}} (1.5 \text{ m})^2}{2} + 10.78 \frac{\text{kN}}{\text{m}} * 1.5 \text{ m} * 0.85 \text{ m} = 1.62 \text{ kNm} \]  

(5.166)

With the simplification that the distance between the centroids of the compression force and the tension force is 0.9 d and with an effective depth, d, of 0.12 m, the tension force in the rebar can be calculated with the following equation.

\[ F_t \approx \frac{M}{0.9d} = \frac{1.62 \text{ kNm}}{0.9 \times 0.12 \text{ m}} = 15.01 \text{ kN} \]  

(5.167)

Therefore, the area of flexural tension reinforcement, \( A_s \), has to be:

\[ A_s = \frac{F_t}{f_y} = \frac{15.01 \text{ kN}}{500 \text{ MPa}} = 30.0 \text{ mm}^2 \]  

(5.168)
The rebar amount calculated in Eq.(5.168) was based on an effective depth of 0.12 m for a 0.15-m thick slab. For an effective depth of 0.07 m (for a 0.10-m slab) the amount would change to (dead load is assumed to be the same):

\[
F_i \approx \frac{M}{0.9d} = \frac{1.62 \text{kNm}}{0.9 \times 0.07 \text{ m}} = 25.71 \text{kN}
\]

\[
A_s = \frac{F_i}{f_y} = \frac{25.71 \text{kN}}{500 \text{ MPa}} = 51.4 \text{ mm}^2
\]

(5.169)

(5.170)

This required rebar amount is also easily fulfilled by the chosen 10-mm rebars at every 200 mm:

\[
A_s = 393 \text{ mm}^2 \geq 51.4 \text{ mm}^2
\]

(5.171)

With Eq.(3.1), Eq.(3.2) and Figure 3-5, it was shown that the hogging and sagging moment are the same for a constant distributed load. Therefore, the amount for the top reinforcement over the support has to be the same as calculated for the bottom reinforcement in Eq.(5.170).

The chosen 10-mm rebars are the principal reinforcement of the slab. Perpendicular to this direction (i.e. in parallel to the T-beam span direction), no structural reinforcement is needed in the slab. In this direction it would be cost-effective to use the steel fibres as crack control. As part of prototype production, crack control tests on the slab with flexural reinforcement transverse to the ribs and steel fibres only in rib direction would be the recommended option. If the NZS 3101 provisions for shrinkage and temperature are invoked, one layer of 8mm diameter rebars at 200 mm spacing for the top and bottom layer would be chosen, giving:

\[
A_s = 2 \times 251 \text{ mm}^2 = 502 \text{ mm}^2
\]

(5.172)

This is also in accordance with 8.8.1 of NZS 3101 (shrinkage and temperature reinforcement):

\[
\frac{A_s}{\text{gross concrete Area}} \geq \begin{cases} 
0.7 \\ f_y \\ 0.0014 
\end{cases}
\]

(5.173)
For a 150 mm slab this gives:

\[
\frac{A_s}{\text{gross concrete Area}} = \frac{502 \text{ mm}^2}{150 \text{ mm} \times 1000 \text{ mm}} = 0.0033 \geq \begin{cases} 0.0014 = \frac{0.7}{500} = \frac{0.7}{f_y} \quad (5.174) \\ 0.0014 \end{cases}
\]

Similar to the shear strength calculation in section 5.2.3 for the web, where the equations are explained and analysed for all three floor tests, the shear strength has to be checked also for the slab, even if shear reinforcement is rarely required for slabs.

In the following, it is shown that no shear reinforcement is necessary for the slab. The dimensions are taken from the most critical third floor test.

The area of flexural tension reinforcement for the slab, \(A_s\), was defined in Eq.(5.158) as:

\[
A_s = 393 \frac{\text{mm}^2}{\text{m}} \quad (5.175)
\]

and, therefore, for a width, \(b_w\), of 1 m:

\[
A_s = 393 \text{ mm}^2 \quad (5.176)
\]

According to section 9.3.9.3.4 of (NZS 3101, 2006), \(v_b\) may be calculated as:

\[
p_w = \frac{A_s}{b_w d} = \frac{393 \text{ mm}^2}{1000 \text{ mm} \times 70 \text{ mm}} = 0.0056 \quad (5.177)
\]

\[
\begin{align*}
v_b &= \min \left\{ \left(0.07 + 10 p_w\right) \left(0.85 \sqrt{f_c}\right) \right\} \\
&= \min \left\{ \left(0.07 + 10 \times 0.0056\right) \left(0.85 \sqrt{13.7}\right) \right\} \\
&= \min \left\{ 0.40 \right\} = 0.40
\end{align*}
\]

and for members with an effective depth equal to or smaller than 400 mm:

\[
k_d = 1.0 \quad (5.179)
\]
The value for $v_c$ and the nominal shear strength resisted by concrete, $V_c$, is given by:

$$v_c = \max \left\{ \frac{0.17 \times k_a \times \left(0.85 \times \sqrt{f'_c}\right)}{k_d k_a \times \nu_b} \right\}$$

$$= \max \left\{ \frac{0.17 \times 0.85 \times \left(0.85 \times \sqrt{13.7}\right)}{1.0 \times 0.85 \times 0.40} \right\}$$

$$= \max \left\{ \frac{0.45 \text{ MPa}}{0.34 \text{ MPa}} \right\} = 0.45 \text{ MPa}$$

$$V_c = v_c A_{cv} = 0.45 \times (1000 \times 70) = 31.5 \text{ kN}$$

(5.180)

(5.181)

For no shear reinforcement, the nominal shear strength of the section, $V_n$, equals the nominal shear strength provided by the concrete, $V_c$.

$$V_n = V_c + V_s = V_c = 31.5 \text{ kN}$$

(5.182)

The dead load, $q_{\text{dead load}}$, and service load, $q_{\text{service load}}$, for the third test are given in Eq.(5.184) and Eq.(5.185).

$$m = 0.1 \text{ m} \times 1.0 \text{ m} \times 1655 \frac{\text{kg}}{\text{m}^3} = 166 \frac{\text{kg}}{\text{m}}$$

(5.183)

$$q_{\text{dead load}} = 166 \frac{\text{kg}}{\text{m}} \times 9.81 \frac{\text{m}}{\text{s}^2} = 1.63 \frac{\text{kN}}{\text{m}}$$

(5.184)

$$q_{\text{service load}} = 5.0 \frac{\text{kN}}{\text{m}^2} \times 1.0 \text{ m} = 5.0 \frac{\text{kN}}{\text{m}}$$

(5.185)

With load factors of 1.2 for the dead load and 1.5 for the live load, the total factored load is:

$$q_{\text{total}} = 1.2 \times q_{\text{dead load}} + 1.5 \times q_{\text{service load}} = 1.2 \times 1.63 \frac{\text{kN}}{\text{m}} + 1.5 \times 5.0 \frac{\text{kN}}{\text{m}} = 9.46 \frac{\text{kN}}{\text{m}}$$

(5.186)

With the dimensions shown in Figure 3-5 and the linear distributed load from Eq.(5.186), the maximum design shear force located at the support is calculated as:
Theoretical Modelling

\[ V' = 9.46 \frac{kN}{m} \times 0.85 \text{ m} = 8.04 \text{ kN} \]  
(5.187)

According to section 7.5 of (NZS 3101, 2006), Eq.(5.188) is fulfilled without any need for shear reinforcement in the slab.

\[ V' = 8.04 \text{ kN} \leq 23.63 \text{ kN} = 0.75 \times 31.5 \text{ kN} = \phi V_n \]  
(5.188)

5.3 CALCULATION OF MOMENT-DEFLECTION CAPACITY

Section 5.2.2 presents the calculation of maximum flexural strength. However, the moment-deflection relationship under increasing loading also requires determination and that is presented in this section. To make it possible to predict the relatively complex behaviour of the floor specimen under bending by means of an analytical model, it is necessary to make some simplifications.

The following assumptions have been made for the entire chapter 5:

1. No slip between steel components and concrete
2. Plane sections are assumed to remain plane (Bernoulli hypothesis)
3. Influence of interaction between bending and shear stress is negligible.
4. The simplified material parameters from sections 3.3.2 and 4.2 are used.
5. The floor is modelled as a simply supported beam, with a span length of 4.5 m. The difference between that simplification and the real support at the SHS is analysed in section 6.4.
6. The shear lag effect and creep or shrinkage effects are neglected.

Example based on Floor Test 2

The procedure for the calculation of the bending-moment capacity is illustrated by detailed reference to an example based on the second experimental floor test. The dimensions of the cross-section are provided in Figure 5-12.
5.3 Calculation Of Moment-Deflection Capacity

For the theory demonstration, the loading situation is chosen where the rebars just start to yield.

The stress-strain diagram for reinforcement for the second and third test according to the test certificate is given in Figure 3-19 which is used in the Abaqus model. To simplify the Excel model, the spread sheet assumes a constant stress increase of 15 % after yielding which is also shown in Figure 3-19. Therefore, the yield strength of the reinforcement used in the Excel sheets is obtained from:

\[ f_y = 1.15 \times 500 \text{ N/mm}^2 = 575 \text{ N/mm}^2 \]  

(5.189)

The strain value in the rebar for this situation is shown in the following equation.

\[ \varepsilon_{\text{reinf}} = \frac{f_y}{E_s} = \frac{575 \text{ N/mm}^2}{205000 \text{ N/mm}^2} = 0.002805 \]  

(5.190)

That gives the first strain point on the strain diagram for the cross-section. Because we assume that the cross-section under load remains planar, we only need one other strain value to draw the strain diagram.
Theoretical Modelling

Usually the position of the neutral axis is unknown and has to be found in an iteration process, where the compression and tension forces have to stay in equilibrium. To initiate the process, the position of the neutral axis has to be roughly estimated at first. With the position of the neutral axis and the strain value from the rebars, the linear strain distribution for the whole cross-section can be drawn.

With the known strain distribution and with the material stress-strain diagrams, the stress distribution in the rebars, steel sheet and concrete can be deduced. Furthermore, with these stresses in the cross-section, the forces above and below the neutral axis can be calculated and their state of equilibrium checked.

If the tension forces are not equal to the compression forces, the assumption of the neutral axis was wrong, and the procedure has to be started again, with a changed assumption for the neutral axis. After a few iterations, the difference between tension and compression forces should be minimal, indicating that the correct neutral axis has been found.

To avoid performing this quite time-consuming iteration process by hand, the assumed position of the neutral axis is calculated beforehand with an Excel program, which is introduced in section 5.6. This computed value is later checked for correctness by the hand calculation in this section.

The Excel program predicts a position for the neutral axis of 90.45 mm from the top of the concrete surface. With that value and the calculated strain in the rebars (Eq.(5.190)), the strain diagram for the cross-section can be drawn as shown in Figure 5-13 to Figure 5-15. Because of the Bernoulli hypothesis, the strain distribution has to be linear.

Figure 5-13 to Figure 5-15 show the strain, stress and force distribution over the height of the cross-section for the reinforcement (Figure 5-13), the steel sheet (Figure 5-14) and the concrete (Figure 5-15). The strain distribution in all three figures has to be the same (since compatibility is assumed).

With the given strain distribution and the material diagrams from sections 3.3.2 and 4.2, the stress distribution for each material can also be drawn.
5.3 Calculation Of Moment-Deflection Capacity

As the rebars are located at only one height of the cross-section, the stress in the rebars at that height can simply be calculated by multiplying the strain at this position with the modulus of elasticity for steel. However, generally, the material stress-strain diagram from the sections are taken and turned by 90 degrees clockwise to show the stress distribution over the cross-section at different height points as seen Figure 5-14 and Figure 5-15. The material stress-strain diagrams used are shown in Figure 3-21 for the steel sheet and the simplified stress-strain diagram from Figure 4-9 for the concrete in tension.

The calculation is illustrated for a section of the floor unit near midspan. Figure 3-12 shows that at midspan of the floor slab there are two “windows”, one with a height of 50 mm and the other with 75 mm, which will weaken the steel sheet. These holes in the steel sheet result in holes in the stress diagram as well, as displayed in Figure 5-14.

As can be seen in the strain diagram for the steel sheet, the yield strain value of 0.002 is reached at one point. At that point the stress value in the steel sheet is:

\[ f_y = E \cdot \varepsilon = 205000 \text{ MPa} \cdot 0.002 = 410 \text{ MPa} \]  
(5.191)

That value is the yielding point for the steel sheet. Therefore, the stress distribution stays constant at a value of 410 MPa for all higher strain values than 0.002, which is demonstrated at the bottom part of the stress diagram for the steel sheet in Figure 5-14.

With the simplified material diagram for concrete in tension from Figure 4-9, the calculated strain distribution results in the plotted stress distribution in Figure 5-15. Because of the different slab and web thickness, which influences the concrete force, the stress diagram for concrete needs an additional separation at 350 mm from the bottom. The concrete fillets at the slab-web junction are neglected.

At this stage the strain and the stress distribution in the entire cross-section for all three materials, reinforcement, steel sheet and concrete, are calculated. In order to predict the bending moment for this cross-section, the stress diagrams from Figure 5-13 to Figure 5-15 have to be transformed to resultant forces. For this, the areas of the stress diagrams are split into simple shapes, which are subsequently replaced by forces acting at the respective centroids of the stressed areas.
Theoretical Modelling

The forces in the cross-section can be calculated by associating stresses with the corresponding areas, which are generally named $A$ in the equations below. Most of them represent triangular or rectangular parts of the stress diagram.

For the reinforcement in Figure 5-13 the resulting force is simply the stress value in the rebars multiplied by the area of the rebars as shown in Eq.(5.192).

For the steel sheet in Figure 5-14 the forces are:

- $F_{\text{sheet}, 1}$: the small part at the top of the stress diagram which is in compression
- $F_{\text{sheet}, 2}$: rectangular shape with a stress value of 45.98 MPa
- $F_{\text{sheet}, 3}$: triangular shape with height of 233.94 mm
- $F_{\text{sheet}, 4}$: rectangular shape with a stress value of 410 MPa (height =11.06 mm)
- $F_{\text{sheet}, 5}$: rectangular shape with a stress value of 410 MPa (height = 55 mm)

For the concrete in Figure 5-15 the forces are:

- $F_{\text{conc}}$: triangular shape at the top of the stress diagram which is in compression
- $F_{\text{conc}, 1}$: triangular shape with a stress value of 4.07 MPa
- $F_{\text{conc}, 2}$: small piece from the stress value at 1.2 MPa after the concrete cracked to the stress value of 1.123, which is the stress in the concrete at the width-change from slab to web (350 mm from bottom surface)
- $F_{\text{conc}, 3}$: very slim rectangular shape with a stress value of 0.144 MPa over a height of 350 mm
- $F_{\text{conc}, 4}$: triangular shape over a height of 350 mm

The precise values for the forces are given below.
Figure 5-13: Strain, stress and force diagrams for the reinforcement, floor test 2
Figure 5-14: Strain, stress and force diagrams for the steel sheet, floor test 2
Figure 5-15: Strain, stress and force diagrams for the concrete, floor test 2
Reinforcement:
Two rebars of 20 mm diameter were used for the second floor test. The force, which represents the stress behaviour for the two bars, is specified below:

\[
F_{\text{reinf}} = f_y * A = 575 \frac{N}{mm^2} * 628 mm^2 = 361.1 kN
\]  
(5.192)

Steel sheet:
The steel sheet has a thickness of 1.6 mm. Eq.(5.193) to Eq.(5.197) define the forces in the sheet.

\[
F_{\text{sheet}, 1} = f_y * A = \frac{1}{2} * \left( 94.06 \frac{N}{mm^2} + 70.72 \frac{N}{mm^2} \right) * 1.6 mm * 15 mm = 1.98 kN
\]  
(5.193)

\[
F_{\text{sheet}, 2} = f_y * A = 45.98 \frac{N}{mm^2} * 1.6 mm * 233.94 mm = 17.21 kN
\]  
(5.194)

\[
F_{\text{sheet}, 3} = f_y * A = \frac{1}{2} \left( 410 \frac{N}{mm^2} - 45.98 \frac{N}{mm^2} \right) * 1.6 mm * 233.94 mm = 68.13 kN
\]  
(5.195)

\[
F_{\text{sheet}, 4} = f_y * A = 410 \frac{N}{mm^2} * 1.6 mm * 11.06 mm = 7.26 kN
\]  
(5.196)

\[
F_{\text{sheet}, 5} = f_y * A = 410 \frac{N}{mm^2} * 1.6 mm * 55.0 mm = 36.08 kN
\]  
(5.197)

Concrete Tension:
The forces for the concrete in the tension zone are evaluated in the following equations.

\[
F_{\text{conc}, 1} = f_c * A = \frac{1}{2} * 4.07 \frac{N}{mm^2} * 1200 mm * 31.88 mm = 77.85 kN
\]  
(5.198)

\[
F_{\text{conc}, 2} = f_c * A = \frac{1}{2} * \left( 1.2 \frac{N}{mm^2} + 1.123 \frac{N}{mm^2} \right) * 1200 mm * 27.67 mm = 38.56 kN
\]  
(5.199)

\[
F_{\text{conc}, 3} = f_c * A = 0.144 \frac{N}{mm^2} * 100 mm * 350 mm = 5.04 kN
\]  
(5.200)

\[
F_{\text{conc}, 4} = f_c * A = \frac{1}{2} \left( 1.123 \frac{N}{mm^2} - 0.144 \frac{N}{mm^2} \right) * 100 mm * 350 mm = 17.13 kN
\]  
(5.201)
Concrete Compression:
If the cross-section is in equilibrium, the sum of all forces has to be zero. Except for the concrete compression force, the forces are calculated above. Therefore, the last missing force can be calculated with Eq.(5.202).

\[ F_{\text{conc}} = F_{\text{reinf}} - F_{\text{sheet}, 1} + F_{\text{sheet}, 2} + F_{\text{sheet}, 3} + F_{\text{sheet}, 4} + F_{\text{sheet}, 5} + F_{\text{conc}, 1} + F_{\text{conc}, 2} + F_{\text{conc}, 3} + F_{\text{conc}, 4} \]

\[ = 361.1 \text{ kN} - 1.98 \text{ kN} + 17.21 \text{ kN} + 68.13 \text{ kN} + 7.26 \text{ kN} + 36.08 \text{ kN} + 77.85 \text{ kN} + 38.56 \text{ kN} + 5.04 \text{ kN} + 17.13 \text{ kN} \]

\[ = 626.4 \text{ kN} \]

Check if assumption of neutral axis is correct:
Alternatively, the concrete compression force, \( F_{\text{conc}} \), may also be calculated with the stress diagram, like all other forces beforehand. If the assumption of the neutral axis in the beginning was correct, the results for \( F_{\text{conc}} \) have to be equal.

\[ \Rightarrow F_{\text{conc}} = f_{c} \times A = \frac{1}{2} \times 11.55 \frac{\text{N}}{\text{mm}^{2}} \times 1200 \text{ mm} \times 90.45 \text{ mm} \]

\[ = 626.8 \text{ kN} \approx 626.4 \text{ kN} \]

For simplicity, the Excel program uses an averaged steel-sheet thickness, \( t' \), without holes. The steel sheet with its cuts for the “windows” is shown in Figure 3-12 and Figure 3-13. The real steel sheet is 1.6 mm thick. According to Eq.(5.204), the 24 holes are “smeared” over the entire length to lead to the reduced average thickness, \( t' \).

\[ t' = \frac{455 \text{ mm} \times 4400 \text{ mm} - 24 \times 50 \text{ mm} \times 75 \text{ mm}}{455 \text{ mm} \times 4400 \text{ mm}} \times 1.6 \text{ mm} = 1.53 \text{ mm} \]

In contrast, the example above uses the real thickness of 1.6 mm and considers the holes. Therefore, although the initial calculated neutral axis with Excel is a good estimate, it will not be exactly the same as the neutral axis calculated in the example. This is the cause of the very small difference in the result of Eq.(5.203).

