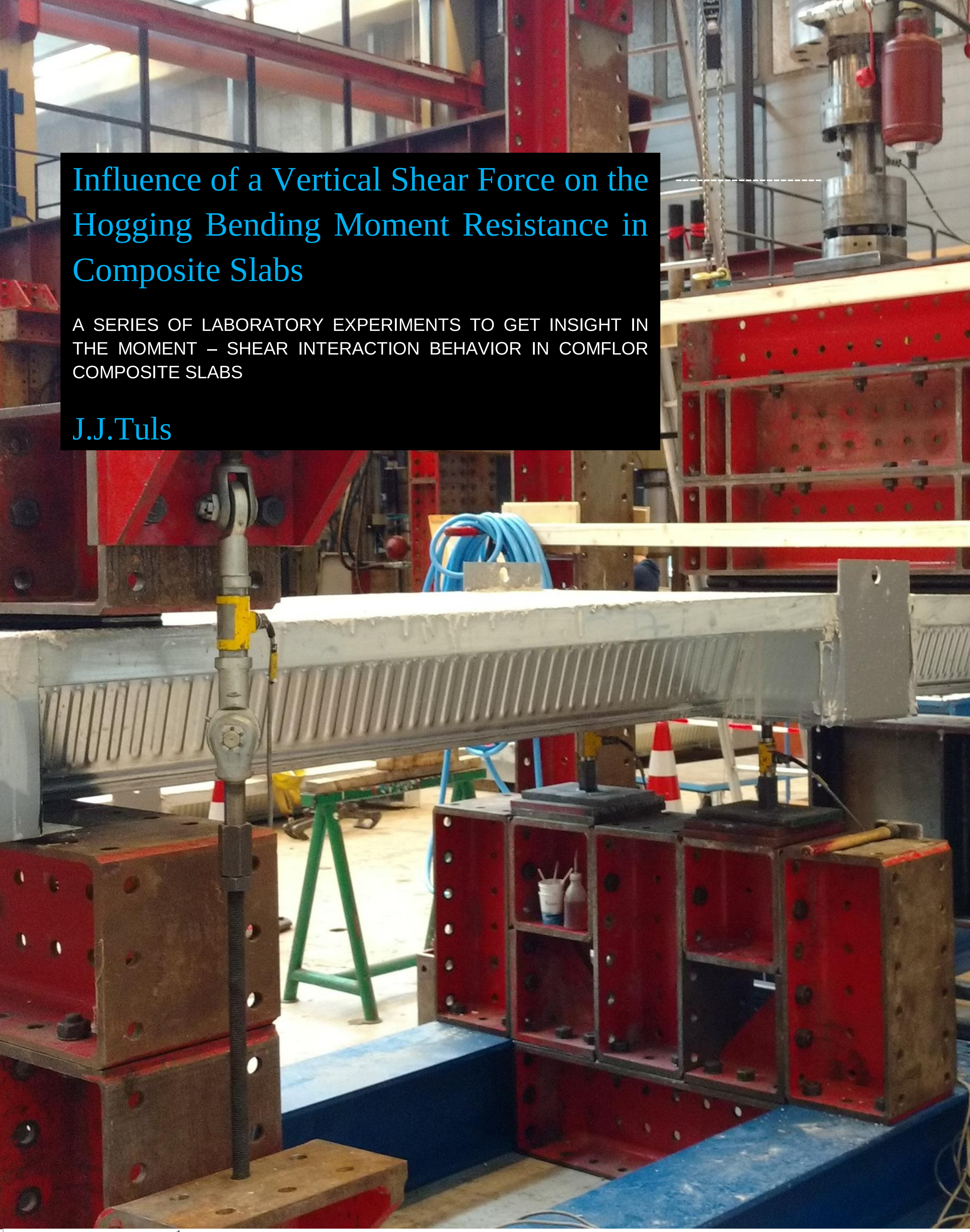
Influence of a Vertical Shear Force on the Hogging Bending Moment Resistance in Composite Slabs

A SERIES OF LABORATORY EXPERIMENTS TO GET INSIGHT IN THE MOMENT – SHEAR INTERACTION BEHAVIOR IN COMFLOR COMPOSITE SLABS

J.J.Tuls



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Influence of a Vertical Shear Force on the Hogging Bending Moment Resistance in Composite Slabs

A SERIES OF LABORATORY EXPERIMENTS TO GET INSIGHT IN THE MOMENT – SHEAR INTERACTION BEHAVIOR IN COMFLOR COMPOSITE SLABS

By

Johan. J.Tuls

in partial fulfilment of the requirements for the degree of

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in Civil Engineering

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II. PREFACE

This thesis is a final work as partial fulfillment for the degree of Master of Science in civil engineering. The report consist of four mayor parts: an initial literature research on the subject, a theoretical part, laboratory tests and a last part with conclusions. The tests were performed in the Stevin II laboratory of the Technical University in Delft, the Netherlands.
The subject of this thesis was conceived in cooperation with Dutch Engineering in Zoeterwoude, The Netherlands and I would like to thank Dutch engineering and the people who assisted on the assemblage of the test specimens for the laboratory experiments.
For the execution of the laboratory test in the Stevin II laboratory of the technical University in Delft, I would gladly thank the people that helped preparing the test rig and allowing a smooth series of tests.
Finally I would like to thank all members of my thesis committee who gave feedback throughout my final project and help me navigate in the right direction when I was stuck or deviating from it.

Johan J. Tuls
Delft, December 2016

III. ABSTRACT

Composite steel-concrete floor systems consist of a trapezoidal shaped steel deck, reinforcement and cast-in-place concrete. Depending on the span, height restrictions and application shallow or deep decks can be chosen. The shallow deck used within this thesis is the ComFlor75. Due to the low self weight, a bundle of steel decks can be lifted to the desired floor and the individual decks are placed by hand to the correct location. Shallow decks normally are placed on top of the supporting beams and one ComFlor75 can cover multiple spans. The deep deck used within this thesis is the ComFlor210, this deck type is usually integrated with the supporting beam by placing it on top of the bottom flange or a steel plate that is welded below the supporting steel beam. This allows for a bigger internal lever arm, while reducing the construction height. After reinforcement is placed in the ribs of the ComFlor75, meshes or additional reinforcement bars are placed in the top layer. This layer does continue over the support beams and creates a continuous floor system. Advantages of this composite steel-concrete floor system are: fast construction, low weight and a small construction height. At the supporting beams of this continuous floor, a hogging bending moment and vertical shear force occur. Within the Eurocode 4, the hogging bending moment and vertical shear resistance are calculated independently. The deep decks are not covered by the scope of the Eurocode 4. At a certain project (Case study: "town hall - Almelo") the authorities asked if the vertical shear could influence the hogging bending moment resistance as both were near the calculated resistance.

This question is being answered by first looking into current researches [1; 2; 3; 4] and calculation methods. These have been used to find a suitable test setup. This test setup has been adjusted to practical values to cover the critical spans where M-V interaction could be a concern. A total of 5 experiments, 3 on deep deck (ComFlor210) and 2 on shallow decks (ComFlor75) have been conducted. In both cases first an experiment is done to determine the hogging bending moment resistance with a low vertical shear force followed by an experiment where the specimen was fully loaded by a vertical shear and a hogging bending moment.

For the shallow as well as the deep decks no reduction in hogging bending moment was found. All specimens failed in bending, even though the specimens were loaded by a vertical shear force surpassing the vertical shear resistance based on the Eurocode 4 and calculations done in practice. For the deep decks a higher hogging bending moment was found compared to the calculated resistance. The steel plate underneath the integrated support beam was not included in the calculation, but did contribute. All 3 specimens failed close to the calculated resistance.

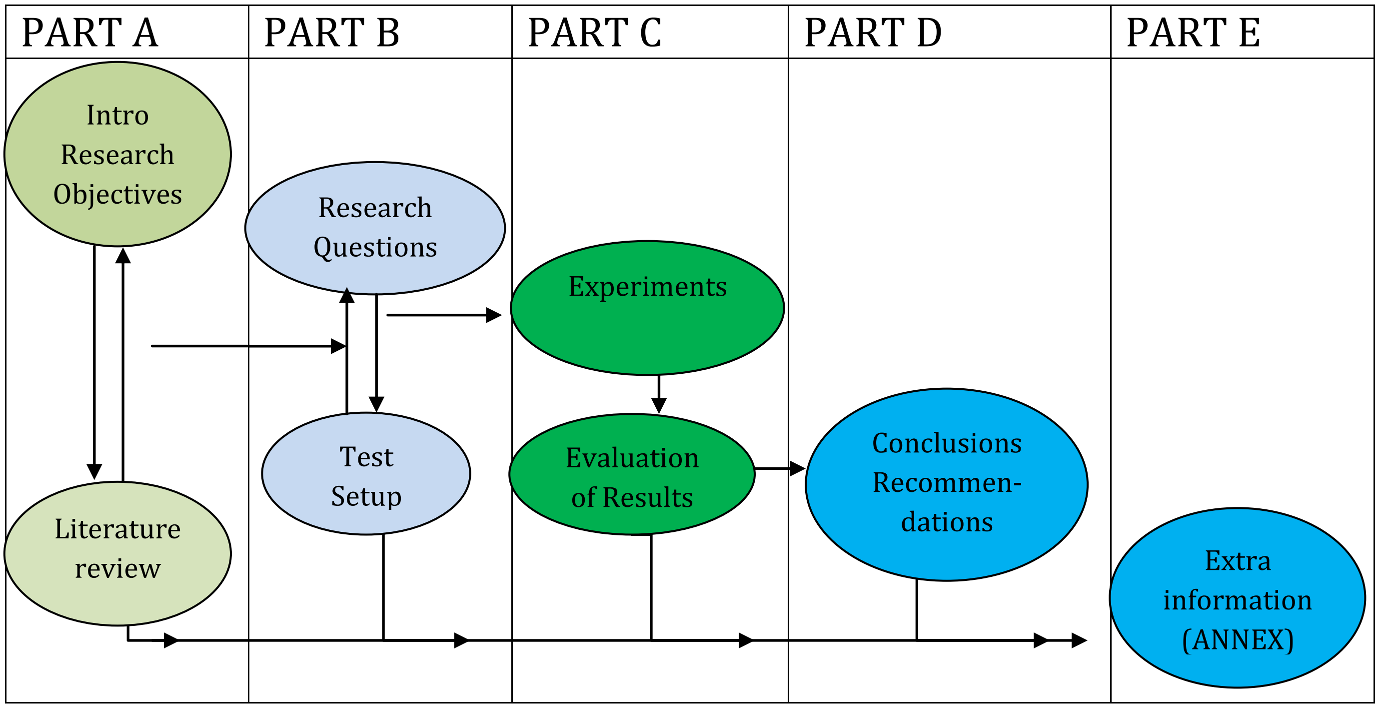
It was concluded that with the maximum shear resistance V_{Rd} used in practice no M-V interaction was found. The actual vertical shear resistance V_U could be far greater than the conservative value of V_{Rd} used according to Eurocode 4 [5]. Under normal distributed load patterns other criteria will govern. Vertical shear can become critical at shorter spans, however this implies a significant high distributed force compared to common values. The question is if it therefore is of interest to know the exact V_U as it generally is not the critical criteria.
There is therefore no influence of a vertical shear force on the hogging bending moment with a $V \leq V_{Rd}$. To get a wider statistical base more experiments are advised.

IV. THESIS OUTLINE

An outline of the different parts of the report is given in the flow chart in Table 1

.

Table 1 Flow chart of how to approach this master thesis



<u>PART A:</u>
The first part of this thesis contains general information about the subject and knowledge gained through existing reports. This includes an introduction to the main objective and to the subject.

<u>PART B:</u>
The second part of this thesis translates the main objective into specific questions needed to determine a way of answering it. It concerns a practical situation and theoretical subjects that lead to possible way of testing. Based on these alternatives a final test setup is chosen to answer the main objective.

<u>PART C:</u>
The third part of this thesis describes the laboratory tests, the test rig and instrumentation used and the results following from these experiments. These can then be used to reflect on the initial study to compare theoretical values with the experiments.

<u>PART D:</u>
The fourth part of the thesis consists of conclusions taken from the results in part c and recommendations regarding uncertainties and practical applications.

<u>PART E:</u>
The final part of the thesis contains the annex with additional information, drawings and calculations. References are made within each chapter to this part.

V. TABLE OF CONTENTS

VI. List of Tables

1 INTRODUCTION

1.1 An introduction to steel-concrete composite structural elements

Many floors are made of reinforced concrete as no other material has the combination of low cost, strength and resistance to corrosion and fire. Besides due to the high self-weight it has a natural sound insulation in case it is applied within buildings. If the span increases the thickness increases past a point where sufficient sound insulation, fire resistance has already been reached and due to the height self-weight it becomes more economical to support the slab on top of a raster of concrete beams. As the construction is made of concrete only, the slab naturally acts as a top flange for the supporting beams below.

At certain applications, especially if fire resistance is not an issue or has been taken care of, steel beams are used instead of concrete beams to provide a lighter, more economic and slender alternative. However the construction now no longer consists of one material and the slab and beam act as separate structural elements without structural collaboration as shown in Figure 1.1

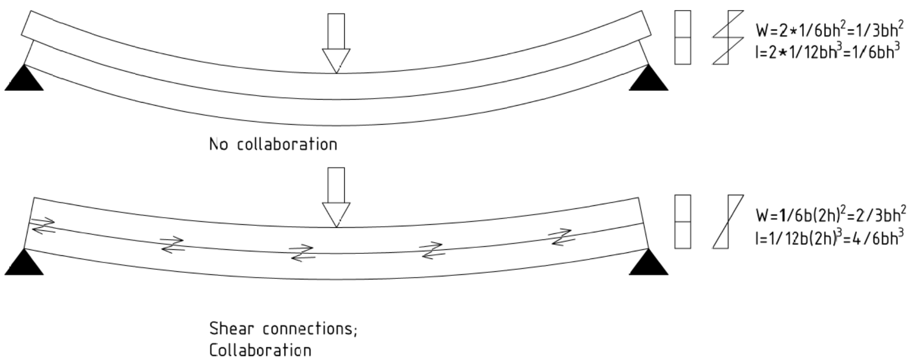


Figure 1.1 By connection both individual members both the moment of inertia I as the section modulus W of the beam. [6]

By the use of shear connectors this structural collaboration, that was naturally present in monolithic concrete members, can now be achieved increasing both the moment of inertia and the section modulus of the structural element. This application is applied both in steel-concrete composite beams as well as in slabs to combine the strengths of both materials.

A new failure mode now appears namely longitudinal shear. In order to prevent this failure mechanism from occurring a shear connection must be present between the different layers capable of transferring this longitudinal shear force.($V_{L,ED}$) [7]

$$V_{L,ED} = \frac{V_{ED} A \bar{Z}}{I_y}$$

V_{ED} Is the shear force in the cross section (related to the change in moment)
A Is the effective area further from the neutral axis than the level considered.
$\bar{Z}$ Is the vertical distance from the neutral axis to the centroid of area A.
I_y Is the second moment of inertia of the effective cross section of the member.

Taking a look to another construction material containing different layers (and possible different material properties) is glued laminated timber. See Figure 1.2. [8]

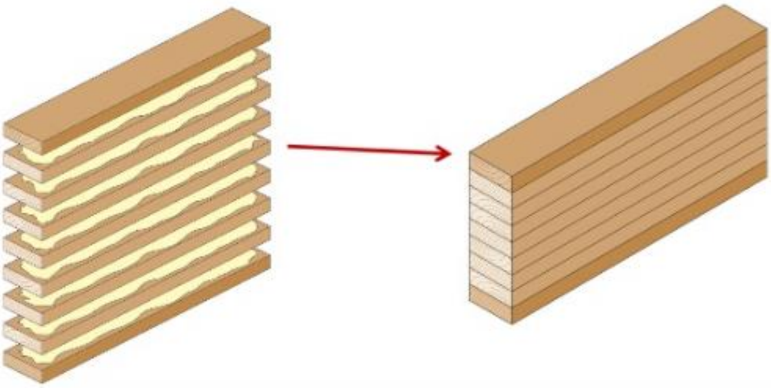
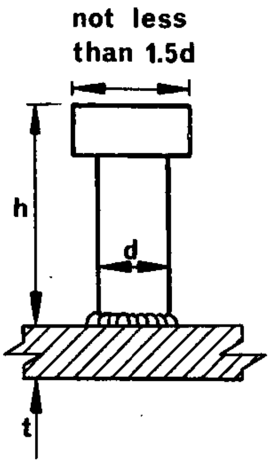


Figure 1.2 Multiple timber beams glued together by adhesives to provide collaboration between all layers. The strength of this layer could be increased by adding additional screws.

Here it concerns different layers of timber (possible different kind of timber) that are connected by adhesives (or additional screws) to allow the beam to collaborate and function as a single cross section.
The same procedure is used in steel-concrete composite beams, however instead of adhesives shear connectors are used to transfer this longitudinal shear force. The most widely used type of connector is the headed stud.



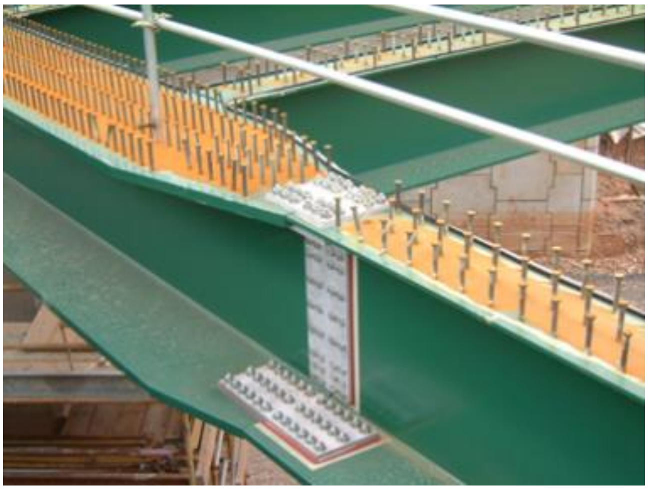


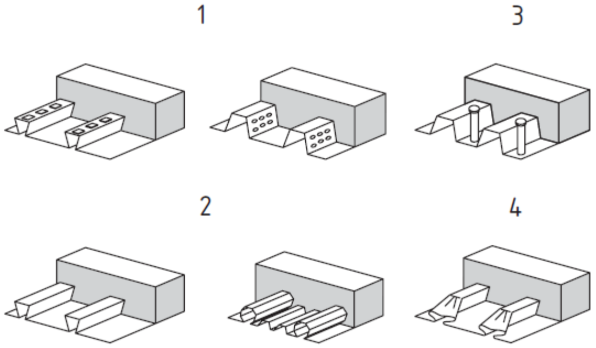
Figure 1.3 Headed stud connector on the left [7] Application of the headed stud on the right [9]

By connecting these two different materials with different properties, advantage can be taken from each material as mentioned below:

-Steel is strong to withstand tension forces, where concrete is strong in compression.
-Steel elements are slender and sensitive to instability, due to the collaboration with concrete these forms of instability are prevented.
-The concrete protect the steel from corrosion
-In case of fire the mass of the concrete slows down the heating of the steel, increasing its fire resistance.
-Steel has a high deformation capacity before finally failing. In combination with the concrete this provides a warning signal instead of a possible brittle failure in case of a pure concrete structure.

1.2 An introduction to steel-concrete slabs and profiles

The same principle used in the composite beams is used for steel-concrete composite slabs. Sometimes in combination with the supporting beams below. A steel profile is connected to the concrete above to provide full collaboration between both materials. Different ways of interlocking in composite slabs is shown in Figure 1.4.



Key

1 mechanical interlock

2 frictional interlock

3 end anchorage by through-deck welded studs

4 end anchorage by deformation of the ribs

Figure 1.4 Typical forms of interlock in composite slabs.

These steel profiles fulfil three different functions:
-It functions as a working floor during the execution.
-It functions as formwork when casting the concrete.
-External reinforcement for the concrete slab to take up tensile forces.

Investments in multi-storey buildings provide no income until finished. This means an increase in construction time is translated in a direct loss. Construction speed is therefore to be considered in the design. The composite floor system can provide a reduced construction especially if it is designed in such a way that no propping is needed. This way the composite floor system provides the following benefits regarding construction speed:

-No propping and no external formwork needed.
-One stack of profiles only requires one lifting movement by crane, local placement is done by hand due to the low self-weight.

The steel profile serves as final external reinforcement as well as formwork. This means it must be designed in two stages:
-Construction stage, while the concrete is still wet and while serving as form work. (Local concentrated loads)
-Final stage

1.3 An introduction to ComFlor210 and ComFlor75 floor system.

The steel profiles that are considered within this thesis are from the ComFlor series of Tata Steel UK. Both decks are shown below in Figure 1.5 & Figure 1.6.

Figure 1.5 Deep deck, ComFlor210

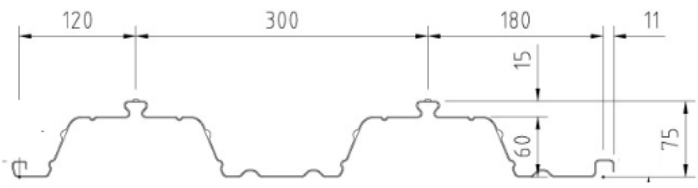


Figure 1.6 Shallow deck,ComFlor75

The shallow decks usually continue over the supporting steel beams and can be connected to by means of shear studs to allow composite behaviour with the supporting beam. Within the trapezium shaped profile embossments have been applied to allow the transfer of shear force between the different materials as shown in Figure 1.7.

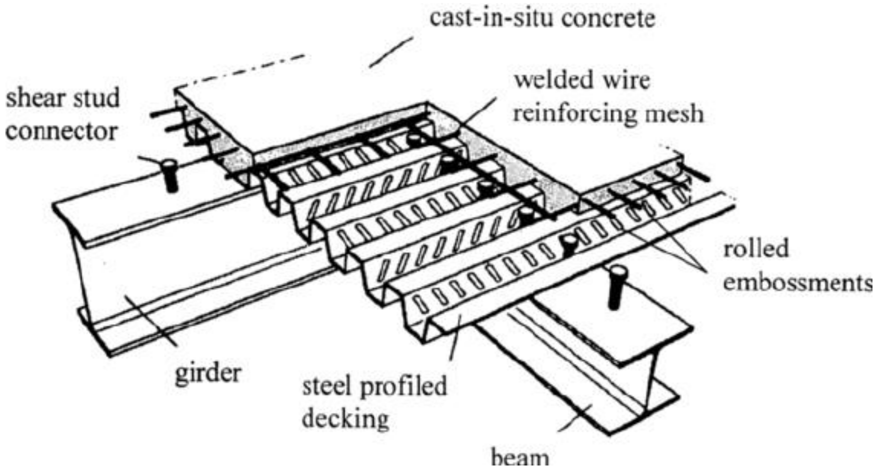


Figure 1.7 Shallow deck with its surrounding construction elements.

In case of the shallow decks as shown above, the steel profile continuous over the supporting beam and can provide additional hogging bending moment resistance. If the load / span increases a deep deck could be a better choice. In order to reduce the construction height, deep decks are often integrated with the supporting beam by placing them on the lower flange of the supporting beam. This means the profile is simply supported and does not continue over the support. It therefore cannot transfer tensile forces to the other profile and the profile may not be included for the hogging bending moment resistance. This detail is shown below in Figure 1.8.

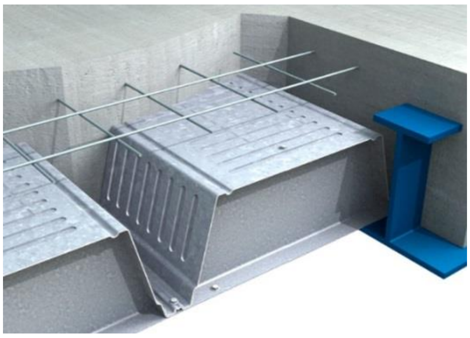


Figure 1.8 Deep deck, ComFlor210, supported by the bottom flange of the steel supporting beam.

2 RESEARCH DESIGN

This chapter will shortly describe what gave rise to this Master Thesis. This will result in a couple of objectives; followed by a build-up of the investigation.

2.1 Problem statement

Within the Netherlands composite slabs take a significant part of the slab market. Due to its fast and easy construction and its economic design, the steel-concrete floor is commonly used. Within the Dutch construction market, there is a huge competition between different builders. In order to get the contract to construct the designed structure, one must make an economic design, efficient in material use and economic when it comes to overall costs. It is therefore desired to design a structure, utilizing close to 100% of the materials capacity in accordance with the standards at force.

At intermediate supports in continuous composite slabs the slab is subjected to a hogging bending moment in combination with a vertical shear force. In a particular project the question was raised if the moment resistance is reduced by the vertical shear force? In other words should the design calculation take into account M-V interaction when applying the ComFlor series (Figure2.1).

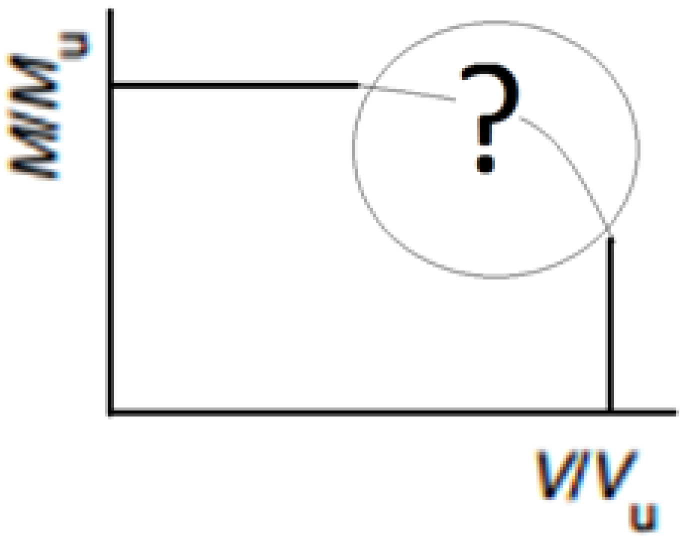


Figure2.1 Does a present vertical shear force influence the hogging bending moment resistance? How is this M-V relation reflected in the interaction diagram?

2.2 Objectives

Within the EN 1994-1-1 [5] & EN 1992-1-1 [10] the calculation of the hogging bending moment and the vertical shear are done separately. Interaction between the hogging bending moment and the vertical shear is not mentioned. This raised the question to the authorities, at the case study “Town hall - Almelo”, if the vertical shear does not influence the hogging bending moment resistance. Is that entirely rightly?

2.2.1 Main objective

To do a limited series of tests, to get insight in the interaction between the hogging bending moment and the vertical shear. Analyzing this data to compare it with the current codes. Thereby providing initial data to support the calculations made by Dutch Engineering according to the current EN 1994-1-1 [5] & EN 1992-1-1 [10].

2.2.2 Sub-objectives

To get insight in the interaction arranged in concrete, steel and composite structures.
To get insight in the behavior of the ComFlor 210, when loaded by vertical shear and a hogging bending moment simultaneously.
To get insight in the behavior of the ComFlor 75, when loaded by vertical shear and a hogging bending moment simultaneously.

2.3 Research question

2.3.1 Main research question

Determine indicatively the influence of the vertical shear on the hogging bending moment resistance of composite steel-concrete floors made with ComFlor75 or ComFlor210 based on a limited series of test

2.3.2 Literature review

- What is the interaction behavior in concrete, steel and other composite structures
- How is the ComFlor slab verified according to the Eurocode?

2.3.3 Sub research questions

- What are the vertical shear and negative bending moment resistances of both the ComFlor210 & ComFlor75?
- What is the area of concern in practice?
- What laboratory tests are needed to answer the main research question and which test rig corresponds with these tests?

2.4 Research methodology

The literature review will have a wide view over the already applicable knowledge. During the thesis it will get more specific towards the actual test series, providing results which will have to be compared to the current formulae. From this data, conclusions can be taken relating consisting projects and new questions can arise for future research. The following parts can be distinguished:

3. **Literature review**
4. **Sub research questions**
5. **Test setup**
6. **Experiments**
7. **Results**
8. **Conclusions**
9. **Recommendations**

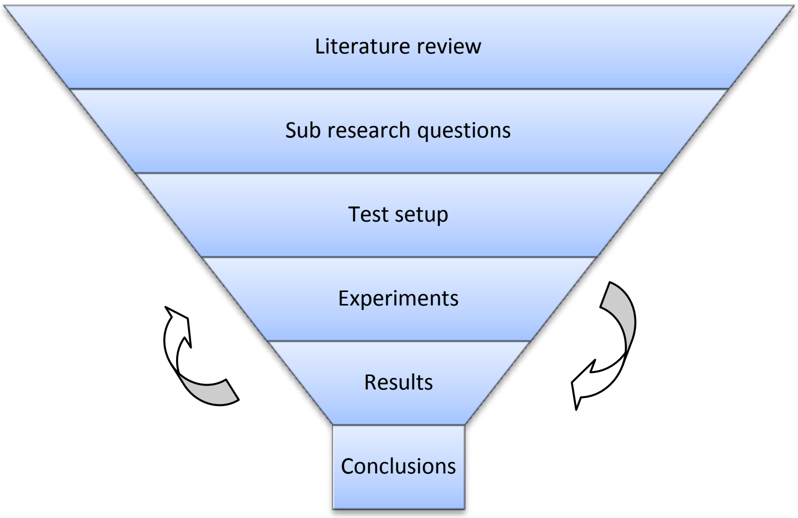


Figure 2.2 Overview research design

3) Literature review

In the literature review the basic knowledge of composite structure will be investigated and how interaction plays a role in steel and concrete structures. First sub-objectives will be answered and worked towards a good basis to determine the test setup. This study will be done based on existing papers and reports.

4) Sub-research questions

Using the knowledge gained in the literature study, analytical calculations will be made regarding resistances and practical values (based on a case study). These calculations form the basis of the test setup used for the laboratory tests. This will ultimately lead to a final test setup used to answer the main research question.

5) Test setup
With the choice of the final test setup and calculations done in the sub-research questions, initial values are known to determine the exact dimensions and properties of both the test specimens and test rig.

6) Experiments
In this chapter the laboratory tests will be described. What will be measured, how the tests are done. There will be 5 different tests. Two tests for ComFlor75 and three for ComFlor210. Using different dimensions and properties to generate various combinations of vertical shear and hogging bending moment on the test specimens.
The laboratory tests will provide data regarding the loading, failure, strength of the concrete used, deformations etc.

7) Results
In this chapter the data gained from the experiments will be compared with each other and with the calculations made beforehand. This will give insight into which parts of the slab were activated during the tests. Comparing this data with the current available formulae, this should give similarities as well as differences and provide data to answer the main objective.

8) Conclusions
After having compared the results with the analytical calculations and between the different tests conducted, conclusions can be taken. Depending on the similarities between the calculations and the results, possible future research must be done or not.

9) Recommendations
Based on the conclusions taken from the results, further investigation might be needed and certain questions might be answered. In this chapter an overview is given of the practical consequences of the test results.

3 LITERATURE REVIEW

3.1 Composite Slabs

Within this thesis the resistance of both bending and shear force are of interest near the intermediate support, as the negative moment will be at its maximum. In order to compare the current calculation rules with the interaction of both components, the standard at force and the approach in practice must be checked. Both for the hogging bending moment as the vertical shear force resistance. This has been done in chapter A.1 in the appendix. Findings are shown in this chapter.

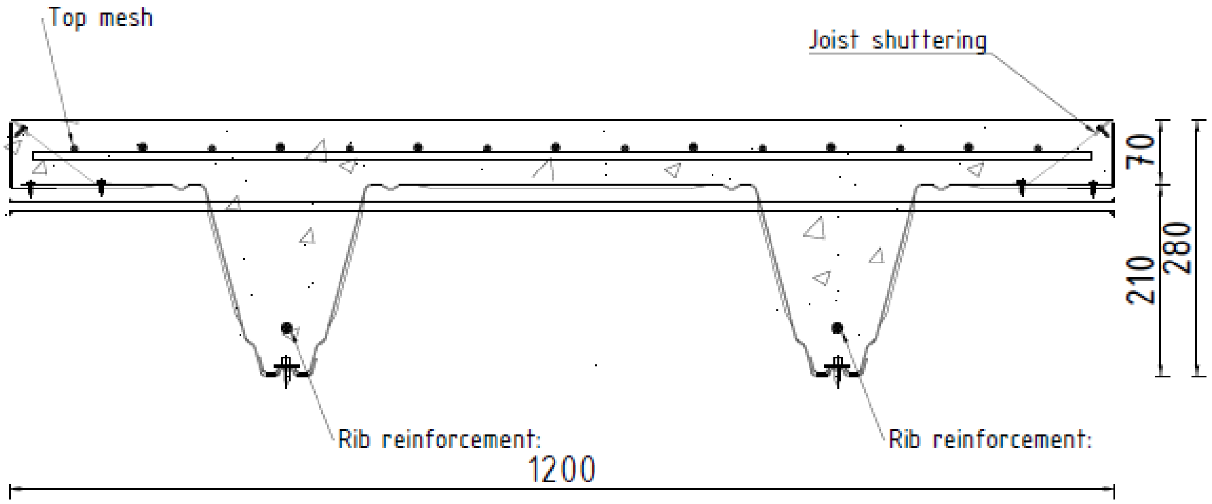


Figure 3.1 Cross section ComFlor210 composite slab, with a width of 1200 mm.

Vertical Shear

Figure 3.1 shows the cross section of a ComFlor210 composite slab. Based on the findings of chapter A.1 in the appendix the vertical shear resistance is calculated according to the shear resistance of the concrete ribs based on a concrete rib not requiring shear reinforcement.
As experiments show that the shear resistance is far greater, in the case study “town hall – Almelo)” is was allowed to include the vertical shear resistance of the ComFlor210 sheet based on experimental results[11]. This results in the following resistance according to the standards:

$$V_{Rd} = V_{Rd,rib}$$

Where:
$V_{Rd,rib} =$ *Shear resistance of the concrete rib, calculated according art. 6.2.2* [10], *elements without shear reinforcement.*

$V_{Rd,rib}$ is based on a member using an empirical formula based on the minimum width of the rib in tension combined with the reinforcement ratio of the reinforcement loaded in tension.

$$V_{Rd,c} = \left[C_{Rd,c} k \left(100\,\rho_1 f_{ck}\right)^{\frac{1}{3}}\right] b_w d$$

With a minimum of:

$$V_{Rd,c} = (v_{min})\,\mathrm{b_w} d$$

Even though vertical shear does not function as a simple summation of contributing parts, it was accepted to include the contribution of the steel sheet to approach the vertical shear resistance if experimental data is available as was the case during the case study “Town Hall - Almelo”, this resulted in:

$$V_{Rd} = V_{Rd,rib} + V_{Rd,sheet}$$

$V_{Rd,sheet} =$ *Shear resistance of the ComFlor steel sheet, based on experiments* [11]

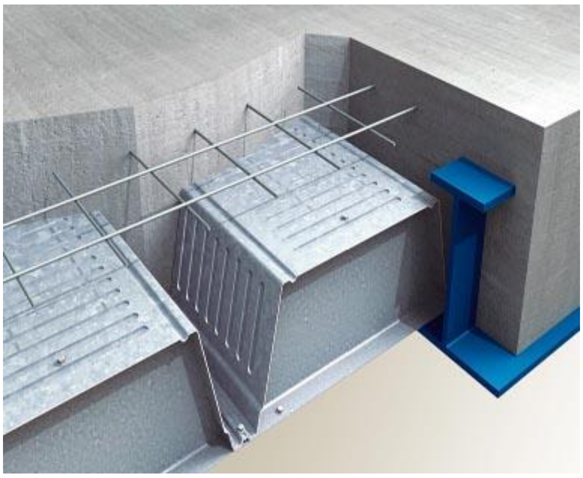


Figure 3.2 Integrated detail of a ComFlor210 sheeting and a hot rolled section functioning as an intermediate support.

Hogging bending moment

The hogging bending moment is calculated based on the reinforcement loaded in tension making equilibrium with the concrete at the bottom of the rib. The ComFlor sheeting can only be taken into account in case it is continuous over the intermediate support. [5] In case of the ComFlor210 (as shown in Figure 3.2) the sheet may not be included. It is usually applied with an integrated beam, as shown in Figure 3.2, while the ComFlor75 is applied as a continuous slab that is placed on top of steel beams, instead of on top of the bottom flange. For more information on the determination of the hogging bending moment according to the standards at force see chapter A.1.2 in the appendix.

M-V interaction in composite slabs.

Concluding the calculation of the vertical shear and hogging bending moment in ComFlor composite slabs the following can be said.

- Top reinforcement in combination with a compression region in the rib takes care of the hogging bending moment.

- The concrete rib (according to the Eurocode 4) [5] and the ComFlor steel deck (according to experiments) [11] take care of the vertical shear resistance.

- The problem with the composite slabs is the uncertain vertical shear resistance as the standard at force, Eurocode 4, only includes the concrete rib as element contributing to the vertical shear resistance.

- In case of continuous decks over the support, the ComFlor sheeting can contribute to the hogging bending moment resistance.