Moment Capacity:
Theoretical Modelling

The moment capacity of the cross-section may be calculated by multiplying all forces by their vertical distance to the neutral axis as in the following equation.

$$
M = F_{conc} * z_{conc} + F_{reinf, +15\%} * z_{reinf} + F_{sheet, 1} * z_{sheet, 1} + F_{sheet, 2} * z_{sheet, 2} + F_{sheet, 3} * z_{sheet, 3} + F_{sheet, 4} * z_{sheet, 4} + F_{sheet, 5} * z_{sheet, 5} + F_{conc, 1} * z_{conc, 1} + F_{conc, 2} * z_{conc, 2} + F_{conc, 3} * z_{conc, 3} + F_{conc, 4} * z_{conc, 4}
$$

$$
= 626.4 \text{kN} * \frac{2}{3} * 0.09045 \text{m} + 361.1 \text{kN} * 0.36955 \text{m}
$$

$$
+ 1.98 \text{kN} * \left( \frac{0.015 \text{m}}{2} + 0.04545 \text{m} \right)
$$

$$
+ 17.21 \text{kN} * \left( \frac{0.23394 \text{m}}{2} + 0.02955 \text{m} \right)
$$

$$
+ 68.13 \text{kN} * \left( \frac{2}{3} * 0.23394 \text{m} + 0.02955 \text{m} \right)
$$

$$
+ 7.26 \text{kN} * \left( \frac{0.01106 \text{m}}{2} + 0.23394 \text{m} + 0.02955 \text{m} \right)
$$

$$
+ 36.08 \text{kN} * \left( \frac{0.055 \text{m}}{2} + 0.05 \text{m} + 0.01106 \text{m} + 0.23394 \text{m} + 0.02955 \text{m} \right)
$$

$$
+ 77.85 \text{kN} * \frac{2}{3} * 0.03188 \text{m} + 38.56 \text{kN} * \left( \frac{0.02767 \text{m}}{2} + 0.03188 \text{m} \right)
$$

$$
+ 5.04 \text{kN} * \left( \frac{0.35 \text{m}}{2} + 0.02767 \text{m} + 0.03188 \text{m} \right)
$$

$$
+ 17.13 \text{kN} * \left( \frac{0.35 \text{m}}{3} + 0.02767 \text{m} + 0.03188 \text{m} \right)
$$

$$
= 208.8 \text{kNm}
$$

(5.205)

Final actuator force in the middle of the span:

If the moment capacity is known, the matching point load in the middle of the span can be recalculated. To compare the result with the load-deflection curve from the experimental test, the dead load of the floor has to be subtracted. The second floor specimen weighed 1994 kg, which results in a self-weight per unit length of:

$$
q = \frac{1994 \text{kg}}{4.5 \text{m}} * 9.81 \text{m/s}^2 = 4.347 \text{kN/m} (5.206)
$$

The moment generated by the dead load of the floor is therefore:
Finally, the maximum load capacity of the floor slab before the rebars start to yield can be calculated as:

\[
F_{\text{machine}} = 4 \times \left( \frac{M - M_{\text{dead load}}}{L} \right) = 4 \times \left( \frac{208.8 \text{ kNm} - 11.0 \text{ kNm}}{4.5 \text{ m}} \right) = 176 \text{ kN}
\]  

(5.208)

This is the maximum load capacity of the floor slab before the rebars start to yield and not the maximum load capacity of the floor overall. The maximum load capacity of the floor overall has to be slightly higher than 176 kN. Even when the rebars are already yielding, the maximum capacity has not yet been reached. If the curvature increases and therefore also the strain, the point in the cross-section where the steel sheet starts yielding is shifted closer to the neutral axis. Therefore, more areas of the steel sheet will have a final stress value of 410 MPa, which results in higher forces and a higher moment resistance. The final load, at which all stress shapes in the steel sheet are rectangles with a value of 410 MPa, is only an idealised theoretical limit value, which is never reached, because it assumes infinite strain.

This hand calculation is presented here mainly to explain the theory. The comparison of the theory with the experimental test result is made in section 5.6.4, but is based on this theory.

5.4 COMPOSITE SECTION PROPERTIES

5.4.1 Moment of Inertia

For calculating the deflection, it is first necessary to calculate the bending stiffness, \( EI \), and therefore the moment of inertia, \( I \).

One method would be to transform the concrete area to an equivalent steel area. Therefore, the concrete area has to be divided by a value \( n \), which is defined as the quotient of the
modulus of elasticity of steel over the modulus of elasticity of concrete. The moment of inertia can then be calculated in the normal way on the transformed cross-section.

For this method, it is assumed that the elastic modulus stays constant and that no slip occurs between the concrete and the steel surfaces. Without modifications, this method is only applicable in the elastic range and can be explained in the following three steps:

1. **Transformed cross-section**
   Before the cross-section can be analysed, it has to be transformed to one single cross-section with one homogenous elastic modulus. The difference of the elastic moduli between steel and concrete is represented by the value $n$:
   
   \[ n = \frac{E_s}{E_c} \]  

   Therefore, the area of the concrete is divided by $n$ to transform the concrete area into an equivalent smaller steel area. Only the width is altered in order to maintain the same strain levels at any given height. All further investigations are based on that transformed cross-section.

2. **Centroid identification**
   Working on the transformed cross-section, the centroid can be located in the same way as for a normal steel section. Therefore, the cross-section is subdivided in several parts, $A_i$, for which the centroid and the moment of inertia are easy to calculate separately.
   
   Thus, the vertical centroid position of the whole cross-section $z^*$ gives:
   
   \[ z^* = \frac{\sum (A_i \cdot z_i)}{\sum A_i} \]  

   where $z$ is measured from the top surface of the cross-section.

3. **Transfer of Axes**
   Knowing the centroid of the cross-section makes it possible to calculate the moment of inertia directly. However, it is usually more convenient to calculate the moment of inertia about a parallel axis, $(I^p)$, for example the top surface of the
5.4 Composite Section Properties

cross-section. From that moment of inertia, $I^p$, the moment of inertia about the centroidal axis, $I$, can be calculated using the parallel axes theorem:

$$I = I^p - A_{total} \left( z^* \right)^2$$  \hspace{1cm} (5.211)

All three steps explained above can be clearly combined in one table, as presented in Figure 5-17 for floor test one. Figure 5-16 illustrates the cross-section of floor test one. The concrete fillets between the web and the slab are ignored.

![Figure 5-16: Cross-section, floor test 1](image-url)

Figure 5-16: Cross-section, floor test 1
Theoretical Modelling

<table>
<thead>
<tr>
<th>No.</th>
<th>Part</th>
<th>A [m²]</th>
<th>z [m]</th>
<th>A²z [m³]</th>
<th>A²z² [m⁴]</th>
<th>I [m⁴]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Steel Sheet Top</td>
<td>2.40E-05</td>
<td>3.75E-02</td>
<td>9.00E-07</td>
<td>3.35E-08</td>
<td>4.50E-10</td>
</tr>
<tr>
<td>2</td>
<td>Steel Sheet Middle</td>
<td>3.50E-04</td>
<td>2.43E-01</td>
<td>9.61E-05</td>
<td>2.31E-05</td>
<td>1.96E-06</td>
</tr>
<tr>
<td>3</td>
<td>Steel Sheet Bottom</td>
<td>8.60E-05</td>
<td>4.43E-01</td>
<td>3.69E-05</td>
<td>1.72E-05</td>
<td>2.22E-06</td>
</tr>
<tr>
<td>4</td>
<td>Reinforcement 1</td>
<td>3.14E-04</td>
<td>4.60E-01</td>
<td>1.45E-04</td>
<td>6.65E-05</td>
<td>7.65E-09</td>
</tr>
<tr>
<td>5</td>
<td>Reinforcement 2</td>
<td>3.14E-04</td>
<td>4.60E-01</td>
<td>1.45E-04</td>
<td>6.65E-05</td>
<td>7.65E-09</td>
</tr>
<tr>
<td>6</td>
<td>Reinforcement 3</td>
<td>3.14E-04</td>
<td>4.20E-01</td>
<td>1.32E-04</td>
<td>5.54E-05</td>
<td>7.65E-09</td>
</tr>
<tr>
<td>7</td>
<td>Reinforcement 4</td>
<td>3.14E-04</td>
<td>4.20E-01</td>
<td>1.32E-04</td>
<td>5.54E-05</td>
<td>7.65E-09</td>
</tr>
<tr>
<td>8</td>
<td>Concrete 500/ft</td>
<td>2.30E-02</td>
<td>7.50E-02</td>
<td>1.72E-03</td>
<td>1.29E-04</td>
<td>4.30E-05</td>
</tr>
<tr>
<td>9</td>
<td>Concrete 400/ft</td>
<td>4.46E-03</td>
<td>3.25E-01</td>
<td>1.45E-03</td>
<td>4.71E-04</td>
<td>4.26E-05</td>
</tr>
</tbody>
</table>

Figure 5-17: Moment of inertia, floor test 1

Cell B3 in the Excel sheet is calculating the value \( n \). The concrete area and the concrete moment of inertia in cells C13-C14 and G13-G14 have to be divided by \( n \).

The centroid identification of step two is done in cell D15. Cell FG16 represents the transfer of axes, and cell FG17 is the final moment of inertia about the centroid.

Figure 5-18 and Figure 5-19 show the same procedure for floor test two. Figure 5-20 and Figure 5-21 show the procedure for floor test three.

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5.4 Composite Section Properties

![Figure 5-18: Cross-section, floor test 2](image)

\[ E_s = \begin{array}{c}
2.05 \times 10^{11} \text{ N/m}^2 \\
E_c = 1.68 \times 10^{10} \text{ N/m}^2
\end{array} \]

\[ n = 12.19 \]

<table>
<thead>
<tr>
<th>No.</th>
<th>Part</th>
<th>( A ) ([\text{m}^2])</th>
<th>( z ) ([\text{m}])</th>
<th>( A^*z ) ([\text{m}^3])</th>
<th>( A^*z^2 ) ([\text{m}^4])</th>
<th>( I ) ([\text{m}^4])</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Steel Sheet Top</td>
<td>2.40E-05</td>
<td>3.75E-02</td>
<td>9.00E-07</td>
<td>3.38E-08</td>
<td>4.50E-10</td>
</tr>
<tr>
<td>2</td>
<td>Steel Sheet Middle</td>
<td>3.92E-04</td>
<td>2.43E-01</td>
<td>9.51E-05</td>
<td>2.31E-05</td>
<td>1.96E-06</td>
</tr>
<tr>
<td>3</td>
<td>Steel Sheet Bottom</td>
<td>8.80E-05</td>
<td>4.43E-01</td>
<td>3.89E-05</td>
<td>1.72E-05</td>
<td>2.22E-08</td>
</tr>
<tr>
<td>4</td>
<td>Reinforcement 1</td>
<td>3.14E-04</td>
<td>4.60E-01</td>
<td>1.45E-04</td>
<td>6.65E-05</td>
<td>7.85E-09</td>
</tr>
<tr>
<td>5</td>
<td>Reinforcement 2</td>
<td>3.14E-04</td>
<td>4.60E-01</td>
<td>1.45E-04</td>
<td>6.65E-05</td>
<td>7.85E-09</td>
</tr>
<tr>
<td>6</td>
<td>Concrete, slab/n</td>
<td>1.48E-02</td>
<td>7.50E-02</td>
<td>1.11E-03</td>
<td>8.31E-05</td>
<td>2.77E-05</td>
</tr>
<tr>
<td>7</td>
<td>Concrete, rib/n</td>
<td>2.87E-03</td>
<td>3.25E-01</td>
<td>9.33E-04</td>
<td>3.03E-04</td>
<td>2.93E-05</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.88E-02</td>
<td>1.31E-01</td>
<td>2.46E-03</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>6.19E-04</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>-3.24E-04</td>
</tr>
</tbody>
</table>

\[ I_{\text{comp}}[\text{m}^4] = 2.95E-04 \]

![Figure 5-19: Moment of inertia, floor test 2](image)
Theoretical Modelling

If the curvature, $\kappa$, and the bending-moment, $M$, are known for a cross-section, it is also possible to calculate the moment of inertia, $I$, or the bending stiffness, $EI$, in a different way.

The curvature is defined as:

$$\kappa = \frac{|\kappa_1 + \kappa_3|}{h} \quad (5.212)$$
where $\varepsilon_t$ and $\varepsilon_b$ are the strain values at the top and bottom of the cross-section with the height $h$ as indicated in Figure 5-22.

![Linear strain distribution in cross-section](image)

Figure 5-22: Linear strain distribution in cross-section

The bending stiffness can be deduced from Eq. (5.213).

$$EI = \frac{M}{\kappa}$$  \hspace{1cm} (5.213)

This second method for calculating the bending stiffness is also used in the Excel program, because the program has already calculated the bending-moment and the strain distribution to find the neutral axis in the cross-section.

### 5.4.2 Shear Area

In a similar way, as the moment of inertia for the composite section is necessary to calculate the bending deflection, the shear area for the composite section is needed to calculate the shear deflection.

For simplification, the shear area, $A_s$, is obtained with the equation for a rectangular cross-section:

$$A_s = \frac{5}{6} A_{\text{comp}}$$  \hspace{1cm} (5.214)

The composite area, $A_{\text{comp}}$, is calculated analogously to $I_{\text{comp}}$ and takes into account that both the steel sheet and the concrete carry part of the shear forces.

Instead of the value $n$, which was used beforehand, the value $n_G$ has to be used now in Eq.(5.215).
Theoretical Modelling

\[ n_G = \frac{G_s}{G_c} \]  
(5.215)

\( G_s \) and \( G_c \) are the shear moduli for steel and concrete and may be expressed using the moduli of elasticity and Poisson’s ratios, \( \nu \), for the respective materials:

\[ G_s = \frac{E_s}{2(1+\nu_s)} \]  
(5.216)

\[ G_c = \frac{E_c}{2(1+\nu_c)} \]  
(5.217)

With a Poisson’s ratio of 0.3 for steel and 0.2 for concrete and with \( E_c \) from Figure 4-4 for the first test, Eq.(5.216) and Eq.(5.217) become:

\[ G_s = \frac{E_s}{2(1+\nu_s)} = \frac{205000}{2(1+0.3)} \frac{N}{mm^2} = 78850 \frac{N}{mm^2} \]  
(5.218)

\[ G_c = \frac{E_c}{2(1+\nu_c)} = \frac{21629}{2(1+0.2)} \frac{N}{mm^2} = 9012 \frac{N}{mm^2} \]  
(5.219)

Figure 5-23 illustrates the concrete and steel area used for the calculation in Figure 5-24. Cell C6 is dividing the concrete area by the value \( n_G \). Note that the (small) contribution of the flanges is ignored.