Within the EN 1994-1-1 [5] M-V interaction is not mentioned when it concerns composite slabs, it is therefore of interest to look at composite beams, steel members and concrete separately and how they behave in different applications where a member is loaded by both a hogging bending moment and vertical shear force simultaneously. .

3.2 Interaction behavior of steel members and composite beams.

The supporting structures in steel structures consist mainly of hot rolled sections in different shapes. Those designed as beams, to withstand bending moments and shear forces, generally consist of flanges and a web(s). The flanges, in combination with the distance between them, are designed to take care of the bending moment, while the web mostly takes care of the vertical shear. For the interaction behavior of steel a more detailed description of is given in chapter A.2 in the appendix.
As different parts of the beam take care of the moment / shear force occurring, (nearly) no reduction takes place if the flanges can take care of the bending moment present.

“If the moment is below the bending resistance based on flanges alone, no reduction is present”.

However if the occurring bending moment is higher, the web must assist in providing resistance. If nearly no vertical shear force is present, this is not a problem. However if a shear force already exists simultaneously, a reduction must be applied.

This same principle applies in case of a steel member supporting a concrete slab, connected by shear connecters to behave as a composite structure. For more information see chapter A.2 in the appendix. An interaction diagram for a composite beam with a class 1-2 steel beam underneath is shown in Figure 3.3. In the appendix another research on the M-V interaction of composite beams is discussed. This one and the diagram shown in Figure 3.3 on the right are both from Australia [2][4] and conclude that the concrete slab has an significant contribution to the vertical shear resistance.

In steel a reduction in resistance occurs once making an appeal on the same construction element. *(For instance the web of an IPE/HEA beam is used for the vertical shear while the flanges are used for the bending moment resistance. In case the bending moment is higher than the resistance of the flanges, appeal is made on the area in the web. This is when interaction occurs.)*

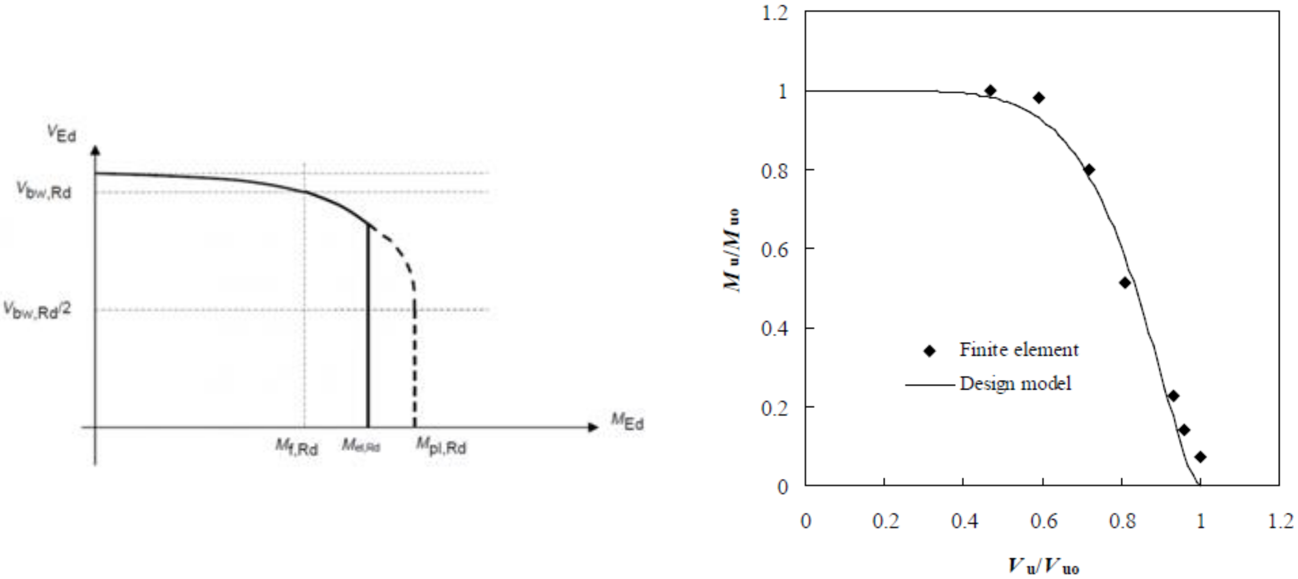


Figure 3.3 Interaction diagram Moment-Shear cross section class 1-2 for composite beams on the left [9] and M-V (hogging bending moment-shear) interaction results for continuous composite beams on the right [2]

3.3 Interaction behavior in concrete.

Concrete structures are nearly always combined with reinforcement steel. Concrete is a brittle composite material and needs the reinforcement steel to assist where tensile forces occur.
Within the Eurocode 2 [12] for concrete a distinction is made between members with and without shear reinforcement (stirrups). Once the occurring shear force is too high, stirrups are applied and are responsible for the shear resistance, this way reinforcement for shear and bending are separated. For more information see chapter A.3 in the appendix. Within composite slabs, the shear resistance is based on a concrete member without stirrups. A thesis [1] has been done to determine the M-V interaction behavior in a rectangular concrete cross section (D-D) with minimal reinforcement present. (and no stirrups) The specimen used is shown in Figure 3.4. [1].

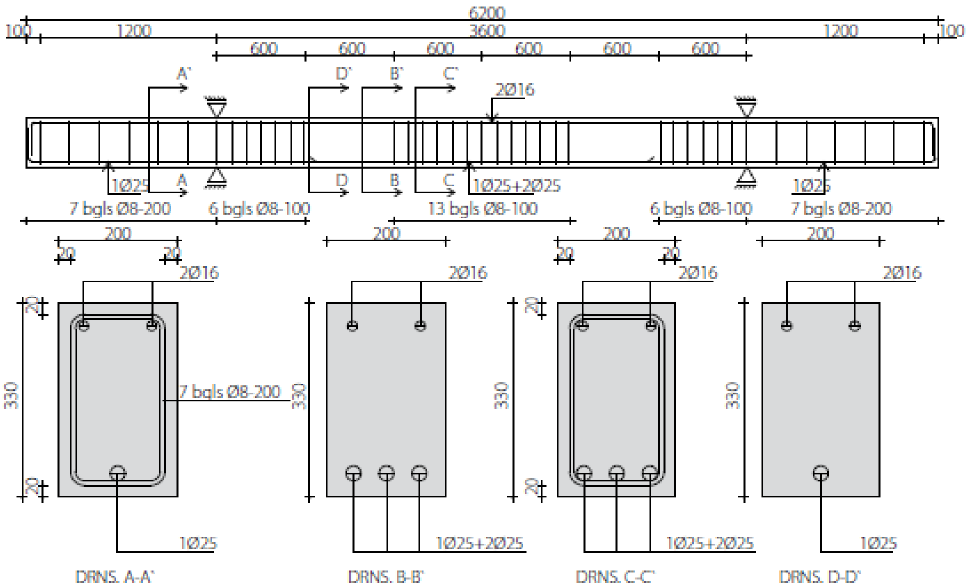


Figure 3.4 Cross sections of the specimen used in the experiments at the TU Eindhoven, The Netherlands. Cross section D-D was critical and used for the results.[1].

The critical cross section contains only 3 reinforcement bars as shown in Figure 3.4. The experiments are therefore based on interaction with one construction element taking care of both the vertical shear and the bending moment occurring.

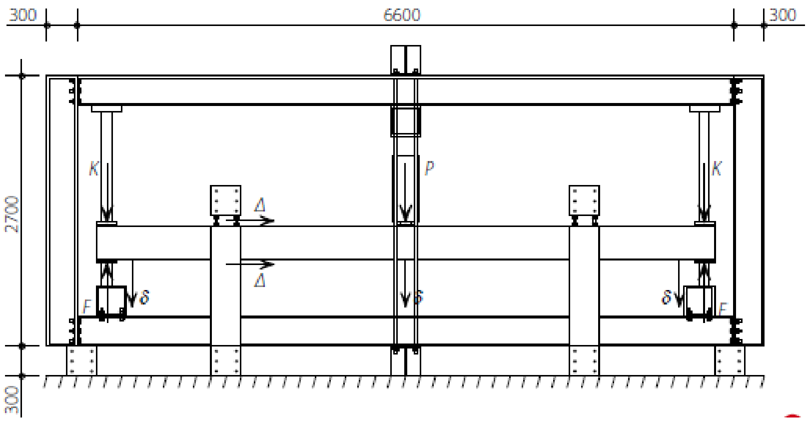


Figure 3.5 Test setup used to preload the specimen by a certain moment by force K, afterwards the specimen is loaded by force P to create the desired M-V combination.

This setup contains 3 different kinds of forces. Force K, P and F. Force K & F are used to apply a constant moment between the supports. In the right combination with the applied force P in the middle, this allows you to vary between M – V combinations in the critical cross section. As other parts are loaded by a higher moment and or shear force, the beam is locally reinforced with extra reinforcement / stirrups as shown in Figure 3.4.

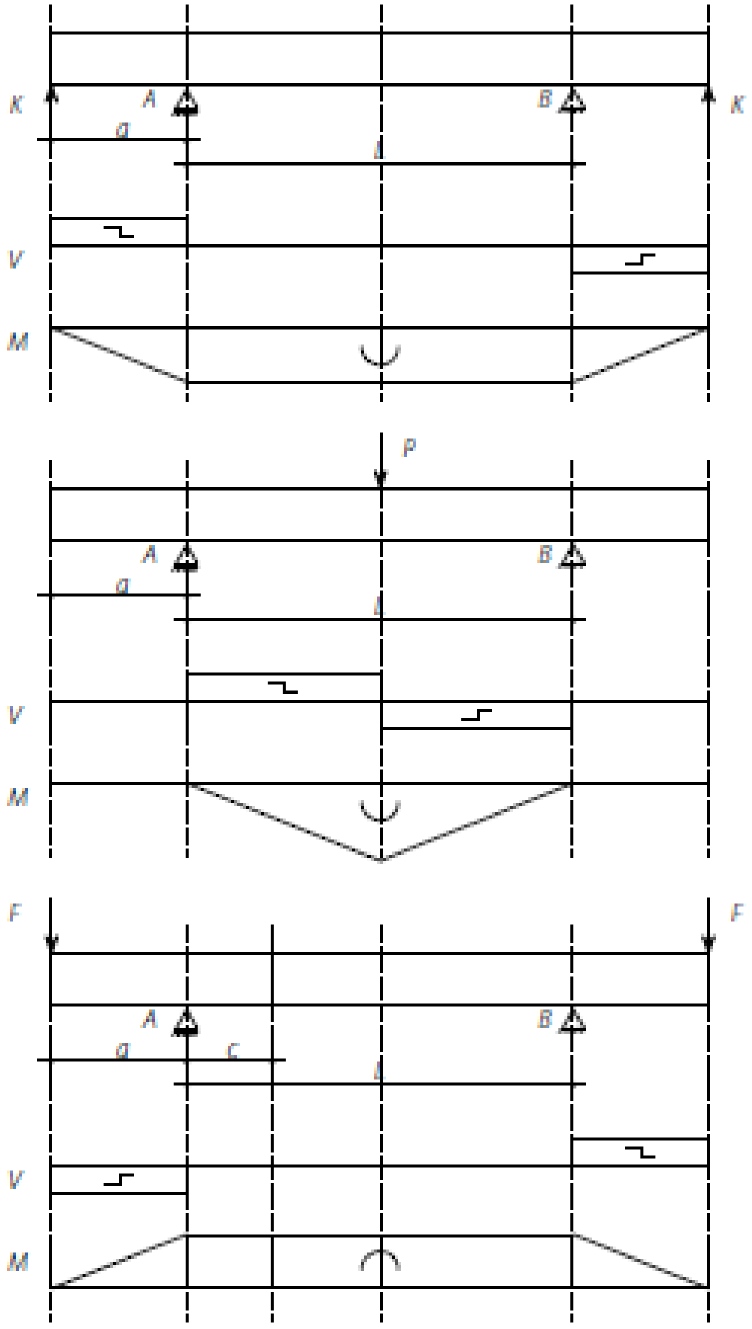


Figure 3.6 M-V lines of the test rig used for the experiments on the concrete beam. With local stirrups and reinforcement the resistance of the beam has been increased locally. This way it always fails in the cross section wanted.

Four experiments were conducted using the test rig shown in Figure 3.5. This gave the results as shown in Figure 3.7.

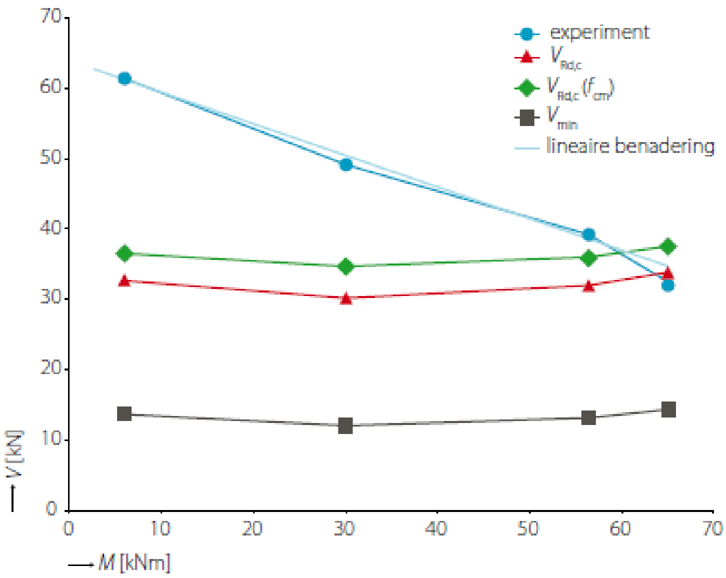


Figure 3.7 Results 4 experiments with different M-V combinations. All failed in vertical shear (in combination with the bending moment),V_{Rdc} = shear resistance using characteristic concrete compressive strength. V_{RDC} (f_{cm}) = shear resistance using mean value and V_{min} = minimum shear resistance of pure concrete.

Within these series of experiments of a concrete member without shear reinforcement, a reduction in shear resistance is found with a relation equal to:

$$V = V_{M=0} - 0.45M$$

Where:
V is the shear force in kN
M is the moment in kNm
$V_{M=0}$ is the shear force capacity, without moment applied.

Based on this research, the standard at force [12]is conservative if a low bending moment is present and only critical if a very high bending moment is present.

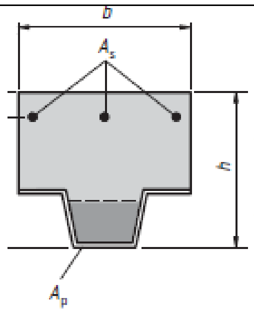
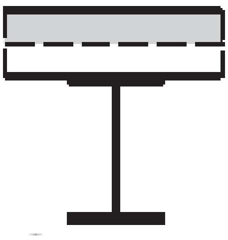
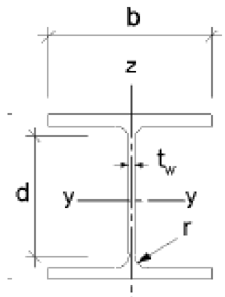
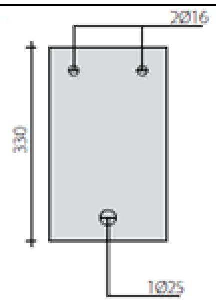
The experiments showed a bending moment has influence on the vertical shear resistance in a concrete beam without shear reinforcement. See chapter A.3 in the appendix for a more detailed overview of this research.

During this experiment the concrete itself had to resist all of the vertical shear and part of the bending moment as the critical cross section had very little reinforcement bars. Comparable to steel, one material is used for creating resistance against both shear and moment. This linear reduction in bending moment resistance might come from an increase in tension zone in the concrete member, which could reduce the shear resistance.

3.4 Overview considered steel-concrete structural elements.

This thesis questions the reduction in hogging bending moment in case the steel-concrete composite slab (ComFlor210 or ComFlor75) is loaded by a vertical shear force. As there is no research done for the ComFlor series and nothing is mentioned about interaction within the Eurocode 4 [5], composite beams, steel profiles and concrete beams have also been looked into.

Table 2 Overview considered structural elements, contributing parts to M & V resistance and M-V interaction.

Cross Section Element	Elements contributing to hogging bending moment resistance	Elements contributing to vertical shear resistance.	M-V interaction
1) 	Tension: • Top reinforcement mesh • Additional reinforcement top • ComFlor sheet (partly & if it is continuous over the support) Compression • ComFlor sheet • Concrete inside the ribs	Eurocode • Concrete rib (effective width [b_w] depends on location neutral axis) [5],[10] Experiments • ComFlor sheet [11]	Eurocode • No interaction mentioned. Experiments • No experiments done.
2) 	Tension: • Top reinforcement mesh • Additional reinforcement top • Structural steel section in tension (flange & web) Compression • Structural steel section, flange • Structural steel section, web	Eurocode • Structural steel section $V_{pl,a,Rd}$, unless a value for the concrete part has been established. [5],[10] Experiments • Concrete slab [2],[4]	Eurocode • If $V_{ED} > \frac{1}{2} V_{Rd}$ given by $V_{pl.Rd}$ *or* $V_{b,Rd}$ effect on moment resistance should be made. [5] Experiments • Interaction present according to Figure 3.3. [2]
3) 	Tension: • Top flange • Part of the web Compression • Bottom flange • Part of the web (if contribution of the flanges alone is not sufficient M-V interaction) [12]	Eurocode • Web of the steel section • Overlap web / flange in case web alone is not sufficient. [12]	Eurocode • If the moment is below the bending resistance based on flanges alone, no reduction is present • If shear force is less than half of the place shear resistance, its effect may be neglected [12]
4) 	Tension: • Top reinforcement Compression • Concrete in compression	Eurocode • Concrete:$V_{Rd,c} = [C_{Rd,c}k(100\,\rho_1 f_{ck})^{\frac{1}{3}} + k_1\,\sigma_{cp}]\,bwd$ [10] • Stirrups handle shear alone if the resistance of the concrete is insufficient. [10]	Experiments • A linear relation between V present and moment reduction (sagging moment) based on a concrete beam without shear reinforcement [1][13]

This overview shows if the elements contributing to the moment and shear resistance are used for both resistances or are contributing separately. Conclusions from the literature review are shown in the next chapter.

3.5 Conclusions literature review

In this chapter the most important conclusions are drawn from the literature review regarding the interaction in composite slabs, beams, steel and concrete.

- Composite slabs: Within the EN 1994-1-1 [5] no interaction is mentioned regarding composite slabs.
- Composite slabs: Within the EN 1994-1-1 [5] the vertical shear resistance of composite slabs is based on a concrete beam without shear reinforcement.
- Composite slabs: According to tests in London [3], the vertical shear resistances of composite slabs according to the EN 1994-1-1 [5] seem conservative and the ComFlor steel sheet seems to contribute significantly.
- Composite beams: Within the EN 1994-1-1 [5] M-V interaction is described and appears after a moment is applied surpassing the bending moment resistance of the flanges.
- Composite beams: According to research done in Australia [4][2] the concrete slab contributes significantly to the vertical shear resistance. Chapter A.2.
- Steel beams: Within the Eurocode [12] M-V interaction is included if the design value of the shear force V_{Ed}exceeds 50% of the design plastic shear resistance$V_{pl.Rd}$.
- Concrete beams: Concrete members without shear reinforcement, have a linear relation between moment and vertical shear force applied. Reduction in hogging bending moment resistance is found if a vertical shear force is present. [1].
- Concrete beams: Reinforcement steel has little to no vertical shear resistance on its own, concrete takes care of the vertical shear resistance; stirrups take over this role if the vertical shear force exceeds the vertical shear resistance of the concrete only.
- Steel beams: Do have a high vertical shear resistance of their own; therefore the same element is used for both moment and shear. This applies to both steel beams as to composite beams where the steel beam is applied underneath the concrete slab.

As M-V interaction in composite slabs is not mentioned in EN 1994-1-1 [5] and interaction is found in other compositions of steel / concrete beams, experiments have been done to determine if a reduction in hogging bending moment resistance is present within the field of application of composite slabs. This could provide initial data to answer the main objective and provide a comparable M-V interaction diagram as present for composite beams.

4 RESEARCH QUESTIONS

4.1 Calculation of the resistances of the ComFlor210 & ComFlor75 (approximated resistance to determine the test setup, not equal to the resistances of the final specimens)

For the composite slabs no research was available regarding M-V interaction. In order to find the region where interaction occurs, the vertical shear'V_U' and hogging bending moment resistance 'M_U^{-}'must be calculated. These calculations have been added in APPENDIX B.

4.1.1 Concluding spread in resistances

The following resistances are calculated to get insight in the calculation procedure and the spread due to uncertainties in the calculation method. These values are used to determine a suitable test setup, afterwards a more detailed calculation is done for the actual test specimens. Both calculations (for the ComFlor210 & ComFlor75) are done with Ø10-75 and C30/37. The aim is to get an idea of the magnitude of the resistances and the spread in uncertainty; these resistances are not similar or comparable to the final test specimen used. According to the initial calculations done in APPENDIX B the resistances are as following:

ComFlor210 slab:
Minimum distance from support to avoid direct support of the force: 771 mm (d = 257 mm)
The hogging bending moment resistance: $M_U = 85kNm/m$
The vertical shear resistance:$V_U = 124\ kN/m$.

ComFlor75 slab:
Minimum distance from support to avoid direct support of the force: 360 mm (d = 120 mm)
The hogging bending moment resistance: $M_U = 57\ kNm/m$
The vertical shear resistance:$V_U = 143\ kN/m$.

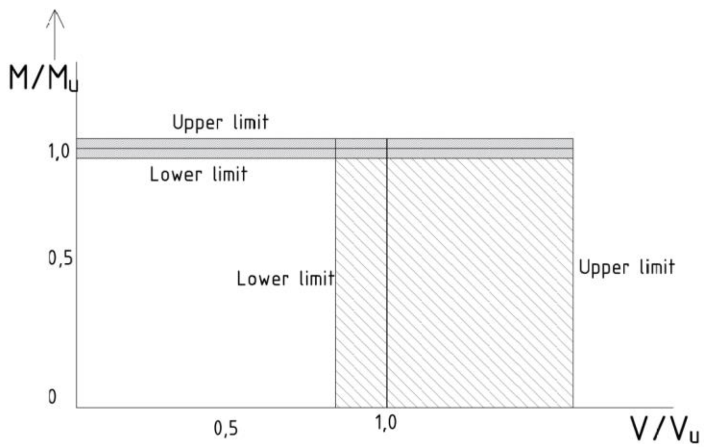


Figure4.1 Spread in resistances ComFlor210

The value of $M_U$has a 15% spread as shown in Figure4.1 due to a spread in material properties. This spread can be greatly reduced by making test cubes and determine the actual strength just before the specimen is tested. The same counts for the reinforcement.
The value of V_U has a huge spread due to the uncertain aspects that have been added in chapter B.3 in the Appendix. There is a big difference between the resistances of only the concrete rib, according to the EN 1994-1-1 [5] and additional contributions according to experiment. [3] Besides in case of an intermediate support the top of the slab is in tension. All tests give information about resistances in combination with a positive bending moment, like the test done in London on the ComFlor75 [3].

4.2 Determination of the test setup

The main objective is to get insight in the influence of a vertical shear force on the hogging bending moment resistance. One way is to do a series of experiments to create an M-V interaction diagram as shown in Figure 4.2. As discussed in previous chapters the hogging bending moment resistance can be predicted by calculations and material tests. The vertical shear is more complicated and to reduce the spread (uncertainties as explained in B.3 in the appendix) an extra test would be required. In this chapter a final choice is made on what test setup should be used to answer the main research question.

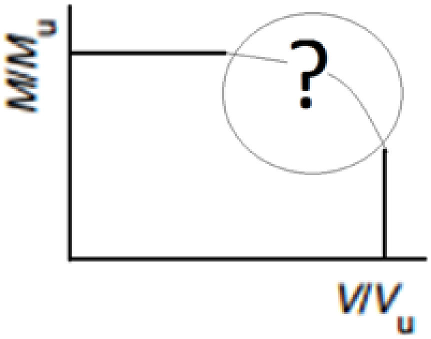


Figure 4.2 Region of interest in the relation between M & V

4.2.1 Data used to determine the test setup.

The test should represent the situation as it is applied in practice, where the maximum vertical shear force is present at the same location as the maximum hogging bending moment.
In practice the ComFlor210 is integrated with for instance a HE200A steel beam to reduce the construction height as shown in Figure 4.3 on the left. This means the ComFlor210 sheet does not continue and cannot contribute to the hogging bending moment resistance.
This means the ComFlor210 sheet does not continue over the intermediate support and the top reinforcement is the only structural element that continues over the support and contributes to $M_{Rd}{}^{-}$.This detail is shown in Figure4.4 on the right. After the concrete above the HE200A has cracked, only the reinforcement continues over the support. The ComFlor210 is however casted in the concrete for 50mm. It therefore could provide some resistance but is neglected.

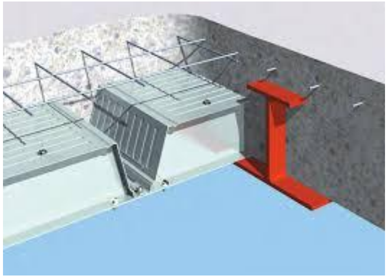


Figure 4.3 Integrated detail of a ComFlor210 composite slab to reduce construction height.

In this chapter possible test setups will be briefly discussed.

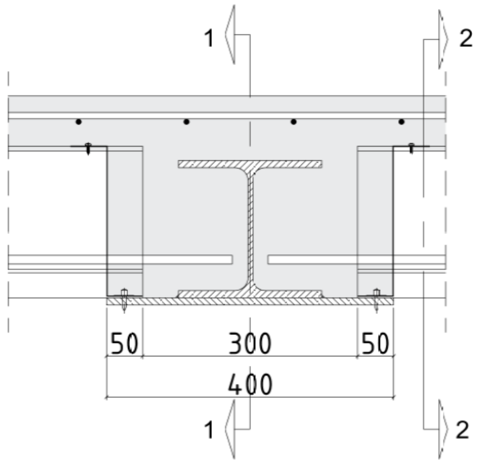


Figure4.4 Cross section of an integrated ComFlor210 sheet in a HE200A with a welded steel plate below.

4.2.2 Considered test setups.

First test setup considered is a simple test rig, 2 supports and a force applied on the cantilever. This setup cannot be used to determine the vertical shear resistance of the specimen, but can be used to let the specimens fail in bending, while having a different vertical shear occurring by simply changing the point of engagement of the force.

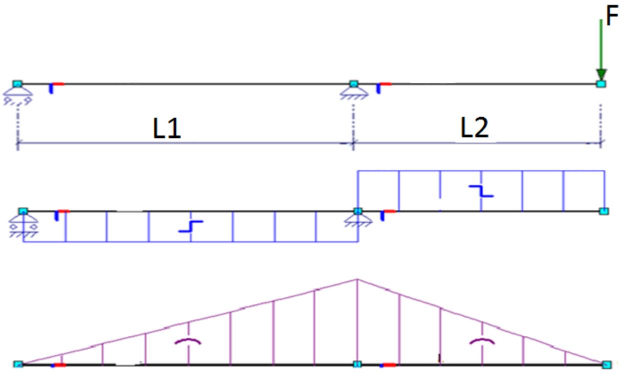


Figure 4.5 Mechanical scheme possible test setup 1

A minimum a/d ratio (L_2/d) of 3 is used to avoid direct support of the load, this setup allows combinations with a negative bending moment occurring equal to 100% and a shear force varying between 10% - 100%. More details considered about this possible test setup can be found in chapter C.2 in the appendix.

The second test setup considered is a slab as applied in practice. Three supports, applying two equal point loads by mean of a spreader beam. This gives the mechanical scheme as shown in Figure 4.6.

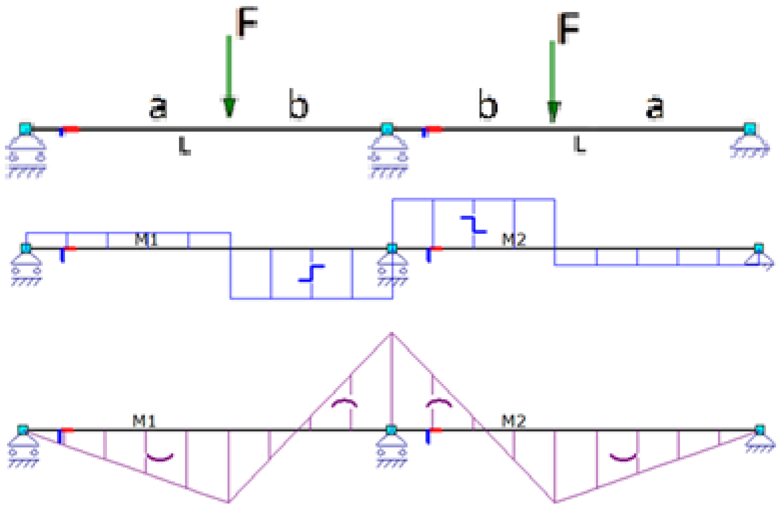


Figure 4.6 Mechanical scheme possible test setup 2.

This setup has a wider range of application, but is statically undetermined, requiring more advanced analyzing methods. Using this setup a minimum of 65% of the M_U will be present, meaning it cannot be used to determine the vertical shear resistance. More details considered about this possible test setup can be found in chapter C.2 in the appendix.

The third setup considered was based on completely controlling the M-V relation. In order to complete the interaction diagram the specimen should fail in vertical shear without (nearly) any bending moment present and avoiding direct transfer of the force to the support. The test setup that can be used is shown in Figure 4.7

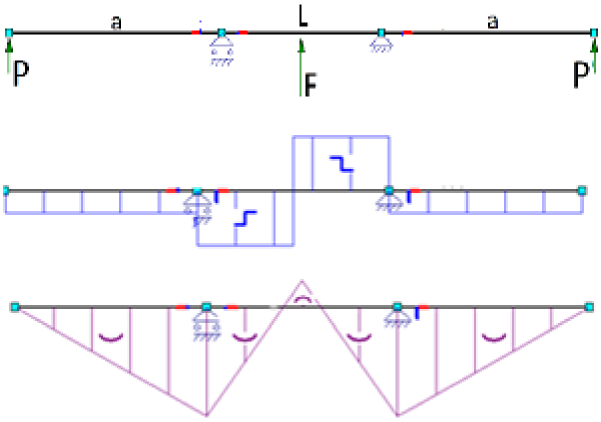


Figure 4.7 Mechanical scheme possible test setup 3.

It has a very wide range of application. (M-V relation). By "prestressing" (prebending) the specimen a minimum of 10% of the Mu can be achieved below the applied force F. This however does give possible other failure locations. To avoid this, the critical section below the applied force F must be weaker compared to other locations. Some sort of stirrups must be applied to strengthen the specimen at different locations. This option is therefore a possible way to determine the vertical shear resistance, but too complicated to use for general testing. More details considered about this possible test setup can be found in chapter C.4in the appendix.

4.2.3 Conclusion test setups

In order to answer the research question as shown below, two possible approaches can be taken.

To do a limited series of tests, to get insight in the interaction between the hogging bending moment and the vertical shear. Analyzing this data to compare it with the current codes. Thereby providing initial data to support the calculations made by Dutch Engineering according to the current EN 1994-1-1 [5]& EN 1992-1-1 [10].

First of all, if interaction plays a role, the failure moment should change if a higher vertical shear force occurs. The amount of interaction depends on the reduction in failure moment. If no interaction plays a role, the negative failure moment should be independent of the vertical shear force present. Changing the vertical shear force should therefore lead to more or less the same failure moment. This approach can be very well done with the mechanical schemes of setup 1.

Secondly if the interaction diagram is wished to be completed, determination of the vertical shear resistance is required as previous chapters showed that the current EN 1994-1-1 [5] does not give a calculation method for the exact vertical shear resistance For this approach the exact vertical shear resistance is required. This could be done with test setup 3, but still requires some attention to prevent other failure modes and more complicated calculation when including a changing modulus of elasticity of the concrete for being a non linear elastic material. Further test could be done according to setup 1.

Final conclusion test setup

As 2 tests on each slab type are wanted, completion of the interaction diagram is too complicated and will not directly answer the questions raised in practice. Therefore the simpler test setup 1 is preferred. However insight must be gained without determination of the exact vertical shear resistance, but based on values used in practice as for example the case study “town hall - Almelo” that raised the question. This will be discussed further in the next chapter. Final setup is shown below:

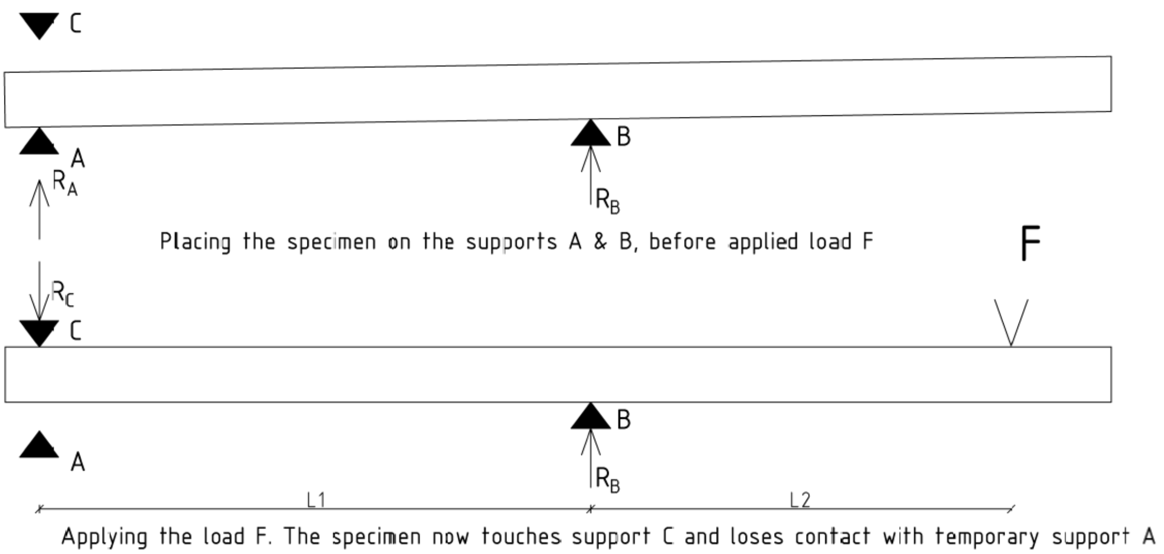


Figure 4.8 Test setup 1, the final test setup used.

5 TEST SETUP

5.1 Test setup and range of application

In order to find the influence of a vertical shear force on the hogging bending moment resistance a test setup is chosen which can both test the hogging bending moment resistance with a small or a high vertical shear force, the relation between occurring vertical shear and negative bending moment is based on practical values. To determine the most suitable a/d ratio's for the setup chosen, a reference is made to the practical values based on the case study described in chapter C.1 and described below. This is based on a ComFlor210 slab.

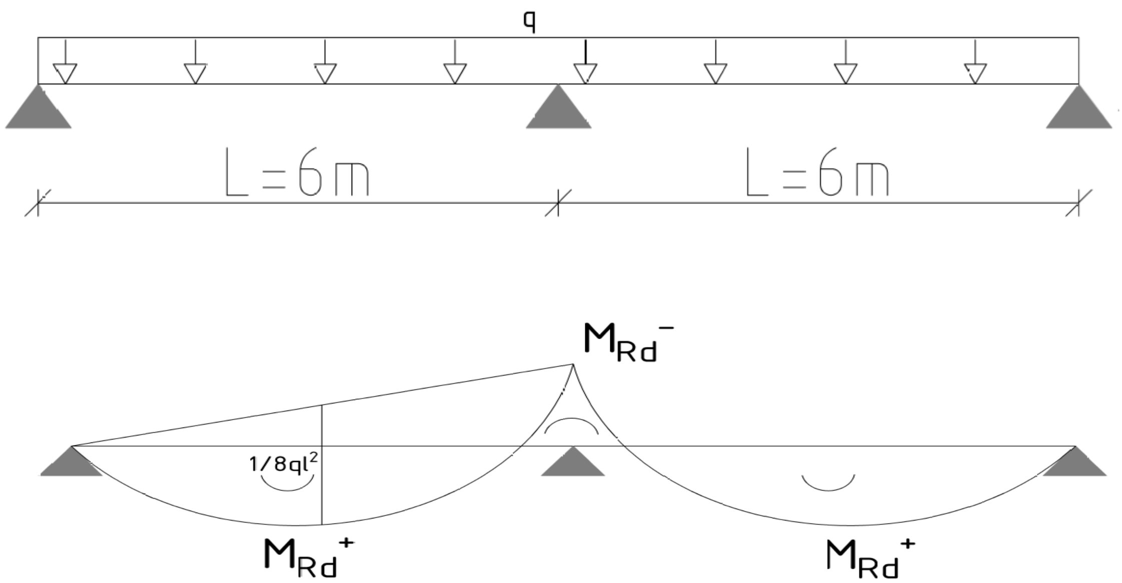


Figure 5.1 Mechanical scheme's in case the slab is supported on 3 supports and plastic hinges form.

The relation between the distributed load and the moments needed to form the mechanism can be found through the following relation:

$$\frac{1}{8ql^2} = \frac{1}{2} * M_{Rd}{}^{-} + M_{Rd}{}^{+}$$

Rewriting this equation gives:

$$q = \frac{4 * M_{Rd}{}^{-} + 8 * M_{Rd}{}^{+}}{l^2}$$

The vertical shear is equal to:

$$V_{pl} = \frac{ql}{2} + \frac{M_{Rd}{}^{-}}{\mathrm{l}}$$

This results in the following relation between V_{pl} and the positive and negative plastic moment resistance.

$$V_{pl} = \frac{4 * M_{Rk}{}^{-} + 8 * M_{Rk}{}^{+}}{2l} + \frac{M_{Rk}{}^{-}}{\mathrm{l}}$$

Using the resistances of the case study as explained in chapter C.1, the $M_{Rk}{}^{-} = 66.5\frac{\text{kNm}}{\text{m}}$,$M_{Rk}{}^{+} = 90.2\frac{\text{kNm}}{\text{m}}$ and . $V_U = 91\frac{\text{kN}}{\text{m}}$ the following distributed load (q) is needed and will result in a particular vertical shear at the intermediate support.

Table 3 Practical spans with corresponding load and vertical shear at the support to create a mechanism.

Span length [m]	q [kN/m]	V at intermediate support [kN]	Comments
3.6	76.19	103.73	Very high q load need for mechanism. Not a practical value.
5.4	33.86	93.36	High q load needed, could fail on shear.
6	27.43	89.81	Around 100% of the shear resistance based on calculation done for the case study. Distributed load q realistic, but high.
7.2	19.05	77.8	Plastic hinges are formed before calculated vertical shear resistance is reached.

Table 3 shows which distributed load is needed to create a combination where both M & V will be critical at practical spans. Constraining the positive and negative moments at their plastic capacity and changing the span (l) gives the critical spans that are sensitive to M-V interaction based on the resistances of the case study “Town Hall - Almelo”:

Table 4 Relation plastic hinges and span based on a percentage of the vertical shear force present with the related distributed load to cause this combination. The colors show the likeliness to occur in practice (these values are excluding all material factors)

V/V_{pl} (%)	Span (l) [m]	q [kN/m]
120%	5.1	37.9
100%	6.15	26.1
80%	7.7	16.7
50%	12.3	6.5

This shows only a region near a span of 6 meters will be in the region of interaction using practical resistances and distributed loads. The a/d ratio for the test setup and relation between M-V will therefore be chosen based on resistances based on the case study.

Based on the setup 1 from Figure 4.5. The following relation can be used to calculate the negative bending moment and vertical shear force applied:

$V = F$ And $M = F * L_2$ ➔ $M = V * L_2$

Using the resistance of the case study, a distance of $L_2 = 720mm$ is found, this is equal to an a/d ratio of 2.92. Where a = L_2, d=251mm as in the case study in chapter C.1 and a/d > 2 to avoid direct transfer of the force applied to the support.

For each slab a length of a/d = 6 is chosen to determine the $M_{Rk}{}^{-}$, followed by an a/d equal to 3 to generate a situation in practice were both the occurring hogging bending moment as well as the vertical shear equal to approximately 100% and to avoid direct transfer to the support.

5.2 Specimen dimensions and capacities

For each specimen the M_{Rk}^{-} and the $V_U$are specified before determination of the exact dimensions of the test setup. All specimens are casted with C30/37 concrete and reinforcement with a tensile strength of approximately 500 N/mm². Before each test 2-3 concrete test cubes are tested to determine the compressive strength of the concrete on the day of testing. The reinforcement is tested by use of tensile tests done on 15-09-2016 at the TU Delft. Each test is described in APPENDIX DMaterial tests.

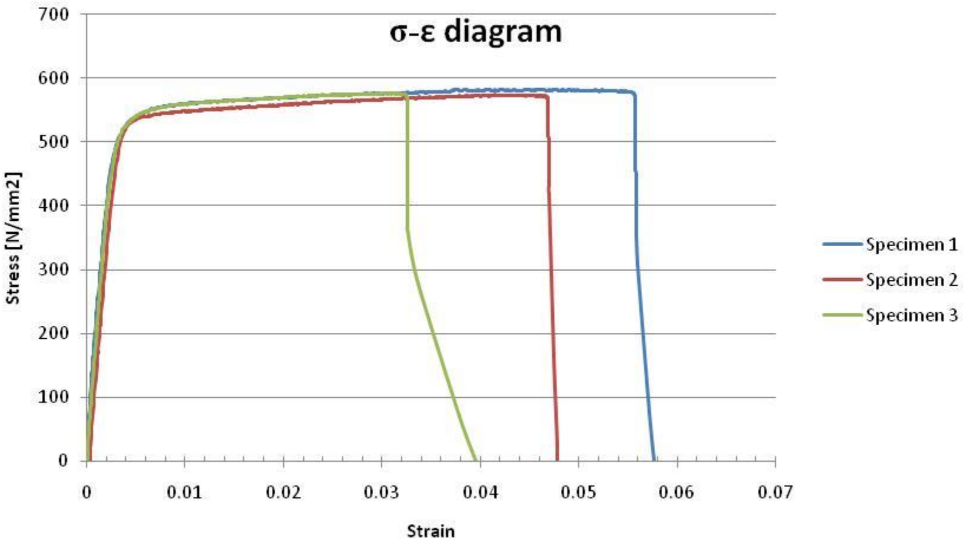


Figure 5.2 Stress - Strain, tensile tests, reinforcement bars.

This resulted in an average $f_u = 574\ N/mm^2$ and a 0,2% yield stress of 540 N/mm². The value of f_u will be further used to calculate the M-V resistances.

Table 5 Compressive strength concrete cubes

	Date / Time	Day	kN	N/mm²	mm²	Average (N/mm²)
Cube 1	04-10-16	21	949	42.18	22500	39.85
Cube 2	04-10-16	21	863.5	38.38	22500	
Cube 3	04-10-16	21	877.6	39.00	22500	
Cube 4	06-10-16	23	934.7	41.54	22500	41.54
Cube 5	12-10-16	29	835.3	41.24	20250	41.76
Cube 6	13-10-16	29	956.8	42.52	22500	
Cube 7	13-10-16	29	917.7	41.52	22100	
Cube 8	19-10-16	35	961.3	42.72	22500	42.093
Cube 9	19-10-16	35	932.9	41.46	22500	
Cube 10	20-10-16	36	981.4	43.61	22500	43.60
Cube 11	20-10-16	36	964.2	42.85	22500	
Cube 12	20-10-16	36	997.8	44.34	22500	

These values will be used to determine the resistances of all 5 specimens.

5.2.1 Dimensions of the specimens

Overview drawings are given of the test specimen. For more detailed drawings see chapter E.1 in the appendix.

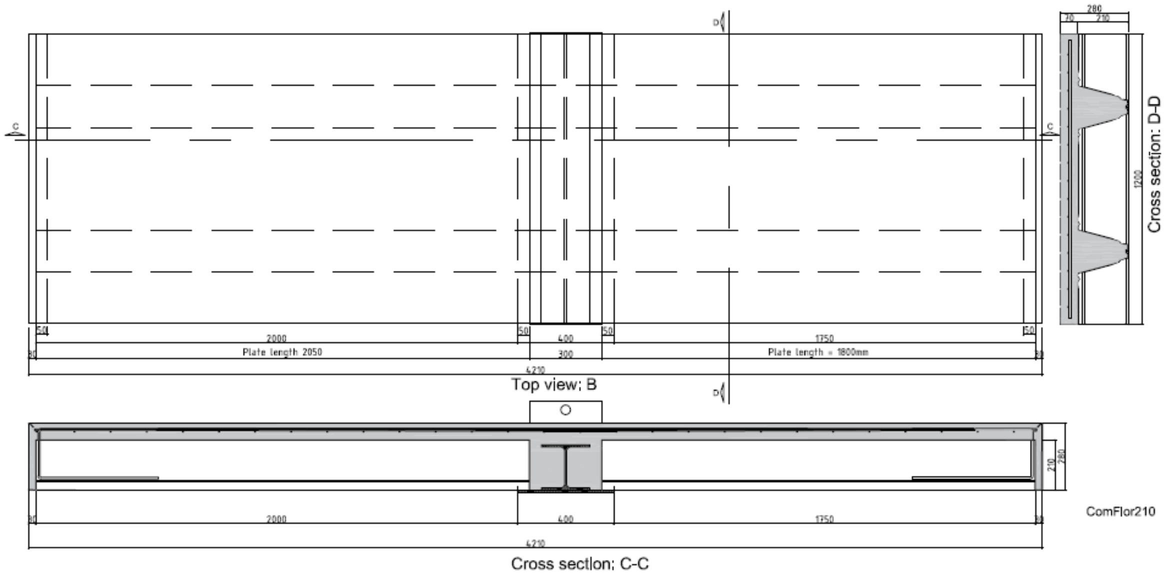


Figure 5.3 Overview ComFlor210 specimens 1-3, see appendix E for detailed dimensions.

A total of three ComFlor210 are casted. Two identical with Ø8-75 and one with 8 Ø8-150 reinforcement bars with an additional 7 Ø10-150 bars.

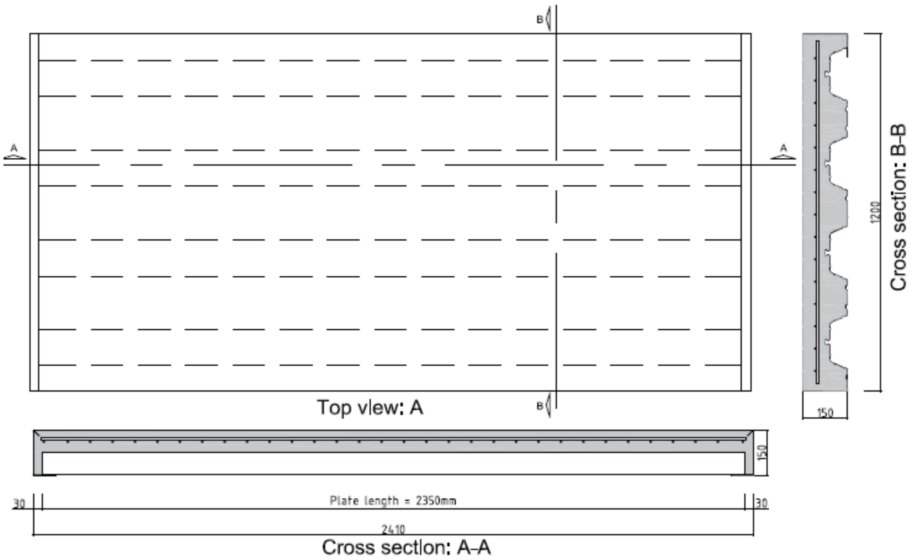


Figure 5.4 Overview ComFlor75 specimens 4-5, see appendix E for detailed dimensions.

Two ComFlor75 are casted, both with Ø8-75 reinforcement bars. (15 bars in total).

Table 6 Overview specimens, number, type, reinforcement.

Number	ComFlor Type	Reinforcement	Bars Ø8	Bars Ø10	A_{sl} [mm²]
Specimen 1	ComFlor210	Ø8 – 75	15	0	754
Specimen 2	ComFlor210	Ø8 – 75	15	0	754
Specimen 3	ComFlor210	Ø8 – 150 + Ø10 – 150	8	7	952
Specimen 4	ComFlor75	Ø8 – 75	15	0	754
Specimen 5	ComFlor75	Ø8 – 75	15	0	754

Table 6 gives an overview of the five specimens used during the experiments.

5.2.2 Overview resistances test specimens

The resistance of each specimen has been calculated based on the measured material properties from the material tests done at the TU Delft laboratory. The detailed calculations can be found in chapter E.2 of the appendix. Below an overview is given of all the specimens and their resistances. The resistance is based on a 1.2m wide slab as used during the tests. For the ComFlor210 two different cross sections can become critical, namely cross section 1 and 2(3) as shown in Figure 5.5. This cross section in more detail can be found in E.1.6. Certain contributions have not been included; these are described in next chapter.

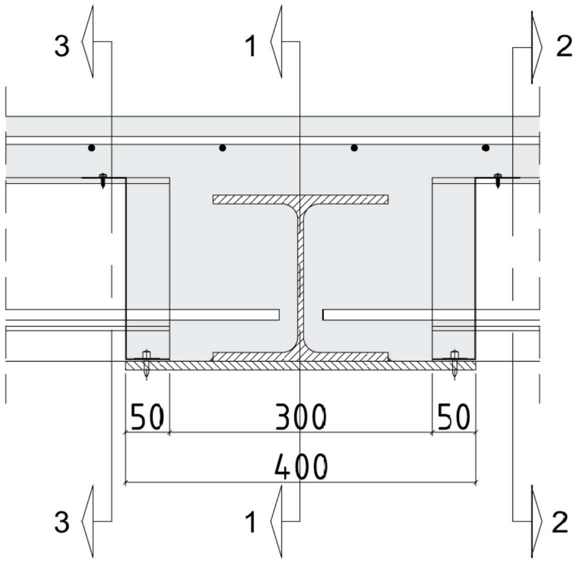


Figure 5.5 Critical cross sections ComFlor210 specimens.

Table 7 Overview expected resistances test specimens, all resistances are calculated with the measured material properties.

Test Specimen	M_{Rd}^- (cross section 1)[kNm]	M_{Rd}^- (cross section 2) [kNm]	$V_U$per slab [kN]
1 ComFlor210	107.3	90.9	111.9
2 ComFlor210	107.3	90.9	111.9
3 ComFlor210	134.2	109.6	120.23
4 ComFlor75	61.51	61.51	125.
5 ComFlor75	61.51	61.51	125

5.2.3 Contributions that have not been included in the calculation.

Additional profiles

The measures (strips and closure profiles) shown in Figure 5.12 could slightly contribute to the hogging bending moment resistance. This is however not included in the calculations as the contribution should be minimal.

Joist shuttering

For the ComFlor210 the joist shuttering on top does not continue of the HE200A steel beam at the support, however it is surrounded by concrete and is fastened at every 500 mm with a small strip. For the ComFlor75 the joist shuttering does continue and is therefore cut out of the specimens before testing. In both cases, this could give a minor contribution to the capacity.

ComFlor210 sheets

The ComFlor210 steel sheets are simply supported and do not continue over the support. However the sheets are fastened by screws and after casting it is anchored in 50 mm concrete. This is shown in Figure 5.6and in more detail in E.1.6in the appendix. Calculations are done based on a simply supported ComFlor210 sheets, they might however contribute to the hogging bending moment.

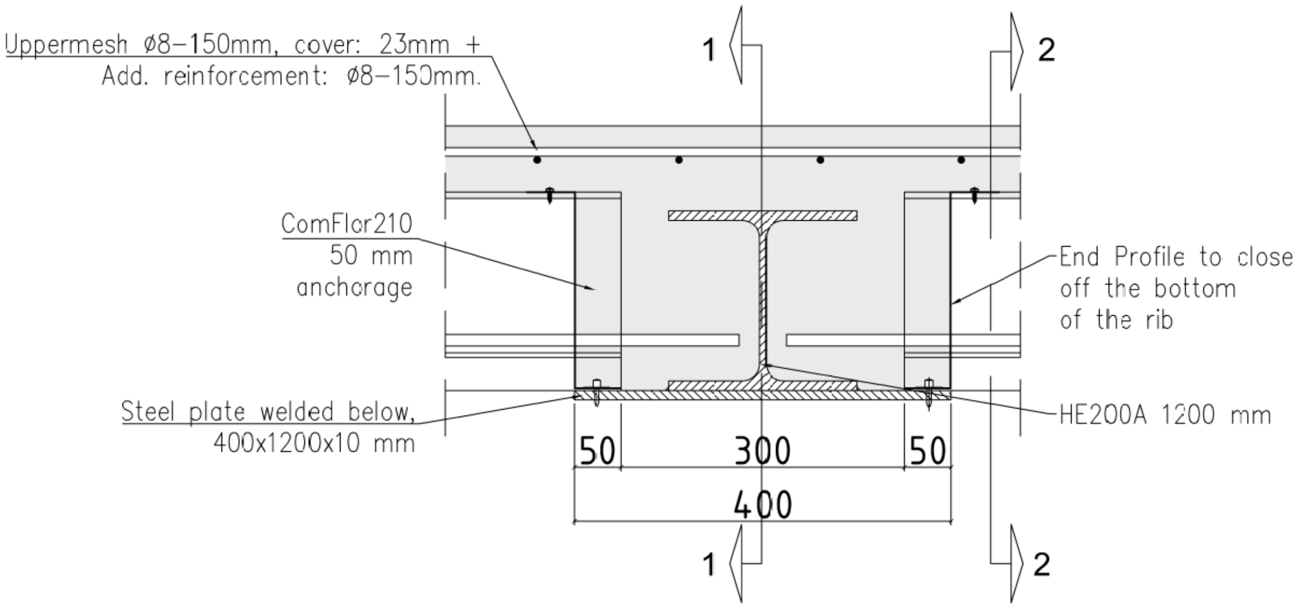


Figure 5.6 Critical cross sections ComFlor210 specimens 1 & 2.

5.2.4 Self weight test specimen and test rig.

The cross section in Figure 5.7can be divided in three parts, each preloading the specimen in its own way.

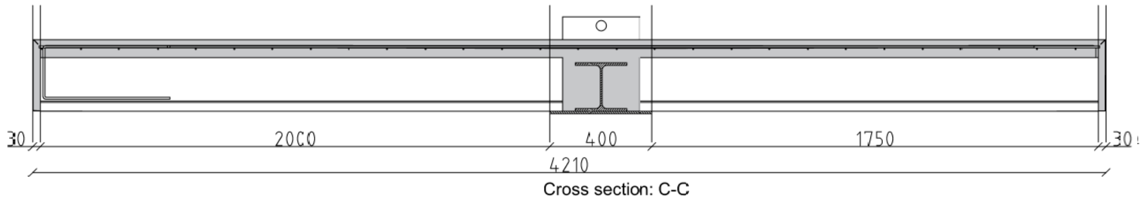


Figure 5.7Cross section test specimen 1-3. Can be divided in three parts: Steel beam, two ribs and two End girders

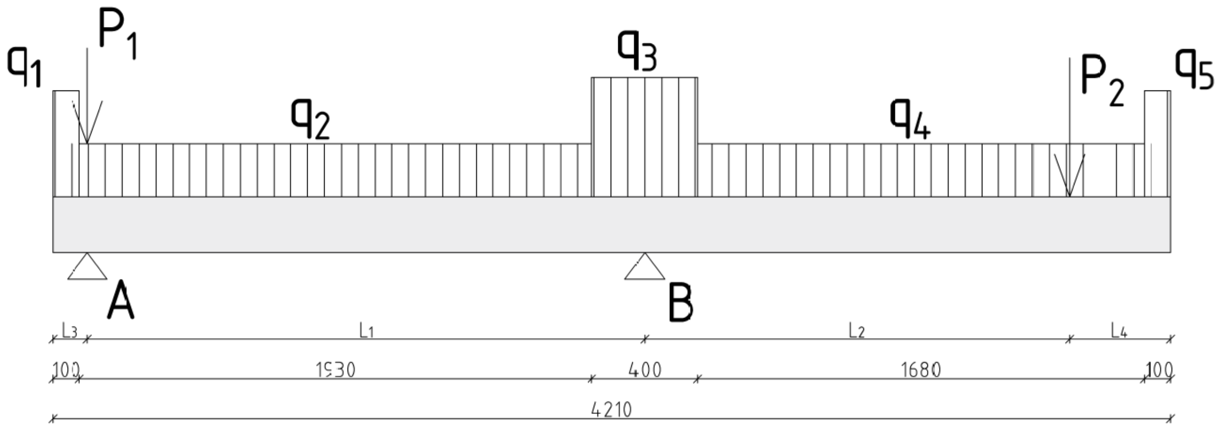


Figure 5.8 Mechanical scheme self weight, ComFlor210

Besides the self weight, one mechano beam is placed at A to prevent the specimen from moving up. Spreader beams are in between the cylinder and the specimen to spread the point load from the cylinder into an equally distributed load. The test rig will be shown and explained in the next chapter. In Table 8 an overview is given of each contributing part to the reaction force and hogging bending moment at the start of the test.R_A is not included in the table but can be calculated based on the difference in $M_{B\ left}$and $M_{B\ right}$.

Table 8 Initial self weight test specimen 1-3

Discription contributing part	Load tag	Value	Length [m]	Weight (q) (kN)	Distance of engagement measured from B [m]	$M_{B\ left}$[kNm]	$M_{B\ right}$ [kNm]
Mechano beam	P_1	2.7 kN	-	2.7	L_1	2.7*L_1	
End girder (Support)	q_1	9 kN/m	0.1	0.9	2.135	1.92	
Slab, ribs (Support)	q_2	3.35 kN/m	1.93	6.5	1.165	7.57	
Spreader beams	P_2	4.8 kN	-	4.8	L_2		4.8*L_2
End girder (Cantilever)	q_4	9 kN/m	0.1	0.9	1.685		1.52
Slab, ribs (Cantilever)	q_5	3.35 kN/m	1.68	5.6	1.04		5.824
Integrated HE200A	q_3	12.65kN/m	0.4	5.06	0	0	
Total value				26.5 kN		9.49+2.7*L_1	7.34+4.8*L_2

The ComFlor75 specimens continue over the intermediate support, the specimens are therefore simpler and have less self weight.

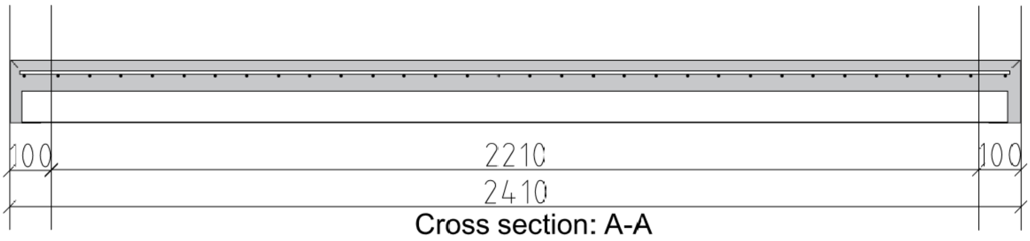


Figure 5.9Cross section test specimen 4-5. Can be divided in two parts: 5 ribs and two End girders

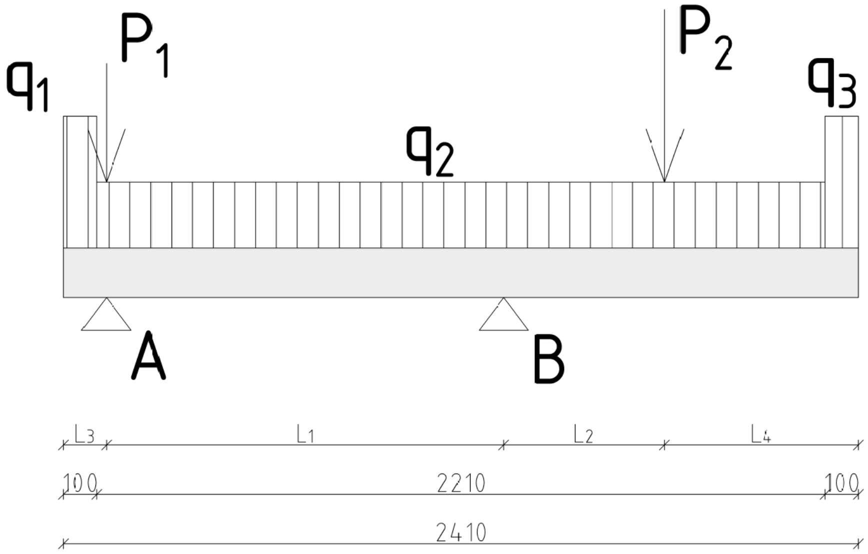


Figure 5.10 Mechanical scheme self weight, ComFlor75

Below in Table 9 an overview is given of the contributing parts and their weight.

Table 9 Initial self weight test specimen 4-5

Discription contributing part	Load tag	Value	Length [m]	Weight (kN)	Distance of engagement measured from B [m]	$M_{B\ left}$ [kNm]	$M_{B\ right}$ [kNm]
Mechano beam	P_1	2.7 kN	-	2.7	L_1	2.7*L_1	
End girder (Support)	q_1	4.5 kN/m	0.1	0.45	1.25	5.25	
Slab, ribs (Support)	q_2	3.5 kN/m	1.2	4.2	0.6	2.52	
Slab, ribs (Cantilever)	q_2	3.5 kN/m	1.01	3.54	0.505		1.79
Spreader beams	P_2	4.8 kN	-	4.8	L_2		4.8*L_2
End girder (Cantilever)	q_3	4.5 kN/m	0.1	0.45	1.06		0.48
				16.1 kN		7.77+2.7*L_1	2.26+4.8*L_2

5.3 Assemblage

In this chapter a short review is given of the assemblage of the test specimens.

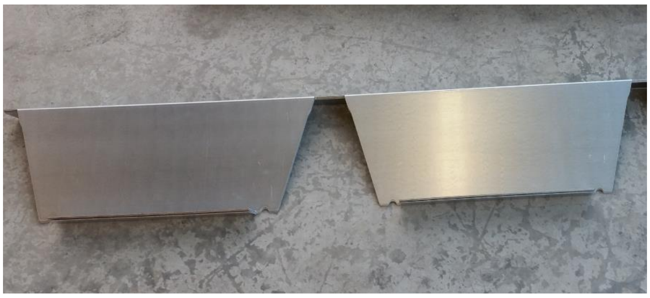
First the “molds” were made to simulate the composite slabs in the region of interest. For the ComFlor75 specimens in particular this means a joist shuttering all around the slab. As this could contribute to the hogging bending moment, the joist shuttering was grinded afterwards to cut it in half. The “molds” are shown in Figure 5.11 below:

a) ComFlor75 sheets in place, including the joist shuttering.

b) All three ComFlor210 specimens in place, including the integrated HE200A steel beam.

Figure 5.11 Assemblage of the ComFlor sheets with the corresponding joist shuttering.

a) Strips c.t.c. 500 mm to prevent bending of the joist shuttering due to wet concrete.



b) End profile to close off the bottom of the ribs.

Figure 5.12 Extra measures to assure the wanted dimensions of the specimen.

The measures shown in Figure 5.12 could slightly contribute to the hogging bending moment resistance. This is however not included in the calculations as the contribution should be minimal.
Secondly the reinforcement is placed. As the concrete spacers delivered are only 30 mm, this has an influence on the internal lever arm (reduction). This has therefore been adjusted in the drawings and calculations.

The placement of the reinforcement and the final configuration are shown in Figure 5.13 below:

a) *ComFlor210 specimen including the joist shuttering, ready to place the reinforcement*.

b) *Reinforcement meshes, sufficient in size to cover the entire test specimen and to avoid overlap*

c) c) *Reinforcement put in place in the ComFlor210 specimen*.

Figure 5.13 Placement of the reinforcement, the mesh is big enough to fit all specimens without overlay.

Once all reinforcement has been put in place and double checked on the right sizes and dimensions, the concrete is poured. In Figure 5.14 the pouring of the concrete is shown.

a) Pouring of the concrete, all specimens will be casted at once.

b) Flattening out of the top side after casting.

Figure 5.14 Pouring of the concrete in the specimens and the test cubes

18 days after pouring of the concrete, the test specimens have been transported to the Stevin II laboratory at the TU Delft. This is done in such a way that the specimens will not be preloaded to avoid unwanted influence on the tests.

5.4 Different scheme's and relation to the M-V diagram per specimen.

In this chapter an overview is given of all 5 test rigs, mechanical scheme's and the interaction diagram of the occurring hogging bending moment and the vertical shear.

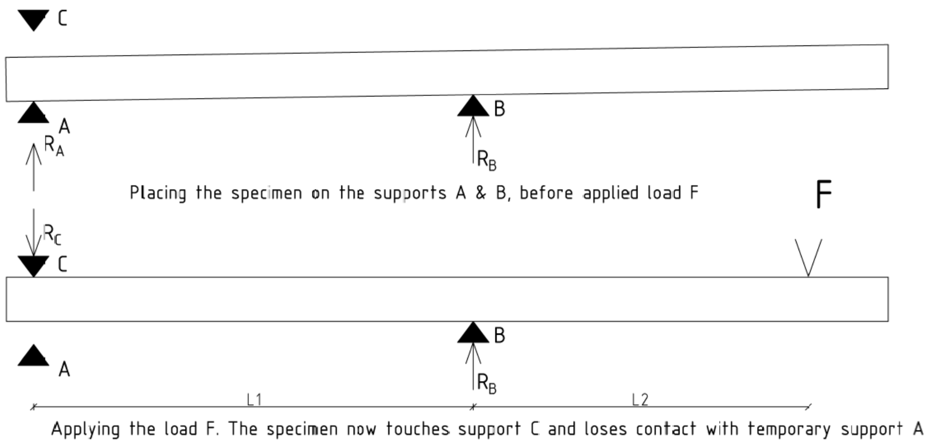


Figure 5.15 Test setup used, with F the applied force at a distance equal to L2 (a) from the support.

All five specimens are tested by using the test setup shown in Figure 5.15. Self weight and dead load on the specimen are shown in Table 8 & Table 9. The properties of each specimen are found in Table 6, these leaded to the resistances shown in Table 7. The ComFlor210 has an integrated HE200A beam with a 1200x400x10 mm steel plate underneath as shown in E.1.6, therefore two different a/d ratios can be found based on cross section 1 & 2. Distances used for all 5 tests can be found in

Table 10 Distances of applying the load and a/d ratios for all 5 specimens.

Test	L_1[mm]	$L_2(a)$ [mm]	d [mm]	a/d (cross section 1)	a/d (cross section 2)
1 ComFlor210	1970	1620	253	6.4	5.6
2 ComFlor210	1950	915	253	3.6	2.8
3 ComFlor210	1970	915	252	3.6	2.8
4 ComFlor75	1100	660	109	6	n/a
5 ComFlor75	1100	330	109	3	n/a

With the chosen distances the occurring forces can be determined. Using the data collected in Table 8&Table 9 the moment at B due to dead load (M_B) can be calculated by filling in L_1&L_2 from Table 10. Combining this moment with the capacity, the force [F] needed to let the specimen fail can be determined.

Table 11 Prediction of the failure load [F] based on the calculated hogging bending moment resistance of the specimen (APPENDIX E.2) and moment due to self weight.

Test	M_{Rd}^- (cross section 1)[kNm]	M_B dead load [kNm]	$M_{Rd}^- - M_B$	F$(M_{Rd}^- - M_B)$ /L$_2$) [kN]	V (force + self weight) [kN]	V/V_U
1 ComFlor210	107.3	15.12	92.18	56.9	70.73	0.63
2 ComFlor210	107.3	11.73	95.57	104.4	118.23	1.06
3 ComFlor210	134.2	11.73	122.47	133.8	147.63	1.23
4 ComFlor75	61.51	5.43	56	84.8	93.59	0.75
5 ComFlor75	61.51	3.84	57.67	174.75	183.54	1.47

The force [F] shown in Table 11 can be used as a predicted failure load for the tests. The expected moment shear relation can be found in the last column, with M/M_U = 1 and V/V_U varying. In APPENDIX F different diagrams are shown in more detail related to the expected forces occurring.

6 EXPERIMENTS

In this chapter an overview is given of the different test rigs, measure devices and the procedure of the experiments.

6.1 Test rig

Before the test specimens can be placed, the test rig must be constructed. The most important details of the test rig will be discussed here; the detailed parts are discussed in APPENDIX G.

The test rig contains 4 major parts. A temporary support (support "A" left side), the end support (support "C" left side), the centre support (support "B") and the location of applying the load (at "F", cantilever side).

Scheme with tags of each support and load is shown in Figure 6.1 and the test rig in the Stevin II laboratory at the TU Delft is shown in Figure 6.2.

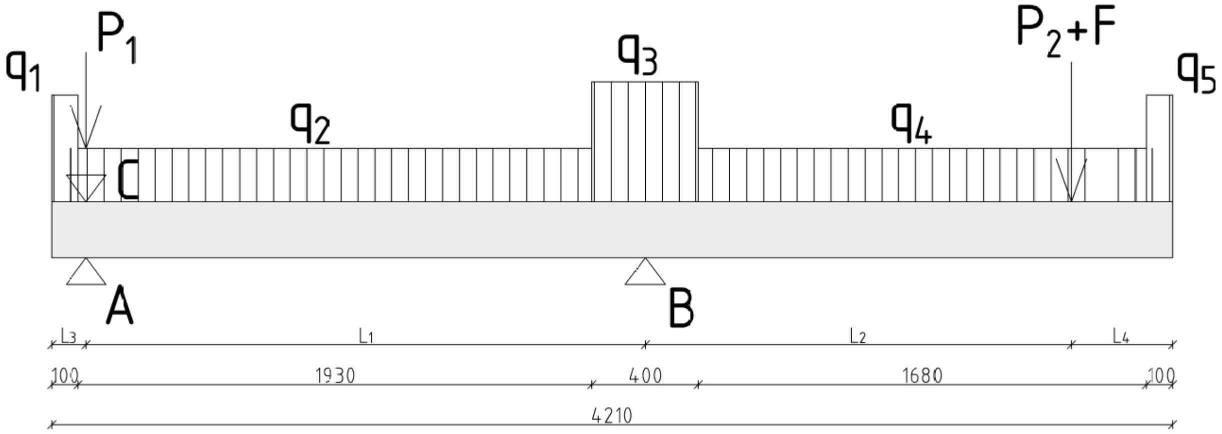


Figure 6.1 Schematic view of the test rig for the ComFlor210 specimens.

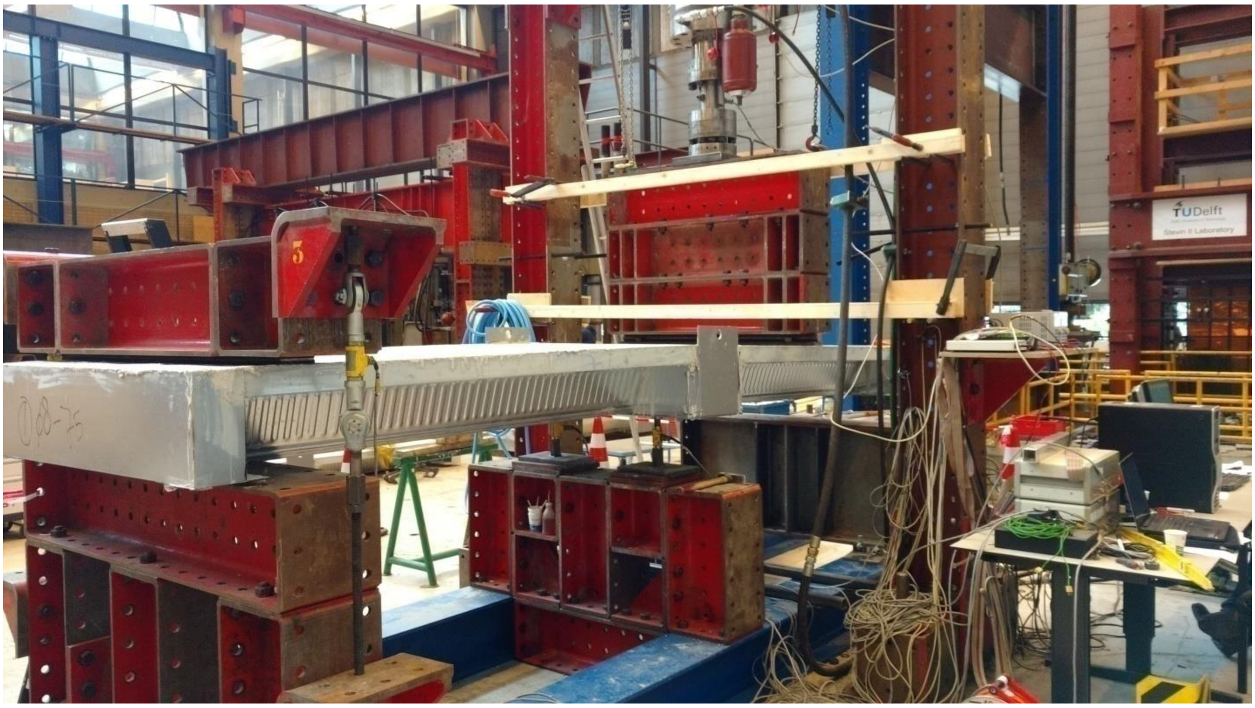


Figure 6.2 Overview test setup for one of the ComFlor210 specimens.

6.1.1 Overview test rig, side view

The complete test rig is shown in Figure 6.3.