Figure 5-23: Shear area floor test 1 and 2
For the second floor test, the shear modulus was calculated as:

\[
G_e = \frac{E_e}{2(1 + \nu_e)} = \frac{14178 \text{ N/mm}^2}{2(1 + 0.2)} = 5908 \text{ N/mm}^2
\]  

(5.220)

The composite area for the second floor test is calculated in Figure 5-25.

According to Figure 5-26, the third floor test has a slightly changed cross-section.
Theoretical Modelling

Figure 5-26: Shear area floor test 3

The shear modulus for the third floor test was:

\[ G_c = \frac{E_c}{2(1 + \nu_c)} = \frac{9895 \text{ N/mm}^2}{2(1 + 0.2)} = 4123 \text{ N/mm}^2 \]  \hspace{1cm} (5.221)

Figure 5-27 provides the table for the last floor test.

<table>
<thead>
<tr>
<th>No.</th>
<th>Part</th>
<th>Area [m²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>G_c</td>
<td>7.89E+04 N/mm²</td>
</tr>
<tr>
<td>2</td>
<td>G_c</td>
<td>4.12E+03 N/mm²</td>
</tr>
<tr>
<td>3</td>
<td>No.</td>
<td>n_g= 19.12</td>
</tr>
<tr>
<td>4</td>
<td>Part</td>
<td>A_comp [m²]</td>
</tr>
<tr>
<td>5</td>
<td>Steel Sheet</td>
<td>6.94E-04</td>
</tr>
<tr>
<td>6</td>
<td>Concrete/n_g</td>
<td>2.61E-03</td>
</tr>
<tr>
<td>7</td>
<td>A_comp [m²]</td>
<td>3.31E-03</td>
</tr>
</tbody>
</table>

As mentioned in Eq.(5.214), the computed composite areas have to be multiplied by a factor of 5/6 to gain the shear area, A.
5.5 Deflection

Figure 5-23 to Figure 5-27 trace the calculation of $A$ or $A_{ncp}$ for the uncracked cross-section. Therefore, all values given in Figure 5-25 can be found also in the Excel sheets as long as the concrete is not cracked. This can be checked in the last seven columns of Figure 5-40. It is assumed that as soon as the concrete cracks the cracked concrete part cannot carry any shear loads anymore. Therefore, the shear area in Figure 5-23 and Figure 5-26 has to be reduced such that only the uncracked concrete part is contributing to the shear area. As the Excel sheet calculates the height of the concrete cracking line for every kN-step, the area is reduced automatically. This can be seen in the “part concrete”-column of Figure 5-40, where the value of $3.75E-03 \text{ m}^2$ is constant up to a load of 24 kN. After that, the concrete is cracked and the shear area reduced.

For the steel sheet, it is assumed that after it has yielded, the yielded part no longer contributes to the shear stiffness. Therefore, the shear area of the steel sheet has to be reduced also after a specific load. The Excel sheet does this automatically in the “part sheet”-column of “Matrix 1”(Figure 5-40). However, Figure 5-40 is only showing a section of “Matrix 1”, which is the top part with the rows up to a load of 60 kN. The steel sheet starts yielding for the second test at 134 kN, therefore, the value of $6.71E-04 \text{ m}^2$ for the steel sheet part stays constant until this load is reached.

5.5 DEFLECTION

The equation to calculate the maximum deflection $u$ for a simply supported beam loaded at midspan neglecting the shear contribution to deflection is:

$$u = \frac{F l^3}{48EI} \quad \text{(5.222)}$$

Eq. (5.222) implies that the bending stiffness, $EI$, stays constant throughout the whole length. This might only be the case at a low load level. As soon as the load increases, the modulus of elasticity, $E$, will slightly decrease, and as soon as the concrete cracks, the second moment of area, $I$, will decrease rapidly as well.

Therefore, the bending stiffness in the middle of the slab is much less, because the concrete is cracked, than it is at the ends over the supports, where the concrete is still uncracked.
Theore
tical Modelling

A more accurate method to calculate the deflection of a beam is to use the unit load method. This method is based on virtual work principles and can also be used when the bending stiffness varies along the length. Apart from other books this method is explained and demonstrated in (Young, 1950).

The equation for the deflection $u_{\text{bending}}$ due to flexure by using the unit load method is given in Eq.(5.223) (Young, 1950, page 116):

$$u_{\text{bending}} = \int_0^l \frac{M}{EI} \, dx$$  \hspace{1cm} (5.223)

$\overline{M}$ is the virtual bending moment due to the virtual force (unit load) and $M$ is the bending moment in the beam due to the real force (Figure 5-28). $EI$ is the bending stiffness of the beam.

And by subdividing the length $l$ into small parts of $\Delta l$, the integral in Eq.(5.223) can be replaced by the summation in Eq.(5.224).

$$u_{\text{bending}} = \sum_i \overline{M}_i \frac{M_i}{E_i I_i} \Delta l$$ \hspace{1cm} (5.224)

The method is demonstrated on an eight-metre-long steel beam 530UB 92.4 which is loaded at midspan with 20 kN, as shown in Figure 5-28.

![Figure 5-28: Bending-moment and virtual bending-moment diagram](image)

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For the example, it is assumed that the modulus of elasticity and the second moment of area stay constant over the length.

\[ E = 200000 \text{ MPa} \]  
\[ I = 554 \times 10^6 \text{ mm}^4 \]

The three equations above inserted in Eq. (5.224) leads to Eq.(5.228).

\[ u_{bending} = \sum_i \frac{M_i}{E_i I_i} \Delta l \]
\[ = \frac{2 \left(0.25m \times 5\text{kNm} + 0.75m \times 15\text{kNm} + 1.25m \times 25\text{kNm} + 1.75m \times 35\text{kNm}\right) \times 1m}{200000 \text{ MPa} \times 554 \times 10^6 \text{ mm}^4} \]
\[ = 1.90 \text{ mm} \]

Because the bending stiffness stays constant in the example, the result in this case can also be checked with the simple Eq.(5.222), which leads to a similar result of \( u = 1.93 \text{ mm} \). The accuracy of the unit load method can be improved if smaller intervals \( \Delta l \) are taken.

Generally, the bending stiffness is not constant over the length. In reality, the concrete will crack under load first in the middle part, and will reduce the second moment of area only locally. At the same time, the second moment of area near the support remains the same. In this case, the distribution of the bending stiffness over the length has to be calculated first, to use the corresponding bending stiffness for each part \( i \) in Eq. (5.224).

For the shear deformation, the unit load method is applicable as well with a similar expression:

\[ u_{shear} = \int_0^l \frac{V}{GA} dx \]  
\[ = \sum_i \frac{V_i}{G_A} \Delta l \]  
\[ = \sum_i \frac{V_i}{G_A s, i} \Delta l \]

For the example calculated, Figure 5-29 shows the shear force diagram and the virtual shear force diagram.
Theoretical Modelling

Only the web of the steel beam is used for the shear area, $A_s$.

$$G = \frac{E_s}{2(1+\nu_s)} = \frac{200000 \text{ MPa}}{2(1+0.3)} = 76900 \text{ MPa}$$  \hfill (5.231)

$$A_s = \frac{5}{6} \times 502 \text{ mm} \times 10.2 \text{ mm} = 4267 \text{ mm}^2$$  \hfill (5.232)

Therefore, Eq. (5.230) leads to Eq.(5.233).

$$u_{\text{shear}} = \sum_i V_i \frac{V_i}{G_i A_{s,i}} \Delta I$$

$$= \frac{0.5 \times 10 \text{ kN} \times 4 \text{ m} + (-0.5) \times (-10 \text{ kN}) \times 4 \text{ m}}{76900 \text{ MPa} \times 4267 \text{ mm}^2}$$

$$= 0.12 \text{ mm}$$  \hfill (5.233)

The total deflection $u$ is simply the sum of $u_{\text{bending}}$ and $u_{\text{shear}}$.

The unit load method is used in the Excel program to calculate the deflection. In the Excel program the modulus of elasticity, $E$, and the shear modulus, $G$, are assumed to be constant to keep the effort in a reasonable limit, but the variable second moment of area, $I$, and the changing shear area, $A_s$, are calculated at 100-mm intervals.
5.6 THEORETICAL MODEL - EXCEL PROGRAM

5.6.1 Introduction

The Excel program uses the theory outlined to analyse the floor unit F1 under concentrated load at midspan. Relevant parameters are listed in the beginning and can easily be adjusted. However, the program makes no claim to be generally applicable. The cells are programmed for the specific situations of the second and third experimental floor tests. Changes to the assumed values are possible, but have to be done with care, as different values can change the arrangement of load cases or the range the cells are programmed for.

The program is able to analyse the floor for every given load. If the floor is able to carry the load, the neutral axis is calculated, and the steel and concrete stress distribution at midspan is presented in diagrams. For the optimisation process to find the neutral axis, a tool called “solver” is used.

The solver tool is a Microsoft Excel add-in, which helps to determine the optimum value for a formula in a particular target cell. To do this, the solver adjusts the values of other cells that are related to the target to find a solution that satisfies all constraints.

The program is also able to calculate the values for the load-deflection curve at midspan. To achieve this, the program starts its calculation at a load value of 0 kN and increases this value in steps of 1 kN. The results of each step are saved, as they are needed to construct the final load-deflection curve.

In order to not have to manually start the solver for every 1 kN, this process is programmed in a macro. A high priority, when developing the Excel program, was keeping it simple and reviewable, and so macros were used only when absolutely necessary.

If the assumed floor parameters in the Excel program should be changed, the macro has to be used with care. As the solver dialog box is turned off, no warning will inform the user, if the solver cannot find a valid solution. Therefore, the results have to be checked separately, or the solver dialog box has to be turned on. This procedure is explained in
section 5.6.3 and will display the solver dialog box after every 1 kN step. This message box will confirm whether a valid solution was found or not.

For simplicity, the macro uses a constant elastic modulus for concrete for all load cases. In section 5.6.5, it will be shown that this assumption is acceptable.

The program does not check the maximum concrete compression strain. Therefore, the calculated strain values in the Excel sheets have to be checked manually to see if the calculated strain at the top surface is less than the maximum strain at the peak stress point given by the experimental tests in Figure 4-4. In section 5.6.4 it is shown that this is only necessary at a load level very close to the peak load. It is also consistent with the visual observation of no concrete compression crushing in any test.

The macro calculates the floor deflection that will satisfy equilibrium for each kN-increment in load. The analysis could alternatively be controlled by incrementing the maximum concrete strain and using equilibrium to calculate the corresponding applied load. This would have the advantage of allowing the analysis to proceed easily into the fully plastic part of the load deflection response. However, in this thesis the load increase is preferred, as for any single given load the strain and stress distribution can be checked instantly. Also, a user of the spreadsheet will generally have a better idea of expected loads rather than concrete strains.

The second and third experimental floor tests are analysed with the Excel program. As the slab thickness is different for these two floors, it was easier to program an individual Excel program for each floor type than trying to combine all different possibilities in one program. Due to the early failure of floor test one, no special case Excel program was written for the first floor test, but the first floor test will be included as one of the cases analysed with the finite-element program in chapter 6. If not otherwise indicated, the following figures refer to the program used to analyse the second floor test.

5.6.2 Excel sheets

The excel program consists of the following 11 different excel sheets:
5.6 Theoretical Model - Excel Program

- Calculation
- Load Cases
- Strain
- Stress Steel Sheet
- Stress Concrete
- Matrix 1
- Matrix 2
- Matrix 3
- Diagram
- Diagram Data
- Example

The left part of the sheet “Calculation” is displayed in Figure 5-30. This is the start screen, where all important parameters for the calculation can be set.

There are two possible ways to use the program:

1. A load is chosen, for which the floor should be analysed. After using the solver from the Excel task bar separately, the neutral axis and every important parameter for the cross-section will be calculated.

2. The macro is chosen, which analyses the floor not only for one given load, but also analyses the floor automatically for every load in the range of 0 kN to 193 kN for the second floor test. 193 kN is the maximum load which can be carried by the second floor specimen, analysed with the first method. If floor parameters are changed, and the first method shows a different maximum load, then, the cells in the corresponding Excel sheets have to be adjusted manually. In addition to this, the results of the macro are used to calculate the deflection of the floor at every kilonewton.

The first method is recommended for analysing the cross-section at a specific stage, or to try out different parameters. The second method should be chosen when the overall performance of the floor is sought, or when deflection is of particular interest.
Theoretical Modelling

![Excel sheet](image)

**Figure 5-30**: Left part Excel sheet “Calculation”, floor test 2

![Image](image)
By using the first method, any load may be chosen by changing the value in the blue field and starting the solver. Thereby, the blue cell is not allowed to be active anymore. That may be arranged by hitting “Enter” or clicking with the mouse in another cell; otherwise the solver will not work.

Typically the solver feature is not installed by default in Excel, but it can be loaded afterwards as explained in Figure 5-31, which is a printout from the Help command of Microsoft Excel 2007.

![Figure 5-31: How to install the Excel solver add-in](image)

The solver can then be started in the “Analysis” group on the “Data” tab for Excel 2007 or from the “Tools” menu for Excel 2003. All solver parameters can be set in one window as shown in Figure 5-32.
The task for the solver is to change the neutral axis depth in sheet “Calculation” (cell B6) and the strain value at the height of the rebar (cell B7) in such a way that the value of cell F45 matches zero.

The concrete compression stress of the cross-section can be calculated in two different ways as explained in section 5.3. If the assumption of the neutral axis is correct, the results of both calculations have to be the same. The Excel sheet analyses these two outcomes in cell B41 and cell B45. Cell F45, the target cell, is the difference between these two values, which should converge to zero.

Two constraints are given in Figure 5-32. The first one makes sure that the cross-section is analysed for the load given in the blue cell at the top, and the second one is necessary to reject some invalid results, which could otherwise arise under certain conditions.

If the load value in the blue cell is set too high (in the actual case over 193 kN), the solver cannot find a solution, which makes sense, because the given cross-section cannot resist that load and, therefore, no solution exists.

However, sometimes, the solver cannot find a solution even if the load level is under the final load level. The problem is the rapid change of material properties when the concrete cracks. This can also be a problem in FEM-programs and the reason why some rapidly
changing material diagrams have to be modified for the FEM-analysis. How to model the concrete cracking behaviour is discussed further in section 6.2.

Figure 5-59 shows the rapid change of the neutral axis location when the concrete cracks.

As the solver needs to calculate the neutral axis to find a solution, this rapid change can be interpreted by the solver that no convergent solution exists. The solver runs several interpolation processes until it finds a convergent solution. If the starting values suggest a load before the concrete cracks the solver adjusts the position of the neutral axis, trying to fulfil the requirements. If the final solution is a load before the concrete cracked the solution is convergent and can easily be found. If the solution is a load after the concrete has cracked it is possible that the solver cannot find a solution. Therefore, the entire load range can be divided into two groups: the load range before the concrete cracks and the load range after the concrete has cracked. The solver always needs starting values in the cells B2, B6 and B7, even if these values are zero. Therefore, the solver always starts in one of these groups. The solution in one group can easily be found, only if the solver starts in one group and the solution is in the other group can the solver not find it.

In this case, the values of the changing cells B6 and B7 are too different from the solution. However, if the cells B6 and B7 are changed manually with values roughly similar to the solution, the solver will be placed in the correct group and will then be able to find the exact solution again.

Therefore, some pre-calculated results are given in Figure 5-33, which is a table placed in the Excel sheet “Calculation”, for different load levels. These results were calculated with the solver and are valid for the second experimental test. They show in which range the neutral axis and the strain in the rebars have to be for different load levels. If the assumed values are not changed the solver should find exactly the same solution. However, the solver does not use the values from the solution example table in any way for its calculation. The table is only an aid for the program user to set the values manually in cells B6 and B7, to put the solver in the correct starting group if that is necessary. Therefore, only two points, one before and one after the cracking load at about 25 kN would be sufficient. However, the other load examples are given to show at least one example for
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each possible combination between the two different load cases for steel and the four different load cases for concrete.

If the assumed values in Figure 5-30 are changed, the solution examples will not match anymore. However, they can still be used as a guideline for similar tasks.

Solution examples are also given in the Excel sheet for the third experimental floor test. However, with the parameter for the third test the solver could always find a solution, no matter at which load level (assuming a load under the maximum valid load). Therefore, the problem around the load where the concrete cracks does not exist for the given values.

<table>
<thead>
<tr>
<th>$F_{machine}$ [kN]</th>
<th>neutral axis $z$ [mm]</th>
<th>$s_{relinf}$ [$10^{-6}$]</th>
<th>Loadcase-Steel</th>
<th>Loadcase-Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>134.65</td>
<td>67.4</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>20</td>
<td>134.65</td>
<td>205.1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>30</td>
<td>125.14</td>
<td>337.5</td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>140</td>
<td>102.97</td>
<td>2048.1</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>160</td>
<td>101.14</td>
<td>2386.1</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>190</td>
<td>76.36</td>
<td>5291.6</td>
<td>2</td>
<td>4</td>
</tr>
<tr>
<td>193</td>
<td>62.54</td>
<td>8353.0</td>
<td>2</td>
<td>4</td>
</tr>
</tbody>
</table>

Dimensions of floor test 2

Figure 5-33: Right part Excel sheet “Calculation”, floor test 2
To keep the Excel sheets clearly laid out, all grey tinted cells in sheet “Calculation” are actually calculated in sheet “Load Cases”, and only the actual used values are transferred back to sheet “Calculation”.

An example for the sheet “Load Cases” is given in Figure 5-34 to Figure 5-36.