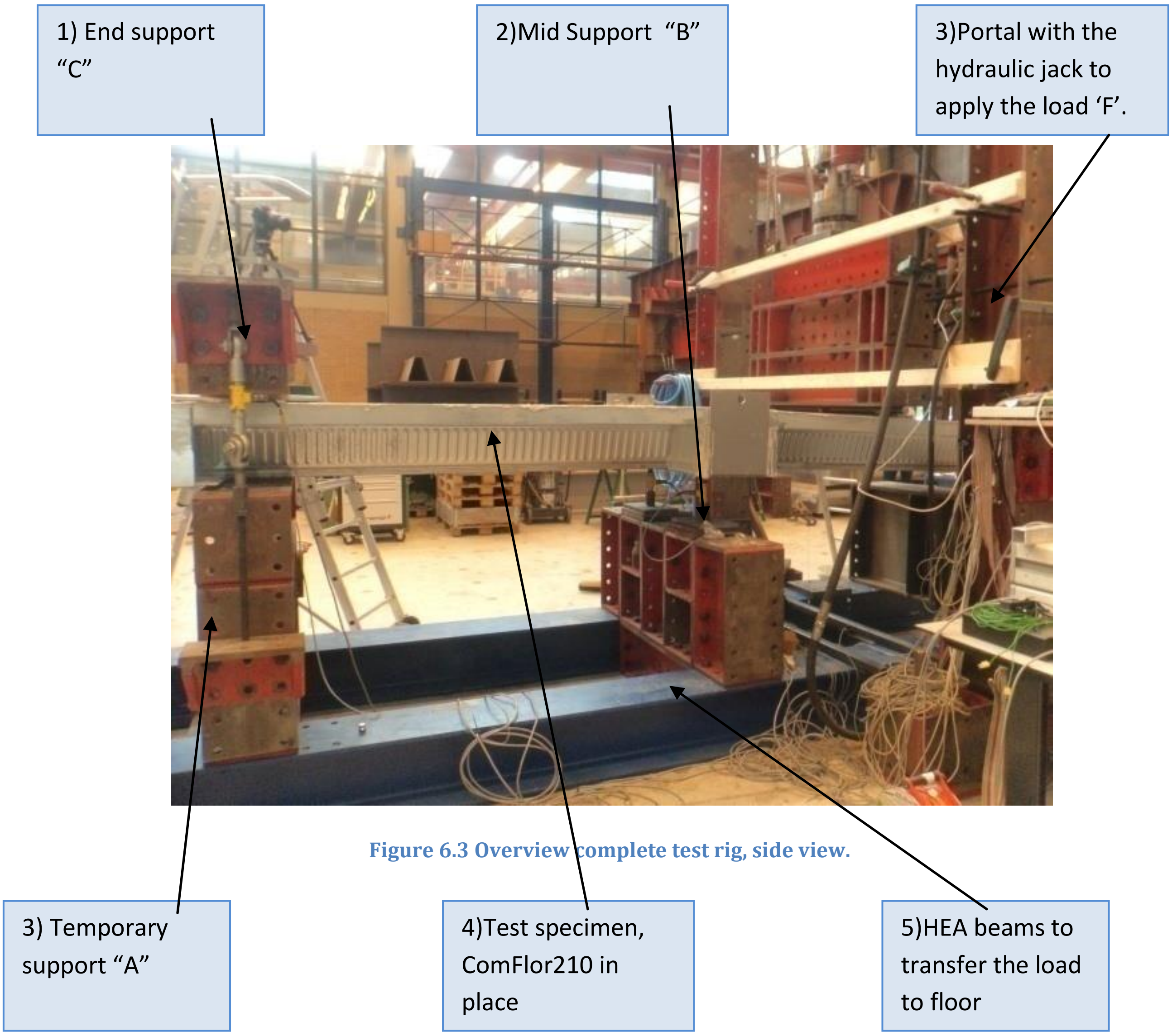


Figure 6.3 Overview complete test rig, side view.

In case of the ComFlor75, test specimens 4 & 5, the distances between the supports and the applied load are smaller. This is shown at the execution of the ComFlor75 experiments. For more detailed information see APPENDIX G.

6.1.2 Overview end support “A” & “C”

A side view of the end support (temporary support “A” and top support “C”) is shown in Figure 6.4. More detailed information on each individual part can be found in APPENDIX G.

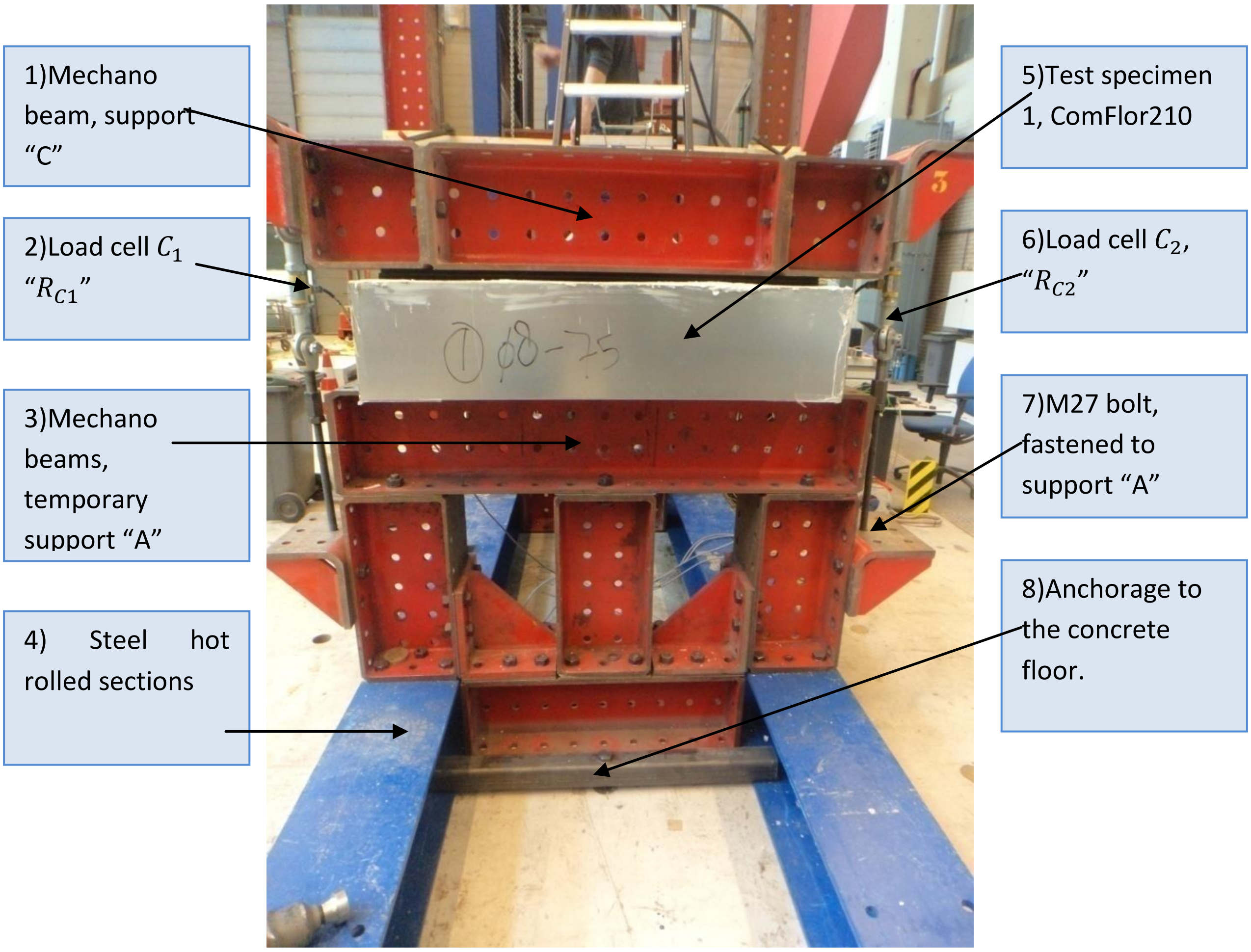


Figure 6.4 Temporary support “A” below the test specimen and the mechano beam on top forming the vertical support “C” during the experiments.

1. The mechano beam prevents vertical movement upwards, forming support “C”, rubber strip of 100 mm is placed in between the beam. Weight of the beam: 2,71 kN.
2. Load cell C1is a measuring device to measure the support reaction (R_{C1}) and prevent vertical displacement.
3. The mechano parts form a temporary support “A”, it supports the specimen before loading. Once loading started, it will lose contact with the specimen.
4. The steel hot rolled sections transfer the force to the 1200 mm thick concrete floor.
5. The test specimen (ComFlor210 in picture)
6. Load cellC2 is a measuring device to measure the support reaction (R_{C2}) and prevent vertical displacement.
7. The load cells are connected to the temporary support “A” by means of a M27 bolt.
8. The anchorage to the concrete slab is realized by a thick bolt fastened at the bottom of the concrete floor.

6.1.3 Overview of the portal applying the distributed line load.

The portal to hold the hydraulic cylinder and to apply the point load on the triple steel beams to distribute the load to an equally distributed line load on the specimen is shown below in Figure 6.5, more detailed information can be found in APPENDIX G.

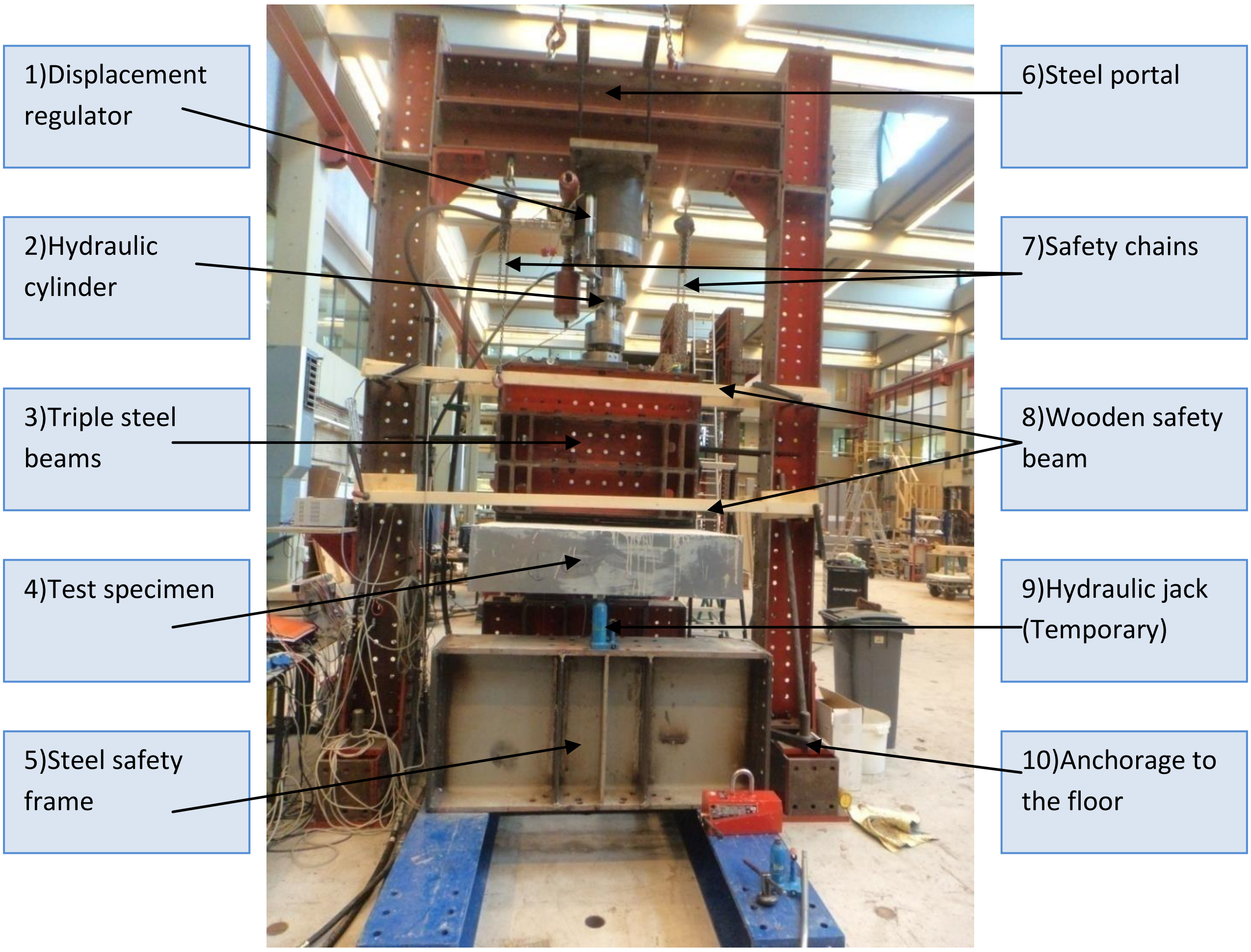


Figure 6.5 Steel frame including the steel beams to redistributed the applied force load to an equally distributed line load / displacement on the specimen.

1. The displacement regulator regulates the displacement of the hydraulic cylinder.
2. The hydraulic cylinder applies a point load on the triple steel beams.
3. The triple steel beams redistribute the point load to a line load
4. The test specimen (ComFlor210 in picture)
5. The steel safety frame avoids a sudden fall in case of brittle failure.
6. The steel portal supports the hydraulic cylinder
7. The safety chains can catch the triple steel beams in case of unwanted failure modes.
8. The wooden safety beams will prevent the triple steel beam from falling sideward.
9. The hydraulic jack temporary supports the test specimen before testing.
10. The steel portal is anchored to the floor to transfer the induced tensile force inside the portal from the hydraulic cylinder.

6.2 Instrumentation

Overview drawings are shown in this chapter and more detailed drawing and information can be found in chapter G.2 in the appendix. Below an overview of the devices used:

2x Load cells (Tension)

5x Load cells (Compression)

4x Thread LVDT's (Deflection)

1x Hydraulic cylinder (Force Applied)

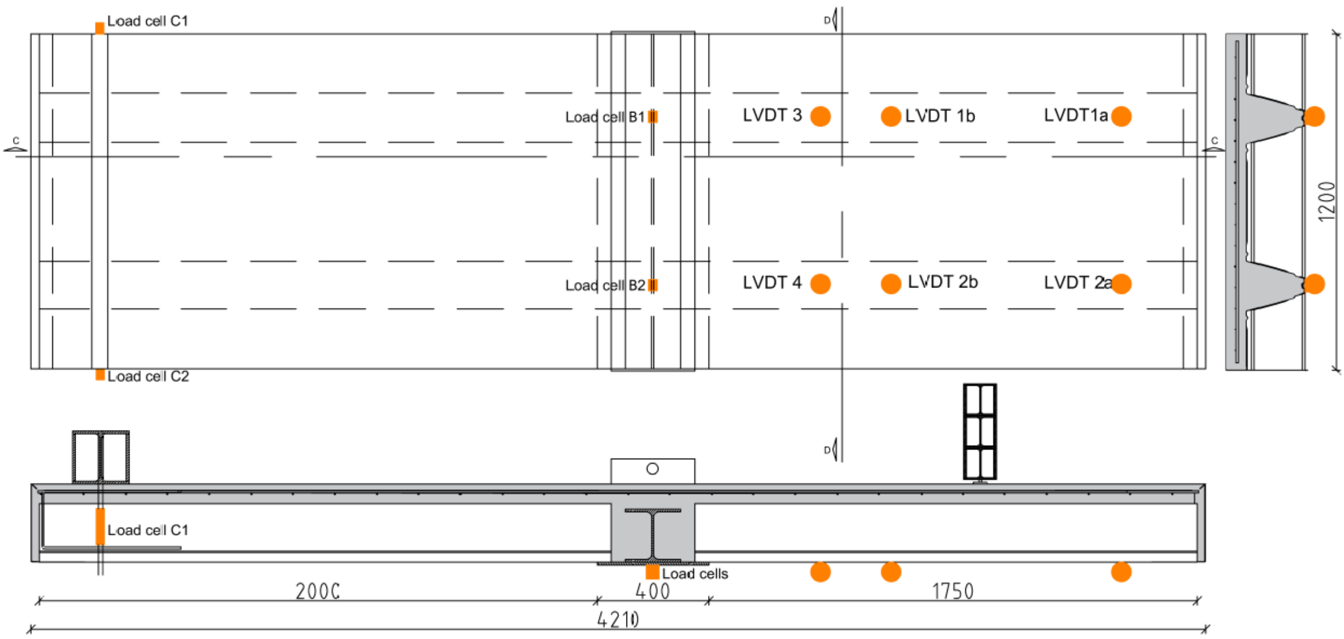


Figure 6.6 Overview of the instrumentation used for the ComFlor210 specimens. LVDT's tagged with 'a' correspond with test specimen 1 and tagged with 'b' correspond with test specimens 2 & 3.

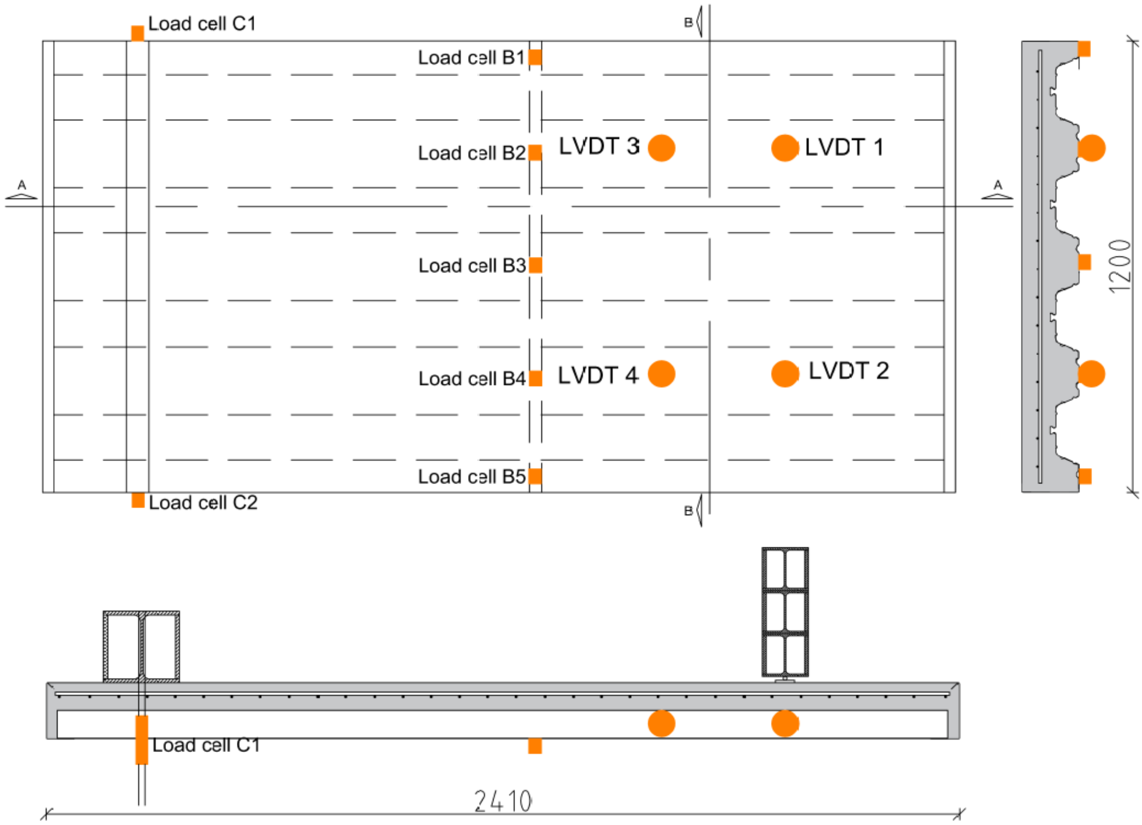


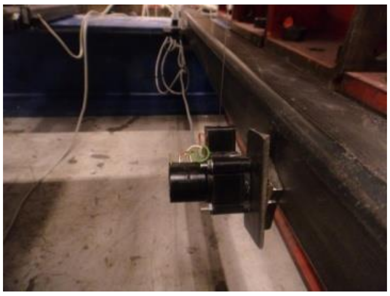
Figure 6.7 Overview of the instrumentation used for the ComFlor75 specimens 4&5.

6.3 Results of the experiments.

All 5 tests have a similar procedure. The hydraulic jack will push downwards, until the slab fails due to the applied hogging bending moment, vertical shear or a combination of both. In order to get correct values, all support reactions will be measured to calculate the self weight and load applied due to the test rig, all supports will be set to zero and at equal height to prevent unequal loading at the location of the supports and finally the test specimen will be loaded until it is lifted up against the top support. From this point the specimen is completely resting on the measured supports and the test can begin.

Assuring exact positioning of the applied load

a) *Lifting the specimen at the beginning of each test to assure free rotation*



b) *Exact positioning of the Thread LVDT's to equalize the 4 measurements between different devices.*

Figure 6.8 Some examples of precautions before testing.

In this chapter an overview is given of the data gained from execution of the 5 tests. After knowing all values, tables and graphs this data can be discussed in chapter 7 Analyzing of the Results.

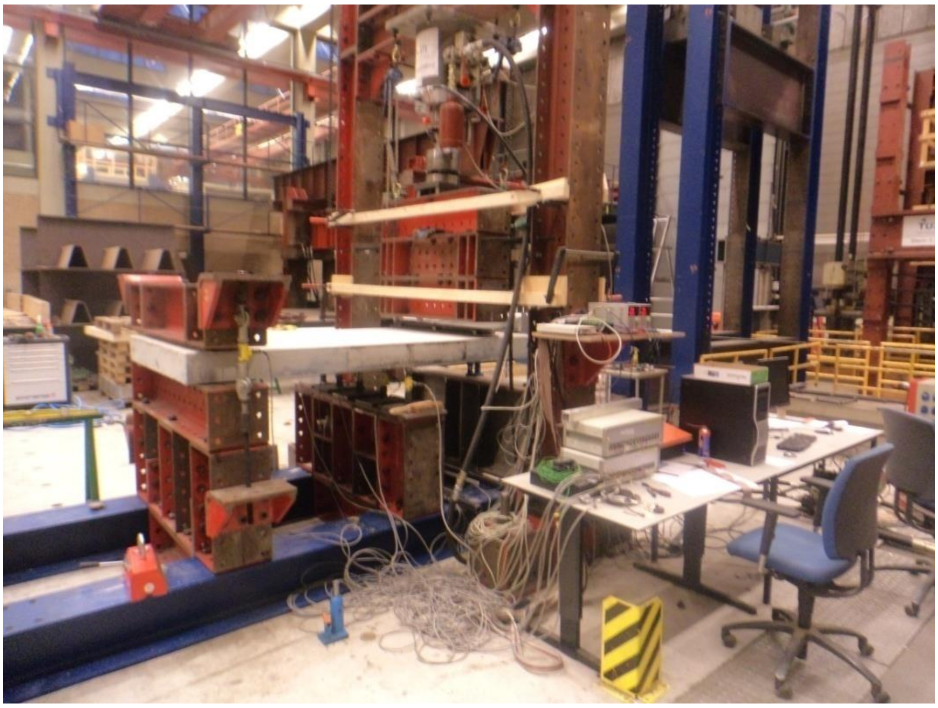


Figure 6.9 Overview test setup for one of the ComFlor75 specimens

6.3.1 ComFlor210, test specimen 1-3

In this chapter an overview is given of the data collected during the experiments for all ComFlor210 specimens combined. For individual data and review of measurement see APPENDIX H.

Table 12 Overview measured reaction force, applied forces and occurring hogging bending moment in B.

0	1	2	3	4	5	6	7	8	9	10	11	12
	R_C[kN]	R_B[kN]	F [kN]	F (Predicted) [kN]	ΔR =-R_B-R_C-F [kN]	q (self weight start of the test) [kN]	L2 [m]	M_B (dead load) [kNm]	M_B (applied) [kNm]	M_B [kNm]	M_{Rd} -- (Resistance) [kNm]	ΔM (Difference between calculated and measured value) [kNm]
1	55.11	-155.97	66.21	56.9	34.65	26.8	1.62	15.12	107.26	122.38	107.3	15.08
2	55.77	-199.36	122.91	104.4	20.68	26.2	0.915	11.73	112.46	124.19	107.3	16.89
3	69.14	-251.16	149.2	133.8	32.82	26.01	0.915	11.73	136.52	148.25	134.2	14.05

In Table 12 the measured values at failure are shown. The force applied and the growing hogging bending moment $M_B$are shown on the next page in Figure 6.10&Figure 6.11.

Comparing specimen 1 & 2 show that an equal $M_B$at the moment of failure is found. (Column 10)

The shear force is nearly twice as high (Column 3 + self weight). The Moment – Displacement graph (Figure 6.11) actually shows a similar path for both specimens. Independent of the shear force applied.

For all 3 specimens a similar difference is found between the calculated resistance and the final hogging bending moment applied at the moment of failure (Column 12) of around 15 kNm.

Vertical equilibrium seems to deviate a bit (Column 5-6), while $R_C$remains the same. (Column 1)

Specimen 1 started cracking at a lower moment applied (Figure 6.11), most likely due to a weak spot in the concrete.

All these aspects are further addressed in chapter 7 Analyzing of the Results.

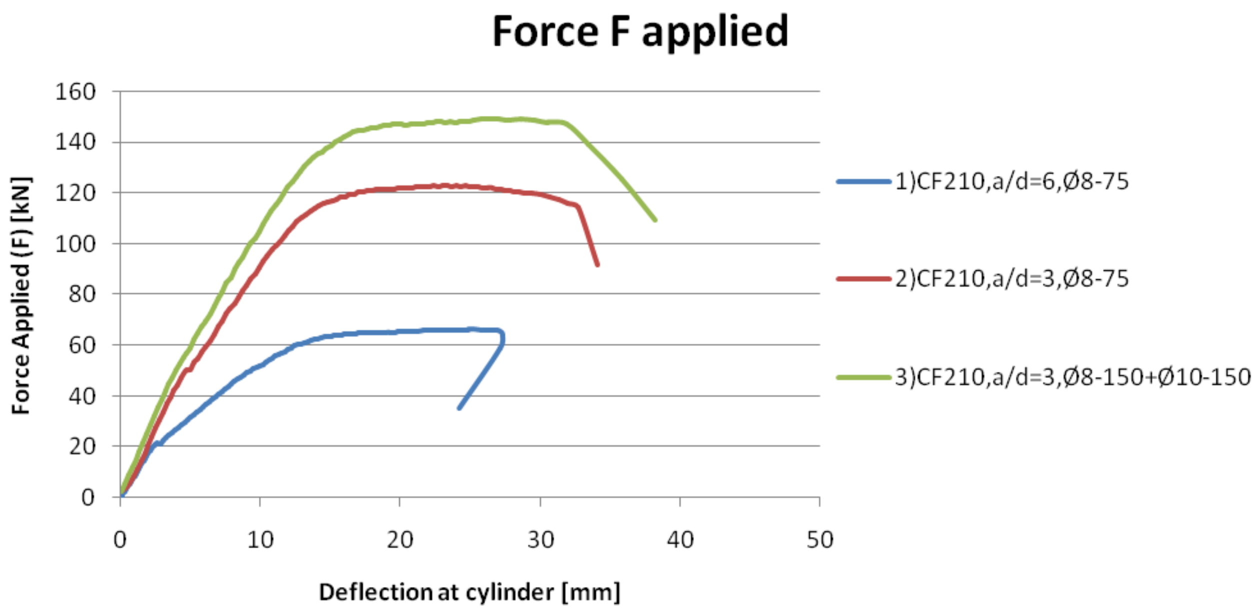


Figure 6.10 Force F applied by the hydraulic jack, with the corresponding deflection below the cylinder.

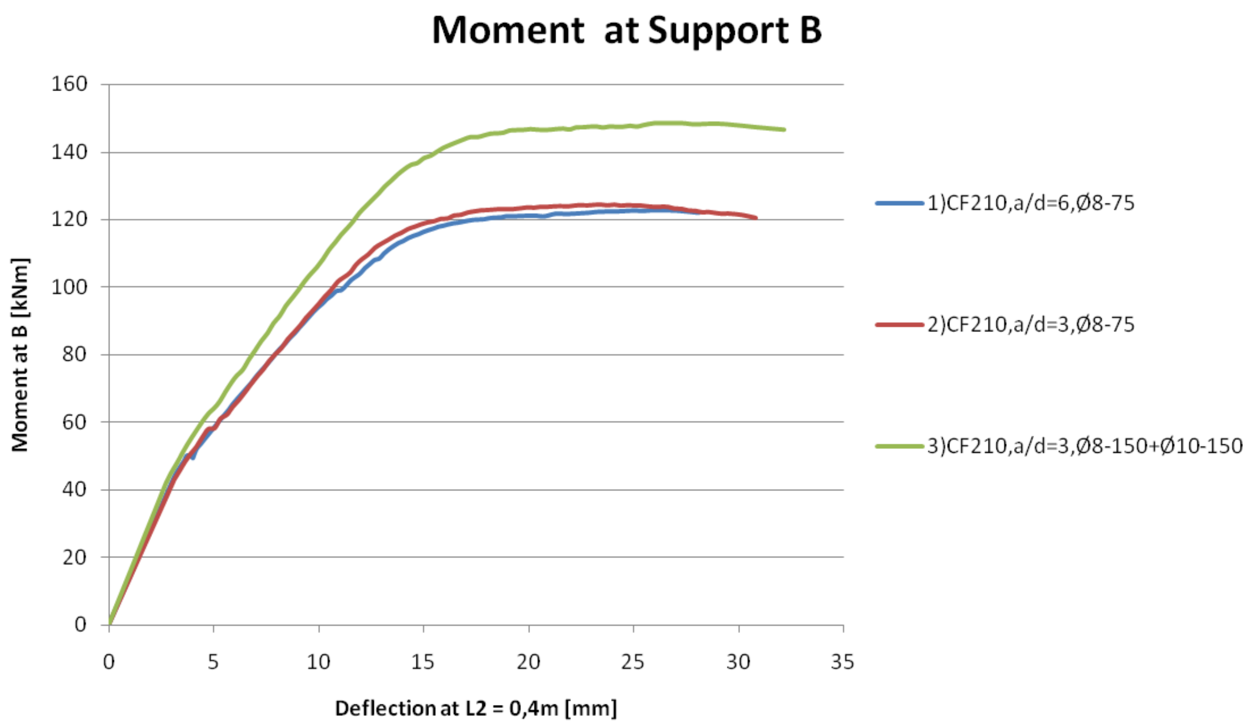


Figure 6.11 Increase in moment MB for specimen 1-3 versus the deflection measured at a distance of 0,4 mm from the steel plate.

Table 13 Approximated values for the cracking moment, yielding of the reinforcement and ultimate moment.

	M_{cr} [kNm]	M_y [kNm]	M_U [kNm]
Test Specimen 1	50	100	122
Test Specimen 2	57	100	124
Test Specimen 3	60	130	148

6.3.2 ComFlor75, test specimen 4-5

In this chapter an overview is given of the data collected during the experiments for all ComFlor210 specimens combined. For individual data and review of measurement see APPENDIX H.

Table 14 Overview measured reaction force, applied forces and occurring hogging bending moment in B.

0	1	2	3	4	5	6	7	8	9	10	11	12
	R_C[kN]	R_B[kN]	F [kN]	F (Predicted) [kN]	$\Delta R = -R_B - R_C - F$ [kN]	q (self weight start of the test) [kN]	L2 [m]	M_B (dead load) [kNm]	M_B (applied) [kNm]	M_B [kNm]	M_{Rd} -- (Resistance) [kNm]	ΔM (Difference between calculated and measured value) [kNm]
4	47.36	-133.65	77.63	84.8	8.66	16.1	0.66	5.43	51.24	56.67	61.51	-4.84
5	52.93	-256.69	181.72	174.75	22.04	18.1	0.33	3.84	59.97	63.81	61.51	2.3

In Table 14 the measured values at failure are shown. The force applied and the growing hogging bending moment $M_B$are shown on the next page in Figure 6.12&Figure 6.13.

Specimen 4 did not reach the calculated hogging bending moment resistance. (Column 11) This may be due to the punching shear failure below the ribs. (H.4 in the appendix)

Specimen 5 reached the expected failure load (Column 4-5) and the moment applied was equal to the resistance. (Column 10-11)

The shear force is nearly twice as high (Column 3 + self weight). The Moment – Displacement graph (Figure 6.13) actually shows a similar path for both specimens. Independent of the shear force applied. Only specimen 4 reaches its maximum resistance (most likely) due to the punching shear leading to a reduction in internal lever arm.

Vertical equilibrium seems to deviate (Column 4-5)

All these aspects are further addressed in chapter 7 Results.

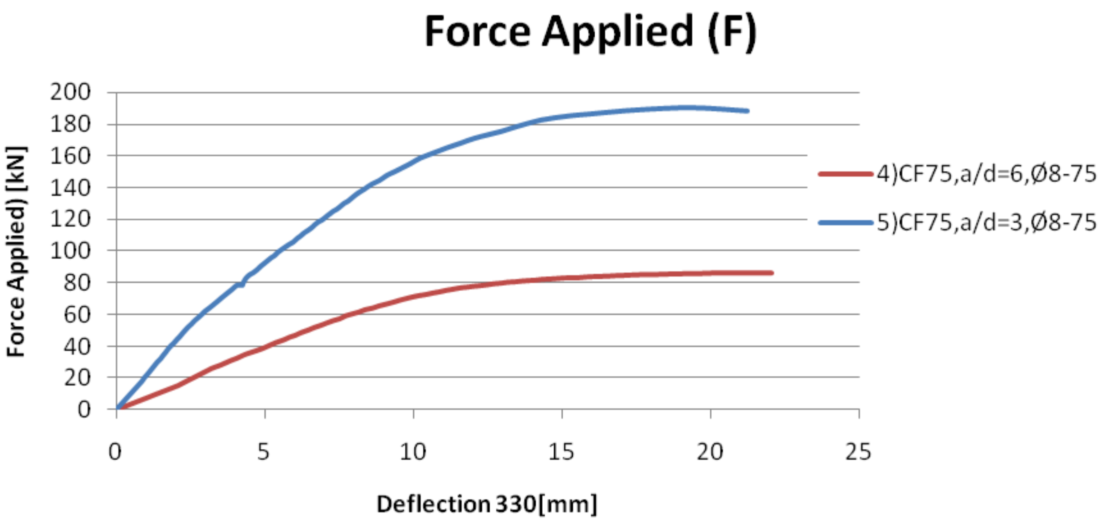


Figure 6.12 Force F applied by the hydraulic jack, with the corresponding deflection below the cylinder.

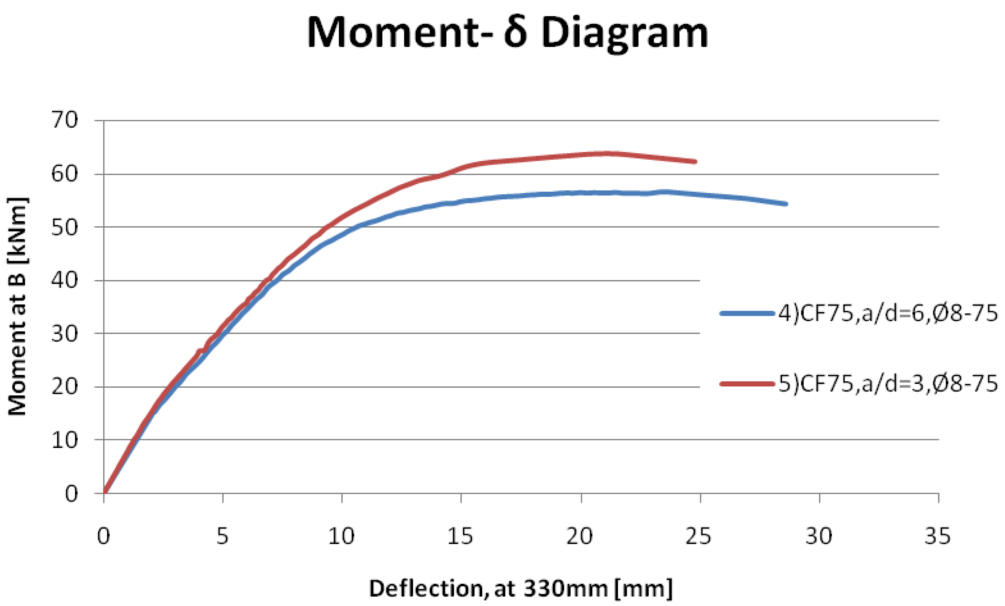


Figure 6.13 Increase in moment MB for specimen 4 & 5 versus the deflection measured at a distance of 0.33 mm from support B.

Table 15 Approximated values for the cracking moment, yielding of the reinforcement and ultimate moment.

	M_{cr} [kNm]	M_y [kNm]	M_U [kNm]
Test Specimen 4	23	42	56
Test Specimen 5	26	50	63

7 ANALYZING OF THE RESULTS

In this chapter the data of the previous chapters is reviewed. Does the outcome of the experiments reflect the predictions made beforehand and if there is a difference: how can this difference be explained and how do the test results answer the main objective: "To get insight in the influence of the vertical shear force on the hogging bending moment resistance."

7.1 Hogging bending moment resistance

In this chapter the specimen will be divided into two groups, the deep decks (ComFlor210) and the shallow decks (ComFlor75).

7.1.1 Hogging bending moment resistance ComFlor75

Specimens 4-5 are discussed first, as the supporting detail is less complex and the calculated values should be closer to the failure moment. For these tests the ComFlor75 sheet was already taken into account in the calculations done in chapter5.2.2 with the full calculation in chapter E.2.3 in the appendix. This iterative calculation gave a M_{pl}equal to 61,51 kNm.

Table 16 Overview results 4-5, Resistances compared to the applied hogging bending moment / vertical shear

	$M_{Rd}{}^{-}$[kNm]	M^{-} present during test[kNm]	$M/M_{Rd}{}^{-}$ *100%[%]	Difference $M_{Rd}{}^{-}$ and M^{-} [kNm]	V_U [kN]	V present during test [kN]	V/V_u *100%[%]
Test 4	61.51	56.6	**91.46%**	**-4.84**	125	86.42	**69%**
Test 5	61.51	63.7	**103%**	**2.3**	125	190.51	**152%**

Test specimen 4 failed at 56.6 kNm (at an applied force F of 77.63 kN), this is at 91% of the calculated resistance, taking the ComFlor75 sheet into account. At this moment a shear force was present of only 86 kN. (Approximated shear resistance equals 125 kN)
Test specimen 5 failed at 63,7 kNm (at an applied force F of 181,72 kN), this is around the calculated value of 61,51 kNm. It therefore reached the hogging bending moment resistance, while 190 kN shear force was acting on the specimen.

Possible explanations for the results: with the main explanations for test 4 under a)

a) In case of specimen 4, punching shear failure was found below some ribs (buckling inwards of the ComFlor75 sheet), reducing the internal lever arm and directly reducing the hogging bending moment resistance of the specimen and explains the gap between test 4 and 5.

Other minor possible deviations could be due to the following influences:

b) In chapter E.2.3 in the appendix the ComFlor75 steel sheet has been simplified in dimensions.
c) No strain gages have been applied, the calculation is based on full plastic behavior of the simplified ComFlor75 sheet, actual strain in the specimen could differ from the values used.
d) Tensile strength of the concrete has been neglected.
e) General spread in material properties (confined concrete compressive strength could be higher for example)
f) The cover is based on the height of the reinforcement before pouring the concrete (based on the concrete bricks maintaining the distance between the sheet and the reinforcement), exact cover height might have changed.
g) Possible imperfections of the test rig, dimensions and measure devices.

In Figure 7.1 the increase in moment is shown of both specimen 4 and 5.

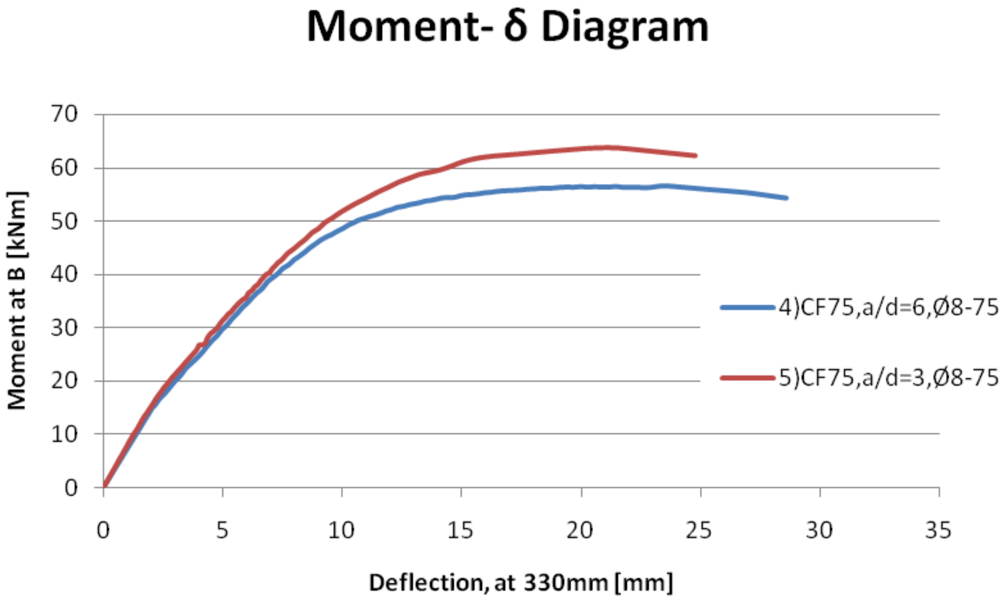


Figure 7.1 Overview of the increase in moment of test specimen 4 & 5 on the y axis and the deflection measured at 330 mm away from the support on the x axis.

The linear elastic part remains the same for both specimen 4 as well as specimen 5, however specimen 4 starts yielding before test specimen 5, resulting in a lower failure load. Specimen 5 reaches the predicted moment at B, specimen 4 fails at 91% of the predicted value.

Figure 7.2 Punching shear above each load cell (inward buckling of the ComFlor75 sheet)

The high concentrated load of the load cells resulted in crushing of the concrete. The effective depth of the reinforcement [d] is equal to 109 mm. Due to crushing of the bottom concrete this could have been reduced causing a reduction of the failure load.

7.1.2 Hogging bending moment resistance ComFlor210

All three test specimens failed in bending, even those with a shear force present of at least 100% of the calculated vertical shear resistance. The failure moment was also slightly higher than the calculated resistance.

Table 17 Overview results 1-3; Resistances compared to the applied hogging bending moment/vertical shear.

	M_{Rd}^{-} above the support[kNm]	M^{-} present during test[kNm]	Difference M_{Rd}^{-} and M^{-}[kNm]	V_U [kN]	V present during test [kN]	V/V_u *100%[%]
Test 1	107.3	122.38	**15.08**	111.9	79.96	**72%**
Test 2	107.3	124.19	**16.89**	111.9	136.66	**122%**
Test 3	136.52	148.25	**14.05**	120.23	162.95	**136%**

All specimens failed in negative bending; even though test specimens 2 & 3 had a vertical shear force exceeded its vertical shear force resistance by 22-36%.

It could be that the actual vertical shear resistance is higher compared to the calculated resistance, based on the EN 1994-1-1 [5], which only takes the rib into account as a beam without shear reinforcement. Besides the resistance is based on an empirical formula that is based on lots of data points with a big spread. (The ComFlor210 is also not in scope of EN1994) This is further discussed in chapter 7.2.2.

In Figure 7.3 an overview is given of the increase in moment of all three specimens and their corresponding deflection 400 mm away from the steel plate and in Figure 7.4on the next page an overview is given of this occurring moment related to the calculated values.

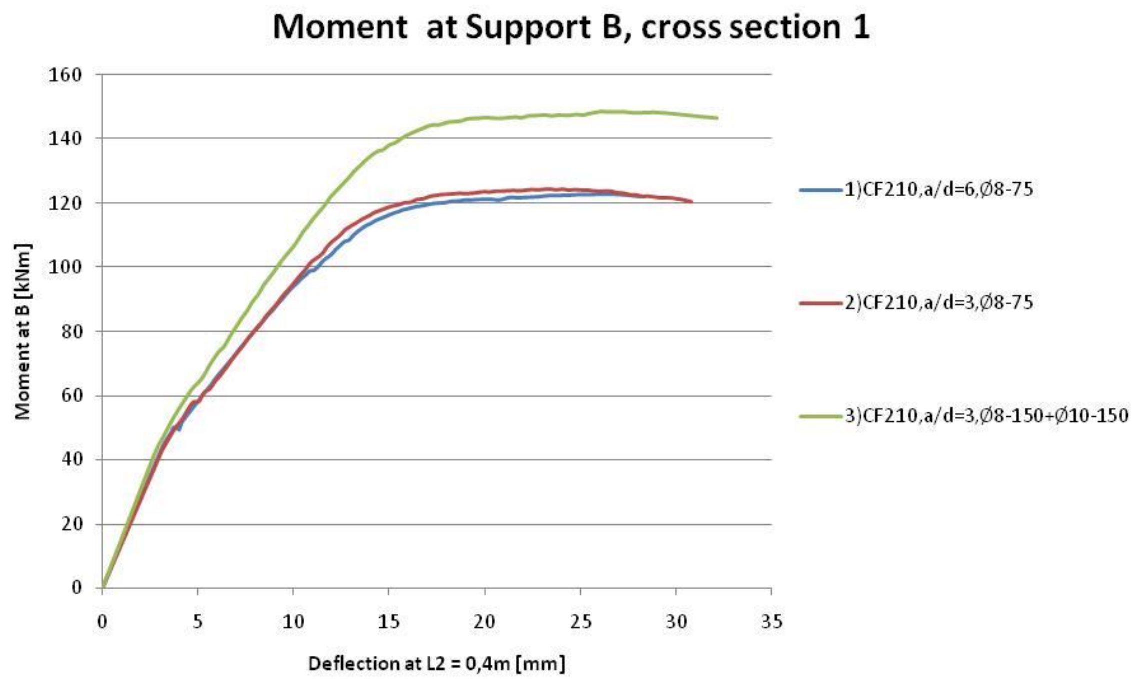


Figure 7.3 Overview of the increase in moment of test specimen 1-3 on the y-axis and the deflection measured at 400mm away from the support on the x axis. Moment based on a lever arm equal to L_2.

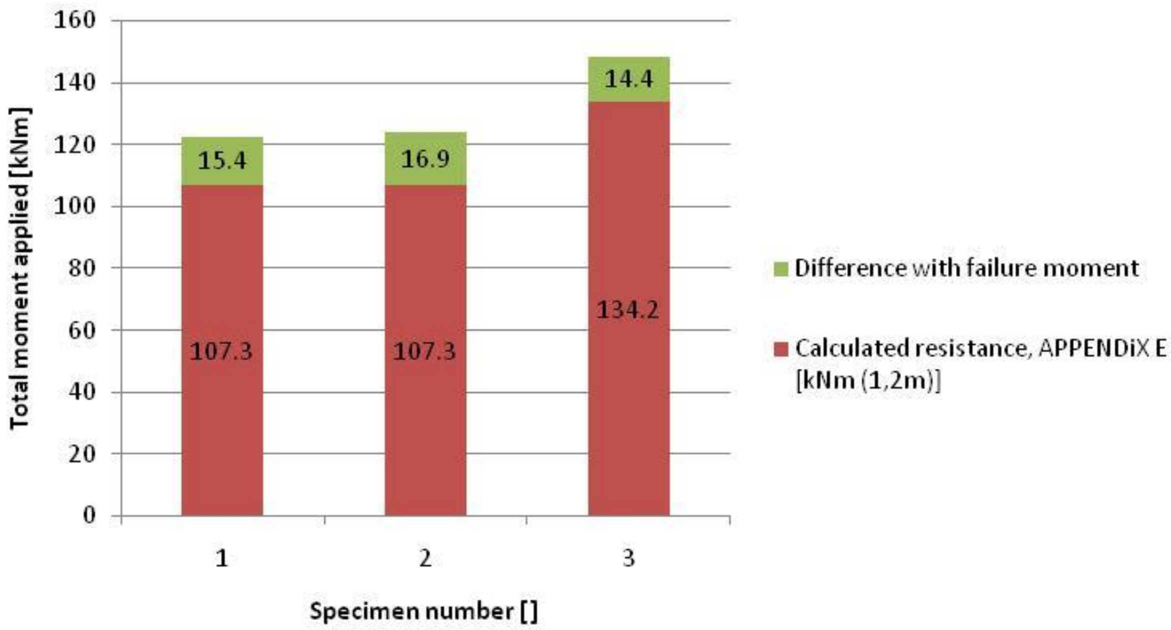


Figure 7.4 Difference between total applied moment and the calculated resistance.

The graphs of the hogging bending moment at B for specimens 1 & 2 are nearly identical (see Figure 7.3), even though nearly twice the vertical shear is present (see Table 17). This means no reduction in negative failure moment is found under the circumstances used for these experiments (reinforcement, test rig, dimensions for example) and the vertical shear resistance calculated.

The difference of 14,4 kNm , 15,4 kNm and 16,9 kNm seems to be a steady difference between the calculated resistance and the actual failure moment. To check whether this could be a spread in the results a short calculation is done to check if the calculated resistance could be the mean value as assumed, with the hypothesizes as following:

$$H_0 = \mu \leq 107.3 \text{ } (or \text{ } 134.2) \text{ } kNm$$

$$H_1 = \mu > 107.3 \text{ } (or \text{ } 134.2) \text{ } kNm$$

n=3 $\bar{x} = 122.87$ s=1,25 $\alpha = 0{,}01$ t = 6.96 (for 1- α)

$$t = \frac{\bar{x} - \mu}{\frac{s}{\sqrt{n}}}$$

Using these values the calculated t value equals 17,53 and the critical limit with a 99% certainty equals t= 6,96. So $H_0 = \mu \leq 107.3 \text{ } (or \text{ } 134.2) \text{ } kNm$ is rejected, H_1 is approved. The gap must therefore be explained by a contribution that has not been included in the calculation or an error in determining the failure moment that occurred.

Possible reasons to explain the gap found are:

a) The ComFlor210 sheet, assumed to be simply supported, somehow contributes or transfers some force to the steel plate welded below the HE200A beam.
b) The steel plate below the HE200A acts as a 400mm wide support instead of the assumed support at the centre of the beam.
c) Concrete compressive strength (cubes are not confined, while the concrete in the test specimen is confined by the steel HE200A and the other steel plates.
d) General spread in resistances of each specimen.
e) Measured cover deviates after pouring of the concrete.
f) Tensile strength of the concrete has been neglected.
g) Error in the test rig.
h) Contribution of small elements that have not been included (joist shuttering, stripes, end profiles)
i) Contribution of the HE200A integrated beam, the corresponding steel plate welded below and the steel plate with lifting eyes at the end face.
j) Contribution of the reinforcement bar in the ribs.

Most of the points mentioned can only influence the hogging bending moment resistance a little bit. In case of point a) the force (F_2 in Figure 7.5) must be transferred to the other side. The concrete between the ComFlor210 sheet and the HE200A beam is cracked and is not able to transfer this force. The ComFlor210 can therefore not contribute to the hogging bending moment resistance directly.

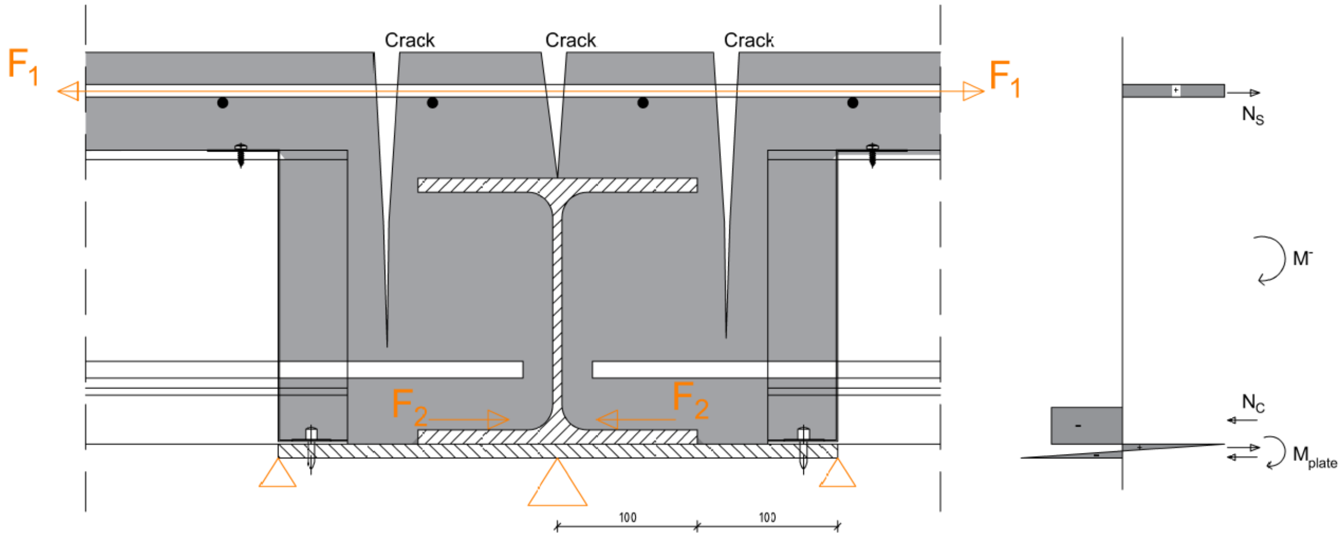


Figure 7.5 Forces at work at support B, intergraded support detail ComFlor210 specimens.

It can however transfer a force to the steel plate (400 x 10 x 1200 mm). This means point i) & b) could actually contribute to the resistance. The top of the steel plate is flat and has a low friction coefficient. It has lost its attachment with the concrete and therefore its composite behavior, separating its stress strain diagram from the concrete / reinforcement, allowing it to transfer a moment. No high strain were observed in the steel plate, meaning the elastic moment resistance has not yet or just been reached. This means a part of the applied force is transferred to the support through the steel plate, reducing the actual occurring moment right at the centre.

The elastic moment capacity of the steel plate is equal to:

$$M_{el} = \frac{1}{6} * b * h^2 * f_y$$

The steel grade is between S235 and S355. Assuming normal construction steel with S235 was used this means the actual strength is normally 20-25% higher. Around 300 N/mm^2.

$$M_{el} = \frac{1}{6} * 1200 * 10^2 * 300 = 6 \text{ kNm (in case of } f_y = 300 \text{ N}/mm^2)$$

In case of 50% M_{el} would be reached, the force F equals: $\frac{M_{el}}{lever\ arm} = \frac{3}{0.2} = 15\ kN$.

The force going through the steel plate is in relation to the vertical shear force. As no strain has been measured in the steel plate, it cannot have been close or past the elastic moment resistance of the plate. However assuming 15 kN going through the steel plate, would reduce the hogging bending moment calculated by 15*0.915=13.7 kNm in case of test specimen 2-3, which is of the same magnitude of the difference found.

The contribution of the steel plate would reduce the hogging bending moment above the support as shown below:

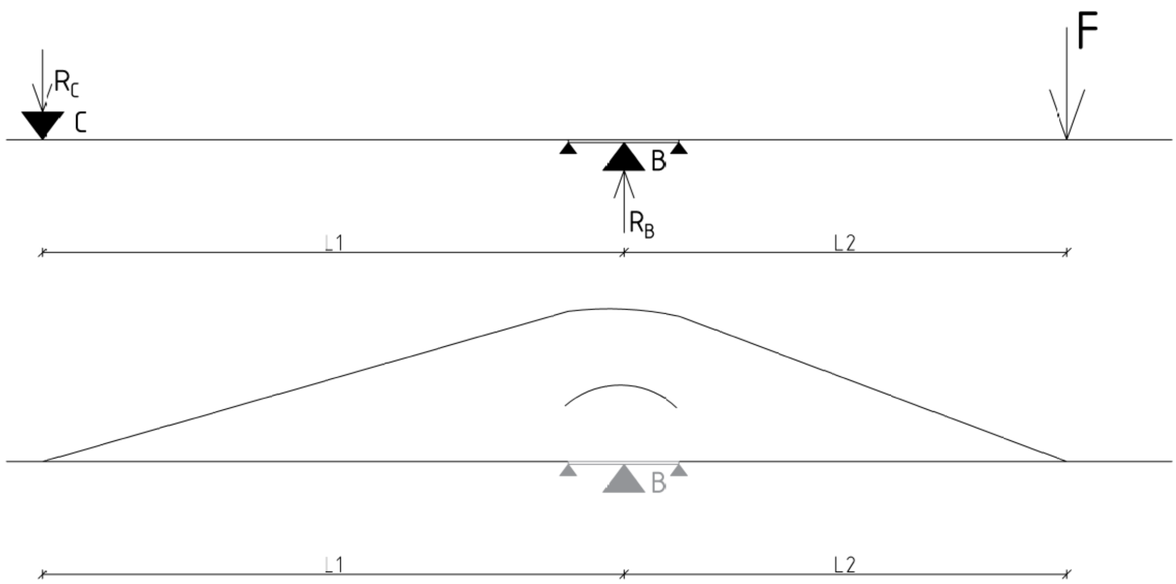


Figure 7.6 M-Line ComFlor210 specimens, with a reduction in hogging bending moment due to contribution of the steel plate.

7.2 Vertical Shear Force Resistance

Even though the tests were successful and no influence of the vertical shear force on the hogging bending moment resistance can be found, the question remains why such high shear forces could occur without causing a shear failure.

7.2.1 Vertical shear force resistance ComFlor75 specimens

For the ComFlor75 tests only one test was conducted with a high vertical shear present. Below an overview of the vertical shear forces occurring.

Table 18 Overview shear resistance and occurring shear forces.

Test Specimen	V_U per slab [kN]	V at failure [kN]	V/V_U
4 ComFlor75	125	86.42	**69%**
5 ComFlor75	125	190.51	**152%**

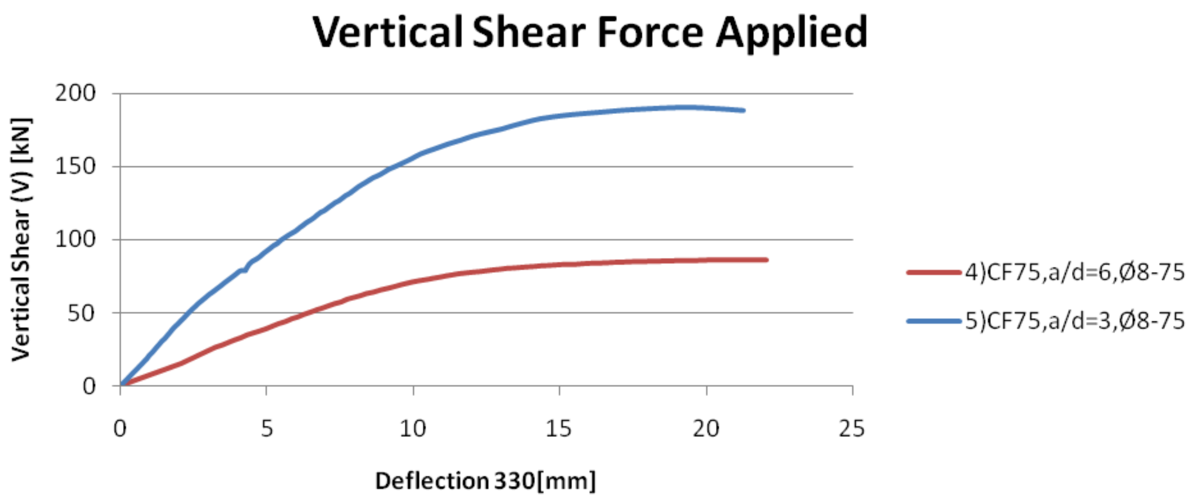


Figure 7.7 Overview test results 4 & 5; increase in vertical shear during the experiments compared using measurements of deflection at the same location.

Specimen 5 had to withstand 152% of the calculated vertical shear force and still did not fail on shear. The vertical shear force is based on the empirical formula provided in the EN 1994. [5]. This is based on a concrete rib without shear reinforcement.

Besides the resistance of the concrete rib, the resistance of the ComFlor75 sheet, based on experiments, has been added to approach the actual vertical shear resistance. Still the resistance seems to be higher.

Table 19 Maximum values load cells during 5th test.

Maximum values load cells [kN]						
Load cell C1	Load cell C2	Load cell B1	Load cell B2	Load cell B3	Load cell B4	Load cell B5
27.6	25.4	-23.4	-69.9	-79.6	-66.1	-26.8

The 152% is based on the shear resistance of the specimen with a width of 1200 mm. The reaction force below each rib has been calculated and load cell B3 had maximum of 79,6 kN. (Table 19) Meaning this rib had to withstand a higher vertical shear force still.

The vertical shear force in concrete is based on the friction between the sliding planes. These planes are hold together by the stirrups in reinforced concrete beams.

An axial force in compression increases the vertical shear resistance, see $k_1\sigma_{cp}$ in the formula below:

$$V_{Rd,c} = [C_{Rd,c}k(100\,\rho_1 f_{ck})^{\frac{1}{3}} + k_1\sigma_{cp}]\, b_w d.$$

A hypothesis could be that the confinement of the concrete due to the ComFlor sheet, keeps the cracked concrete together in longitudinal and horizontal direction, to increase the friction between possible sliding planes. This could mean the vertical shear resistance is now conservative as it is calculated based on a beam without shear reinforcement and to approach the actual resistance the slab should be considered to be a beam with shear reinforcement. This however is only a possible way of explaining the gap between the calculated value and the actual vertical shear resistance.

Other possible reasons for the difference in vertical shear resistance are:

a) Concrete strength is now based on concrete cubes of 150 x 150 mm, not confined, while the concrete in the ribs / slab is confined and could result in a higher resistance.
b) The $C_{Rd,c}$ factor used in the EN 1992-1-1 [10] equals $\frac{0.18}{\gamma_c}$ as discussed in chapter A.1.1, this factor lies in reality between 0,12-0,16 [14]. 0,15 was used. A higher value could contribute to the vertical shear resistance of the concrete parts.
c) The load was placed at a distance of an a/d ratio of 3. Some direct transfer towards the support could be possible.
d) The vertical shear resistance is now based on the summation of different parts as no calculation method was available to fill the gap between standard at force and experimental results.
e) General spread in material properties.
f) Possible imperfections in the test rig, dimensions and measure devices.

7.2.2 Vertical shear force resistance ComFlor210 specimens.

The values used to calculate the shear resistances resulted in values that are far above the values used in the EN 1994-1-1 [5], after removing all safety factors. Even with the calculation methods from Dutch Engineering based on laboratory tests, the specimens did not fail in shear. Below a small overview is given in Table 20 and Figure 7.8 regarding the vertical shear force on the ComFlor210 specimens 1-3.

Table 20 Overview shear resistance and occurring shear forces

Test Specimen	V_U per slab [kN]	V at failure [kN]	V/V_U
1 ComFlor210	111.9	79.96	**71%**
2 ComFlor210	111.9	136.66	**122%**
3 ComFlor210	120.23	162.95	**136%**

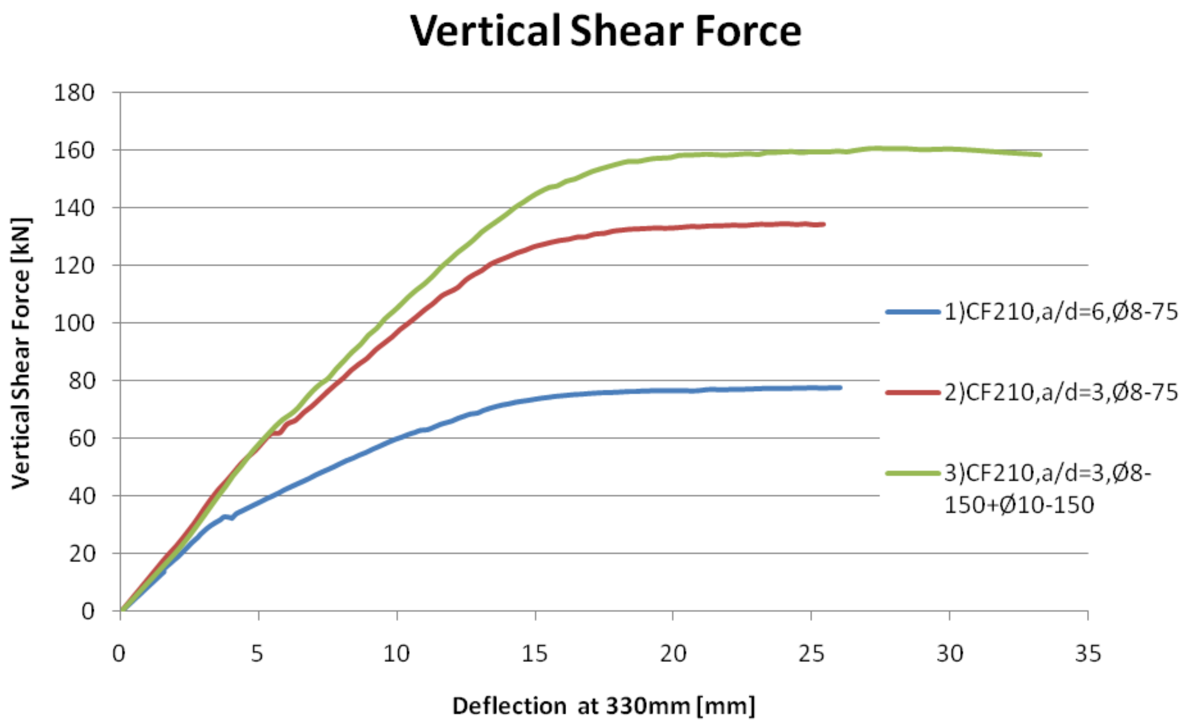


Figure 7.8 Overview test results 1-3; increase in vertical shear during the experiments, compared using measurements of deflection at the same location.

Interesting are the high values, why didn't the specimen fail once passing the 111 – 120 kN? During the second and third test the maximum shear force in the specimen was equal to 122% and 136% respectively.

In the beginning of the thesis, it was concluded that an extra test setup was required in case of exactly determining the vertical shear resistance. Instead a high value was chosen to approach the vertical shear resistance as an extra test rig would deviate too much from the main research question. All though the exact mechanism of vertical shear is not part of this thesis, here follow some possible reasons why the shear value could be higher. (some are identical to the ComFlor75 series)

a) Concrete strength is now based on concrete cubes of 150 x 150 mm, not confined, while the concrete in the ribs / slab is confined and could result in a higher resistance.
b) The $C_{Rd,c}$ factor used in the EN 1992-1-1 [10] equals $\frac{0.18}{\gamma_c}$ as discussed in chapter A.1.1, this factor lies in reality between 0,12-0,16 [14]. 0,15 was used. A higher value could contribute to the vertical shear resistance of the concrete parts.
c) The load was placed at a distance of an a/d ratio of 3. Some direct transfer towards the support could be possible.
d) The vertical shear resistance is now based on the summation of different parts as no calculation method was available to fill the gap between standard at force and experimental results.
e) General spread in material properties.
f) Possible imperfections in the test rig, dimensions and measure devices.
g) At the location of support B, contribution of the HE200A and steel plate has not been included. Locally they could contribute to the vertical shear resistance.

8 CONCLUSIONS& RECOMMENDATIONS

8.1 Conclusions ComFlor210 specimens.

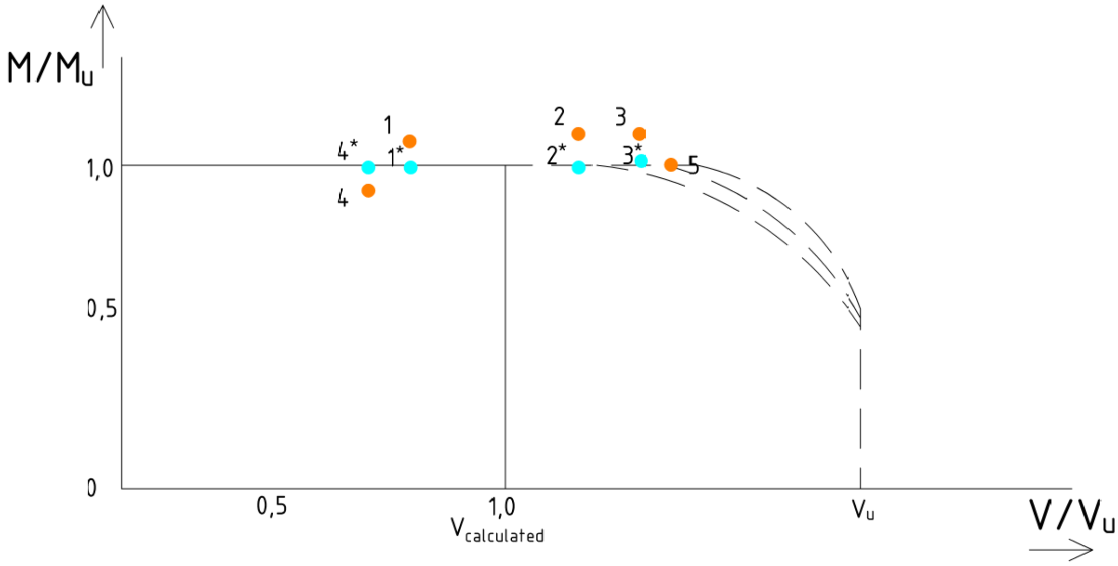


Figure 8.1 M-V interaction diagram, no reduction in moment resistance was found. It could be that $V_U \gg V$.

The calculated moment resistance of 107.3 kNm for the specimens 1)CF210,a/d=6,Ø8-75 and 2)CF210,a/d=3,Ø8-75 and 134.2 kNm for specimen 3)CF210,a/d=3,Ø8-150+Ø10-150 was surpassed an average of 15 kNm. As the steel plate acts like a 400 mm wide support, the calculated failure moment was inaccurate. The adjusted hogging bending moment right above support B is shown as 1*, 2* and 3*. All specimens CF210 failed on bending, even while the vertical shear force was above the calculated vertical shear resistance.

Hogging bending moment resistance:
The calculated failure moment was too high or was inaccurate due to the influences a) – i) as mentioned in chapter 7.1.2. Most of these points only have a small influence. The difference in moment found can be explained by the contribution of the steel plate that is welded below the integrated HE200A beam. By acting as a 400mm wide support, it reduced the failure moment calculated. This means all specimens failed around the maximum calculated resistance

Vertical shear resistance
The vertical shear force found was 136% of the calculated value, taking the ComFlor210 sheet into account. This means the actual vertical shear is higher than the calculated value. The deep decks are not covered by the standard at force EN 1994-1-1 [5], but the same principle is used as for the ComFlor75. As with the ComFlor75, the method of calculation is based on a rib without shear reinforcement. The sheet contributes and could act as shear reinforcement. More research is needed to get insight in the actual vertical shear resistance and the calculation method. Influences on the vertical shear resistance are mentioned in chapter 7.2.2 under a)–g)

M-V interaction for specimen:
No reduction in the calculated hogging bending moment has been found. Varying the vertical shear force, while keeping the hogging bending moment resistance the same (specimen 1 and 2), gave a nearly identical growth in moment. (See Figure 7.3) Taking the circumstances of the experiments into account, interaction plays no role. It is possible that the $V_U \gg V_{Rd}$, meaning the area of interaction has not been reached and that interaction plays no role in the area of application. (as $V_{Rd} \ll V_U$) Due to the conservative calculation method of the V_{Rd}, the area of interaction is not reached in practice.

8.2 Conclusions ComFlor75 specimens.

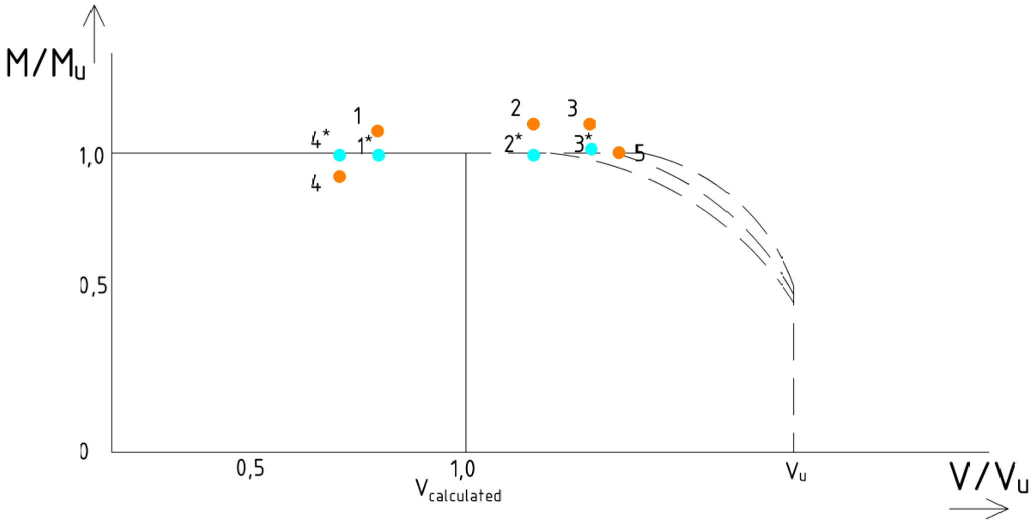


Figure 8.2 M-V interaction diagram, no reduction in moment resistance was found. It could be that $V_U \gg V$.

The calculated resistance of 61,51 kNm approaches the failure moment found during the test on specimen 5)CF75,a/d=3,Ø8-75. The punching shear found contributed to the early failure of specimen 4)CF75,a/d=6,Ø8-75. The 5 independent supports punched for about 1 cm through the ComFlor75 due to the high concentrated reaction forces. This resulted in a reduction of the internal lever arm of 1 cm, equal to around 10%. Taking this reduction into account, the failure moment from specimen 4 resulted in a similar failure moment as specimen 5. (4* in Figure 8.2)

Hogging bending moment resistance:
The calculated resistance can be compared with the failure moment found during the experiment, taking into account the influences a) – g) as mentioned in chapter 7.1.1.
Due to punching shear and exclusion of additional contributions, the difference in hogging bending moment resistance between test specimen 4 and 5 can be explained. Both test specimens failed in hogging bending moment without any influence of the vertical shear force present.

Vertical shear resistance
The vertical shear force found was 152% of the calculated value, taking the ComFlor75 sheet into account. Locally this was even higher (at load cell B3), meaning the actual vertical shear is higher than the calculated value. The method of calculation is based on a rib without shear reinforcement. The sheet could contribute and act as shear reinforcement. More research is needed to get insight in the actual vertical shear resistance and the method of calculation. Influences on the vertical shear resistance are mentioned in chapter 7.2.1 under a) – f).