![Example for Loadcase Steel 1](image1)

![Example for Loadcase Steel 2](image2)

![Example for Loadcase Steel 3](image3)

Figure 5-34: Left part, Excel sheet “Load Cases”, floor test 2
Theoretical Modelling

<table>
<thead>
<tr>
<th>Stress concrete</th>
<th>x [N/mm²]</th>
<th>y [mm]</th>
<th>F_{concrete} [kN]</th>
<th>Δz_{concrete} [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load Case Concrete 1</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>134.97</td>
<td>32.57</td>
</tr>
<tr>
<td>0</td>
<td>500</td>
<td>161.14</td>
<td>175</td>
<td></td>
</tr>
<tr>
<td>-9.53</td>
<td>500</td>
<td>677.16</td>
<td>233.33</td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>288.86</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.60</td>
<td>350</td>
<td>11.28</td>
<td>221.02</td>
<td></td>
</tr>
<tr>
<td>37.59</td>
<td>0</td>
<td>5.09</td>
<td>46.02</td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>4.60</td>
<td>350</td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>4.07</td>
<td>355.67</td>
<td></td>
</tr>
<tr>
<td>0.32</td>
<td>0</td>
<td>1.2</td>
<td>356.78</td>
<td></td>
</tr>
<tr>
<td>Load Case Concrete 2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>134.97</td>
<td>32.57</td>
</tr>
<tr>
<td>0</td>
<td>500</td>
<td>-2.61</td>
<td>46.02</td>
<td></td>
</tr>
<tr>
<td>-9.53</td>
<td>500</td>
<td>0.15</td>
<td>45.08</td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>388.86</td>
<td>11.28</td>
<td>221.02</td>
<td></td>
</tr>
<tr>
<td>4.60</td>
<td>350</td>
<td>15.70</td>
<td>161.75</td>
<td></td>
</tr>
</tbody>
</table>

Figure 5-35: Right top part, Excel sheet “Load Cases”, floor test 2
The loading spectrum from zero to the final load is separated into three different load cases for the steel sheet and five different load cases for the concrete. During loading, the shape of the stress distribution over the cross-section will vary. For example, in the beginning the shape for the sheet part which is in tension will be a triangle, as seen in the example for load case one in Figure 5-34. After increasing the load, the steel sheet will start yielding at the bottom part. Consequently, the stress in this region cannot be further increased. The
stress shape will now change from a triangle to a trapezoid, as seen in the example for load case two.

After initially yielding, the cross-section analysis changes from one force representing the triangle in tension to two forces representing the trapezoid. It would be very confusing to have a lot of these “if-then” alternatives pyramiding in one Excel equation. Therefore, the different load cases have been established, where each of them represents a load area with the same geometrical shape as the stress distribution in the cross-section.

The Excel program finds out automatically which load case is relevant for the given values in sheet “Calculation”, and displays the actual load case in the red cells at the top of the sheet “Load Cases”. Only the actual load case is marked red, and the corresponding values are marked bold and black. The results of this sheet are the values in the grey tinted cells, which are transferred to sheet “Calculation”, where they are needed for the calculation.

The shapes for the concrete load cases are based on the concrete stress-strain diagram from the flexural tensile strength tests in section 4.2.1. At the cross-section height of 350 mm, measured from the bottom, the concrete load cases need an additional separation, because the width changes here from the web of 100 mm to the slab of 1200 mm.

Figure 5-37 to Figure 5-39 show the sheets “Strain”, “Stress Steel Sheet” and “Stress Concrete”. Each of these sheets’ diagrams shows the actual strain or stress distribution for the given parameters in sheet “Calculation”. Meanwhile, the diagrams in sheet “Load Cases” are fixed and will not change, since they are only examples for a type of load case.
5.6 Theoretical Model - Excel Program

Figure 5-37: Excel sheet “Strain”, floor test 2
Figure 5-38: Excel sheet “Stress Steel Sheet”, floor test 2

<table>
<thead>
<tr>
<th>$F_{machine}$</th>
<th>180.00 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>$x$ [N/mm²]</td>
<td>$y$ [mm]</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>0</td>
<td>500</td>
</tr>
<tr>
<td>0</td>
<td>410</td>
</tr>
<tr>
<td>$-10^8$</td>
<td>470</td>
</tr>
<tr>
<td>$491.19039^*$</td>
<td></td>
</tr>
<tr>
<td>$410$</td>
<td>$139.94623$</td>
</tr>
<tr>
<td>$410$</td>
<td>30</td>
</tr>
<tr>
<td>0</td>
<td>30</td>
</tr>
</tbody>
</table>

Current stress distribution in the steel sheet over the cross section for the load specified in sheet "Calculation".
The following sheets in the Excel program from “Matrix 1” onwards will only change if the macro is used. The macro is doing the same thing that the solver does, if it is started manually. The only differences are that the loading force is increasing automatically from zero to 193 kN, and that the results of the macro are printed in the green-tinted cells in sheet “Matrix 1”.

Figure 5-40 shows the top part of “Matrix 1”. This figure and the following ones show, for demonstration purposes only, the first few rows up to a actuator load of 60 kN. The real sheet has more rows up to the final load of 193 kN.

“Matrix 1” collects the results from the macro in the green-tinted cells and calculates the curvature and the bending stiffness for each row in the next two adjacent columns. The
columns that follow are more or less ancillary columns, which calculate the shear area for each given load.

“The Matrix 2” calculates the total bending deflection of the floor slab for every given load. Therefore, the bending-moment over the floor length is calculated first at every 100-mm increment, as seen in Figure 5-41, by assuming a linear bending-moment distribution over the length of the floor.

The bending stiffness is also calculated for every 100-mm increment, as seen in Figure 5-42, and so the Excel function VLOOKUP is used. Because the macro only calculated the cross-section in the middle part of the slab at 2.25 m, a comparison is used between the moment value anywhere in the slab closer to the support and the moment value at 2.25 m. The cross-section is originally the same everywhere in the floor. Only at the places where the moment has reached a level where the concrete is already cracked, will the bending stiffness be reduced locally. While increasing the load of the floor, the bending-moment at a low load level in the middle part of the slab will be reached by a cross-section that is further away from the middle part and only a little bit later at a higher load level. It is assumed that the two cross-sections behave similarly under the same bending-moment.

The right most column in Figure 5-42, $E_{I_{2.25m}}$, shows for the first rows up to a load of 24 kN a constant value of $5.31 \times 10^7$ Nm$^2$, which is the bending stiffness for the uncracked floor. The same value can be seen in Figure 5-57, where the bending stiffness at midspan is plotted as a function of the increasing load. If the load increases, the concrete at the bottom of the floor will reach its tension capacity and will crack. For the second floor test the values from the simplified stress-strain diagram from Figure 4-9 are used for characterising this behaviour and this material diagram is implemented in the Excel sheets as shown in Figure 5-35 and Figure 5-36, where the material diagram from Figure 4-9 is turned by 90 degrees to match the sketches in Figure 5-35 and Figure 5-36. Initially, it is only the concrete at the bottom that is affected, as the strain has its maximum value there. When the load is increased further, the critical strain where the concrete starts cracking will move upwards in the cross-section increasing the area of cracked concrete. Therefore, the bending stiffness decreases, as shown in Figure 5-57. The material properties for the steel sheet and the rebars are also implemented in the Excel sheets. Therefore, if the steel sheet
or the rebars start yielding, the bending stiffness will be affected as shown in Figure 5-57. This diagram shows the bending stiffness at midspan for varying load levels. Therefore, the values from the y-axis are the same as the values in the last column, EI_{2.25m}, in Figure 5-42.

This column, EI_{2.25m}, calculates the bending stiffness for different load levels only at midspan, which is 2.25 m apart from the support. At a load level of 29 kN the concrete at midspan is already cracked, resulting in the reduced bending stiffness of 4.75E+07 Nm². The bending moment for any given load has its maximum at midspan. Further away from midspan the bending moment is reduced until it reaches zero at the supports. Therefore, the cross-sections closer to the support are utilised less. This is reflected in Figure 5-42, where all other sections from EI_{0.05m} to EI_{2.15m} apart from midspan still have their uncracked bending stiffness of 5.31E+07 Nm² for a load level of 29 kN. For example, at the load level of 38 kN, the area where the bending stiffness is reduced has increased from section EI_{2.25m} to EI_{1.85m}. At this load level only the sections from EI_{0.05m} to EI_{1.75m} are showing the uncracked bending stiffness of 5.31E+07 Nm².

The values in Figure 5-42 are calculated with the help of the Excel function VLOOKUP. Therefore, the bending moments at all cross-sections from M_{0.05m} to M_{2.25m} and for all load levels have to be calculated as shown in Figure 5-41. In this figure only the first 60 x 1-kN load steps of the Excel sheet are shown, which should be sufficient to explain the procedure. At a load level of 44 kN the value for the section at 0.75 m from the support (M_{0.75m}) shows a bending moment of 20.17 kNm. The Excel function VLOOKUP takes this value of 20.17 kNm and searches in the last column (M_{2.25m}) of Figure 5-41 for the closest value, in this case 20.01 kNm at a load level of 8 kN. As all bending stiffness values have been calculated for the midspan section at EI_{2.25m} for all load cases as shown in the last column of Figure 5-42, VLOOKUP takes the bending stiffness at section EI_{2.25m} for a load level of 8 kN, which is the uncracked value of 5.31E+07 and writes it into the requested cell at section EI_{0.75m} for a load level of 44 kN.

In this way, the characteristics of the cross-section under load have to be calculated only once, as shown in Figure 5-58. All other bending stiffness values from different cross-
sections are calculated by comparing the actual bending moment at this cross-section with the diagram (Figure 5-58).

Therefore, for example, the function VLOOKUP takes the moment at location 1.05 m and searches for the same or next closest moment in the calculated values at 2.25 m. Because the moments are the same at these two locations, the bending stiffness has to be the same as well. In this way, it is possible to calculate the bending stiffness at every location in the slab and at every load level.

In Figure 5-43, it is illustrated how the total bending deflection is calculated. Based on the theory explained in section 5.5, each column represents a part of Eq. (5.228).

According to Figure 5-44 and Figure 5-45, the total shear deflection in “Matrix 3” is calculated in a similar way to the total bending deflection in “Matrix 2” and is based on the same theory from section 5.5.
Figure S.40: Top part Excel sheet “Matrix 1”, Floor test 2 (only the last 60 KN are shown)
Figure 5-41: Top-left part, Excel sheet “Matrix 2”, floor test 2 (only the first 60 kN are shown)
Figure 5-42: Top-middle part, Excel sheet “Matrix 2”, floor test 2 (only the first 60 kN are shown)
Figure 5-43: Top-right part, Excel sheet “Matrix 2”, floor test 2 (only the first 60 kN are shown)
Figure 5-44: Top-left part, Excel sheet "Matrix 3", floor test 2 (only the first 60 kN are shown)
Figure 5-45: Top-right part, Excel sheet “Matrix 3”, floor test 2 (only the first 60 kN are shown)
Figure 5-46 shows the Excel sheet “Diagram”, where the results from the Excel calculation are graphically compared with the load-deflection curve from the experimental test. The data for this diagram are saved in the sheet “Diagram Data” (Figure 5-47).

![Diagram](image)

Figure 5-46: Excel sheet “Diagram”, floor test 2

![Diagram Data](image)

Figure 5-47: Excel sheet “Diagram Data”, floor test 2
Theoretical Modelling

The sheet “Example” in Figure 5-48 to Figure 5-50 was created to give an overview of the floor behaviour. It shows the strain and stress diagrams for selected load levels from to the maximum load.
Figure 5-48: Excel sheet “Example”, floor test 2 (top part)
Figure 5-49: Excel sheet “Example”, floor test 2 (middle part)
5.6 Theoretical Model - Excel Program

Figure 5-50: Excel sheet “Example”, floor test 2 (bottom part)
The theoretical modelling of the Excel program is based on the second floor test. The values under “Assumptions” in the sheet “Calculation” can be changed to simulate different floor models, as long as the load cases stay valid. The third floor test had a slab thickness of 100 mm, against a 150 mm slab thickness in floor test two. Because the slab thickness has a significant impact on the type and number of load cases, it is not programmed as a changeable variable. Therefore, a separate Excel program was established for the third floor test.

The assembly of the sheets are similar to the previous one, and the values are updated for the third floor test. In particular, the concrete load cases in sheet “Load Cases” had to be changed to the new dimensions and are shown in Figure 5-51 and Figure 5-52.

As mentioned earlier in section 1.1, the goal of the thesis was to analyse the 4.5-m long floor specimen experimentally and theoretically. Therefore, the Excel sheets were written for the very particular geometry used in the second and third experimental test only for a length of 4.5 m. The construction of the sheets “Matrix 2” and “Matrix 3” (Figure 5-41 to Figure 5-45) are based on this length. Therefore, the columns correspond to half of the floor section at every 0.1-m interval. For example: \(M_{0.05m}, M_{0.15m}, M_{0.25m},\ldots\) or \(EI_{0.05m}, EI_{0.15m}, EI_{0.25m},\ldots\) or \(u_{0.0-0.1m}, u_{0.1-0.2m}, u_{0.2-0.3m},\ldots\) in Figure 5-41 to Figure 5-43.

As these columns in the Excel sheets are fixed, the sheets can only analyse the behaviour of the experimental tests with a floor length of 4.5 m. However, most of the material parameters under “Assumptions” in Figure 5-30 are changeable as long as the load cases stay valid. This facilitates analysing the floor behaviour for different concrete, steel sheet or rebar parameters. The intention of the Excel table was to use the same equations used for a hand-calculation, but obtain rapid results, while understanding at the same time what the program is doing and where the values are coming from. The visualisation of different load cases was very helpful in understanding the floor behaviour under increasing load. However, the drawback is that the sheets are not very flexible. A more flexible program would have split the cross section in finite elements or strips so that the program would not have to work with different load cases. However, it was not intended to write a finite element program as the analysis with finite elements is undertaken with the finite element modelling program Abaqus in chapter 6. Abaqus is able to model the floor unit more
closely than the Excel-based calculation method, however, the Abaqus model does not easily offer the same insights into the internal actions as the first-principles calculation does.

The two Excel files which analyse the behaviour from the second and third experimental floor test can be found in the Appendices.
Table 5-51: Load cases for concrete top part, floor test 3

<table>
<thead>
<tr>
<th>Load Case Concrete 1</th>
<th>Load Case Concrete 2</th>
<th>Load Case Concrete 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stress concrete x [N/mm²]</td>
<td>y [mm]</td>
<td>F_{concrete} [kN]</td>
</tr>
<tr>
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<td>0</td>
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</tr>
<tr>
<td>-10.06</td>
<td>500</td>
<td>0</td>
</tr>
<tr>
<td>0</td>
<td>408.58</td>
<td>44.96</td>
</tr>
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</tr>
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<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

Figure 5-51: Load cases for concrete top part, floor test 3
5.6 Theoretical Model - Excel Program

5.6.3 Macro

The advantage of using a macro is that the solver does not have to be started separately for every increased kilonewton step. Figure 5-53 lists the short macro code which is used in the Excel program for the second floor test.

At the beginning, the two yellow cells in sheet “Calculation” are set to values close to the estimated solution. They do not have to be very accurate, because even when both values
are set to zero, the solver will usually find a solution. Only when the solver has been used before with a very high load, might the values be too different from the solution for 0 kN, which is the first load step used by the macro, so that the solver will not find a solution. That was mentioned earlier in section 5.6.2, and that is also the reason why the macro is split in two halves. The first half calculates the first 25 rows in the for-next loop, which are for the load levels from zero to 24 kN for the actuator load. The second half calculates in a second for-next loop the rest of the rows, which are for the load levels from 29 kN to the final load. The part in between 25 kN and 28 kN, which is in most cases the range where the concrete starts to crack and which could create problems for the solver, is left out.

The next lines set the solver parameters, similar to the solver parameter window in Figure 5-32. At the end of the for-next loop, the macro saves all important parameters in the green tinted cells in sheet “Matrix 1”. After that, the loop starts again with the next row, which increases the load by one kilonewton.

Checking the results is recommended when parameters from the assumptions part are changed on sheet “Calculation”. The macro turns the solver dialog box off, which means that no error message occurs, even when the solver cannot find a solution. Alternatively, the SolverSolve UserFinish in the macro could be changed from True to False. Then, the solver dialog box will appear after every analysed kilonewton step. The starting values for the cells B6 and B7 at the beginning of each for-next loop are designed for the given parameters. Therefore, these values have to be adjusted accordingly, if parameters are changed.
Figure 5-53: Macro code for Excel program, floor test 2
5.6.4 Results

The results of greatest interest were the load-deflection curves of the theoretical model generated by the Excel program, as they allowed immediate comparison with the experimentally determined plots. If the theoretical model was to be used as a tool to assist design, it was important that it gave reasonable values for stiffness and ultimate load and generally provide a “good fit” to the experimental data. Load-deflection diagrams are printed in Figure 5-54 for floor test two and in Figure 5-55 for floor test three. Because the first floor test failed in an unexpected mode far below its intended capacity, no Excel program was established for the first test. However, in the finite-element analysis, all three floor specimens were analysed.

The load-deflection curve in Figure 5-54 consists primarily of two straight lines and a curved end. The first line reaches from the origin to the state where the concrete starts to crack. The little gap in the graph before the concrete cracks is the missing kilonewton steps in the macro from 25 kN to 28 kN. After the concrete cracked, the slope changes and stays relatively constant, until the rebars start to yield. Even after this point, the maximum load can be increased by the spread of yield into the steel sheet.
The final load of 194 kN is slightly higher than the measured 186 kN in the experimental test.

By using 1.0-kN load increments, the solver in Excel can find solutions up to 193 kN. Only when finding the last point at 194 kN was manual intervention needed. Because the solution for 194 kN is so different from the solution for 193 kN, the solver requires starting values closer to the solution of 194 kN to resolve the problem.

Figure 5-55: Load-deflection curve, floor test 3

Figure 5-55 shows a very good agreement between the experimental test results of floor test three and the results from the Excel simulation between zero and 60 kN. For loads higher than 60 kN, the load-deflection curve from the experimental test will separate more and more from the theoretical simulation by bending to the right. This behaviour is even stronger for floor test three than observed for floor test two.

Before discussing possible reasons for the variations in the diagrams between theoretical and experimental results, some other interesting results from the Excel program are highlighted below. The situations are explained using examples from the second floor test, but the statements are applicable to the first and third floor tests as well.
Figure 5-56 displays the strain in the rebars at midspan for the second floor test. The last point at 194 kN in the load-deflection diagram is a situation with an extremely high-strain value, where the strain gauge (type KFW) would be at the limit of its performance, which the manufacturer describes to be at $28000 \mu \varepsilon$. Also, the theoretical assumptions made for the calculation might not apply anymore at this stage.

Figure 5-56: Rebar strain at midspan, floor test 2

Figure 5-57 displays the bending stiffness of the floor at midspan at different load levels. This diagram contributes to the understanding of the fundamental floor behaviour under load. Up to a load of 27 kN, the bending stiffness stays constant. After that, the concrete tension strength at the bottom of the specimen is reached and the concrete cracks. This has a significant influence on the bending stiffness, which rapidly decreases. With increasing load, the bending stiffness approaches a constant value again, before the yielding of the steel sheet causes a very slight change in slope. At a load level of 182 kN the rebars also start yielding, causing the final steep decline of the bending stiffness.
The bending stiffness diagram shown is valid for the cross-section at midspan. At locations closer to the support, the bending-moment and the strain at the bottom of the floor are lower. Therefore, the bending stiffness stays constant longer, before the concrete cracks at this location when the load is increased.

Independently from the cross-section location, the bending stiffness at any location can be shown as a function of the actual bending moment at this location (Figure 5-58).

The location of the neutral axis depends on the tension and compression forces in the cross-section. When the concrete cracks, for instance, the forces in the cross-section will change simultaneously. Figure 5-59 shows the location of the neutral axis at midspan. The graph is very similar to the graph of the bending stiffness and shows how closely connected these values are.
Figure 5-58: Bending stiffness dependent on the bending moment for any location, floor test 2

Figure 5-59: Neutral axis at midspan, floor test 2
The Excel program calculates the bending and shear deflections separately and adds them together for the total deflection. In Figure 5-60 the results are shown for the bending deflection alone and for the total deflection for comparison. It can be seen that the shear deflection for the second floor test is much less than the bending deflection, but not so small that it is negligible.

In Figure 4-4, the stress-strain curves for the concrete in compression were printed. Each curve was simplified to three straight lines with different elastic moduli from $E_1$ to $E_3$. $E_1$ defines the stress strain behaviour at the beginning of the curve and has the highest value. $E_3$ defines the end of the curve with the lowest value and $E_2$ is somewhere between. For simplicity, the Excel sheets are written for one constant concrete elastic modulus in compression only. Therefore, only one of the three elastic moduli ($E_1$ to $E_3$) can be used in the Excel sheets. In order to cover the average slope of the stress-strain diagram, the medium elastic modulus, $E_2$, was used for all calculations. For example, for the second floor test this would be $E_2 = 14178$ MPa (Figure 4-4).

In Figure 5-61, the results of all three elastic moduli are shown. In reality, a combination of all three elastic moduli, or, even more accurately, the real curve of the stress-strain diagram
defines the concrete behaviour. However, the Excel program was used to calculate the extreme cases of the whole floor consisting of a constant low elastic modulus of 11677 MPa or a constant high elastic modulus of 16816 MPa (for the second floor test). From Figure 5-61, it can be observed that, for these cases, the load-deflection curves do not differ much and, therefore, the Excel simulation with a constant medium elastic modulus of 14178 MPa should be accurate enough.

In a standard compression test, the concrete peak stress value typically occurs at a strain value of about 0.002. This agrees well with the recordings from the three different concrete types in Figure 4-4. Normally, a falling branch of the stress-strain curve can be observed after this peak load, reaching an ultimate strain value between 0.003 and 0.004. However, this post-peak behaviour is not modelled in the Excel sheets. As mentioned previously, the stress-strain curve of concrete in compression is modelled as one straight line with the medium elastic modulus $E_2$ from Figure 4-4. The post-peak behaviour of the concrete is only important if some concrete parts in the cross section exceed the peak stress value. From Figure 4-4 it can be seen that the peak stress value for the second test has a value of
20.8 MPa and a corresponding strain value of 1780 με. This is still significantly less than the 0.003 to 0.004 strain at which concrete compression failure is initiated.

The maximum compression strain value for the floor test should be at any time at the top surface of the cross-section. The macro, programmed in the Excel sheets, calculates the strain distribution in the cross-section for every given load in “Matrix 1” (Figure 5-40). The second column, “strain top”, shows the calculated strain value at the top concrete surface. However, Figure 5-40 is only showing a selection of the sheet “Matrix 1”, which is the part of the first 60-kN steps. Figure 5-62 shows a selection of the same sheet for the last 5-kN steps and for the first six columns. It can be seen that only in the last row, at a load of 194 kN, is the peak strain value of 1780 με exceeded. Although the peak compression strain is not checked automatically, this manual check shows that the calculations up to 194 kN are correct, as they are totally independent of the post-peak behaviour of the concrete.

Figure 5-40 shows also the rapid strain increase for the last kN-steps. The corresponding points in the load-deflection curve can be seen in Figure 5-54, where the curve is nearly horizontal at the maximum load level.

<table>
<thead>
<tr>
<th>F_{machine} [kN]</th>
<th>ε_{strain top} [10^{-6}]</th>
<th>ε_{strain bottom} [10^{-6}]</th>
<th>z [mm]</th>
<th>ε_{reinf} [10^{-6}]</th>
<th>M [kNm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>190</td>
<td>-1053.58</td>
<td>5847.88</td>
<td>76.33</td>
<td>5295.77</td>
<td>224.76</td>
</tr>
<tr>
<td>191</td>
<td>-1119.78</td>
<td>6612.64</td>
<td>72.41</td>
<td>5994.05</td>
<td>225.89</td>
</tr>
<tr>
<td>192</td>
<td>-1201.97</td>
<td>7640.00</td>
<td>67.97</td>
<td>6932.65</td>
<td>227.01</td>
</tr>
<tr>
<td>193</td>
<td>-1315.30</td>
<td>9207.65</td>
<td>62.50</td>
<td>8365.81</td>
<td>228.14</td>
</tr>
<tr>
<td>194</td>
<td>-2415.47</td>
<td>34796.75</td>
<td>32.46</td>
<td>31819.77</td>
<td>229.26</td>
</tr>
</tbody>
</table>

Figure 5-62: Extract from “Matrix 1” for the last kN-steps

The stress-strain diagram for concrete in tension for the second floor test is shown in Figure 4-9. The Excel program uses a very similar graph as demonstrated in Figure 5-63. Generally, the elastic modulus in tension is either similar or the same as the elastic modulus in compression at a low stress level. Therefore, Figure 4-9 and the graph from the experiment in Figure 5-63 both use a value of 16816 MPa. Instead, the Excel program uses
The medium elastic modulus of 14178 MPa, which causes the small difference in the slope in Figure 5-63 before the concrete cracks.

FEM programs like Abaqus have difficulties in modelling concrete behaviour in tension in the way that the Excel program has been set up to do. Therefore, different ways are needed to model such behaviour in Abaqus, as will be discussed in detail in section 6.2.

Figure 5-63 shows, with the red line, a different concrete diagram where the maximum tension strength of 1.3 MPa is close to the strength after the concrete cracks with 1.2 MPa. As the concrete strength rapidly decreases after cracking, this behaviour causes problems when using finite-element programs. Different approaches to avoid these problems are discussed in the finite-element chapter in section 6.2. To use the material behaviour as displayed with the red lines in Figure 5-63 is one possibility. The small jump from 1.3 MPa for the maximum stress value to 1.2 MPa is only used in the Excel program to obtain a diagram shape the program is intended for. In Abaqus, this model would match a diagram with a maximum strength of 1.2 MPa. Figure 5-64 displays the result of the two concrete tension models shown in Figure 5-63 (Excel 1 and 2).

This is an advantage from the Excel program, as the results from different models are able to be compared with the results from the experimental model, with the steep stress decrease after cracking. This behaviour cannot be modelled directly with Abaqus.
Figure 5-63: Different concrete tension diagrams, floor test 2

Figure 5-64: Results of different concrete tension diagrams, floor test 2

Figure 5-64 shows that the slopes of the graphs at low load levels are different, as a result of the use of different tensile strengths in the models. But generally, the differences in the load-deflection curves are very small.
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Therefore, the influence of the maximum tension force on the load-deflection curve is not of major interest. More important is the area under the stress-strain curve, which is known as fracture energy. As mentioned in the theory of the Excel calculation, this area affects the force which determines the bending-moment. The area under the graph in Figure 5-63 before the concrete cracks is relatively small. Therefore, it causes a small force and bending-moment difference as well.

5.6.5 Discussion of the load-deflection curve

The load-deflection curve at midspan, as simulated with the Excel program, can be closely approximated by two lines before the rebars start yielding. The two different slopes are demonstrated in Figure 5-65 and Figure 5-66 for floor test two and three.

Figure 5-65: Two line simplification, floor test 2
The point of intersection of the two lines coincides with the initiation of concrete cracking. For the third floor test, this cracking behaviour simulated with the Excel program matches the curve from the experimental test very closely. Up to about 60 kN, the graph from the Excel simulation is nearly identical to the experimental graph.

In Figure 5-65, the graph from the second experimental floor test is almost perfectly linear for up to about 50 kN and does not show the typical slope change predicted by the theory when the concrete starts cracking. The question is: why not?

The steel beam which is loading the floor was seated in all experimental tests on fast curing plaster to the top floor surface. This was to ensure, there was a solid support of the beam without any gaps. In the second test, the beam was pressed by the testing machine against the floor to distribute the wet plaster. By mistake, the load used was significantly higher than necessary. As soon as the mistake was recognised, the load was immediately removed. The loading time was about five seconds. It was not recorded how high the load was. No visible displacement or deformation was observed.
After analysing the load-deflection curve from Figure 5-65, it is likely that the floor was loaded with at least 50 kN. This would explain why the curves from the test and the Excel program differ. If the floor was loaded with 50 kN before the recorded test was started, the concrete would have been loaded above the crack initiation threshold. Therefore, the recorded graph is not the first loading behaviour of the floor as simulated in the theory. Instead, the recorded data from the experimental test show the load behaviour of the second load cycle, which is less stiff and follows the slope of the second model line for the post-cracking regime.

It has to be noted that there are other parameters which are neglected in the theoretical model, which may also influence the cracking behaviour. For example, the concrete shrinkage characteristics lead to tensile stresses in the concrete. Especially at low loads, this could influence the results. The movement of the floor element under the testing machine prior to the test also influences the floor behaviour at a low load level.

The slope of the load-deflection curve for the second experimental test at a load of about 60 kN matches very well the slope from the Excel simulation, as shown in Figure 5-65. At higher loads, the graph from the experimental test shows decreasing stiffness and increases the gap between experiment and theory. The same behaviour can be observed in the third floor test, as shown in Figure 5-66. At loads over 60 kN, the deflection from the experimental test does not increase in proportion to the load. The gap between the results of the experimental test and the Excel simulation increases at higher load levels.

The reason for the different results is that the Excel simulation assumes no slip between the rebar and the concrete at all times, however, as detailed in section 4.3.5.4 and 4.3.6.4, slip occurred during the experimental floor tests. It was shown that the slip started abruptly at a specific load. Therefore, the graphs from the experimental test and from the Excel simulation are very similar at low load levels, where the assumption of no slip is true. At higher load levels, the influence of the slip behaviour increases and also, therefore, the gap between experiment and theory.

For the third floor test in Figure 5-66, the gap is bigger than in the second floor test in Figure 5-65. The results from the second experimental floor test show a graph which is
relatively close to a straight line before the rebars start yielding. However, in the third floor test, the graph is smoothly curved up to the point of failure. No slope change, as the rebars start yielding, can be observed in this test. This is caused by the fact that the slip region in the third test extended to midspan, which did not happen in the second floor test. When slip extends to midspan, the maximum theoretical load cannot be reached. This behaviour was explained in section 4.3.6.4.

For the second floor test, slip was detected only at the ends and not at midspan, and so the maximum load in the experimental test and the Excel simulation was nearly identical. The difference between the experimental and theoretical graph is moderate. However, in the third floor test, where slip was detected at midspan, the maximum load from the experimental test was less than the result from the Excel program. Because of the bigger spread of slip in the third test compared to the second test, the gap between theory and experiment is also bigger.

Rebar slip was not included in the theoretical model, as it was the intention from the outset to detail the reinforcing and its interconnection in such a way that the slip was either prevented or confined to a small region. This condition was only achieved in test number two and requires further research, as noted in section 7.

5.6.6 Conclusions

The outcomes of the theoretical model as implemented in the Excel program compared with the observed performance of test two and three can be summarised as follows:

- With the help of the Excel program, it is possible to theoretically model the floor behaviour. The load-deflection diagram at midspan, which is one of the main interests, can be predicted by the program.
- In reality, slip was detected during the three experimental tests. Slip at the ends of the floor, but not at midspan, did not affect the maximum load of the floor for test number two, although it did affect the deflections of the floor, which were higher than the results from the model which assumes no slip.
- In the third floor test, slip was also detected at midspan. This resulted in a decrease of the maximum load capacity of the floor. Therefore, the maximum load in the
third experimental test was less than the results from the theory. As slip is more widespread in the third floor test compared to the second test, the gap between theory and experiment is also bigger in the third test.

- When the floor cracks the graph from the load-deflection curve significantly changes its slope. In the third floor test, this behaviour corresponds very well between theory and experiment. In the second floor test, this behaviour could not be observed. This is very likely due to a preload of at least 50 kN leading to cracks at midspan prior to the loaded deflections of the floor being recorded. Therefore, the measured deflections of the second experimental test were the second load cycle (up to 50 kN) and not the first. Only the first load cycle can show the typical crack initiation behaviour.

- With the Excel program, some situations can be modelled which cannot be modelled in a finite-element program. The after-crack behaviour of concrete often leads to severe numerical problems in finite-element programs. Normally, it is difficult to model the rapid decrease of the concrete tension strength immediately after cracking. This behaviour can be modelled with the Excel program without difficulty. Therefore, different concrete tension models were analysed and compared with the Excel program. A simplified model which best agrees with the real behaviour can also be transferred to, and used in, the finite-element analysis.

- It was shown that the simulation of the load-deflection curve, with the constant medium modulus of elasticity of concrete, is sufficiently accurate.

### 5.7 NATURAL FREQUENCY

The transverse natural frequency $\omega_1$ in radius/ sec for a simply supported beam is given in Eq.(5.234).

$$\omega_1 = \frac{\pi^2 EI}{l^4 \rho A}$$

(5.234)

where $\rho$ is the density of the beam.

To use the natural frequency, $f_1$ (Hz), and the self-weight, $q$, Eq.(5.234) can be transformed to Eq.(5.235).
5.7 Natural Frequency

\[ f_i = \frac{\omega_i}{2\pi} = \frac{\pi}{2} \sqrt{\frac{EI}{I^4 \rho A}} = \frac{\pi}{2} \sqrt{\frac{EI}{I^4 \frac{q}{g}}} = \frac{\pi}{2} \sqrt{\frac{gEI}{I^4 q}} \]  (5.235)

where \( g \) is the standard gravity.

With the moment of inertia for the composite section, \( I_{\text{comp}} \), calculated in section 5.4.1, and the line load, \( q \), for the first floor test from section 4.3.4.1, Eq.(5.235) becomes:

\[
\left( \frac{9.81 \text{ m} \cdot \text{s}^2}{\text{s}^2} \right) \frac{205000 \text{ N mm}^2}{\text{mm}^2} \cdot \frac{0.000465 \text{ m}^4}{(4.5 \text{ m})^4} \cdot 5 \frac{0.000465 \text{ m}^4}{(4.5 \text{ m})^4} = 33.5 \text{ Hz} \]  (5.236)

With the corresponding values for floor test two and three, Eq.(5.235) can be transformed to Eq.(5.237) and Eq.(5.238).

\[
\left( \frac{9.81 \text{ m} \cdot \text{s}^2}{\text{s}^2} \right) \frac{205000 \text{ N mm}^2}{\text{mm}^2} \cdot \frac{0.000295 \text{ m}^4}{(4.5 \text{ m})^4} \cdot 4.35 \frac{0.000295 \text{ m}^4}{(4.5 \text{ m})^4} = 28.7 \text{ Hz} \]  (5.237)

\[
\left( \frac{9.81 \text{ m} \cdot \text{s}^2}{\text{s}^2} \right) \frac{205000 \text{ N mm}^2}{\text{mm}^2} \cdot \frac{0.000223 \text{ m}^4}{(4.5 \text{ m})^4} \cdot 2.82 \frac{0.000223 \text{ m}^4}{(4.5 \text{ m})^4} = 30.9 \text{ Hz} \]  (5.238)

The measured values for the natural frequency in section 4.4 were 26 Hz, 19.5 Hz and 22 Hz, for floor tests one to three. This shows that the natural frequencies in the experimental tests are all lower than in the theory.