M-V interaction for specimen:
No reduction in the calculated hogging bending moment has been found. Taking these circumstances into account, interaction plays no role. It is possible that the $V_U \gg V$, meaning the area of interaction has not been reached. See Figure 8.1. As $V_{Rd} < V_{calculated}$ due to the conservative approach in the EN 1994-1-1 [5], the area of interaction is not reached in practice.

8.3 Suggestions for further research.

The main objective of this thesis was:
To determine indicatively the influence of the hogging bending moment on the vertical shear resistance of composite steel-concrete floors made with ComFlor75 or ComFlor210 based on a limited series of test

With the results gained from these 5 experiments, it may be concluded that no influence on and no reduction in hogging bending moment resistance was to be found. However the experiment raised new questions that could be the base of further research.

- The vertical shear resistance of a ComFlor210 and ComFlor75 is not yet fully known, especially not when loaded by a negative bending moment. In order to know the exact vertical shear resistance of these slabs, specific test should be done with a test setup that can test on pure vertical shear, without direct transfer of the forces towards the support.

- The vertical shear resistance has now been calculated according to the standards at force, in combination with test results (of specimens loaded by vertical shear and positive bending) in order to include the contribution of the steel ComFlor sheets. Within the standards at force the ComFlor75 is calculated based on a concrete beam without shear reinforcement. However it might be the case that the steel ComFlor sheet acts as continuous longitudinal shear reinforcement and increases the shear resistance far above the current approach. Further research is needed to investigate the influence of the ComFlor sheets on the vertical shear resistance .

- The ComFlor210 (deep decks) is not yet covered by the standards at force, however results from the experiments show comparable moment resistance to the EN 1994-1-1 [5], (if neglecting the integrated detail), maybe the deep decks could at some point be covered by the current EN 1994-1-1 [5].

- The experiments show a uniform result, in all 5 tests no interaction was found and similar hogging bending moment resistances are found compared to the calculations. However in order to create a full interaction diagram with the exact $M_U \& V_U$ more investigation is needed to provide a larger amount of statistical data.

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APPENDIX A. Literature Review

A.1. Verification of the composite floor according to the EN 1994-1-1 [5].

A.1.1. Vertical shear force

Composite slabs must be verified according to the EN 1994-1-1 [5](design of composite steel and concrete structures).Within this code (as deep decks are not covered by the Eurocode 4) the vertical shear force is calculated based on the shear resistance of the concrete rib only. However as experiments in London [3]& Germany [11] show the ComFlor steel sheet contributes to the vertical shear resistance.
The vertical shear resistance follows from:

$$V_{Rd} = V_{Rd,rib} + V_{Rd,sheet}$$

Where:
$V_{Rd,rib} =$ Shear resistance of the concrete rib, calculated according art. 6.2.2 of [10], elements without shear reinforcement.
$V_{Rd,sheet} =$ Shear resistance of the ComFlor steel sheet, based on experiments Zulassung [11]. . *For the ComFlor210 this gives a Vu of 44,98 kN/m¹ and for the ComFlor75 this results in a* $V_U = 55\ kNm/m^1$

To calculate the vertical shear resistance both participating elements should be considered.

Shear resistance $V_{Rd,sheet}$ of the ComFlor210 steel sheet

The vertical shear resistance of the ComFlor210 sheet is based on test done in Germany [11]. ComFlor 210/1,00 mm in Z350 steel gives a Vu of 44,98 kN/m^1.
This value will be used further throughout the report to calculate the vertical resistance of the ComFlor210 specimens.

The vertical shear resistance of the ComFlor75 is also based on experiments. In London test have been done to determine the shear resistance of the ComFlor75 composite slabs. [3]. Tests have been done with and without ComFlor75 sheeting below. The difference can be used to approach the contribution of the steel sheet. This mean a $V_U = 55\ kNm/m^1$.

Shear resistance $V_{Rd,Rib}$ of the concrete rib

To determine the shear resistance of the rib, EN 1994-1-1 [5] is to be followed. At the chapter 9.7.5 Vertical Shear the following is written:

9.7.5 ***Vertical shear***
EN 1994-1-1 (1) The vertical shear resistance Vv,Rd of a composite slab over a width equal to the distance between centers of ribs, should be determined in accordance with EN 1992-1-1, 6.2.2.[5]

This means the vertical shear resistance of a composite slab is based on the already known concrete code. It refers to a chapter in EN 1992-1-1 [10] regarding members not requiring design shear reinforcement. This will therefore neglect the contribution of the steel sheet.
Within 6.2.2 from EN 1992-1-1 [10] a formula is given to calculate the design value for the shear resistance $V_{Rd,c}$, namely the same as in chapter A.3.1A.3.1:

$$V_{Rd,c} = [C_{Rd,c}k(100\ \rho_1 f_{ck})^{\frac{1}{3}}]\ bwd$$

With a minimum of:

$$V_{Rd,c} = (v_{min})\ bwd$$

Where ρlrefers to the percentage of the tensile reinforcement in the considered section. This section equals: $b_w * d$, where $b_w$is the smallest width under tension and d the internal lever arm of the centre of the reinforcement in tension to the bottom of the steel sheet.
This could imply an extra usage of the longitudinal tension reinforcement by shear and which, at the position of the intermediate support, is already loaded by the hogging bending moment.
With this formula the shear resistance of the rib can be determined, but this would ignore the possible overloading of the reinforcement loaded in tension in case the slab is also subjected to a hogging bending moment.

At [10 p. 6.2.1 (7)] of EN 1992-1-1 [10] under the general shear verification procedure is mentioned:
The longitudinal tension reinforcement should be able to resist the additional tensile force caused by shear (see 6.2.3(7)).

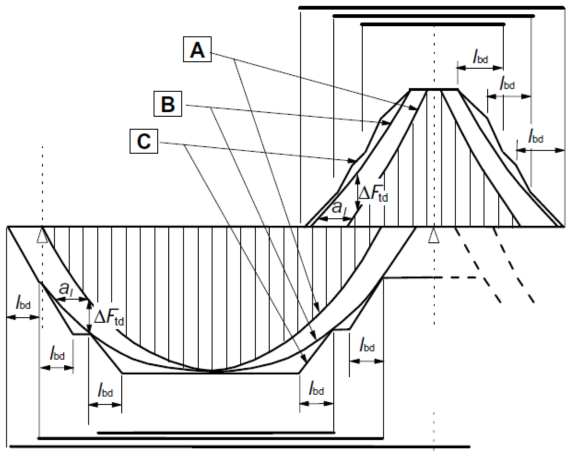
Looking up on 6.2.3(7) of [10]it says:
(7) The additional tensile force, ΔFtd, in the longitudinal reinforcement due to shear V_{Ed} may becalculated from:
ΔFtd= 0,5 V_{Ed} (cot θ- cot α) (6.18)
(M_{Ed}/z) + ΔFtd should be taken not greater thanM_{Ed},max/z, whereM_{Ed},max is the maximum moment along the beam.

However the angles θand αassume shear reinforcement to be present. It is therefore under 6.2.3. , which implies member requiring shear reinforcements. This general verification procedure is therefore not applicable for our composite slab, which does not contain shear reinforcement apart from the sheet.

Returning to the 6.2.2 of EN 1992-1-1 [10] we find under 6.2.2.(5) the following:
(5) For the design of the longitudinal reinforcement, in the region cracked in flexure, the M_{Ed} -line should be shifted over a distance al = d in the unfavorable direction (see 9.2.1.3 (2))

In 9.2.1.3(2):

EN 1992-1-1 (2) For members with shear reinforcement the additional tensile force, ΔFtd, should be calculated according to 6.2.3 (7). For members without shear reinforcement ΔFtd may be estimated by shifting the moment curve a distance al = d according to 6.2.2 (5).



A - Envelope of $M_{Ed}/z + N_{Ed}$ B - acting tensile force F_s C - resisting tensile force F_{Rs}

Figure A-1Illustration of the curtailment of longitudinal reinforcement

This implies that in the case of a slab without shear reinforcement, the longitudinal reinforcement will be exposed to forces due to the shift in theM_{Ed} line over a distance aI = d.

This refers to the diagonal cracks occurring due to the Shear force. The longitudinal reinforcement calculated at the maximum moment, located at the intermediate support, will be required at full capacity up to a distance *d* away from the support. This criterion therefore covers the length over which this longitudinal reinforcement is present due to this phenomenon. It however does not say anything about the interaction between Moment and Shear force. It also does not increase the area of reinforcement. It only increases the length over which is it required.

Calculations regarding the $V_{Rd,rib}$ therefore is confined to the formula given in EN 1992-1-1 [10] , art. 6.2.2. Calculated completely separately from the moment capacity and moments applied.

A.1.2. Hogging bending moment resistance

At the intermediate support a negative bending moment is present. According to the EN 1994-1-1 [5] art. 9.7.2 (2)

the contribution of the steel sheeting shall only be taken into account where the sheet is continuous (in case of shallow sheets) and when for the construction phase redistribution of moments by plastification of cross-sections over supports has not been used.

Most of the time this will be the case, as the sheets give their advantageous when used as reinforcement for the sagging bending moment and as formwork. Neglecting of the steel sheeting will leave the concrete, possible reinforcement at the bottom and top longitudinal reinforcement to resist the hogging bending moment.

The tensile resistance of concrete is very small and therefore neglected also.

According to EN 1994-1-1:[5], art. 9.7.2 (7), the stress distribution for the hogging bending moment will be calculated as in Figure A-2.

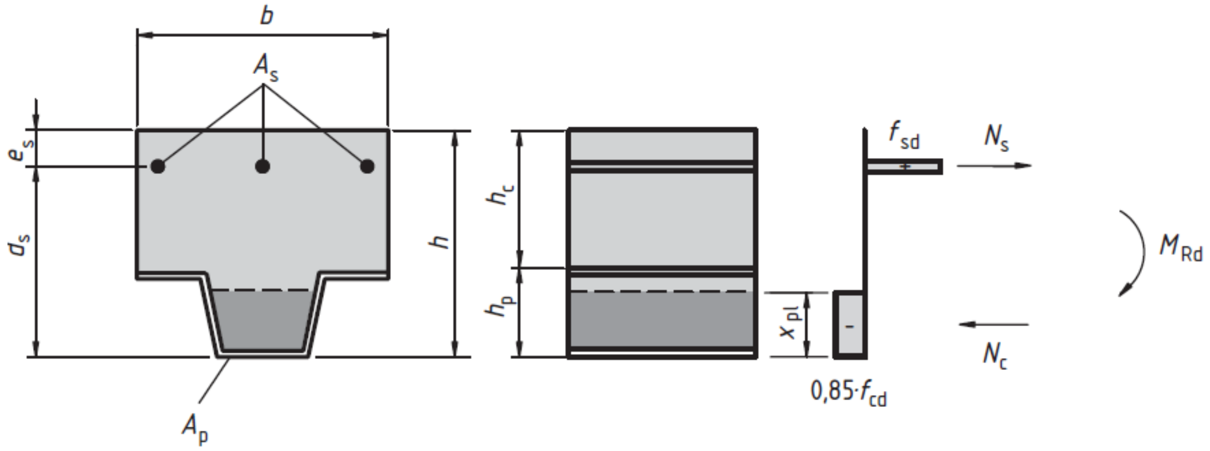


Figure A-2Stress distribution for hogging bending

Within this chapter, there is no reference made towards the influence of a possible shear force present in the cross section.

Within the part of the Eurocode concerning composite slabs, nothing is mentioned regarding possible interaction and therefore a reduction of capacity in case of a high hogging bending moment and vertical shear.

In search for research regarding this interacting on composite slabs, did not give any results

A.2. Interaction behavior of steel and composite slabs with steel beams.

Most of the time, the values of moment and shear resistance are approached separately. These effects however, coexist and therefore interaction must be considered. As insufficient information is present for composite slabs, it`s worth considering a cross section build-up of steel only.

Within steel more research is done compared to composite structures. As with composite structures, the interaction between bending and axial force is more familiar and is shown in Figure A-3.

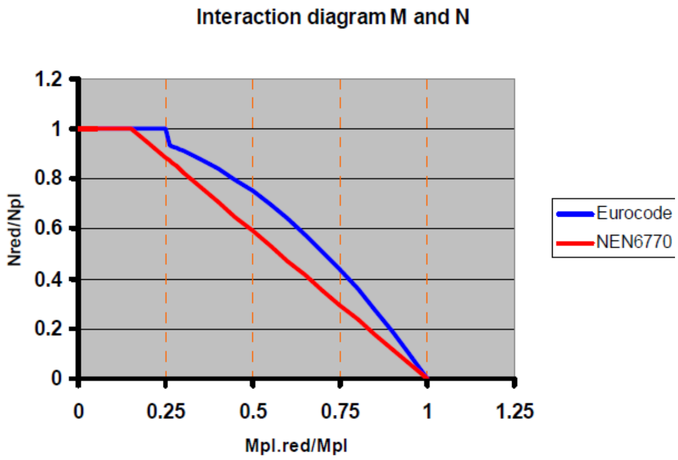


Figure A-3 Interaction diagram for Moment and Axial Force

The axial force has an influence on the bending moment capacity. Bending means an axial tensile force on one side, combined with an axial compressive force on the other side. As an additional axial force is in the same direction, this influence can be explained. If it comes to shear, the additional force present is no longer in the same direction, but is active in the same cross section. In the next chapter this interaction is further investigated.

Within the steel code EN 1993-1-1 [12] , the interaction between bending and shear is ignored, if the design shear force is less than: $\frac{V_{pl,Rd}}{2}$. (art. 6.2.10 (2))

There where the shear force exceeds 50%, the shear will be taken into account, by reduction of the yield strength of the web. The yield strength of the web will be taken as: $(1-\rho)*f_y$. in calculating the plastic moment resistance. See 6.2.8 [12].

With: $\rho = (\frac{2*V_{Ed}}{V_{pl,Rd}} - 1)^2$.

Where:

V_{Ed}= the design shear force

$V_{pl,Rd}$= the plastic shear resistance

So once applying over 50% of the maximum plastic shear resistance, a reduction in bending moment is found. If a shear force is applied, equal to the maximum plastic shear resistance, only 50% of the maximum bending moment resistance remains.

It therefore suggests, that if the cross-section would be completely be build-up out of steel (depending on the cross section class still), a failure would occur in case both maximum shear and maximum moment would be applied.

Within the EN 1994-1-1 [5], under the composite slab section 9.7.5 a reference is made to the concrete code EN 1992-1-1 [10] to determine the vertical shear. If looking to 6.2.2.4 [5] a description is given, similar to that used in the research of determining the resistance of an IPE/concrete composite beam. [9].

In EN 1994-1-1 [5] 6.2.2.4 (3) a reference is made to the steel code in case of class 3 or 4 cross sections.
Under chapter 7.1 of[15] interaction between shear force, bending moment and axial force is mentioned. Based on an I or box girder a formula is given to calculate the interaction between moment and shear force.

$$\bar{\eta}_1 + \left(1 - \frac{M_{f,Rd}}{M_{pl,Rd}}\right)\left(2\bar{\eta}_3 - 1\right)^2 \leq 1{,}0 \quad for\ \bar{\eta}_1 \geq \frac{M_{f,Rd}}{M_{pl,Rd}}$$

With:

$$\bar{\eta}_1 = \frac{M_{Ed}}{M_{pl,Rd}}$$

$$\bar{\eta}_3 = \frac{V_{Ed}}{V_{bw,Rd}}$$

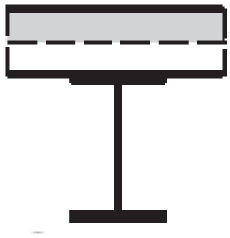


Figure A-4Cross section considered in 6.2.2.4[5]

In this chapter, in case of a steel section bearing a concrete floor, the shear resistance is reduced in case of simultaneous loading by both shear and bending. Resulting in the graph provided in Figure A-6.

In case of steel girders or composite beams (mainly consisting of a steel girder), a reduction of the moment capacity will be present in case a shear force over 50% of the maximum shear resistance is applied.

c

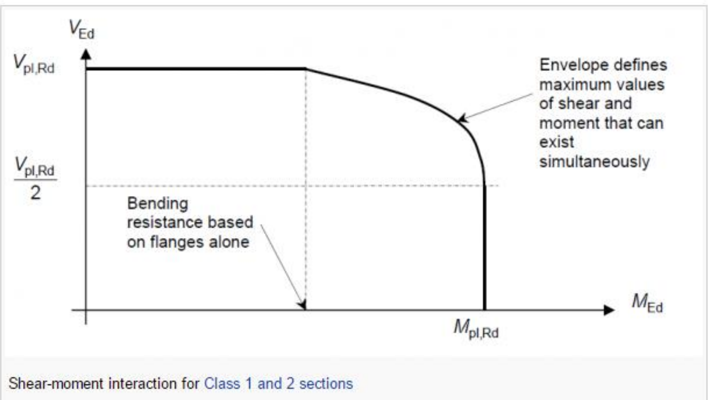


Figure A-5Interaction diagram Moment-Shear cross section class 1-2[9]

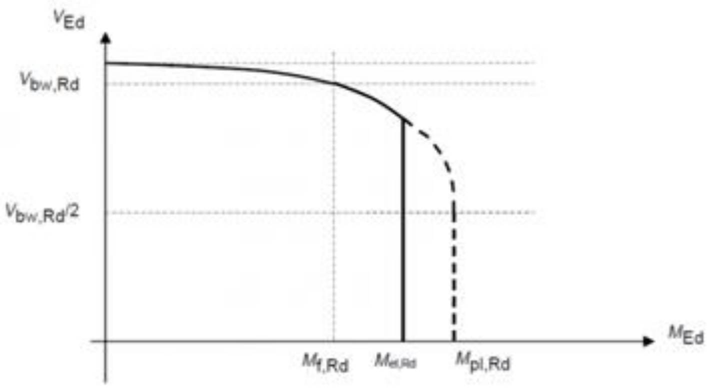


Figure A-6Interaction limits for cross section class 3 and 4

A research done by a lecturer at the University of Edinburgh U.K. and a professor at the University of New South Wales in Sydney Australia did do a research regarding composite beams. In this particular study [4], a steel IPE330 beam was used, connected to a concrete slab of 150mm thick.

Within this research the numerical results combined with the test results leaded to the following graph:

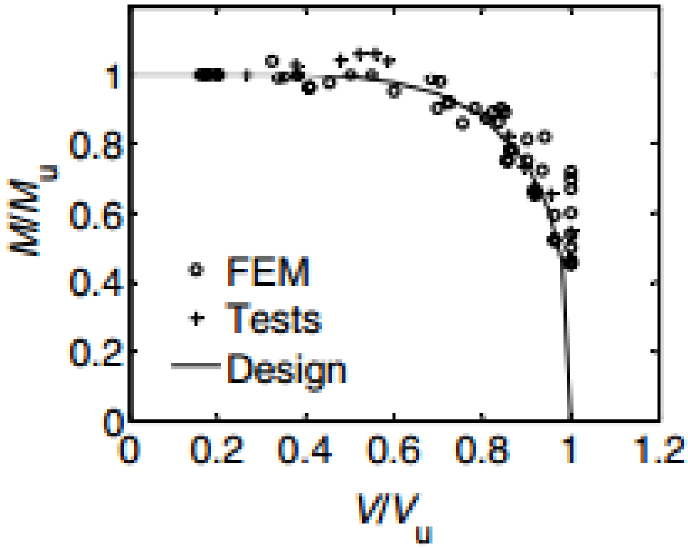


Figure A-7Moment-Shear interaction diagram resulting from the parametric analyses

Within the EN 1994-1-1 [5], the shear resistance of the composite beam is calculated according to the shear resistance of the steel web. With a capacity of: $V_{pl;Rd} = 0.6 * A_v * f_{yw}$. Where Av is the shear area of the steel member and $f_{yw}$is the yield strength of the steel of the web. Within this research they concluded that the slab contributes up to 33-56% of the total shear capacity. As shown in Figure A-8.

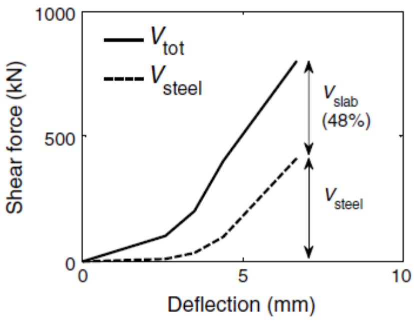


Figure A-8 Calculation of slab contribution to the total shear strength of the composite beam, based on the data of the strain gauges.

As an IPE330 is used in the research done in Sidney [4], half of the shear resistance is received from the steel beam. 52%. The total resistance is formulated as:

$$V_{comp} = V_{pl,Rd} + V_{slab}$$

Where $V_{pl,Rd}$ is the contribution of the steel section and V_{slab} the contribution of the concrete slab. According to this research, there should be a relation comparable to the diagram used in steel members. Within the steel codes, distinction is made between the different cross section classes. Reduction of the sagging bending moment occurs, once 50% - 100% of the maximum shear force is applied simultaneously. Reduction in capacity also starts once $\frac{V_{pl,Rd}}{2}$ is applied. It might therefore be the case that the interaction between shear and moment is comparable to the steel cross sections, as the composition of the cross section is also comparable of that of a hot rolled IPE or HEA steel beam.

A.3. Interaction behavior in concrete

A.3.1. Verification of a concrete member according to Eurocode 2

Within the EN 1992-1-1 [10], the shear capacity of a member without shear reinforcement is calculated according to a cross section design method. Here by the applied vertical shear force V_{Ed} may not exceed the shear capacity $V_{Rd,c}$ of the beam. This is also the section where the EN 1994-1-1 [5] refers to when checking a composite floor on vertical shear resistance.

The shear capacity ($V_{Rd,c}$)has initially been determined in the beginning of 19th century by Arthur N. Talbot [16]. After much further research, this lead to an empirical formula as used within the Eurocode:

$$V_{Rd,c} = [C_{Rd,c}k\left(100\,\rho_1 f_{ck}\right)^{\frac{1}{3}} + k_1\sigma_{cp}]\,b_w d.$$

With a minimum of

$$V_{Rd,c} = (v_{min} + k_1\sigma_{cp})b_w d$$

Where:

fck is in MPa

$$k = 1 + \sqrt{\frac{200}{d}} \leq 2.0 \text{ with } d \text{ in mm}$$

$$\rho 1 = \frac{Asl}{b_w d} \leq 0.02$$

Asl is the area of the tensile reinforcement, which extends ≥(lbd + d) beyond the section considered.

b_w is the smallest width of the cross-section in the tensile area [mm]

σ_{cp} = NEd/Ac < 0,2 fcd [MPa]

Ned is the axial force in the cross-section due to loading or prestressing [in N] (NEd>0 for compression). The influence of imposed deformations on NE may be ignored.

AC is the area of concrete cross section [mm2]

VRd,c is [N]

Note: *The values of CRd,c, vmin and k1 for use in a Country may be found in its National Annex. The recommended value for $C_{Rd,c}$ is 0,18/ γc, that for v_{min} is vmin $= 0{,}035\ k^{3/2} f ck^{1/2}$ and that for k1 is 0,15.(in the Dutch National Annex)*

In case it surpasses the shear capacity of the concrete cross section, the entire shear force must be carried by the applied shear reinforcement (stirrups). Within this empirical formula, the variables are based on the dimensions of the cross section as well as the properties of the materials. This excludes influences of other forces within the element such as an applied moment. It is assumed that the moment reinforcement takes care of the moments applied and the shear reinforcement / cross section takes care of the shear force applied.

A.3.2. Interaction moment and shear within a concrete member

As explained before is the shear capacity within the EN 1992-1-1 [10] determined using empirical formulae and is independent of the applied moment. However a recent thesis at the TU Eindhoven in the Netherlands [1], shows that the moment in reality does influence the shear resistance. This however was an experimental research with only 4 tests with a variety in strength properties, but still the results show that a certain interaction is present.

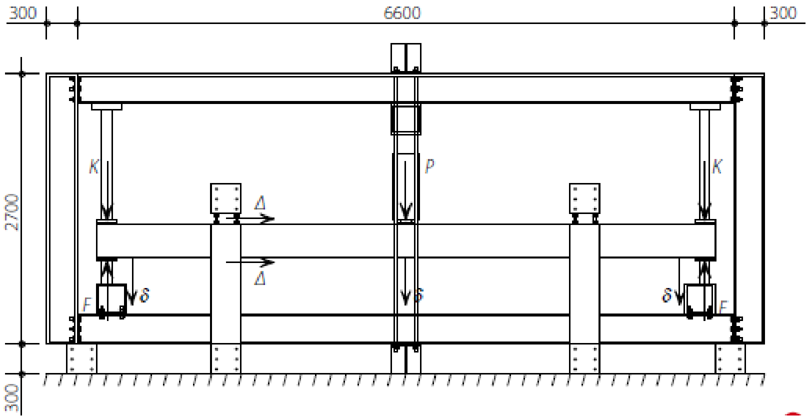


Figure A-9Test setup used to preload the specimen by a certain moment by force K, afterwards the specimen is loaded by force P to create the desired M-V combination.

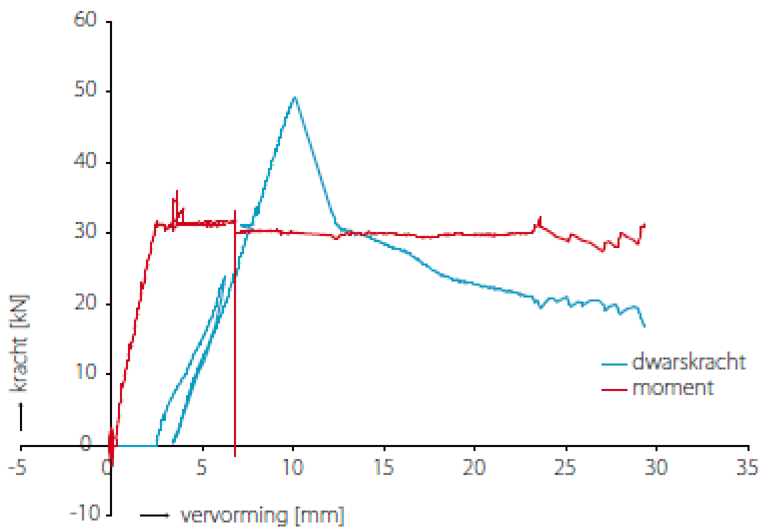


Figure A-10 Force [kN] on the y-axis and deflection [mm] on the x-axis. The red line shows the applied moment, remaining constant during the test, the blue line the applied shear force. All in the critical cross section.

Above the setup used, and how the Moment / Vertical shear force developed during one of the experiments.

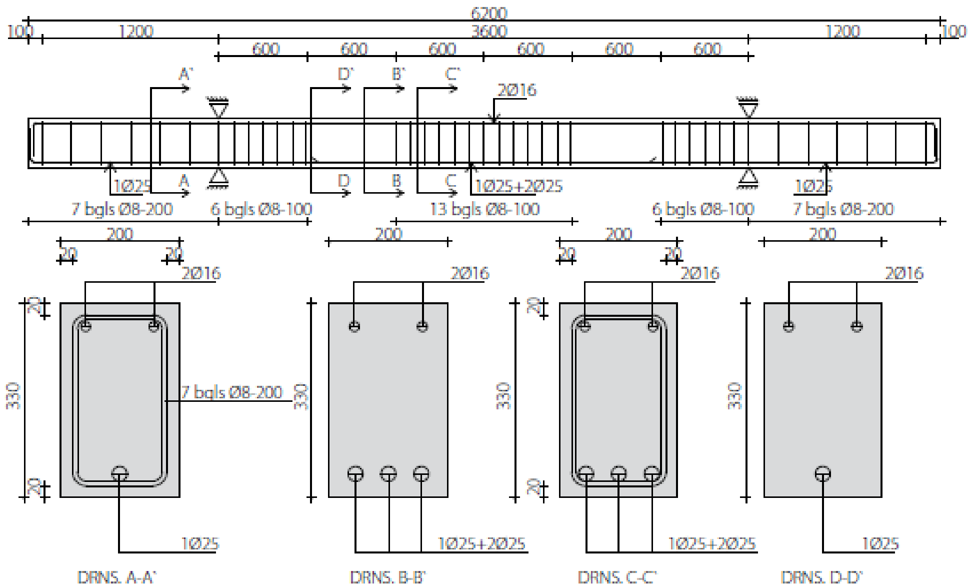


Figure A-11 Cross sections of the specimen used in the experiments at the TU Eindhoven, The Netherlands.[1].

The specimen used was determined based on a weak cross section D-D, resulting in a controlled location of failure, allowing identical failure locations for all four experiments. The results of these experiments are shown below in Figure A-12.

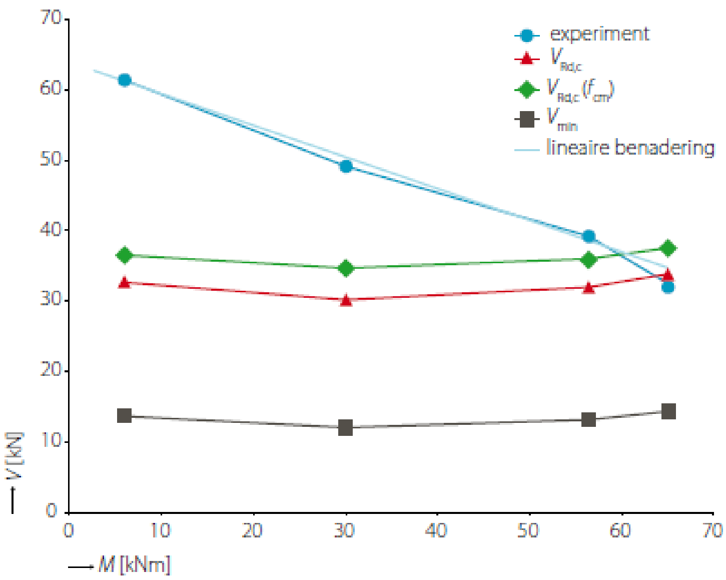


Figure A-12 M-V interaction graph, comparing results with the Eurocode

Even with a variety in properties a relation can be found between the applied moment and the shear capacity.

Within a publication in Cement [13] at cement online [17] by the involved professor of the reviewed master thesis, a linear relation has been determined based on the test results:

$$V = V_{M=0} - 0.45M$$

Where:
V is the shear force in kN
M is the moment in kNm
$V_{M=0}$ *is the shear force capacity, without moment applied*

This would suggest a linear relation and therefore a huge influence of the different forces applied. Comparing this to the relation obtained in 3.2, it shows that both in concrete and steel, it is not possible to apply both maximum moment and shear force at the same time, using the same material to resist both vertical shear and bending moments to it maximum. Within EN 1993-1-1 [12] of the steel department, a reduction is applied the moment 50% of the maximum shear capacity is reached. Taking the linear relation obtained from the thesis done at the TU Eindhoven [1] a reduction should be present the moment both forces are applied simultaneously.

Looking into steel and concrete separately, both showing an influence once both forces are applied simultaneously, the doubts regarding interaction of M-V in composite structures, can be taken to be in place.

APPENDIX B. Initial calculations

Initial calculations are done to get insight in the minimum span, hogging bending moment resistance and vertical shear resistance. These can be used to determine the correct test rig that can answer the main research question.

B.1. Calculation of the ComFlor210 (not equal to the resistance of the test specimens, resistances used to investigate test setup to be used.)

B.1.1. Minimum span ComFlor210

During the tests a shear force is wanted between 70% - 100% of its total capacity. To avoid that the force is directly transmitted to the support a minimum span is required. According to 6.4.1 of EN 1992-1-1 [10] the angle to determine the minimum distance away from the support is equal to $\theta = \arctan\left(\frac{1}{2}\right) = 26.6°$ as shown in Figure B-1. This means a minimum distance between load applied and support is equal to 2d. In case of the ComFlor210, is equal to approximately 257mm. $2d = 514\ mm$, to be conservative and including the supporting edge a minimum span length of 600 mm is preferred. However dr.ir.Yuguang Yang said that some force can still be directly transmitted to the support at $^{a}/_{d}$ratios of up to 5. This means a minimum$^{a}/_{d}$ratio of 3 is preferred to get a real vertical shear value. A minimum $^{a}/_{d}$ratio of 3 will therefore be used in this thesis.

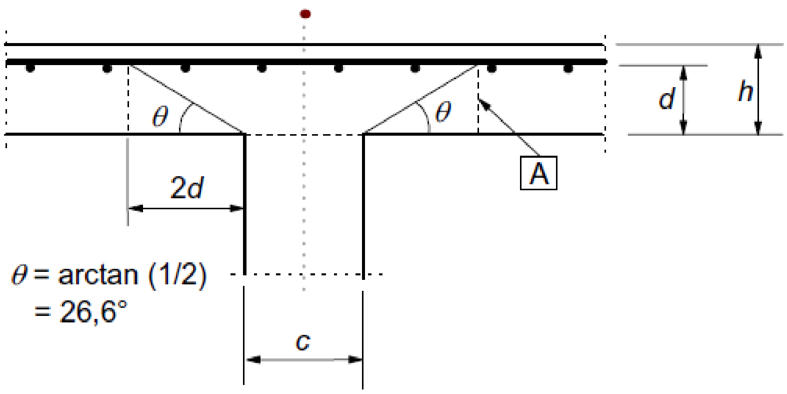


Figure B-1 Punchingangle 6.4.1. General [10].

B.1.2. Moment capacity ComFlor210

For the calculations of the hogging bending moment capacity a ComFlor210 with a total height of 280mm is used, based on the case study “town hall Almelo” that raised the question. In the table below an overview is given of the reinforcement and concrete applied:

Table 21 Properties ComFlor210 M calculation

Name	Unit	Name	Unit
$f_{ck} = 30$	$N/mm^2/$	$f_{cd} = f_{ck}/\gamma_c$	N/mm^2
$f_{cm} = 38$	N/mm^2	$\gamma_s = \gamma_c = 1$	
$Mesh_{ctc} = 150$	mm	$c = 25$	mm
$Mesh_{\emptyset} = 10$	mm	$h = 280$	mm
$Add, R_{ctc} = 150$	mm	$w_{ribbot} = 56$	mm
$Add, R_{\emptyset} = 10$	mm	$w_{ribtop} = 175$	mm
$f_{yk} = 500$	N/mm^2	$h_{sheet} = 210$	mm
$f_{yd} = f_{yk}/\gamma_s$	N/mm^2	$effective\ width = 600$	mm

These values are obtained by using partial material factor for steel of γs = 1 and γc = 1 for concrete. with respect to the variables regarding the vertical shear resistance. Parameters like the characteristic strength for concrete (f_{ck}), are values taken from a normal distribution. In Figure B-3an overview is given of the compressive strength distribution of C30. The characteristic strength (f_{ck}) is the 5% value and the mean strength at 28 days is the f_{cm}value. Within this calculation values of f_{ck} are replaced by the f_{cm}as given in Table 21. At the end of the calculation, the influence of this spread is taken into account.

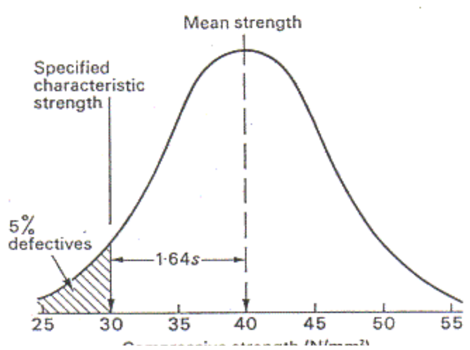


Figure B-2 Normal distribution compressive strength C30

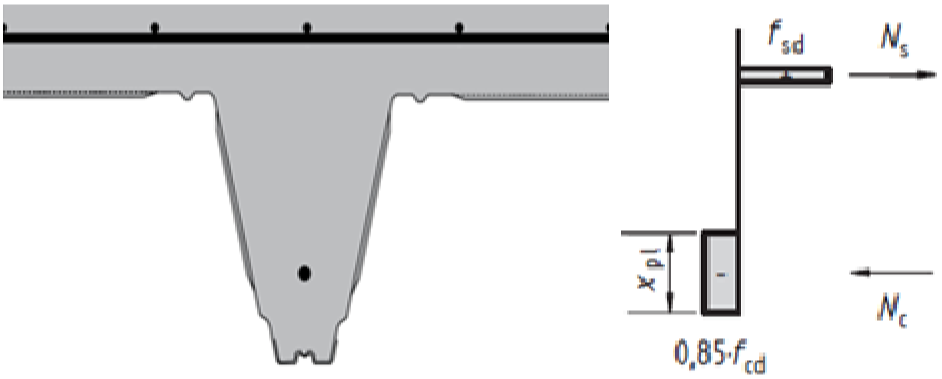


Figure B-3 Hoggingbending moment equilibrium

The upper reinforcement will make equilibrium with the concrete rib in compression. This equilibrium is shown in Figure B-3, where $N_S$refers to the resistance of the upper reinforcement and N_C to the concrete rib in compression.