The theory assumes an unloaded, and therefore uncracked, cross-section with the highest possible value for the composite moment of inertia. But the floor at the experimental test is at least loaded with its dead load, which still should not change the composite moment of inertia. Figure 5-57 shows the moment of inertia decreased rapidly after the concrete cracked in the cross-section at a relative low load level. During preparations for the load test, the floor element is moved under the test machine and the formwork was taken off. During that procedure, the floor had to resist some unmeasured pressure at a low level on the top of its own dead load.
One explanation of the lower natural frequency in the experimental test could be that these preloading additional to the dead load could cause microcracks, which decreased both the moment of inertia and the natural frequency.

Microcracks produced by concrete shrinkage are another process which is not further analysed in this thesis and which would also decrease the moment of inertia.

The natural frequencies will also be measured with the Abaqus model in section 6.6.

The natural frequency was experimentally measured in section 4.4 and theoretically modelled in this section for a floor length of 4.5 m by a width of 1.2 m. However, only the frequency of the final 9-m product is of real interest. From Eq. (5.235) it can be seen that if the length, $l$, is multiplied with the factor 2, the natural frequency has to be multiplied with the factor $\frac{1}{4}$ if everything else stays the same. Therefore, the natural frequencies for the 9-m floor can be estimated from the results of the 4.5-m model. For further examination the experimentally measured frequencies from section 4.4 are used to predict the 9-m floor behaviour in Table 5-3.

<table>
<thead>
<tr>
<th></th>
<th>Test 1</th>
<th>Test 2</th>
<th>Test 3</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>4.5m</strong> experimental tested in section 4.4</td>
<td>26 Hz</td>
<td>19.5 Hz</td>
<td>22 Hz</td>
</tr>
<tr>
<td><strong>9.0 m</strong> factor 1/4</td>
<td>6.5</td>
<td>4.9</td>
<td>5.5</td>
</tr>
</tbody>
</table>

The design of the floor regarding the vibration behaviour is generally a serviceability issue. The floor will not break or fail if the natural frequency will not perfectly match the recommended values. However, recommended values are given in several publications to prevent discomfort for people inside the building. Mast, for example, discussed the vibration behaviour for precast concrete floors (Mast, 2001). In this paper the research of
Allen and Murray in (Applied Technology Council, 1999), (AISC, 1997) and (Allen and Murray, 1993) is emphasised.

These publications use a frequency versus peak acceleration diagram (Figure 5-67) to define the vibration acceptability for different environments.

![Figure 5-67: Recommended permissible peak vibration acceleration levels acceptable for human comfort while in different environments (Applied Technology Council, 1999)](image)

The floor system is satisfactory if the following equation is fulfilled (Applied Technology Council, 1999, equation 2-3) or (AISC, 1997, equation 4.1):

\[
\frac{a_p}{g} = \frac{P_r e^{-0.35f_a}}{\beta W g} \leq \frac{a_0}{g}
\]

(5.239)

where \( a_p \) = estimated peak acceleration due to walking

\( a_0 \) = acceptable peak acceleration due to walking

\( g \) = acceleration due to gravity
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\[ P_0 = \text{constant force representing the excitation in kN} \]
\[ = 0.29 \text{ kN for buildings} \]
\[ = 0.41 \text{ kN for footbridges} \]

\[ f_n = \text{fundamental natural frequency in Hz} \]

\[ \beta = \text{modal damping ratio} \]
\[ = 0.01 \text{ for footbridges} \]
\[ = 0.03 \text{ for a typical office area} \]
\[ = 0.05 \text{ for office floors with full-height room partitions} \]

\[ W = \text{effective weight of the floor in kN} \]

The effective weight, \( W \), might be calculated as follows:

\[ W = wBL \tag{5.240} \]

where \( w = \text{uniformly distributed weight per unit area (actual dead and actual live load)} \)

\( L = \text{floor span} \)

\( B = \text{effective width} \)

with

\[ B = C \left( \frac{D_{\text{perp}}}{D_{\text{par}}} \right)^{0.25} \leq \frac{2}{3} B_{\text{total}} \tag{5.241} \]

where \( C = \text{perpendicular flexural continuity factor from 1.0 (edge) to 2.0 (middle)} \)

\( D_{\text{par}} = \text{transformed moment of inertia per unit width of floor parallel to the span} \)

\[ = \frac{I_{\text{par}}}{b} \text{ with } b = \text{beam spacing} \]

\( D_{\text{perp}} = \text{transformed moment of inertia per unit width of floor perpendicular to the span} \)

with

\[ D_{\text{perp}} = \frac{d_e^3}{12n} \tag{5.242} \]

where \( d_e = \text{effective depth} \)

\[ n = \frac{E_s}{1.35E_c} = \text{dynamic modular ratio} \]
5.7 Natural Frequency

The theory above is applied for the parameters from the second floor test with the dimensions of one real size floor element (9 m long and 3 m wide). With the effective depth of the floor slab assumed to be \( d_e = 0.12 \text{ m} \) and

\[
\frac{n}{1.35} = \frac{E_s}{1.35 \cdot 16816 \text{ MPa}} = 8.81
\]  

(5.243)

the transformed moment of inertia per unit width for the perpendicular floor span is given by:

\[
D_{\text{perp}} = \frac{d_e^3}{12n} = \frac{(0.12 \text{ m})^3}{12 \cdot 8.81} = 1.63 \times 10^{-5} \text{ m}^3
\]  

(5.244)

For the transformed moment of inertia per unit width for the parallel floor span it is assumed that the effective moment of inertia for the 3-m wide specimen is about two times the amount calculated in Figure 5-19.

\[
D_{\text{par}} = \frac{I_{\text{par}}}{b} = \frac{2 \cdot 2.95 \times 10^{-4} \text{ m}^4}{3.0 \text{ m}} = 1.97 \times 10^{-4} \text{ m}^3
\]  

(5.245)

Therefore, the effective width may be calculated as:

\[
B = C \left( \frac{D_{\text{perp}}}{D_{\text{par}}} \right)^{0.25} L = 2.0 \left( \frac{1.63 \times 10^{-5}}{1.97 \times 10^{-4}} \right)^{0.25} * 9 = 9.65 \text{ m} \leq 2.0 \text{ m} = \frac{2}{3} B_{\text{total}}
\]  

(5.246)

With the line load of 4.35 kN/m from (4.25) for the 1.2-m wide floor specimen, the effective weight of the full-size floor yields to:

\[
W = wBL = \frac{4.35 \text{ kN}}{1.2 \text{ m}} * 2.0 \text{ m} * 9.0 \text{ m} = 65.3 \text{ kN}
\]  

(5.247)

Therefore, the estimated peak acceleration ratio from (5.239) may be calculated with the following equation.

\[
\frac{a_p}{g} = \frac{P_f e^{-0.35f_f}}{\beta W} = 0.29 * e^{-0.35+4.9} \approx 0.027 = 2.7\%
\]  

(5.248)
From Figure 5-67 it can be seen that the peak acceleration ratio for offices is limited to 0.5%. Therefore, the vibration acceptance criterion for human comfort in office buildings is not fulfilled. However, for car parks or buildings with less strict requirements the floor may be suitable.

If the proposed floor product is intended to be used in office buildings, further research is necessary to show that the human response to floor vibration can be made acceptable. Frequency measurements on experimental real-size floor slabs are recommended, following for example the testing protocol in (AISC, 1997) or (Smith et al., 2009).

In Eq.(5.248), the modal damping ratio, $\beta$, was assumed to be 0.03 for a typical office area as recommended in (AISC, 1997). However, for a more accurate approach it is recommended to conduct real size floor tests and calculate the modal damping ratio from the measured response curve or to use finite element analysis. Furthermore, with the finite element program, the real floor support conditions at the longitudinal side could be modelled, which would positively increase the damping ratio compared to a single floor element. With the finite element results, the floor could be analysed according to the general assessment method given in (Smith et al., 2009).

### 5.8 CONCLUSIONS

The conclusions of the chapter 5 can be summarised as:

- The theoretical model implemented in the Excel program is able to analyse the floor behaviour under load and model the load-deflection curve at midspan based on the assumption of no slip between rebar and concrete. The detailed conclusions of the Excel program are given in section 5.6.6.

- To explain and verify the theory, a hand calculation was done in an example in section 5.3. The bending and shear deflection could be calculated with the unit load method, as explained in section 5.5.

- The natural frequencies measured in the experimental tests were between 22% and 32% lower than the results from the theory. It is not totally clear where this
difference comes from. In section 6.6, the behaviour has been analysed further with the finite-element program.
Theoretical Modelling
6 FINITE-ELEMENT ANALYSIS

6.1 INTRODUCTION

To investigate the behaviour of the experimental test specimens, the finite-element program Abaqus, version 6.7-1, was used. The objectives of the finite-element analysis were:

- To analyse and compare its results with the experimental tests and with the results from the theoretical modelling.
- To model and analyse the support condition as actually implemented. The Excel program assumed a simply supported beam, but was not able to model the support on the SHS. Abaqus was expected to be able to calculate a model supported either on the SHS or on the bottom of the concrete web. The difference should show how accurate the assumption in the Excel simulation was.
- To model the floor behaviour into the inelastic range more accurately than the hand method could achieve, taking account of bearing, moment and shear interaction.

The Abaqus model assumes, for simplicity, that the steel parts are totally connected to the concrete throughout the entire time. Therefore, the model neglects any slip between these parts. Even with this simplification, finite-element programs often have a problem modelling the behaviour after the concrete cracks. Section 6.2 explains different possibilities to avoid this problem. It was intended to develop a floor reinforcing system that was detailed and interconnected in such a way that slip was prevented or confined to small end regions. Consequently, it was not regarded as a high priority for inclusion in the theoretical or finite-element models.

Section 6.3 comments on how the floor is simulated in Abaqus and defines all parameters used in the model.

The results of the Abaqus analysis can be found in section 6.4.
In section 6.5, the load-deflection curve obtained from the Abaqus model is discussed and compared with the results from the Excel program and with the results from experimental testing.

Section 6.6 calculates the natural frequencies for all three floor tests and compares these values also with the results from the theory and experimental section.

6.2 MODELLING REINFORCED CONCRETE

A simply supported concrete beam with rectangular cross-section and one or two rebars loaded at midspan is a very simple construction. However, in Abaqus, it is a highly complex process to model this construction in detail with exactly the same structural characteristics. The following statements are applicable to a range of finite-element programs and are not limited to Abaqus alone.

The difficulty lies in modelling the composite-material reinforced concrete. The concrete material itself is normally modelled with enough accuracy as a homogenous material. Steel or concrete parts under compression generally lead to good results in Abaqus, if the materials are not mixed in the structure. The problems begin when the different materials are modelled in one construction together, as in a reinforced concrete structure, and when the concrete starts to crack under tension stresses:

1. The connection between the two materials needs to be modelled in Abaqus. The easiest way to do this is to “tie” these parts together so that they are totally connected, similar to a stiff welding line. If the concrete cracks, the finite concrete elements can separate. But as soon as the cracking gap reaches the rebar, the FEM program has an unsolvable problem: On one hand, the tension forces in the concrete try to separate the two finite concrete elements. On the other hand, the tied connection between the rebar and the concrete holds the concrete elements in place. The finite concrete elements connected to the rebar cannot separate and stay in place at the same time.

2. When a single material cracks, it is often seen as the failure point, and the FEM model normally stops. However, in a reinforced concrete beam, the failure point of the beam is not reached when the concrete cracks under bending. The tension
There are different ways to prevent the first problem. One solution is to model the rebar as an “embedded” element, as is done in this thesis. The embedded element technique is used to specify a group of elements which lie embedded in a group of host elements and can be used to model reinforcement. The response of the host elements are used to constrain the translational degrees of freedom of the embedded nodes.

Different approximation methods are possible for the second problem as well. Most methods modify or avoid the sudden decrease of stress after cracking in the material diagram of concrete.

After the concrete has cracked, some material between the cracks, where the bond has not been destroyed, can still carry tension forces from the rebar to the surrounding concrete. This behaviour may be considered as an increase in stiffness in the rebar and is therefore known as the “tension-stiffening effect”. This effect can be modelled either in the material diagram of the rebar or in the material diagram of the concrete, where it defines the after-cracking behaviour. Therefore, the same topic can also be understood as “concrete-softening behaviour”. But it has to be noted that the most perfect theoretical model is not necessarily the best choice for the material definition in an FEM program. To keep Abaqus running, it is sometimes necessary to model a smoother after-cracking behaviour of concrete than that which results from a material test.

Many researchers are working on a concept to model concrete in tension in general or specifically in an FEM program. Winkler (2001) gives a general overview of concrete behaviour under tension, and the Comite Euro-International du Beton (1993) presents a model code which provides more details of a concrete model than most national codes do. Carpinteri and Aliabadi (1999) and Maekawa et al. (2003) describe and analyse different models for concrete and reinforced concrete structures. A collection of papers relevant to this topic are given in (Hendriks et al., 2002), (Konsta-Gdoutos, 2006) and (Meschke, 2006).
Several papers discuss the after-cracking behaviour in the stress-strain diagram of the concrete. In particular, if the graph in the stress-strain diagram after the crack should be linear, bilinear or parabolic. However, a lot of parameters influence the after-cracking behaviour in a reinforced concrete construction, which makes it difficult to find a general solution. Specifically, steel fibres or other material fibres added to the concrete mix do not normally increase the tension strength of concrete, but they do influence the after-cracking behaviour significantly. Therefore, in most cases individual material tests are necessary to define the after-cracking behaviour.

Abaqus offers three different concrete models as described in the Abaqus Analysis User’s Manual (Simulia., Abaqus Version 6.7-1):

1. The “smeared crack concrete model”, which is available in Abaqus/Standard only.
2. The “brittle cracking model”, which is available in Abaqus/Explicit only.
3. The “concrete damaged plasticity model”, which is available in Abaqus/Standard and Abaqus/Explicit.

Each of these three models is designed to provide a general capability for modelling concrete. The most flexible model, which can also be used in both Abaqus programs, Standard and Explicit, is the concrete damaged plasticity model. For the floor analysis in this thesis, the concrete damaged plasticity model was used.

The input values for Abaqus are split into elastic and plastic data. The elastic modulus and Poisson’s ratio have to be defined separately; the rest is defined in the concrete damaged plasticity model. Some general plasticity parameters have to be defined in this model first. Therefore, the default values or the values recommended from Abaqus were used. After this, the compressive behaviour has to be given in tabular data of stress and plastic strain values. The tensile behaviour can be modelled in three different ways: First, the stress and plastic strain table is one option. Secondly, the table may be filled with stress and plastic displacement values. Or, as a third option, it may be filled with stress and fracture energy values. All three options describe the same thing: the post-failure behaviour of the concrete. Therefore, it is not relevant for the data definition which method is being used.
The concrete material was tested in section 4.2.1 with the results summarised in a stress-strain diagram. Therefore, the stress and plastic strain table was used in this thesis.

The simplified stress-strain diagram from the second floor test, as analysed in section 4.2.1, is repeated in Figure 6-1. Abaqus recommends in the Abaqus Analysis User’s Manual (Simulia., Abaqus Version 6.7-1) a linear decreasing stress value after the concrete cracking point. The strain at zero stress should be about ten times the strain value at the crack point. This recommendation is also plotted in Figure 6-1.

Due to the fact that Abaqus cannot model the real concrete behaviour in tension, this recommendation at least keeps the model running. But Figure 6-1 shows as well that although agreeing at maximum tensile stress, the Abaqus model does not align closely with the experimental data in the post-cracking region.

In this thesis, a different approach, which is based more on the experimental test data was used as demonstrated in Figure 6-2. The difference in the load-deflection curve between this model and the real crack behaviour is very small as this comparison was already

---

Figure 6-1: Abaqus recommendation for after crack behaviour, floor test 2
analysed with the Excel program in section 5.6.4 and shown in Figure 5-63 and Figure 5-64.

In order to understand the impact which the concrete material behaviour in tension has on the load-deflection curve of the floor, four different concrete models were analysed with the Excel program (Figure 6-3). The first one is the same as the one used in the Excel calculation for the second floor test. The next one overestimates the concrete post-cracking stress with a linear stress reduction after the crack point. The blue line underestimates the concrete behaviour with a stress decrease to zero after the concrete cracks. And finally, the green line simulates a concrete material with nearly no tension strength.

Because the Excel program normally requires input data with a jump in the stress value after cracking, the jump distance is set for the red example to a very small value of 0.01 MPa, from 4.07 MPa to 4.06 MPa. The influence of this small adjustment on the results is insignificant, and can be ignored.
The load-deflection curves for the four models are shown in Figure 6-4. This diagram demonstrates the influence of the concrete material definition in relation to the load-deflection curve of the floor. A final strain value of 3500 $\mu \varepsilon$ was kept constant. Otherwise, even more concrete diagram variations would be possible.

Figure 6-3: Different concrete cracking models, floor test 2
Figure 6-4: Load-deflection curve for different concrete models, floor test 2

It can be seen that the modelling of the concrete tensile behaviour has a significant influence around the initial cracking load. It has relatively little influence on either subsequent stiffness or ultimate load.

### 6.3 FINITE-ELEMENT MODEL

Because the tested T-section in the experimental test is double symmetric, only one quarter of it (half length and half width) was modelled in Abaqus as seen in Figure 6-5.
The floor is supported at the SHS as a simply supported beam and loaded with a uniform area load over a small rigid beam of 50 mm by 600 mm.

The main intention of the Abaqus model was to find the maximum floor capacity, and also to monitor the deflection of the slab as the load increases. This can be achieved best with the “Riks method”, which is a displacement-controlled analysis procedure implemented in Abaqus.

In most analysis cases, the load level is given and the displacement is the unknown, which the program should calculate for that particular situation. However, the Riks method uses the load magnitude as an additional unknown. In the beginning of the Abaqus calculation, the model requires the type of load, the location and also the magnitude of the load. The magnitude of the load is not constant instead it is going to be increased during analysis by
the load proportionality factor (LPF). Abaqus starts with a low LPF and increases that factor step by step. For example, if the highest factor equals two, the maximum load for that floor is two times the given load value at the beginning. Because the Abaqus results should be easily comparable with the load-deflection curve of the test, the load proportionality factor is set to 1.0 at the maximum load reached at the experimental test.

For the first experimental floor test with a maximum measured load of 121 kN, the beam pressure, \( p \), in the model is therefore set to:

\[
p = \frac{121000 \text{ N}}{4 \times 50 \text{ mm} \times 600 \text{ mm}} = 1.01 \frac{\text{N}}{\text{mm}^2}
\]

(6.1)

The pressure for the second and third test is calculated accordingly in Eq.(6.2) and Eq.(6.3).

\[
p = \frac{157000 \text{ N}}{4 \times 50 \text{ mm} \times 600 \text{ mm}} = 1.31 \frac{\text{N}}{\text{mm}^2}
\]

(6.2)

\[
p = \frac{186000 \text{ N}}{4 \times 50 \text{ mm} \times 600 \text{ mm}} = 1.55 \frac{\text{N}}{\text{mm}^2}
\]

(6.3)

As only one quarter of the experimental tested floor is modelled, only one quarter of the final load from the experiment equals the final load in the Abaqus simulation, which is indicated by the number four in the denominator.

All steel parts are modelled individually with their corresponding material behaviour. The steel sheet, the SHS and the 6-mm steel plate are “merged” together into one part as seen in Figure 6-6, which implies that they cannot separate anymore. The same was achieved in the experimental test by the welding lines between the steel parts. The rebar is constrained with the type “tie” to the 6-mm steel plate, which simulates a stiff welding connection as well.
All steel parts were “embedded” in concrete, which constrains the translational degree of freedom between the steel part and the concrete. In other words, no slip is possible between these parts. The main reason for the “casement doors”, according to Figure 3-14, is to connect the concrete to the steel sheet and to prevent translational slip. Because that behaviour is already simulated in Abaqus, the “casement doors” can be left out in the model, so that only the holes are simulated as demonstrated in Figure 6-6.

FEM programs often have problems with the corners of rectangular holes, therefore, the corners of the holes are rounded out with a radius of 5 mm. Depending on the selected mesh size, these rounded corners were simplified automatically by Abaqus to linear polylines in the analysis.
The rebar was modelled as a “truss” element, which supports loading only along the axis. Therefore, it is particularly suitable for long slender elements. The steel sheet is modelled as a “shell” element, which is generally used for structures where one dimension, the thickness, is significantly smaller than the other dimensions. All other materials were generated as common “solid” elements.

The material data given in section 3.3.2 are “nominal” stress and strain data. The nominal stress is also called engineering stress and is the force per unit un-deformed area. But, most FEM programs, including Abaqus, require “true” stress and strain values for their material definition, where the true stress is the force per deformed or current area. Therefore, the material values from section 3.3.2 need to be transformed, before they can be used as input values in Abaqus. Eq.(6.4) converts nominal stress, \( \sigma_{\text{nom}} \), to true stress, \( \sigma_{\text{true}} \).

\[
\sigma_{\text{true}} = \sigma_{\text{nom}} (1 + \varepsilon_{\text{nom}}) \tag{6.4}
\]

And Eq. (6.5) calculates the true strain, \( \varepsilon_{\text{true}} \), which is also called logarithmic strain, out of the given nominal strain value.

\[
\varepsilon_{\text{true}} = \ln (1 + \varepsilon_{\text{nom}}) \tag{6.5}
\]

Instead of the total strain, which is the sum of the elastic and the plastic strain, Abaqus requires the plastic strain values plus the elastic modulus, \( E \). The input value for the plastic true strain can be obtained from:

\[
\varepsilon_{\text{pl,true}} = \varepsilon_{\text{total,true}} - \varepsilon_{\text{el,true}} = \varepsilon_{\text{total,true}} - \frac{\sigma_{\text{true}}}{E} \tag{6.6}
\]

From Eq.(6.4) and Eq.(6.5), it can be concluded that the difference between nominal and true data is only significant for high strain values and could therefore be neglected in the elastic range. However, this thesis uses true values for all material definitions in Abaqus.

Additional to the supports conditions, the symmetric boundary conditions also had to be modelled. Therefore, the displacement in x direction, the rotation about the y-axis and the rotation about the z-axis have to be zero for the surface at the cut of 2.25 m of the floor length. Similarly, the displacement in the z direction, the rotation about the x-axis and the rotation about the y-axis have to be zero for the surface at the cut of 0.6 m of the floor width. The orientation of the coordinate system is displayed in Figure 6-5.
A typical mesh of the steel parts is displayed in Figure 6-7, and a typical concrete mesh is displayed in Figure 6-8.

Figure 6-7: Example of meshed steel parts
An 8-node linear brick reduced integration hourglass controlled element (C3D8R) was used for all solid sections. These sections are the concrete, the SHS and the 6-mm steel plate. The C3D8R element is a basic brick element with nodes only at the corners and is commonly used in 3-D models. The steel sheet is modelled with a similar element type for shells (S4R) and, therefore, this element has four nodes at the corners. Two-node linear 3-D truss elements (T3D2) are used for the rebar.

**6.4 RESULTS**

In order to compare the Abaqus results with the results from the Excel program and with the results from the experimental test, the load-deflection curve generated from Abaqus results is of major interest.
The results of an FEM program depend on a lot of parameters. Therefore, the results always have to be checked carefully. For example, if the maximum floor capacity of a floor element is investigated, and the model stops at a certain stage, it has to be analysed, if the model stops, because the final load is reached or because of other problems in the model. An important part in the model is the rebar. If the rebar has not reached its yielding stress at midspan, it is an indication that the maximum floor capacity has not been reached, and that the floor model has stopped because for other reasons. In the Abaqus simulation, the maximum stress and strain value in the rebar was checked at the step, where the Riks method stopped. The program shows with the values of 587 MPa and $8368 \mu e$ for the simulated second floor test an expected stress distribution, where the rebar reached the yielding stress of 576.6 MPa (true stress) in the middle of the floor slab.

Figure 6-9 shows the von Mises stress distribution in the steel sheet. The red part presents yielded parts or parts close to the yielding stress measure of 410.8 MPa (true stress) at midspan. The right part closer to the support is still in the elastic range.
Finite-Element Analysis

Figure 6-9: Von Mises stress distribution in the steel sheet at maximum load, second floor test

Figure 6-9 is displayed in the deformed condition, but the deformations are so small compared to the floor length that it is difficult to notice the deformations. Figure 6-10 shows the same stress measure distribution in the steel sheet, but with a 10-times bigger deformation.
Figure 6-10: Deformation with a deformation scale factor of 10 at maximum load, second floor test

Figure 6-11 displays the load-deflection curve from the model compared with the load-deflection curve from the experimental test. As mentioned in the previous section, the Riks method uses a Load Proportionality Factor (LPF) which is displayed on the y-axis. A LPF of 1.0 equals the final load from the second experimental test of 186 kN.

The point where the rebars start to yield and the curve changes its slope rapidly is shown clearly in the graph.
The floor from the experimental test has to carry its own dead load in addition to the load applied from the test machine. In contrast to that, the Abaqus model is not carrying the dead load. In order to compare both results, the diagram from the Abaqus model is shifted in the point of origin approximately by the influence from the dead load as seen in Figure 6-11. With Eq.(6.7), the dead load (line load) is transferred to a corresponding point load.

\[
\begin{align*}
        u_{\text{line load}} &= \frac{5ql^4}{384EI} = \frac{Fl^3}{48EI} = u_{\text{point load}} \\
        \Rightarrow F &= 0.625ql
\end{align*}
\]  

(6.7)

With the corresponding dead loads, \( q \), Eq.(6.7) changes to Eq.(6.8) for floor test one to Eq.(6.10) for floor test three.

\[
\begin{align*}
        F_1 &= 0.625 \times 5.01 \frac{\text{kN}}{\text{m}} \times 4.5 \text{ m} = 14.1 \text{ kN} \\
        F_2 &= 0.625 \times 4.35 \frac{\text{kN}}{\text{m}} \times 4.5 \text{ m} = 12.2 \text{ kN} \\
        F_3 &= 0.625 \times 2.82 \frac{\text{kN}}{\text{m}} \times 4.5 \text{ m} = 7.93 \text{ kN}
\end{align*}
\]  

(6.8)  

(6.9)  

(6.10)
6.4 Results

The slope from the graph can be calculated from the first calculated point in Abaqus for all three floor elements as:

\[
slope_1 = \frac{0.1805}{0.7417 \text{ mm}} = 0.243 \frac{1}{\text{mm}}
\] (6.11)

\[
slope_2 = \frac{0.1829}{1.729 \text{ mm}} = 0.106 \frac{1}{\text{mm}}
\] (6.12)

\[
slope_3 = \frac{0.01563}{0.1203 \text{ mm}} = 0.130 \frac{1}{\text{mm}}
\] (6.13)

Therefore the dead load equals a shifted coordinate system with the amount on the x- and y-axis of:

\[
\Delta y_1 = \frac{14.1 \text{ kN}}{121 \text{ kN}} = 0.117
\] (6.14)

\[
\Delta x_1 = \frac{0.117}{0.243 \frac{1}{\text{mm}}} = 0.481 \text{ mm}
\] (6.15)

\[
\Delta y_2 = \frac{12.2 \text{ kN}}{186 \text{ kN}} = 0.0656
\] (6.16)

\[
\Delta x_2 = \frac{0.0656}{0.106 \frac{1}{\text{mm}}} = 0.619 \text{ mm}
\] (6.17)

\[
\Delta y_3 = \frac{7.93 \text{ kN}}{157 \text{ kN}} = 0.0505
\] (6.18)

\[
\Delta x_3 = \frac{0.0505}{0.130 \frac{1}{\text{mm}}} = 0.389 \text{ mm}
\] (6.19)

During the development of the Abaqus model, it was convenient to work with one step only, the Riks method. To avoid repeating the adjustment at the point of origin, it would also be possible to analyse the dead load in an individual step, before the Riks step.

To model the floor behaviour by hand or with an Excel program, some simplifications were necessary, as explained in chapter 5. One of the simplifications was that the floor is modelled as a simply supported beam. In reality the support conditions are slightly different, but it was assumed that the support condition mostly influences the local behaviour and does not much influence the load-deflection curve.
With Abaqus both situations were simulated: one situation with the real support conditions at the SHS and one situation with a 50 mm square support plate under the concrete web to simulate a simply supported beam without the hanger system. The load mid-deflection curves for both models are plotted in Figure 6-12. It can be seen that the difference between these curves is very small, and so the assumption made is therefore acceptable.

The concrete in compression was modelled by three straight lines as shown in Figure 4-4 and repeated in Figure 6-13. The post-peak behaviour was neglected. However, concrete models normally include post-peak behaviour. The model from Hognestad (Canadian Portland Cement Association, 1989) for example is shown in Figure 6-14 which assumes a linear behaviour between the peak stress and ultimate strain.
Figure 6-13: Simplification of stress-strain curve for second floor test without post-peak behaviour

Figure 6-14: Idealized stress-strain curve for concrete in uniaxial compression due to Hognestad (Canadian Portland Cement Association, 1989)
In section 5.6.4 it was shown that the post-peak behaviour has little influence on the load deflection curve, as the concrete strain is reaching the peak strain only just before the maximum load is applied. However, the post-peak behaviour of the concrete, as shown in Figure 6-15 for the second test, is modelled in Abaqus to confirm this result. The behaviour before the peak stress was modelled as beforehand and measured in the experimental tests. The post-peak behaviour was modelled, according to Figure 6-14, linearly to a strain value of 0.0038 with a reduction of the stress value of 15%.

![Stress-strain curve](image)

**Figure 6-15: Simplification of stress-strain curve for second floor test with post-peak behaviour**

The Abaqus result of the load deflection curve for the second floor test by modelling the concrete in compression with a post-peak behaviour as simplified in Figure 6-15 is shown in Figure 6-16. For comparison the original Abaqus result from Figure 6-11 without the modelled post-peak behaviour is drawn in the same diagram. However, the two curves are so identical in Figure 6-16 that they are drawn exactly on top of each other. This confirms the statement from section 5.6.4 that the post-peak behaviour of the concrete does not have an influence on the result of the load-deflection curve for the cases considered and the simplified concrete models used in the Excel sheets and the Abaqus analysis are accurate.
enough and acceptable, provided a check is made to ensure that maximum concrete strains
do not exceed the value at peak stress.

![Figure 6-16: Abaqus result for concrete modelled with post-peak behaviour from Figure 6-15](image)

Without the post-peak behaviour the first and third floor test have also been modelled with
Abaqus. The load-deflection curves for the first and third floor tests are shown in Figure
6-17 and Figure 6-18 respectively. In the first floor test, the rebars were not connected
properly to the rest of the floor, which resulted in the specimen failing far below its
theoretical capacity. But, up to that failure point, the experimental test results agreed very
well with the Abaqus results. Therefore, the thin black line, which is normally used for the
experimental test results, is changed to thick red to make the line easily seen in Figure
6-17.
Figure 6-17: Abaqus results for floor test 1

The load-deflection curve for the third floor test is slightly stiffer at the beginning than the Abaqus model as demonstrated in Figure 6-18. At about half of the load, this changes, and the deflections from the experiment are larger than in the Abaqus model. The final load calculated with Abaqus is about 23% larger than the maximum load observed in the experiment.
6.5 DISCUSSION OF THE LOAD-DEFLECTION-CURVE

In Figure 6-19 and Figure 6-20, the load-deflection curves of the experimental tests, the Excel and the Abaqus simulations are shown in one diagram for the second and third floor test. In order to compare the two diagrams, the starting point of the Abaqus result is shifted as the Excel calculation has the dead load already included at a load level of 0 kN and the Abaqus simulation not. In direct comparison, some characteristics can be observed:

- In Figure 6-19, it looks for the load range from 0 kN to 40 kN as if the results from the Abaqus simulation are closer to the experimental test results than the results from the Excel program. However, this can only be observed for the second floor test and is caused by the fact that the typical slope change after the concrete cracked is missing in the second experimental test. This was discussed in section 5.6.5 and is a result of preloading the test specimen, before the measuring instruments were put in place. The Excel simulation shows, at a low load level, how the experimental test would appear, if the preload had not been applied. In
Figure 6-20 the Excel simulation shows a better correlation to the experimental test than the Abaqus model does.

- The reason why the Abaqus deflections are higher in the beginning, before the concrete cracked, is that the concrete model is a simplification of the real concrete behaviour in tension. As explained in section 6.2, the stress-strain diagram after the concrete has cracked is modelled in Abaqus to gain a smooth running model. However, the floor performance before the concrete cracks cannot be modelled accurately in this way.

- The Abaqus results give a larger deflection than the Excel model. One reason for this is the different concrete model for tension that has been discussed in section 6.2. Figure 5-64 demonstrates the results for the Excel model (Excel 1) and the concrete model used in Abaqus (Excel 2) in one diagram, simulated with the Excel program. It can be noted that the deflection for the Abaqus model (Excel 2) always has to be slightly higher than the deflection of the Excel program (Excel 1). Similar to the Abaqus diagrams in Figure 6-19 and Figure 6-20, the diagram also shows the missing break in the slope when the concrete cracks (the graph with the concrete tension strength of 1.3 MPa).

- The difference between the Excel results and the Abaqus results slightly increases at higher loadings. A reason for this could be that the Excel program analyses the bending and shear deflection separately with de-coupled stresses, before the deflections are added together to obtain the total deflection. The FEM program Abaqus, instead, works with fully interacting 3-D stresses and can superpose several stress conditions at the same time. Specifically, at loads close to the maximum load, this difference will become noticeable.

- The calculated maximum load or the load where the rebars start yielding is very similar in the Excel and Abaqus model. However, the maximum deflection for the maximum load is difficult to compare. Due to the fact that the load-deflection curve is nearly horizontal at this stage, very small load increases can result in large deflection increases. Therefore, the maximum deflection between Excel and Abaqus is more easily compared at the stage where the rebars start yielding.