The maximum lever arm of the reinforcement follows from:

- $d = h - c - \max(\frac{Mesh_{\emptyset}}{2}; \frac{Add,R_{\emptyset}}{2})$ $\qquad d$=250mm

In order for the reinforcement to yield and avoid a brittle behavior, a maximum compressive zone $X_{u,max}$ is defined:

- $X_{u,max} = \frac{500*d}{500+f_{yd}}$ $\qquad X_{u,max} = 125$ mm

Using this data the hogging bending moment resistance per meter floor can be calculated.

- $w_{Xu} = w_{ribbot} + \frac{(w_{ribtop} - w_{ribbot})}{h_{sheet}} * X_{u,max}$ $\qquad w_{xu} = 126.83$mm

w_{xu} equals the width of the maximum compressive area in the concrete rib.

- $A_{xu} = \left(\frac{(w_{xu}+w_{ribbot})}{2} * X_{u,max}\right) * \frac{1000}{effective\ width}$ $\qquad A_{xu} = 19045\text{mm}^2/\text{m}$

A_{xu} equals the area of the maximum compressive area in the concrete rib.

- $A_{mesh} = ((\frac{Mesh_{\emptyset}}{2})^2 * \frac{\pi*1000}{Mesh_{ctc}})$ $\qquad A_{mesh} = 335.1\ \text{mm}^2/\text{m}$

A_{mesh} equals the area of the mesh reinforcement

- $A_{Add,R} = ((\frac{Add,R_{\emptyset}}{2})^2 * \frac{\pi*1000}{Add,R_{ctc}})$ $\qquad A_{Add,R} = 523.6\text{mm}^2/\text{m}$

$A_{Add,R}$ equals the area of the additional support reinforcement.

- $A_{total} = A_{mesh} + A_{Add,R}$ $\qquad A_{total} = 858\text{mm}^2/\text{m}$

$A_{total}$is the total area of reinforcement above the support.

- $N_s = A_{total} * f_{yd}$ $\qquad N_s = 429$ kN/m

N_s equals is the maximum force the reinforcement can withstand.

- $N_c = A_{xu} * f_{cm} * 0.85$ $\qquad N_c = 647$ kN/m

$N_c$is the maximum compressive force the concrete rib can withstand

- $N_{max} = \min(N_s; N_c)$ $\qquad N_{max} = 429\ k\text{N/m}$

Knowing the maximum force the cross section can withstand, a compressive region in the rib makes equilibrium with the force in the reinforcement. To determine the area of the concrete in compression and therefore the centre of gravity of the compressive area, the height of the compression zone musts be calculated.
Height compression zone = y.

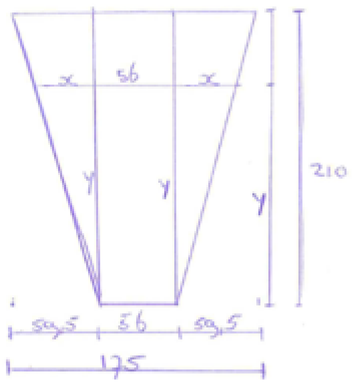


Figure B-4 Compression zone rib Comflor210

- $x = \frac{w_{ribtop} - w_{ribbot}}{2*h_{sheet}} * y$ $\qquad x$= 0.28*y mm

- $N_{max} = \frac{f_{cm}*0.85*1000}{effective\ width} * (w_{ribbot} + x) * y$

Solving this equation gives a total height [y] of 92.2 mm
So the width of x is equal to $x = 0.28 * y = 26.14\ mm$

Using this data, the centre of gravity from the bottom of the rib can be calculated:

- $X = \frac{(\frac{w_{ribbot} * y^2}{2} + \frac{x * y^2 * 2}{3})}{(w_{ribbot} + x) * y}$ $\qquad X = 51\ mm$

Now all data is known and the Final Hogging Bending Moment Capacity per meter slab can be calculated:

- $M_U = (N_s * (d_{rib} - X)$ $\qquad M_U = 85.4\text{kNm/m}$

Within this calculation not many uncertainties are present. The compressive strength of the concrete is to be determined right before testing is done to get inside in the actual strength. This changes the compressive resistance of the rib. The same counts for the reinforcement used in tension.
As explained based on Figure B-2 a spread in compressive strength is present in the concrete, this could lead to a stronger or weaker concrete. To include this normal distribution, minimum and maximum values are submitted into the calculation, results will be given below:

The upper limit: $M_U = 98.5\ kNm/m \quad (f_c = 55 \frac{N}{mm^2} \& f_{yd} = 550\ N/mm^2)$
The lower limit: $M_U = 76.8\ kNm/m \quad (f_c = 30 \frac{N}{mm^2} \& f_{yd} = 475\ N/mm^2)$

This means a total of 15% uncertainty (15% higher or lower) due to spread in material properties. Later on this uncertainty is taken into account when determining the length of the specimen. At the moment of testing, the cubes and reinforcement can be tested, providing a more accurate compressive strength for the concrete and yield strength of the reinforcement.

B.1.3. Vertical shear resistance ComFlor210

The vertical shear is more complicated to determine, as the Eurocode does not have an exact calculation of composite slabs. (Besides the deep deck floor are not covered in the Eurocode 4) It refers to art. 6.2.2 of EN 1992-1-1 [10], which is a calculation for concrete members without shear reinforcement. The ComFlor210 contains three components which contribute to the resistance, namely, the steel sheet, the concrete rib and the concrete slab that could possibly contribute to the vertical shear resistance. The values of these contributing parts depend on the way of supporting. The shear resistance V_{su} of the steel sheeting is determined by testing. The values can be found in the Zulassung [11] of a German research. In case of the ComFlor210 with a thickness of 1 mm this equals 44.98 kN/m. The contribution of the ribs determined according to the Eurocode and the concrete flange is assumed to be a separate beam to contribute as an individual part (to be sure to approach a high shear resistance). The Eurocode takes both the concrete and the top reinforcement into account. However the rib reinforcement bar is not included.

To calculate the Vertical Shear Resistance of the composite slab the following data is used:

Table22 Properties ComFlor210 V calculation

Name	Unit	Name	Unit
$f_{ck} = 30$	$\mathrm{N/mm^2}$/	$f_{cd} = f_{ck}/\gamma_c$	$\mathrm{N/mm^2}$
$f_{cm} = 38$	$\mathrm{N/mm^2}$	$C_{rd,c} = 0.15$*	
$Mesh_{ctc} = 150$	mm	$c = 25$	mm
$Mesh_{\emptyset} = 10$	mm	$h = 280$	mm
$Add, R_{ctc} = 150$	mm	$w_{ribbot} = 56$	mm
$Add, R_{\emptyset} = 10$	mm	$w_{ribtop} = 175$	mm
$f_{yk} = 500$	$\mathrm{N/mm^2}$	$h_{sheet} = 210$	mm
$f_{yd} = f_{yk}/\gamma_s$	$\mathrm{N/mm^2}$	$effective\ width = 600$	mm

The value for$C_{rd,c}$ in the formula to determine the vertical shear resistance consists of $\frac{0.18}{\gamma_c}$ where γ_c is the partial factor for concrete of 1,5. To determine the real vertical shear resistance for testing, instead of the design value, it is not possible to simply put the partial factor for concrete equal to 1. After a discussion with dr.Ir. Yuguang Yang, a reference was made to his thesis [14], page 21. A realistic value should be 0.15 and using f_{cm}instead of the f_{ck}value in the formula given in the EN 1992-1-1 [10]:

$$V_{Rd,c} = \left[C_{Rd,c} k \left(100\, \rho_1 f_{ck}\right)^{\frac{1}{3}}\right] bwd$$

This however means a first uncertainty when it comes to determining the actual vertical shear resistance.

B.1.3.1. Vertical Shear Resistance of the Rib

The vertical shear resistance of the rib is based on a concrete beam, not requiring shear reinforcement. Calculating its capacity is based on the EN 1992-1-1 [10] chapter 6.2.2. The capacity is given as explained in 0; $V_{Rd,c} = \left[C_{Rd,c} k \left(100\, \rho_1 f_{ck}\right)^{\frac{1}{3}}\right] bwd$ with a minimum of: $V_{Rd,c} = \mathrm{v_{min}} * \mathrm{b_w} * d$.

The Vertical Shear calculation is shown below:

- $\mathrm{b_w} = \frac{\mathrm{w_{ribbot}} + \mathrm{w_{ribtop}}}{2} * \frac{1000}{\text{effective width}}$ $\quad\quad$ $\mathrm{b_w} = 192.5\mathrm{mm/m}$

b_w is the minimum width in tension in the rib.

- $\mathrm{b_{w2}} = \mathrm{w_{flange}} * \frac{1000}{\text{effective width}}$ $\quad\quad$ $\mathrm{b_{w2}} = 708.3\ \mathrm{mm/m}$

b_{w2} is the minimum width in tension in the flange.

- $d_{flange} = h - c - \max\left(\frac{mesh_{\emptyset}}{2}; \frac{Add, R_{\emptyset}}{2}\right)$ $\quad\quad$ $d_{flange} = 40\ mm$

d_{flange} is the maximum internal lever arm in the flange (slab between ribs)

- $Ratio_{rib} = \frac{b_w * d_{rib}}{b_w * d_{rib} + b_{w2} * d_{flange}}$ $\quad\quad$ $Ratio_{rib} = 0.63\ \%$

Here the ratio is assumed to be related to the effective area of both the flange as the rib itself. The stiffer part will deform less compared to the flange, resulting in taking over load from the flange. This will lead to a redistribution of the reinforcement contributing to the vertical shear resistance of the rib and the flange. This gives a rise of ρ| in the rib and a decrease in the flange.

- $A_{sl,rib} = Ratio_{rib} * A_{total}$ $\quad$ $A_{sl,rib} = 540.5\text{mm}^2/\text{m}$

$A_{sl,rib}$ is the amount of reinforcement contributing to the vertical shear resistance of the rib.

- $\rho_{rib} = \min(\frac{A_{sl,rib}}{b_w * d_{rib}}; 0.02)$ $\quad$ $\rho_{rib} = 0.011$

ρ_{rib} is the reinforcement percentage within the rib per meter slab.

- $k_{rib} = \min(1 + \sqrt{\frac{200}{d_{rib}}}; 2)$ $\quad$ $k_{rib} = 1.89$
- $v_{c,rib} = C_{Rd,c} k_{rib} \left(100\, \rho_{rib} f_{cm}\right)^{\frac{1}{3}}$ $\quad$ $v_{c,rib} = 1.01\ \text{N/mm}^2$

In this formula the $C_{Rd,c}$ and f_{cm} are used as explained at the beginning of B.1.3 [14].

- $v_{min} = 0.035 * k_{rib}{}^{3/2} * f_{cm}{}^{0.5}$ $\quad$ $v_{min} = 0.577\text{N/mm}^2$

v_{min} represents the minimum shear stress of the concrete alone.
This leads to completing the final formula to determine the vertical shear contribution of the concrete rib:

- $V_{U,rib} = \max\left(v_{c,rib}; v_{min}\right) * \ bw * d_{rib}$ $\quad$ $V_{U,rib} = 48.6kN/m$

This is also the value used in the Eurocode (excluding all safety and material factors)

B.1.3.2. Vertical Shear Resistance of the Flange

Even though discussion exist if the flange may be included or not, for initial calculations it is included to gain a high vertical shear resistance. If later on the flange appears to not contribute, at least the test rig is still useful to answer the main research question. Below the calculation is given with the properties provided in Table22:

- $Ratio_{flange} = \frac{b_{w2} * d_{flange}}{b_w * d_{rib} + b_{w2} * d_{flange}}$ $\quad$ $Ratio_{flange} = 0.37\ \%$

The ratio is assumed to be related to the effective area of both the flange and the rib itself. The stiffer part is most likely to carry more loads.

- $A_{sl,flange} = Ratio_{flange} * A_{total}$ $\quad$ $A_{sl,flange} = 318.2\ \text{mm}^2/\text{m}$

$A_{sl,flange}$ is the amount of reinforcement contributing to the vertical shear resistance of the flange.

- $\rho_{flange} = \min(\frac{A_{sl,flange}}{b_{w2} * d_{flange}}; 0.2)$ $\quad$ $\rho_{flange} = 0.0112$

ρ_{flange} is the reinforcement percentage within the flange per meter slab.

- $k_{flange} = \min(1 + \sqrt{\frac{200}{d_{flange}}}; 2)$ $\quad$ $k_{flange} = 2$
- $V_{c,flange} = C_{rd,c} * k_{flange} * (100 * \rho_{flange} * f_{cm})^{1/3}$ $\quad$ $V_{c,flange} = 1.066\text{N/mm}^2$

In this formula the $C_{rd,c}$ and f_{cm} are used as explained at the beginning of B.1.3 [14].

- $v_{min} = 0.035 * k_{flnge}{}^{3/2} * f_{cm}{}^{0.5}$ $\qquad v_{min} = 0.62\text{N/mm}^2$

v_{min} represents the minimum shear stress of the concrete alone.

This leads to completion of the final formula to determine the vertical shear contribution of the concrete flange:

- $V_{U,flange} = \max(v_{c,flange}; v_{min}) * \ bw * d_{rib}$ $\qquad V_{U,flange} = 30.2\text{kN/m}$

B.1.3.3. Shear resistance of the complete cross section

The total vertical shear resistance is a summation of all the contributing parts.

- The concrete rib $V_{U,rib}$
- The concrete flange $V_{U,flange}$
- The ComFlor210 sheet V_{su}

The first two parts have been determined;
The value V_{su} of the steel sheeting is determined by testing. The values can be found in the Zulassung [11] of a German research. This leads to a contribution of an additional 44.98 kN/m without safety factors.

The total vertical shear resistance is as following:

$$V_{U,210} = V_{U,rib} + \ V_{U,flange} + \ V_{su}$$

Resulting in:

$$V_{\text{U},210} = 48.6 + \ 30.2 + \ 44.98 = 123.78\ kN/m$$

According to the Eurocode (without safety factors), this would lead to a $V_{U,210}$ of 48.6 kN/m as only the rib is taken into account. Which leads to a gap of almost 80 kN/m compared to the other calculation.

B.2. Calculation of the ComFlor 75(not equal to the resistance of the test specimens, resistances used to investigate test setup to be used.)

The difference between the ComFlor 75& the ComFlor210 can be found in the height, ratio rib height vs slab height, concrete/steel ratio, continuous sheet etc.
The rib will be provided with an M10 bar, resulting in Ø10-300 reinforcement in the bottom of the slab.
Most of the procedure will be the same as used for the ComFlor210. Starting with determination of the minimum span to avoid direct force transmission to the supports, followed by a calculation of the different capacities. Based on these values, the final test specimen can be determined. As most of the calculation of the ComFlor75 is comparable to those of the ComFlor210, a shortened calculation is shown in this chapter.

B.2.1. Minimum span ComFlor75

The minimum span (distance from the support) must be equal to a minimum of $3 * \frac{a}{d}\ ratio$.

$$3 * d = 3 * 120 = 360\ mm$$

B.2.2. Moment Capacity ComFlor75

Instead of the integrated detail used for the ComFlor210, shallow floors are commonly placed continuously over the support as shown in Figure B-5. This means the sheet is in compression and can take up some extra compressive force in case of a negative hogging moment. This differs from the ComFlor210 where the sheet could only transmit the force to the concrete as it did not continue.

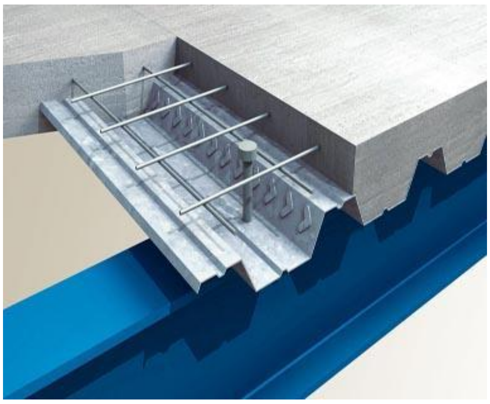


Figure B-5 Support detail ComFlor75

In the table below the specific properties of the concrete, reinforcement and ComFlor75 are given:

Table 23 Properties ComFlor 75 M Calculation

Name	Unit	Name	Unit
$f_{ck} = 30$	N/mm²/	$f_{cd} = f_{ck}/\gamma_c$	N/mm²
$f_{cm} = 38$	N/mm²	$C_{rd,c} = 0.15$	
$Mesh_{ctc} = 150$	mm	$c = 24$	mm
$Mesh_{Ø} = 8$	mm	$h = 150$	mm
$Add, R_{ctc} = 150$	mm	$w_{ribbot} = 120$	mm
$Add, R_{Ø} = 12$	mm	$w_{ribtop} = 265$	mm
$f_{yk} = 500$	N/mm²	$h_{sheet} = 60$	mm
$f_{yd} = f_{yk}/\gamma_s$	N/mm²	$effective\ width = 300$	mm
$\gamma_s = 1$		$\gamma_c = 1$	

In Figure B-6 a cross section is shown of the ComFlor75 sheet. The spread in material properties, also plays a role when using the ComFlor75. This influence will be shown at the end of the calculation.

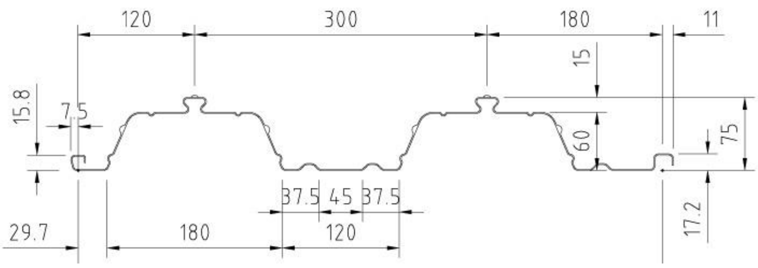


Figure B-6 Cross section ComFlor 75 steel sheeting1.

Using a total height of 150 mm, cover of 24 mm and reinforcement of Ø12, this will result in a maximum leverarm of d_{rib} = 120 mm.As material factors are set to 1, this leads to a maximum compression height$X_{u,max}$ of:

- $X_{u,max} = \frac{500*z}{500+f_{yd}}$ $\quad$ $X_{u,max} = 60$ mm
-

Using these parameters, the capacity calculation is as follows:

- $w_{Xu} = w_{ribbot} + \frac{(w_{ribtop} - w_{ribbot})}{h_{sheet}} * X_{u,max}$ $\quad$ $w_{xu} = 178$mm
- $A_{xu} = \left(\frac{(w_{xu} + w_{ribbot})}{2} * X_{u,max}\right) * \frac{1000}{effective\ width}$ $\quad$ $A_{xu} = 29800\ \text{mm}^2/\text{m}$

A_{xu} equals the area of the maximum compressive area in the concrete rib.

- $A_{mesh} = ((\frac{Mesh_{\emptyset}}{2})^2 * \frac{\pi*1000}{Mesh_{ctc}})$ $\quad$ $A_{mesh} = 335.1\ \text{mm}^2/\text{m}$
- $A_{Add,R} = ((\frac{Add,R_{\emptyset}}{2})^2 * \frac{\pi*1000}{Add,R_{ctc}})$ $\quad$ $A_{Add,R} = 753\ \text{mm}^2/\text{m}$
- $A_{total} = A_{mesh} + A_{Add,R}$ $\quad$ $A_{total} = 1089\text{mm}^2/\text{m}$

Here the total area of reinforcement above the support is calculated, resulting in a maximum force the reinforcement can withstand:

- $N_s = A_{total} * f_{yd}$ $\quad$ $N_s = 544$ kN/m

N_s equals is the maximum force the reinforcement can withstand.

- $N_c = A_{xu} * fcd * 0.85 + (w_{ribbot} + h_{sheet,effective}) * S350 * \frac{1000}{effective\ width}$ $\quad$ $N_c = 1060\ kN/m$
- $N_{max} = \min(N_s; N_c)$ $\quad$ $N_{max} = 544\ k\text{N/m}$

The maximum force is now known and the exact compressive region must be found to determine the lever arm and finally the hogging bending moment resistance:
Height compression zone = y.

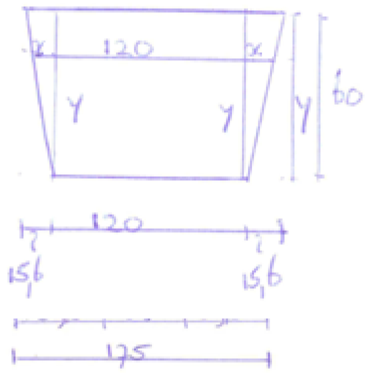


Figure B-7 Compression zone ComFlor75

- $x = \frac{w_{ribtop} - w_{ribbot}}{2*h_{sheet}} * y$ $\quad$ x= 1.02 *y mm
- $N_{max} - \left(w_{ribbot} + \frac{h_{sheet}}{4}\right) * S350 = \frac{f_{cm}*0.85*1000}{effective\ width} * (w_{ribbot} + x) * y$

Solving this equation gives a total height [y] of 28.43 mm

So the width of x is equal to $x = 1.02 * y = 13.74\ mm$

The total height of the compression zone with the given reinforcement is now known.
Using this data, the centre of gravity from the bottom of the rib can be calculated:

- $X = \frac{(\frac{w_{ribbot}*y^2}{2}+\frac{x*y^2*2}{3})}{(w_{ribbot}+x)*y}$ $\qquad X = 14.7\ mm$

All data is now known and the final Hogging Bending Moment capacity per meter slab can be calculated:

- $M_U = (N_s * (d_{rib} - X)$ $\qquad M_U = 57.34 \text{kNm/m}$

With the spread in concrete compressive strength and steel tensile strength this can vary between:

The upper limit: $M_U = 65\ kNm/m \quad (f_c = 55\frac{N}{mm^2} \& f_{yd} = 550\ N/mm^2)$
The lower limit: $M_U = 52.86\ kNm/m \quad (f_c = 30\frac{N}{mm^2} \& f_{yd} = 475\ N/mm^2)$

This is around 9% spread above and below the calculated moment, but based on the strength of the test cubes, the moment resistance can be calculated just before the specimen is tested.

B.2.3. Vertical Shear Resistance ComFlor 75

To determine the shear capacity of the ComFlor75 a small difference is present compared to the ComFlor210, namely: the ComFlor210 has a very high and stiff rib, with a limited width and a very slender flange with a big width compared to the rib. Meaning not all force will be transmitted to the rib and the flange contributes with a separate resistance. In the ComFlor75 the distance between each rib is a lot smaller, so the slope of the ribs is extended to the top of the floor. This way the floor can be seen as multiple high ribs next to each other with a higher shear resistance.

The distribution of the forces can be seen in Figure B-9below:

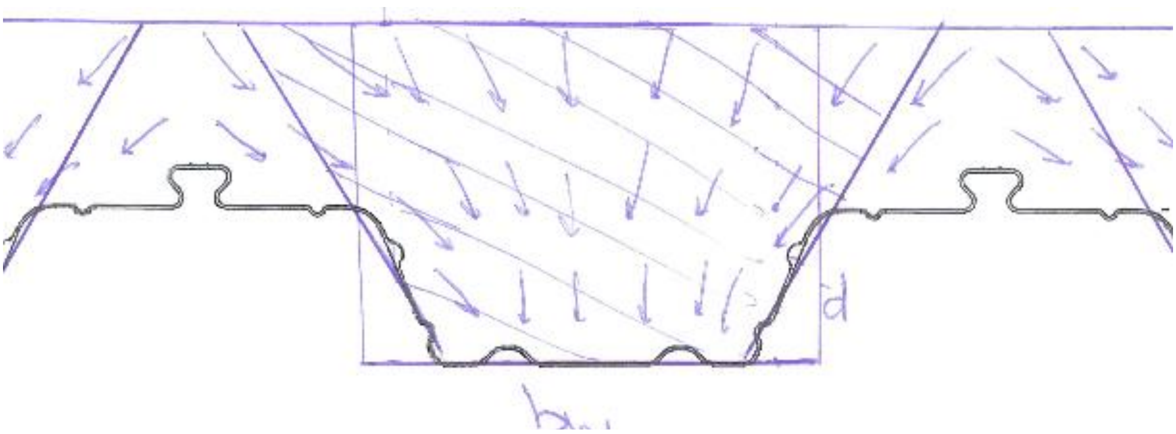


Figure B-8Force Flow of the ComFlor75

In Table 24below an overview of the properties is given:

Table 24 Properties ComFlor75 V Calculation

Name	Unit	Name	Unit
$f_{ck} = 30$	N/mm²/	$f_{cd} = f_{ck}/\gamma_c$	N/mm²
$f_{cm} = 38$	N/mm²	$C_{rd,c} = 0.15$	
$Mesh_{ctc} = 150$	mm	$c = 24$	mm
$Mesh_{\emptyset} = 8$	mm	$h = 150$	mm
$Add, R_{ctc} = 150$	mm	$w_{ribbot} = 120$	mm
$Add, R_{\emptyset} = 12$	mm	$w_{ribtop} = 265$	mm
$f_{yk} = 500$	N/mm²	$h_{sheet} = 60$	mm
$f_{yd} = f_{yk}/\gamma_s$	N/mm²	$effective\ width = 300$	mm
$\gamma_s = 1$		$\gamma_c = 1$	
$S_{sheet} = 350$	N/mm²	$V_{su,75} = 55$	kN/m

The total capacity will be calculated by a summation of the contribution parts, the sheet and the concrete floor. In order to calculate the vertical shear resistance the same procedure is used as by calculation of the ComFlor210.

- $\mathrm{b_w} = \frac{\mathrm{w_{ribbot}+w_{ribtop}}}{2} * \frac{1000}{\text{effective width}}$ $\mathrm{b_w}$ =642 mm/m

This width is almost 2,5 times higher compared to the ComFlor210 due to the small center to center distance of each rib.

- $\rho_{rib} = \min(\frac{A_{sl,rib}}{b_w * d_{rib}}; 0.02)$ $\rho_{rib} = 0.0141$

This is the reinforcement ratio in the tensile region. In practice this can be increased by placing an extra mesh right on top of the sheet. One extra mesh would increase $\rho_{rib}$by 0.005 to a total of 0.019. This is a 30% increase of tensile reinforcement, leading to almost 13% extra shear capacity.

- $k_{rib} = \min(1 + \sqrt{\frac{200}{d_{rib}}}; 2)$ $k_{rib} = 2$
- $v_{c,rib} = C_{Rd,c} k_{rib} \left(100\, \rho_{rib} f_{cm}\right)^{\frac{1}{3}}$ $v_{c,rib} = 1.15\ \mathrm{N/mm^2}$

In this formula the $C_{Rd,c}$and $f_{cm}$are used as explained at the beginning of B.1.3 [14].

- $v_{min} = 0.035 * k_{rib}{}^{3/2} * f_{cm}{}^{0.5}$ v_{min} =0.626 N/mm²

This is the minimum shear stress without reinforcement. All together this leads to a vertical shear resistance of the floor of:

- $V_{U,rib} = \max\left(v_{c,rib}; v_{min}\right) * \ bw * d_{rib}$ $V_{U,rib} = 88.6\ kN/m$

The vertical shear resistance of the sheet V_{su}(ComFlor75) is higher compared to the ComFlor210 sheet, due to the smaller center to center distance between each rib. This also gives more effective steel to withstand the shear force leading to a vertical shear capacity of the sheet of 55 kN/m.

In 2012 multiple test were carried out in London to determine the contribution of different parts of the ComFlor75. From the test results on test specimen L12 & L13 to be found in Table 2b & Table 4.2 of the test done in London [3] , a ComFlor75 was tested with and without deck. 2 tests were carried out on each, both giving a difference in vertical shear capacity of 61 kN/m& 53 kN/m respectively.

This $V_{U,rib}$ would represent a resistance according to the Eurocode, but not using a higher rib. This however excludes the contribution of the steel sheeting. To include this to the resistance this resistance will be added to the resistance of the rib:

Final Vertical Shear Resistance [kNm/m]

- $V_U = V_{U,rib} + V_{su,75}$ $\qquad$ $V_U = 143\text{kN/m}$

The tests done in London [3] are based on a floor loaded by a positive bending moment. This means in London the sheet and rib reinforcement were under tension, while in the case of a negative moment the ribs are in compression and the top mesh and reinforcement are in tension. According to the formulae provided by different codes, this should result in a different capacity. Nevertheless the vertical shear capacities derived from these tests with a height of 150mm, using C20/25, a single mesh on top and a rebar in the rib vary between 130 kN/m and 200 kN/m when applying a thicker rib bar or 1.2mm sheet instead of 0.9. All higher compared to the theoretical capacity. This could be due to the low $^a/_d$ ratio. In case of the test done in London, this $^a/_d$ ratio was 2.25. So in reality this capacity could be lower in case of some direct transmission of the applied force towards the support. The test rig used is shown in Figure B-9.

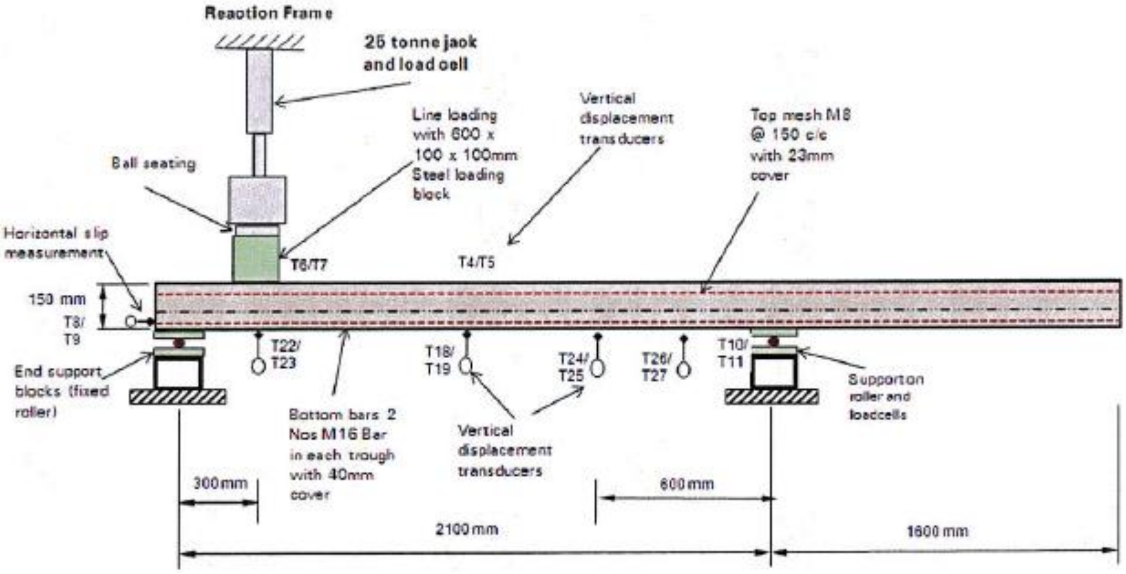


Figure B-9 Test setup London

This means the slab is loaded by a positive moment. As the shear capacity of concrete is weak in the tensile zone. This zone is now reinforced by the M16 bar in the rib as well as the ComFlor75 sheet. This is significant more steel compared to the mesh reinforcement in the top as in our case. Another reason that could suggest this capacity of 200 kN/m might not be reached.

B.3. Uncertainties vertical shear resistance.

There are a couple factors that give some uncertainties. They are listed below:

Summation of the contributing parts

The summation of the different contributing parts, would suggest all contributing parts will reach their full capacity at the same time / and have sufficient deformation capacity to redistribute the force. This is however debatable as shear is a brittle failure mode. Also the influence on one another is not included.

Value of Crdc for the calculation of the shear stress

As explained in the thesis of dr.Ir. Yuguang Yang [14], the real value for the Crdc without safety factors is under discussion. Using Crdc equal to 0.15 would be a good estimation to approach reality. The value lies somewhere between 0.14-.163.

Ratio of the reinforcement

In this calculation the stiffer parts will carry more loads and therefore have a higher reinforcement ratio. Also here an uncertainty to this relation is present.

Capacity of the ComFlor210 sheeting in combination with the concrete present

According to the German research:[18], the vertical shear resistance of the ComFlor210 with a thickness of 1.25 mm is 25 kN/m in the construction phase. Question is how this value changes in case of summation with the concrete in the final phase.

Influence of the reinforcement bar in the rib

In practice a reinforcement bar is placed inside the rib to assure fire safety. This rebar is however, not included in any of the previous calculations. First of all, because it does not continue through the integrated beam and second of all, because it is in the compressive side of the rib. So it is not included in the vertical shear capacity. This could improve the shear a little, but will not cause a mayor difference.

All together these factors lead to an upper and lower limit of the maximum vertical shear capacity. These factors combined could give a huge difference between mean and upper or lower limit.

Conclusion

The gap between the value found through the Eurocode calculation, tests done to determine the contribution of the sheet [11] and the calculation shown above is very big. Due to this big range between the lower and upper limit of the vertical shear resistance, it is of interest to cast a third specimen. For this specimen the same parameters, concrete, reinforcement and time of casting should be used. Only with a short span, in order to test it on vertical shear only. By casting specimen based on the upper and lower limit of the vertical shear capacity. The support can be adjusted to the right position once the resistance is known.

APPENDIX C. Determine test setup

C.1. Area of concern in practice

In order to find a test setup that reflects situations in practice, a case study "town hall - Almelo" is reviewed, checking common resistances in combination with its spans. This gives a distributed load needed to form plastic hinges and interaction between the high vertical shear and hogging bending moment occurring can become governing.

The ComFlor210 and the ComFlor75, both have their typical applications. The way of construction, integration, span length and need for props determine the choice of slab to be used. Each span, combined with a function will result in a critical failure mode.

For example:

-In case of a high point load at mid span, in case of a big span, it is likely to fail in bending. (or in case of a slab, punching shear could be governing).
-In case of a relatively high moment at a small span, longitudinal shear could become governing.
-In case of a short span with a high load, shear failure could become governing.
-In combination with a high load and span length a failure mode could govern on a combination of a hogging bending moment and vertical shear.

In case of the case study that raised questions. A ComFlor210 slab was used continuous over 3 supports with a span of 6 meter each as shown in Figure C-1.

Figure C-1 Situation in Case Study "Town hall - Almelo", two spans of 6000 mm.

At the right distributed load, this could lead to both a high negative bending moment and vertical shear at the mid support. Especially after redistribution and yielding of the top reinforcement, due to the extra deflection a plastic hinge can be formed at mid span to increase the maximum distributed load and thereby the occurring shear force.

In this chapter the failure mode based on a present hogging bending moment and a vertical shear force will be further discussed. At which spans and what distributed load is it likely to occur in case of full plastic redistribution. For this case study reinforcement is used based on the case study, instead of the values used in previous chapter.

C.1.1. Plastic behavior and M-V combination at a realistic loading pattern

Common lengths for the ComFlor210 are 3600 mm – 7200mm, where propping is needed from a length of 5400 mm. In case of the case study "Town hall - Almelo", where the questions were raised, the mechanical scheme was according Figure C-2.

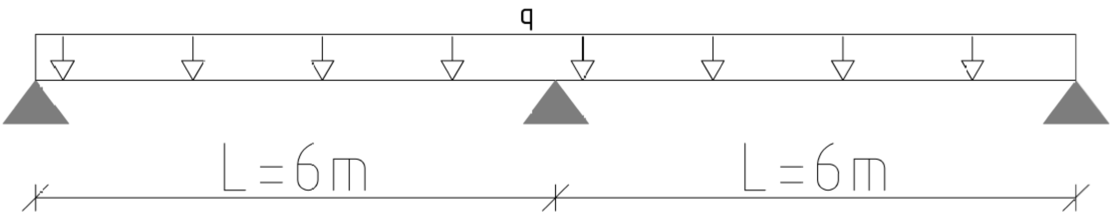


Figure C-2 Mechanical scheme according to case study "Town hall - Almelo"

In case of full plastic redistribution, plastic hinges could occur at 3 locations, one at the mid support and two at the middle of each span. In the case of Figure C-2, the point of yielding would occur first at the intermediate support. While yielding, this plastic hinge provides deformation capacity to redistribute the extra load towards the hinges in the middle of the span. If no limits are given to the maximum redistribution of loads, a final moment line with 3 plastic hinges will occur as shown in Figure C-3.

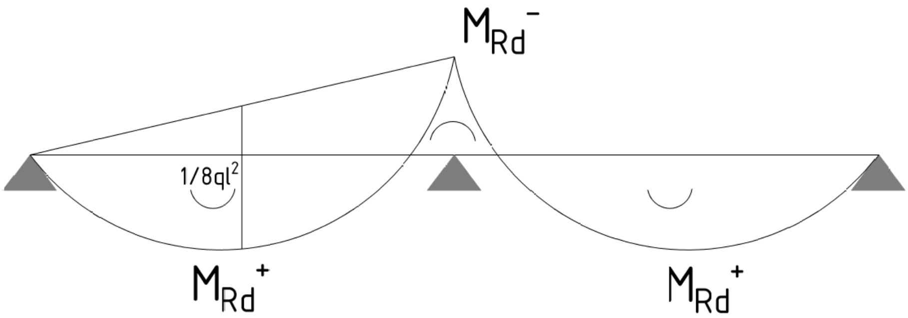


Figure C-3 Final M-line after formation of 3 plastic hinges.