- In Figure 6-19, the results from the experimental test differ more and more from the Excel and Abaqus simulation at higher load levels. This is a result of the slip between the rebars and the concrete, which was noted and discussed in section
Due to the fact that slip only occurred close to the support and not in the middle of the rebar length, the slip only influences the deflection of the floor. In the middle of the floor, the specimen can still carry the maximum bending-moment, which is confirmed in Figure 6-19, due to the fact that the maximum loads from the experiment, the Excel simulation and the Abaqus simulation are similar.

- In Figure 6-20, the differences between the results of the experimental test and the Excel and Abaqus simulations also increase at higher load levels. This time, the slip also appears in the middle of one rebar as monitored in Figure 4-84. As a result, the deflection difference from the theory is, first of all, much larger in the third test than it was in the second test. Secondly, the maximum bending-moment from theory cannot be reached in the experimental test. This can be seen in Figure 6-20, where the maximum load of the third experimental test is much lower than the maximum load of the Excel and Abaqus simulation. This is due to the fact that the strain in the rebars cannot reach the yield point of about 2800 με at midspan, because one rebar starts slipping beforehand at a strain value of 1773 με (Figure 4-84).

![Figure 6-19: Second floor test, load-deflection curve](image)
Sometimes, simulating exactly the load-deflection behaviour from an experiment with a FEM simulation is attempted, regardless if the experiment showed “correct” results or not. The case of slip between the rebar and the concrete is relatively difficult to simulate with an FEM program if the overall response of the floor is important.

A very simplified way to simulate the slip of the rebar would be by changing its material behaviour. It should be clearly pointed out that this method will only demonstrate a very rough simplification, which is mainly based on the empirical values found by comparison of the results. Figure 6-21 shows in blue the normal, true stress-strain diagram used in Abaqus. The red graph is used for the slip simulation. The difference between the graphs is very small, such that they lie nearly on top of each other in Figure 6-21. The point at 272 MPa is the stress value where slip was observed for the first time in the second floor test. The strain value at the yielding point was found by comparing the results. The strain value with the result that most closely matched the experimental test was selected. The load-deflection curve from the modified Abaqus simulation can be seen in Figure 6-22.
6.5 Discussion of the Load-Deflection-Curve

Figure 6-21: Modified stress-strain diagram of rebar for slip simulation

Figure 6-22: Slip simulation for floor test 2
6.6 NATURAL FREQUENCY

The natural frequency can also be calculated with Abaqus. The Abaqus model which was used for the Riks analysis can easily be transformed to a natural frequency calculation by replacing the Riks procedure in the step manager with the frequency procedure. In this procedure the Lanczos eigensolver is used, which is the default setting. The densities of all used materials must be defined in the property module.

In order to find out the influence of the support, the floor is modelled in two ways: one model uses the real support under the square hollow sections and one model simulates the support under the concrete web.

Table 6-1 shows an overview from all calculated frequencies. The frequencies from the experimental tests refer to section 4.4, while the theoretically modelled frequencies refer to section 5.7.

<table>
<thead>
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<th>Natural Frequency in Hz</th>
<th>Test</th>
<th>Theory</th>
<th>Abaqus</th>
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<tbody>
<tr>
<td>Floor Test 1</td>
<td>26</td>
<td>33.5</td>
<td>32.6</td>
</tr>
<tr>
<td>Floor Test 2</td>
<td>19.5</td>
<td>28.7</td>
<td>28.1</td>
</tr>
<tr>
<td>Floor Test 3</td>
<td>22</td>
<td>30.9</td>
<td>28.8</td>
</tr>
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</table>

The natural frequencies determined by Abaqus are similar to the values calculated from first principles in the theory section. In particular, there is a close match between the theoretical and the finite-element model with the support under the concrete web. The values predicted from Abaqus for support under the SHS are closer to the experimentally tested frequencies, but still overpredict these values.
The reasons for the difference between the measured and predicted natural frequencies cannot be conclusively established from the work undertaken. However, some possibilities about the reason for this have already been given in section 5.7. Furthermore, the support conditions under the SHS are idealised in Abaqus with a support distance of 30 mm to the concrete surface, which was best estimated from the experimental testing. The assumption in the theory of a fully symmetric specimen was not given in the experimental testing. Instead, the hollow sections at the supports were slightly twisted during the welding process and the concrete pour so that the final floor element was not fully supported at both ends at the same time. This behaviour would have decreased the bending stiffness of the floor specimen and also the natural frequency.

6.7 CONCLUSIONS

The conclusions of chapter 6 are:

- The results of the Abaqus model and the Excel program are in general very similar, as shown by the load-deflection curves in Figure 6-19 and Figure 6-20.
- The concrete tension behaviour demonstrated in the flexural tensile strength test in section 4.2.1 cannot be incorporated into Abaqus without causing numerical modelling problems. Therefore, a simplification was used, avoiding the rapid decrease in the stress following the peak load being reached. The simplified model is shown in Figure 6-2 and uses a maximum tensile strength of 1.2 MPa, instead of 4.07 MPa. As a result, the Abaqus graph cannot show the typical slope change in the diagram when the concrete starts cracking, whereas the Excel graph can.
- For both of the tests analysed, the difference between the Abaqus and Excel results increases at higher loads. In the Excel program, the flexural and shear deflection are analysed separately at the full cross-section. Abaqus considers 3D-stresses and takes account of all interactions at the same time. This difference is most noticeable at high loads.
- The maximum load in the Abaqus and Excel simulation were very close, with a difference under 2%.
- The reason for the larger deflections in the experimental test, when compared with the Abaqus model, is the same as the reason that the deflections in the experimental
test are larger than predicted by the Excel program. The difference is due to the slip between rebar and concrete and was analysed in section 5.6.5.

- The fact that the maximum load of the third experimental test was lower than the Abaqus model was also caused by slip at midspan, as discussed in section 5.6.5.
- By readjusting the stress-strain diagram of the rebars, it was possible to simulate the slip between the rebars and the concrete. As a result, the Abaqus program was able to simulate the same load-deflection curve as measured in the experimental test. But it has to be noted that this was based on a trial and error method to find the best fit to the experimental test and does not give insight into the actual slip characteristics.
- The results of the natural frequency analysis with Abaqus agree well with the results from the theory section. However, the results from the experimental tests are in all three cases about 25% less. No conclusive reasons for this could be found; some suggested reasons are given.
7 DISCUSSION AND SUGGESTIONS FOR FURTHER WORK

Chapters 4 to 6 each contain specific discussion and conclusions sections, keeping the results and discussions close together. This chapter does not discuss the details again, but focuses on bringing together the main results and making suggestions for improvements.

The new “F1” floor element has been shown to be a feasible contender as a long-spanning, precast flooring system component and to fulfil the brief from the CSA partnership.

The innovative use of sheet-steel reinforcing had demonstrable benefits in:

- ensuring a robust, slab-level support detail.
- providing steel in an adaptable form, able to contribute to the demands of both shear and flexure.
- eliminating conventional stirrups and allowing a very compact reinforcing assembly leading to a reduced overall web thickness.
- permitting a self-supporting reinforcing assembly, capable of being lifted and placed in the formwork as a single unit.
- providing a support detail with a robust load path and ability to provide a level top concrete surface on deforming supports.

The use of lower density, fibre-reinforced concrete resulted in good composite performance of the concrete/ sheet-steel combination and allowed construction of a unit with sufficient strength and stiffness to meet normal industrial loading requirements.

The F1 development program was constrained by limitations of funding, time and physical resources, allowing only three large-scale floor test specimens to be constructed and tested. Poor concrete quality and rebar anchorage in the first test specimen led to reduced performance, but enough information was obtained to confirm the effectiveness of the sheet-steel and dispel any worries about web splitting. The need for properly compacted concrete and effective rebar anchorage and attachment were addressed in the second test, allowing the target load to be borne and showing suitable ductile behaviour. The third test
Discussion and Suggestions for further Work

was intended primarily as a confirmation of the second test, although, a thinner slab was used (100 mm rather than 150 mm) with lower density concrete. The connection of rebars to the steel sheet with conventional reinforcing tie-wire proved to have less capacity than necessary, leading to some separation and limiting the post-maximum load performance.

Theoretical modelling, based on a simple assumption of linear strain, together with careful consideration of concrete-cracking behaviour, applied with precision both over the cross-section and along the length of the floor unit, gave behaviour predictions that agreed well with observed experimental tests. The assumption of no slip on the concrete/steel interface limited the model’s predictions to some extent. Despite this, the peak load prediction was unaffected, provided slipping did not approach the midspan region. Stiffness, however, would be affected by any occurrence of slip. Slip was not incorporated in the model, which was aimed primarily at peak load prediction and the load-deflection behaviour up to that point. Good detailing and interconnection of the reinforcement was intended to ensure that slip was effectively suppressed and sufficient experimental evidence was gathered, especially from the second test, to suggest that this goal was achieved for that test and can be achieved in practice.

Finite-element analysis generally confirmed the results of the theoretical model. Concrete cracking behaviour, always difficult to model in finite-element programs, was approximated based on the results of the theoretical model, which was able to experiment with different tensile stress-strain relationships. Full composite action was also assumed in the finite-element model. Incorporating treatment of slip into the theoretical or the Abaqus model may be worth considering for future development.

There are some improvements and further work which could be conducted in continuation of this project:

- If the floor development process is finalised and several 4.5 m long specimens have confirmed the concept, a full-scale floor test with the dimensions of nine metres in length and three metres in width is essential, with strain gauges at intervals between the midspan and end to monitor slip.
- The rebar connection to the surrounding concrete is very important and full development of this rebar at the point of maximum moment in the centrally loaded
4.5 m span test is required. It will be an important research area to determine appropriate embedment details, then build representative models and undertake pull-out tests to determine the anchorage details and development length required. However, this will not be practical to undertake until the preferred form of reinforcement and anchorage into the end plate is determined.

- Repeat tests of the same kind as the second test but with revised rebar anchorage details are then required to determine variability.
- A 5000 cycle load test is required on the final planned detail to determine resistance to shakedown in service.

One conclusion from this research is that the steel sheet is very effective for carrying shear forces. Especially at the support, the sheet is able to fulfil complex load conditions and is able to utilise its advantages against other floor types. This is of particular importance, as the support detail is often critical in conventional precast floor units as discussed in section 2.4.

It was not planned to develop a better support detail for double-Tee floors, but the ideas of this thesis can be used to do so. The sheets could be only one metre long at each support and the thickness can easily be adjusted to the requirements. The use of two parallel steel sheets for wider concrete webs would also be possible.

Unlike double-Tee floors or hollowcore slabs, the F1 unit always has the steel sheet in the cracking plane, which prevents brittle cracking at the support. Brittle cracking mechanisms are critical for floor systems, as the sudden local failure can lead to total failure of the floor system, which can in turn result in total collapse of the building if the floor below collapses as well, due to the extra load of the first crushed floor.

This thesis presents experimental tests, theoretical and finite-element models that demonstrate the feasibility of the F1 unit. However, at the moment the F1 unit is still a research project and is not yet at the stage of a “prototype” for commercial production. Apart from more experimental tests, some other details have to be investigated further. These include, for example, the levelling system at the SHS, which should allow the adjustment of the height with a screwed bolt to make a concrete topping unnecessary.
Furthermore, the connection between the units along their transverse edges has to be developed and investigated, especially with regards to the diaphragm shear transfer under seismic loading.

One possibility for further development could be to replace the reinforcing bars in the bottom of the ribs with prestressing tendons. These would be installed in the former and would pass through the holes in the end plate, as is shown in Figure 4-44 for the rebars. The tendons would be stressed against the formwork, as they are for a conventional double-Tee. However, dead end anchors would be installed at the steel plates before the concrete was poured. When the prestressing bars are cut after the concrete has hardened sufficiently, the tension force in the tendons would be resisted through compression in the steel end plate and in the concrete and steel sheet at the bottom of the rib. This would have the following advantages:

- The compression would eliminate visible cracking in the base of the ribs under serviceability conditions.
- The dead-end anchorage would ensure the solid bond of the tendon into the concrete and prevent premature slip.
- The prestressing would decrease the deformation, which is important in toppingless systems.
- The floor height could be minimised.
8 CONCLUSIONS

Specific conclusions from the experimental testing and numerical modelling are given in each of the chapters 4 to 6. This chapter describes in general terms the principal conclusions of the research project:

- A new composite precast floor concept which uses a 1.6-mm perforated steel sheet as part of the reinforcing system has been developed, analysed and tested.

- The steel sheet is able to carry forces at any location and in any direction in its plane and is an effective replacement for conventional stirrups. The continuously distributed forces in the sheet are beneficial for the concrete-cracking behaviour. Stirrups can only carry loads in a vertical direction and have the disadvantages of stress concentrations at the locations where they are placed.

- The steel sheet also contributes an increase in the bending-moment resistance of the floor, which is not possible with stirrups.

- Experimental testing has verified that the floor concept is constructible. The floor behaviour was analysed and evaluated after each test to improve the floor concept. The rebar connection to the concrete and to the rest of the floor was insufficient in the first test. The best results were obtained in the second test, when the rebar ends were welded to the 6-mm steel end plate, and when the rebars were also spot-welded to the steel sheet. Further developments are feasible to improve this end anchorage and attachment between rebar and steel sheet along the length of the member.

- Before the first experimental floor test was conducted, the main concern was that the concrete at the web might separate at each side from the steel sheet when loaded. The experimental testing has shown that this failure mode does not occur, at least for the steel-fibre dosage and with the percentage of openings used in the steel sheet, and that the steel sheet remains well connected to the concrete at all times.

- Strain gauges glued to the rebars indicated slip between the rebar and the surrounding concrete at a specific load stage. If the slip occurred only at the ends of the rebars, it affected only the deflection behaviour, but not the maximum load capacity of the floor. In the third floor test, slip was also detected in the midspan region, which let to a lower maximum load capacity of the floor.
Conclusions

- Predictions from theoretical modelling implemented in an Excel spreadsheet and from finite-element analysis with the program Abaqus were confirmed by the results from the experimental tests. In particular, the load-deflection curve was calculated and compared with the data from experimental testing. In order to make the theory accessible to hand calculation, some simplifications had to be made, such as calculating the flexural and shear deflections separately. Abaqus is able to superpose these actions and can model this behaviour more precisely, which is most important for behaviour beyond the elastic range. However, some material behaviour can be modelled more easily using Excel, in particular, the rapid change in stresses at a given level when the concrete cracks.

- At the support and especially, for floor types with a hanger system, the stress distribution is very complex and may change for different load conditions. The steel sheet is able to resist these complex stress distributions, even when the stress directions change. The developed support detail with the square hollow section and the 6-mm steel plate welded to the steel sheet showed a very robust and reliable behaviour under experimental testing.

- The use of steel-fibre reinforcement improved the tensile strength of the concrete and added to its ability to connect through the “windows” in the steel sheet.

- The entire reinforcing system can be, advantageously, fabricated as a self-supporting rigid assembly, which can be lifted by its ends and placed in the formwork.
9 REFERENCES


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CONCRETE INSTITUTE OF AUSTRALIA. & NATIONAL PRECAST CONCRETE ASSOCIATION AUSTRALIA. 2002. Precast concrete handbook, North Parramatta, N.S.W., National Precast Concrete Association Australia : Concrete Institute of Australia.


Dissertation, Fachbereich Bauingenieurwesen, Universitaet Kaiserslautern (in German).


NZS 3112 1986. Specification for Methods of Test for Concrete, Standards New Zealand.


Appendices

APPENDIX 1  REBAR TEST CERTIFICATES

The first test certificate with the cast number 51168-01 corresponds to the rebars used in the first experimental test. The second test certificate with the cast number 53888-01 corresponds to the rebars used in the second and third experimental test.
CERTIFICATE OF TEST

Customer Number: 320040
Customer Name: FLETCHER REINF OLD ROD MILL
Delivery Address: Fraser Street Whangarei

Cast Number: 51168-01
Specification: AS/NZS 4671 GRADE 500
Product: ROUND M 20 SEISMIC 500 DEF 9M
Method of Manufacture: Micro-Alloyed
Certificate Number: 296841
Issue Date: 10/08/2007

Chemical Analysis (% by mass)

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We certify that the above information is in accordance with the records of the company and conforms to the specifications as stated.

Pacific Steel Authorized Signatory:

Janel Roberts
Site Metallurgist
## CERTIFICATE OF TEST

**Customer Number:** 320040  
**Customer Name:** FLETCHER REINF OLD ROD MILL  
**Delivery Address:** PACIFIC STEELS OLD ROD BLOCK 259 JAMES FLETCHER DRIVE OTAHUHU

**Cast Number:** 53888-01  
**Specification:** AS/NZS 4671 GRADE 500  
**Product:** ROUND M 20 SEISMIC 500 DEF 9M  
**Method of Manufacture:** Micro-Alloyed  
**Certificate Number:** 306157  
**Issue Date:** 10/01/2008

### Chemical Analysis (% by mass)

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### Mechanical Tests

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We certify that the above information is in accordance with the records of the company and conforms to the specifications as stated.

Pacific Steel Authorised Signatory:  

Bruce Roberts  
Site Metallurgist.
APPENDIX 2  EXCEL PROGRAM TEST 2

The Microsoft Excel file for analysing the second experimental floor test as discussed in section 5.6 can be found on the attached CD-ROM.

APPENDIX 3  EXCEL PROGRAM TEST 3

The Microsoft Excel file for analysing the third experimental floor test as discussed in section 5.6 can be found on the attached CD-ROM.