Once all 3 hinges have developed, a mechanism is formed and the construction can no longer receive extra load and will fail. This is therefore the maximum distributed load that the construction could withstand, in case no limits are given to the maximum deformation and redistribution of loads. The value of the occurring moments with respect to the distributed load 'q' as shown in Figure C-3 can be written as:

$$\frac{1}{8ql^2} = \frac{1}{2} * {M_{Rk}}^- + {M_{Rk}}^+$$

Rewriting this equation results in:

$$\frac{4 * {M_{Rk}}^- + 8 * {M_{Rk}}^+}{l^2} = q$$

The only step left to determine the maximum possible distributed load the construction can take is the calculation of the ${M_{Rk}}^-$ and ${M_{Rk}}^+$ of the applied slab system.

Calculations M_{Rk}^{-}&M_{Rk}^{+} case study to determine maximum possible distributed load.

For the calculation of the maximum possible distributed load data is used based on the case study “Town hall - Almelo” to get realistic values. The calculation is done with mean values and without any material or safety factors. A handwritten calculation is shown below in Figure C-4.

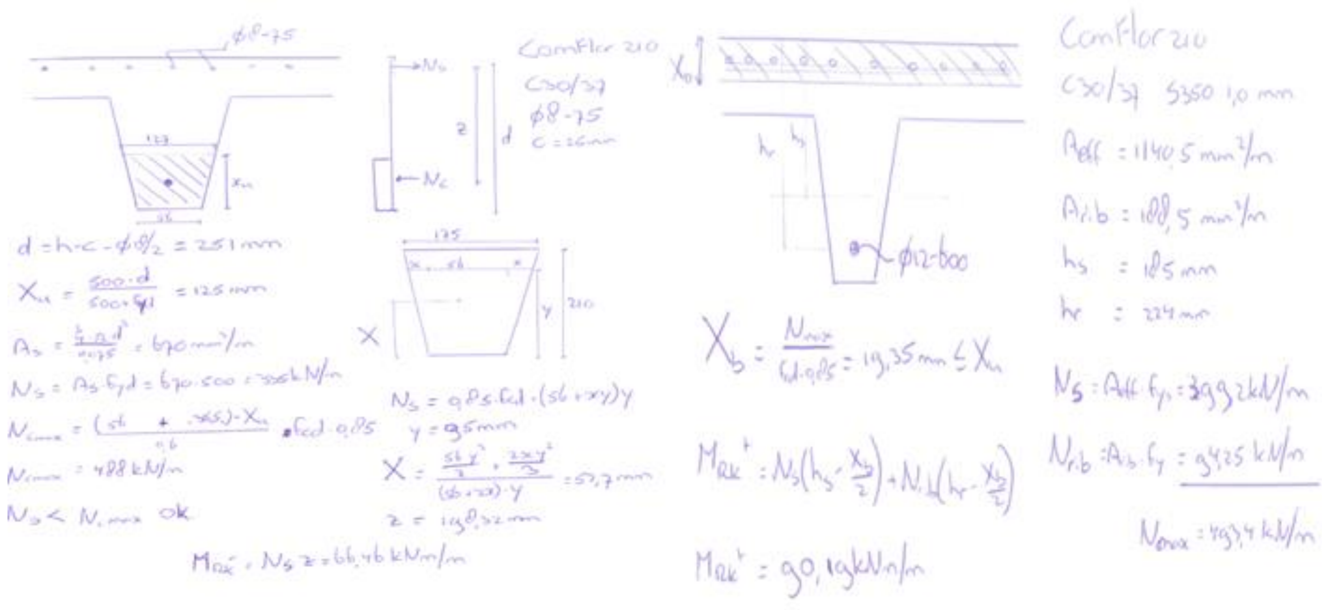


Figure C-4 handwritten calculations to get approximate values to determine the maximum possible distributed load before a mechanism is formed.

This results in the following values: $M_{Rk}^{-} = 66.5\,\frac{\text{kNm}}{\text{m}}$ and $M_{Rk}^{+} = 90.2\,\frac{\text{kNm}}{\text{m}}$.
Including these resistances in the formula below, results in the distributed loads [q] needed to form this mechanism.

$$q = \frac{4 * M_{Rk}^{-} + 8 * M_{Rk}^{+}}{l^2}$$

Table 25 Overview distributed load needed to form a mechanism at different spans

Span length [m]	Distributed load, q [kN/m]
3.6	76.19
5.4	33.86
6	27.43
7.2	19.05

As Table 25 shows, for spans smaller then 5,4 meter the distributed load needed to form both plastic hinges at the intermediate support and at mid span is way above normal values. It therefore will fail in vertical shear only or by means of a different failure mechanism.

These distributed loads give an extra vertical shear load at the intermediate support.
The vertical shear present at the intermediate support can be calculated as shown below:

$$V = \frac{ql}{2} + \frac{M_{Rk}^{-}}{l}$$

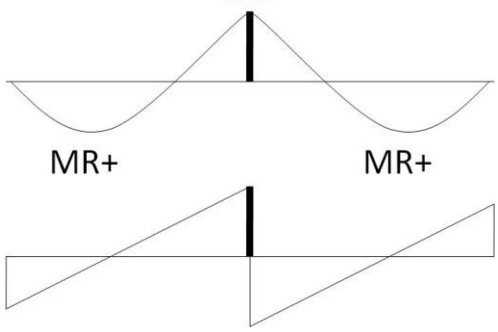


Figure C-5 M-V lines in case of a full plastic redistribution

The maximum shear force at the intermediate support (Figure C-5), are shown in Table 26 below.

Span length [m]	Vertical shear force at the intermediate support [kN]
3.6	103.73
5.4	93.36
6	89.81
7.2	77.8

Table 26 Maximum present vertical shear force at the intermediate support in case of full redistribution at different span lengths

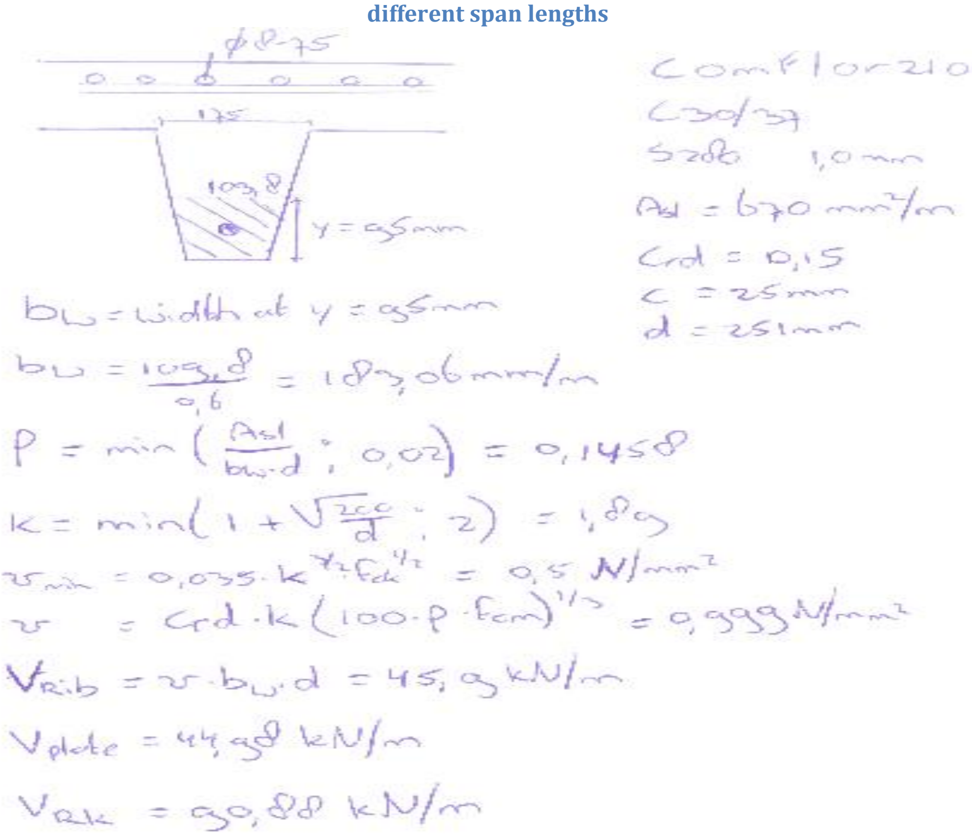


Figure C-6 handwritten approximation of the vertical shear resistance of the ComFlor210 slab used in the case study "Town hall - Almelo"

The occurring vertical shear forces in case of a plastic mechanism are shown in Table 26. In this situation the M_{Rk}^{-} is loaded to its maximum and if the vertical shear force occurring is also near a 100%, interaction can play a role. In order to check this, an approximation must be made of the vertical shear resistance of the composite slab. This is done in Figure C-6.
The vertical shear resistance is equal to approximately 91 kN/m.

In the case of L = 6 meter, the occurring vertical shear force is equal to 89.81 kN, therefore approaching the 100% of both the hogging bending moment and the vertical shear force.

Based on the calculated distributed load of 27 kN/m and full redistribution a situation is created where interaction could play a role. The only thing left is to compare the calculated distributed load with the distributed load used in practice.

In case of the case study “Town hall - Almelo” the distributed loads are as following:

Self weight floor	$q_{sw} =$	2.8 kN/m^2
Dead load	$q_{dl} =$	2.0 kN/m^2
Variable load	$q_{vl} =$	5.0 kN/m^2
	Total load:	9.8 kN/m^2

As all material factors have been removed by the calculation of the resistances, the distributed load should also be multiplied by 2 in order to compare these values. Resulting in a total distributed load of 20 kN/m.
In the case of a 6 meter span a distributed load of 27 kN/m is needed to form 3 plastic hinges and form a mechanism. Comparing the distributed loads from Table 25, it can be concluded that interaction starts playing a role at spans of L > 5,4 meter. In the case study “Town hall - Almelo” the span was equal to 6 meter, using only 20% redistribution instead of full redistribution will close the gap between the maximum possible distributed load of 27 kN/m and the applied 20 kN/m (without safety factors).

Conclusion
In order to find a test setup that refers to the practical values as discussed in this chapter, a test setup must be found that can represent the situation of a continuous beam, representing a mechanical scheme as shown in Figure C-5, zoomed in at the location of the intermediate support. The combination between full vertical shear and hogging bending moment is most likely to occur if the span length is between 6 and 7 meter. If the span is smaller the distributed load is unrealistically high and will not occur in practice. However these values are based on the shear resistance calculated according to the current accepted standards, while the real vertical shear resistance could be much higher.

C.2. Possible test setup 1

To simulate the M-V combination that occurs in practice at an intermediate support, a cantilever is used as shown in Figure C-7. The force will be simulated by a cylinder pushing downwards on the test specimen. Once the floor is pushed against the steel beams, the slab will have lost contact with the temporary support.

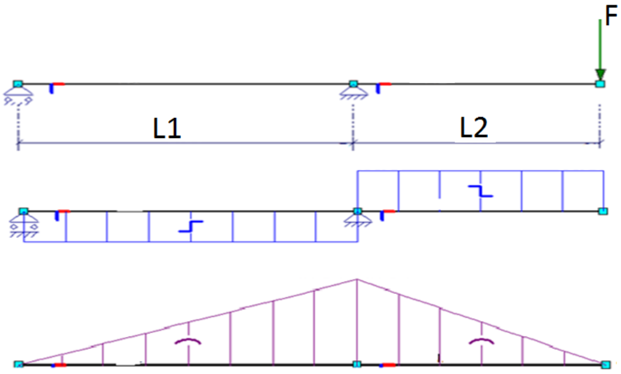


Figure C-7 Test setup simply supported

This way the M & V – line will be as shown in Figure C-7.
The maximum moment and shear force will be above the support. With the integrated beam system (ComFlor210) the cross section right above the applied force F will be very stiff and strong, shifting the critical cross section a bit to the side, just next to the steel beam. The reduced moment is represented by $F * (L_2 - x)$ where x is the distance from the centre of the cylinder.

Possible failure modes

- Vertical Shear Failure
- Hogging bending moment Failure
- Longitudinal Shear (not wanted)

During the test series of Patrick van Erp in May 2016 in Stevin II laboratory at the TU Delft, most of the specimens failed in longitudinal shear. The floor that had an End girder failed in bending. Even though these specimens were loaded by a positive bending moment, it is not wanted to have a failure on longitudinal shear and the ComFlor210 steel sheets should be anchored in the concrete to prevent unwanted loss of composite behavior.

Positive and negative aspects of test setup 1

Positive	Negative
+ Short specimens	- Complicated supports
+ No positive bending moment	-Always a moment present
+ Simple relation between V and M	-M-V relation depends on specimen length or location of loading
+ Statically determined	
+ No other plastic hinges can occur	

The longitudinal shear failure mode can be prevented. For small specimens, this setup could be an option. Changing the supports is not so labor intensive. As the length of the specimen depends on the M-V relation, all require a different location of applying the load.

Ratio between moment and shearComFlor210

The setup gives a relation between the force applied and the occurring vertical shear / hogging bending moment.

$V = F$

$M = F * L_2$

Combining these two formulae gives:

$\alpha * M = * \beta * V * L_2$

Where α relates to the percentage of $M_U$applied and β to the percentage of $V_U$applied.

With this formula, the only variable is the length L of the specimen. The resistances of the ComFlor210 are based on the initial calculations done in APPENDIX B:

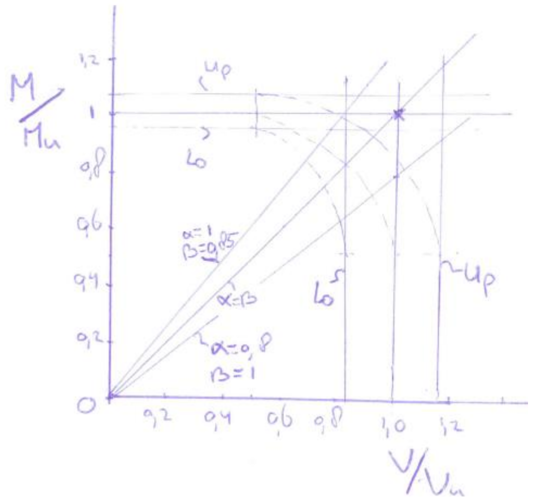


Figure C-8 M-V interaction diagram, including uncertainties

The hogging bending moment capacity $M_U = 85.4\ kNm/m$

The vertical shear resistance$V_U = 128.8\ kN/m$.

Using these values and the relation between the M_U& V_U ,this leads to a specific span (L_2) [m]. The red cells show when the forces is within the $^a/_d \leq 3$ region. This would mean a part of the applied load is directly transmitted towards the support and the failure load cannot be related to the V_U.

Table 27 Test setup 1 specimen length and M-V ratios ComFlor210

Moment(α)	Shear (β)	α/β	L2 (100% Vu)	L 2(125% Vu)	L 2(150%*Vu)	Min L2
0.3	1	0.3	0.20	0.16	0.13	0.75
0.4	1	0.4	0.26	0.21	0.17	0.75
0.5	1	0.5	0.33	0.26	0.22	0.75
0.6	1	0.6	0.39	0.31	0.26	0.75
0.7	1	0.7	0.46	0.37	0.31	0.75
0.8	1	0.8	0.52	0.42	0.35	0.75
0.9	1	0.9	0.59	0.47	0.39	0.75
1	1	1.0	0.65	0.52	0.44	0.75
1	0.9	1.1	0.73	0.58	0.48	0.75
1	0.8	1.3	0.82	0.65	0.54	0.75
1	0.7	1.4	0.93	0.75	0.62	0.75
1	0.6	1.7	1.09	0.87	0.73	0.75
1	0.5	2.0	1.31	1.05	0.87	0.75

This test setup is not valid to determine the shear resistance for the ComFlor210. It will most likely not fail in shear but on bending. Or if it fails on shear, it either already passed 50% of the moment capacity leading to possible interaction or the span is so small that part of the force is directly going to the support. It is however a suitable setup to determine interaction based on specimens failing on bending with different shear forces present.

Ratio between moment and shear ComFlor75

The same relation between the moment and shear applies:

$\alpha * M = \beta * V * L_2$

The hogging bending moment capacity $M_U = 57.4\ kNm/m$

The vertical shear resistance$V_U = 143\ kN/m$.

The same applies as for the ComFlor210:

Using these values and the relation between the M_U& $V_U$this leads to a specific span (L_2) [m]. The red cells show when the forces is within the $^a/_d \leq 3$ region. This would mean a part of the applied load is directly transmitted towards the support and the failure load cannot be related to the V_U.

Table 28 Test setup 1 specimen length and M-V ratios ComFlor75

Moment(α)	Shear (β)	α/β	L (100% Vu)	L (125% Vu)	L (150%*Vu)	Min L
0.3	1	0.3	0.12	0.09	0.08	0.33
0.4	1	0.4	0.16	0.13	0.10	0.33
0.5	1	0.5	0.20	0.16	0.13	0.33
0.6	1	0.6	0.24	0.19	0.16	0.33
0.7	1	0.7	0.28	0.22	0.18	0.33
0.8	1	0.8	0.31	0.25	0.21	0.33
0.9	1	0.9	0.35	0.28	0.24	0.33
1	1	1.0	0.39	0.31	0.26	0.33
1	0.9	1.1	0.44	0.35	0.29	0.33
1	0.8	1.3	0.49	0.39	0.33	0.33
1	0.7	1.4	0.56	0.45	0.37	0.33
1	0.6	1.7	0.66	0.52	0.44	0.33
1	0.5	2.0	0.79	0.63	0.52	0.33

Also with the ComFlor75 with extra top reinforcement to increase the $M_U$to 57 kNm/m the minimum moment is still around 70%. This is lower due to the lower height and therefore the $^a/_d$ ratio requires a smaller distance from the support generating a smaller moment. Same conclusion applies as with the ComFlor210.

C.3. Possible test setup 2

The other option is a setup with 2 spans, 3 supports and to apply a force by use of a spreader beam as shown in Figure C-9. In this setup all test specimens can have the same length. Only the position of applying the force by means of a spreader beam has to be changed between experiments.

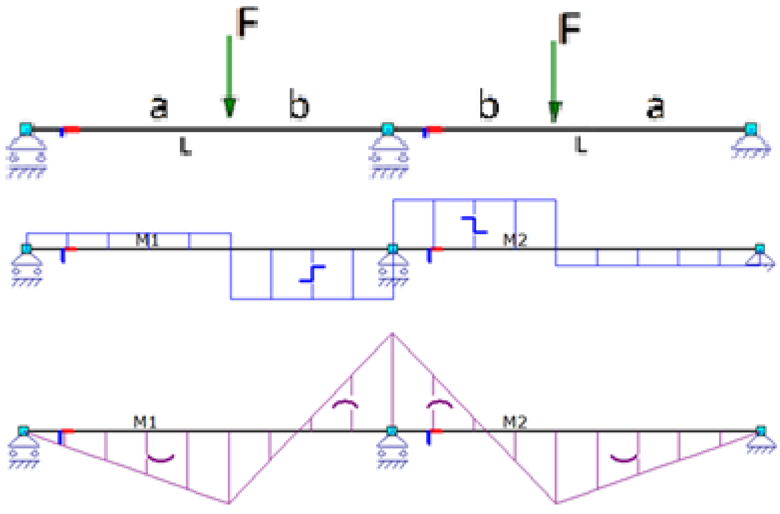


Figure C-9 M-V line test setup 2

The relation between the span, position of load and the occurring vertical shear and negative bending moment can be derived using basic mechanics. Where the angle φA is the angle left of the support and φB the angle right of the support.

$$\varphi A = \frac{Fa_1b_1(L+a_1)}{6EIL} + \frac{MB * L}{3EI} \qquad \varphi B = \frac{Fa_2b_2(L+b_2)}{6EIL} + \frac{MB * L}{3EI}$$

Combining these two equations as $\varphi A = \varphi B$ for **small angles** results in the following hogging bending moment MB, **(this however neglects the fact that concrete is not a linear elastic material after cracking):**

$$MB = \frac{Fa_1b_1(L+a_1)}{2L^2}$$

The vertical shear at the intermediate support can be calculated the following equation:

$$VB = \frac{Fa_1}{L} + \frac{MB}{L}$$

By means of substitution this will lead to:

$$VB = \frac{Fa_1}{L} + \frac{Fa_1b_1(L+a_1)}{2L^3}$$

Both the moment and the shear force will increase linearly by increase of the force of the cylinder F. The position of the force is restricted to a region depending on possible direct transmission of the force to the support and other failure mechanisms occurring. For instance a plastic hinge occurring right under the load due to the deformation capacity of the intermediate support.

Possible failure modes

- Vertical Shear Failure
- Hogging bending moment Failure
- Longitudinal Shear (not wanted)
- Flexural bending moment (not wanted)

The longitudinal shear can be avoided by not exceeding half of the span and by applying End girder to avoid loss of composite behavior and unwanted failure modes.

The Flexural bending moment elastic or plastic due to the ductility of the plastic hinge occurring above the intermediate support, can be avoided by also limiting the position of the applied force F. Especially in the case of the ComFlor210 this means the force should not be further away than 40% of the span and in case of the ComFlor75 it should not exceed half of the span.

Positive	**Negative**
+ All specimens same length	- Possible failure mechanisms increase
+ Easily adjustable	- Long specimens
+ Equal to application in practice	- Low M_{pl}, means F very close to the support
+M-V relation depends on easily adjustable point loads.	-Influences length on the M to the power 3
	-Concrete is not a linear elastic material E changes during tests

The minimum and maximum distances from the support are as follows:

Minimum:
- ComFlor210 >2d = 520 mm ≈ 600 mm.
- ComFlor75 >2d = 244 mm ≈ 250 mm.

Maximum:
- ComFlor210 <1440 mm≈ 1400 mm
- ComFlor75 <.1350 mm ≈ 1300 mm

Length of the specimen

In case of test setup 2, for each floor type 2 tests will be conducted. This means different positions of the applied load, will give two points in the interaction diagram. It is therefore of essence to choose, based on the first shear capacity test, 2 ratios that give the most insight. In Figure C-10 a schematic overview is given for the distances in test setup 2.

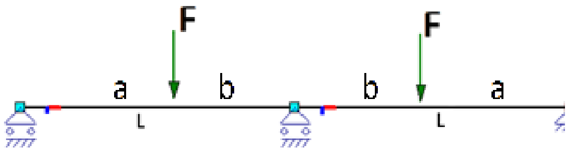


Figure C-10 Schematic model test setup 2

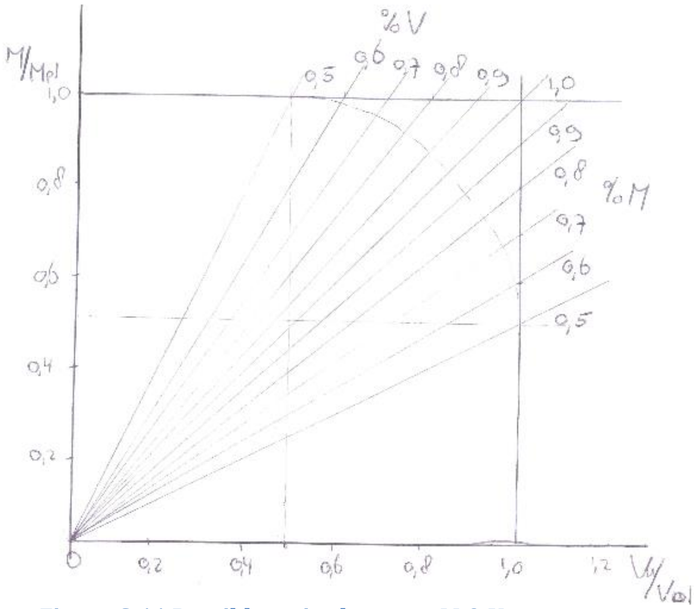


Figure C-11 Possible ratios between M & V.

Each ratio is related to a different distance a and b as shown in the figure above. With the limits determined for each floor, this will exclude certain ratios as can be seen on next page.

Below different ratios as shown in Figure C-11are given:

Table 29 M-V ratios of the ComFlor210/75 for setup 2, limited by a minimum a/d ratio and a maximum due to unwanted failure modes.

ComFlor 210	L=3600mm	ComFlor75	L=1800mm
V 100%	M 60%	V 100%	M 60%
F [kN/m]	123	F [kN/m]	137
a [mm]	3073	a [mm]	1460
b [mm]	526	b [mm]	339
V 100%	M 75%	V 100%	M 75%
F [kN/m]	129	F [kN/m]	147
a [mm]	2927	a [mm]	1362
b [mm]	672	b [mm]	437
V 100%	M 80%	V 100%	M 85%
F [kN/m]	131	F [kN/m]	155
a [mm]	2878	a [mm]	1292
b [mm]	722	b [mm]	507
V 100%	M 100%	V 100%	M 100%
F [kN/m]	141	F [kN/m]	170
a [mm]	2665	a [mm]	1180
b [mm]	934	b [mm]	620
V 90%	M 100%	V 90%	M 100%
F [kN/m]	132	F [kN/m]	165
a [mm]	2507	a [mm]	1060
b [mm]	1092	b [mm]	740
V 80%	M 100%	V 0.8%	M 100%
F [kN/m]	125	F [kN/m]	172
a [mm]	2281	a [mm]	870
b [mm]	1318	b [mm]	930

The ratios M/V as provided above show only certain combination are possible. For the ComFlor210 this varies between 70% Moment resistance and 90% Vertical Shear resistance.
For the ComFlor75 this varies between 65% Moment resistance and 65% Vertical Shear resistance.

As the influence of the exact resistances is big, these distances will change once the first test is conducted and the concrete test cubes have been tested. At that stage the final resistances will be determined and the exact location of the applied point loads will be decided.

Conclusion test setup 2

This setup might be an easy way of testing, as only the cylinders applying the force musts change position. But as it is statically undetermined and has a minimum and maximum range to the interaction diagram of about 70% moment – 90% shear. The range of possible test combination is very limited. Therefore test setup 1 is preferred over test setup 2.

C.4. Possible test setup 3.

As previous test setups both have a problem with low $^{a}/_{d}$ ratios and a pure vertical shear test is difficult. In order to increase this $^{a}/_{d}$ ratio the moment capacity must be increased. As all parameters have already been improved (amount of reinforcement, concrete quality etc.), something else must be done. If test setup 1 would be loaded by pure positive bending, this would give some sort of prestressed moment, increasing the hogging bending moment capacity. As applying pure bending is difficult, and all specimens must have a cantilever in order to be practical a following setup is suggested: (Figure C-12)

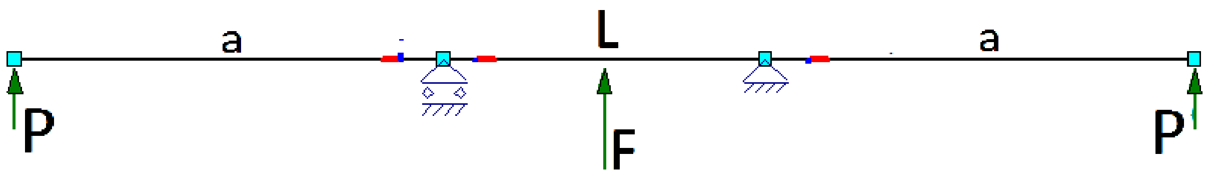


Figure C-12 Test setup 3, setup 1 with an applied positive moment

By first applying force P at a certain distance "a", this will apply a positive moment, while F generated a negative moment. Both shear forces will be separated due to the supports. This way the relation between moment and shear force can be controlled as shown in Figure C-13.

The moment M in between the support will be equal to: $M = \frac{1}{4} * F * L - P * a$ and the vertical shear force will be as in setup 1: $V = 0.5 * F$

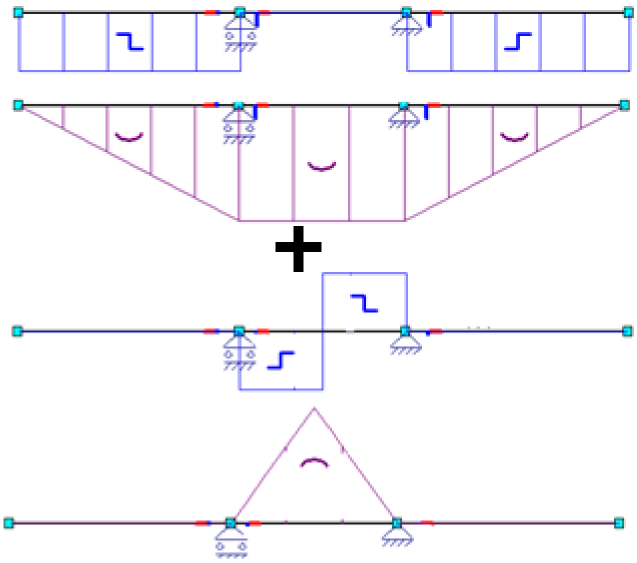


Figure C-13 M - V line of separate forces P and F

Submitting these equations in one another gives:
$\alpha * M_U = 0.5 * \beta * V_U * L - P * a$

Where:
L is restricted by the $^{a}/_{d}$ $ratio \geq 2$ at least.
P*a is restricted by the positive bending moment capacity of the slab. This should avoid any yielding and therefore be on the safe side.

Also the combination between the positive moment and the applied force F should not cause a failure at the support due to interaction between M – V.
This would lead to the following moment and shear line:

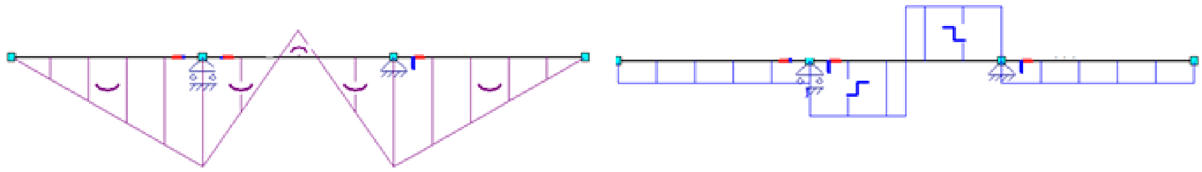


Figure C-14 M-V line test setup 3

Possible failure modes

- Vertical Shear Failure
- Hogging bending moment Failure
- Longitudinal Shear (not wanted)
- Positive bending moment failure at support (not wanted)
- Combination positive moment and vertical shear failure at support (not wanted)

To avoid the last 3 failure modes from happening, the end must be anchored (Longitudinal Shear failure) and the positive moment applied should not exceed the design value of the moment. Also where possible this capacity could be increased by for instance a thicker rib bar. This will not increase the hogging bending moment capacity as the top reinforcement will be governing. The combination of a positive moment and vertical shear at the support is an issue. This would imply extra shear reinforcement locally to avoid failure due to interaction.

Positive and negative aspects of test setup 3

Positive	**Negative**
+ Equal specimens, change in moment applied and span L	- New possible failure modes
+ Freedom in αM_U allowing a shear test to be possible	-ComFlor210 integrated beam cannot take up any positive moment, can be countered by stepwise loading
+ Simple relation between V and M	- Complicated supports
+ Same setup for all slabs / tests	-New failure mechanism of V-M interaction at support

The applied moment depends on the capacities of each floor. In general the positive bending moment capacity is higher due to a bigger compressive zone (concrete slab) and the higher amount of steel in tension (steel sheet + reinforcement inside the rib). Though a combination of this applied moment and the present shear force could become governing. In case of a normal concrete beam, this can be solved by using stirrups inside the entire beam, except from the desired failure cross section. This however in case of a composite slab is not as simple as only a limited space is available for some stirrups.

Ratio between moment and shear ComFlor210

The hogging bending moment capacity $M_U = 85.4\ kNm/m$
The vertical shear resistance$V_U = 128.8\ kN/m$.
The positive bending moment capacity $M\ =\ 100\ kNm/m$using aØ12-600 in the rib.

It is possible to either change the span length L or the applied force P*a. This follows in the following 2 tables. One with P*a equal to 50 kNm/m and the other of 75 kNm/m.

Figure C-15 Possible spans in relation to M-V interaction ratios ComFlor210

Applied moment equal to 50 kNm/m					
Moment(α)	Shear (β)	α/β	L (100% Vu)	L (125% Vu)	L (150%*Vu)
0.1	1	0.1	0.94	0.75	0.62
0.2	1	0.2	1.07	0.86	0.71
0.3	1	0.3	1.21	0.97	0.81
0.4	1	0.4	1.34	1.08	0.90
0.5	1	0.5	1.48	1.18	0.99
0.6	1	0.6	1.62	1.29	1.08
0.7	1	0.7	1.75	1.40	1.17
0.8	1	0.8	1.89	1.51	1.26
0.9	1	0.9	2.02	1.62	1.35
1	1	1.0	2.16	1.73	1.44
1	0.9	1.1	2.40	1.92	1.60
1	0.8	1.3	2.70	2.16	1.80
1	0.7	1.4	3.09	2.47	2.06
1	0.6	1.7	3.60	2.88	2.40
1	0.5	2.0	4.32	3.46	2.88

Applied moment equal to 75 kNm/m					
Moment(α)	Shear (β)	α / β	L (100% Vu)	L (125% Vu)	L (150%*Vu)
0.1	1	0.1	0.49	0.39	0.33
0.2	1	0.2	0.57	0.46	0.38
0.3	1	0.3	0.65	0.52	0.43
0.4	1	0.4	0.73	0.58	0.49
0.5	1	0.5	0.81	0.65	0.54
0.6	1	0.6	0.89	0.71	0.59
0.7	1	0.7	0.96	0.77	0.64
0.8	1	0.8	1.04	0.83	0.70
0.9	1	0.9	1.12	0.90	0.75
1	1	1.0	1.20	0.96	0.80
1	0.9	1.1	1.33	1.07	0.89
1	0.8	1.3	1.50	1.20	1.00
1	0.7	1.4	1.71	1.37	1.14
1	0.6	1.7	2.00	1.60	1.33
1	0.5	2.0	2.40	1.92	1.60

The red indicates a ratio of $^a/_d \leq 2$. Depending on the maximum moment allowed to be applied a relation up to only 10% of the maximum M_U is possible. This however requires extra caution at the supports avoiding a failure due to positive bending and a combination with the applied shear force.

Ratio between moment and shear ComFlor75

For the ComFlor75 the same applies with different capacities. Due to a lower height it is easier to satisfy the $^{a}/_{d}$ratio. Applying different moments gives the relations between span and $M_U - V_U$ratios as shown in Figure C-16.

Figure C-16 Possible spans in relation to M-V interaction ratios ComFlor75

Applied moment equal to 30 kNm/m						Applied moment equal to 40 kNm/m					
Moment(α)	Shear (β)	α/β	L (100% Vu)	L (125% Vu)	L (150%*Vu)	Moment(α)	Shear (β)	α/β	L (100% Vu)	L (125% Vu)	L (150%*Vu)
0.1	1	0.1	0.49	0.39	0.33	0.1	1	0.1	0.63	0.50	0.42
0.2	1	0.2	0.57	0.46	0.38	0.2	1	0.2	0.71	0.57	0.47
0.3	1	0.3	0.65	0.52	0.43	0.3	1	0.3	0.79	0.63	0.53
0.4	1	0.4	0.73	0.58	0.49	0.4	1	0.4	0.87	0.69	0.58
0.5	1	0.5	0.81	0.65	0.54	0.5	1	0.5	0.94	0.76	0.63
0.6	1	0.6	0.89	0.71	0.59	0.6	1	0.6	1.02	0.82	0.68
0.7	1	0.7	0.96	0.77	0.64	0.7	1	0.7	1.10	0.88	0.73
0.8	1	0.8	1.04	0.83	0.70	0.8	1	0.8	1.18	0.94	0.79
0.9	1	0.9	1.12	0.90	0.75	0.9	1	0.9	1.26	1.01	0.84
1	1	1.0	1.20	0.96	0.80	1	1	1.0	1.34	1.07	0.89
1	0.9	1.1	1.33	1.07	0.89	1	0.9	1.1	1.49	1.19	0.99
1	0.8	1.3	1.50	1.20	1.00	1	0.8	1.3	1.67	1.34	1.11
1	0.7	1.4	1.71	1.37	1.14	1	0.7	1.4	1.91	1.53	1.27
1	0.6	1.7	2.00	1.60	1.33	1	0.6	1.7	2.23	1.78	1.49
1	0.5	2.0	2.40	1.92	1.60	1	0.5	2.0	2.68	2.14	1.78

When limiting the moment to 33% this would provide a good setup to determine the vertical shear capacity and followed by other tests.

Conclusion test setup 3

It is a test setup with a very wide range, namely: 10% moment as a minimum and all other combinations are possible. There is however a downside, namely the more expensive test setup and the new possible failure mode which is harder to tackle. The local increase of vertical shear resistance to control the location of failure is more complicated compared to a concrete beam specimen. The ComFlor steel sheet is in the way to locally control the vertical shear resistance.

APPENDIX D. Material tests

D.1. Reinforcement tensile tests:

On the 15th of September 2016 three tensile tests have been conducted at the TU Delft laboratory to determine the grade of the applied reinforcement. Below in Figure D-1 the test rig used to determine the tensile strength of the reinforcement bars is shown.

a) Test rig before testing the individual reinforcement bars

b) Reinforcement bar at the moment of failure.

Figure D-1 Material tests on the applied reinforcement bars.

In total three random bars of reinforcement have been tested. The findings can be found on the next page.

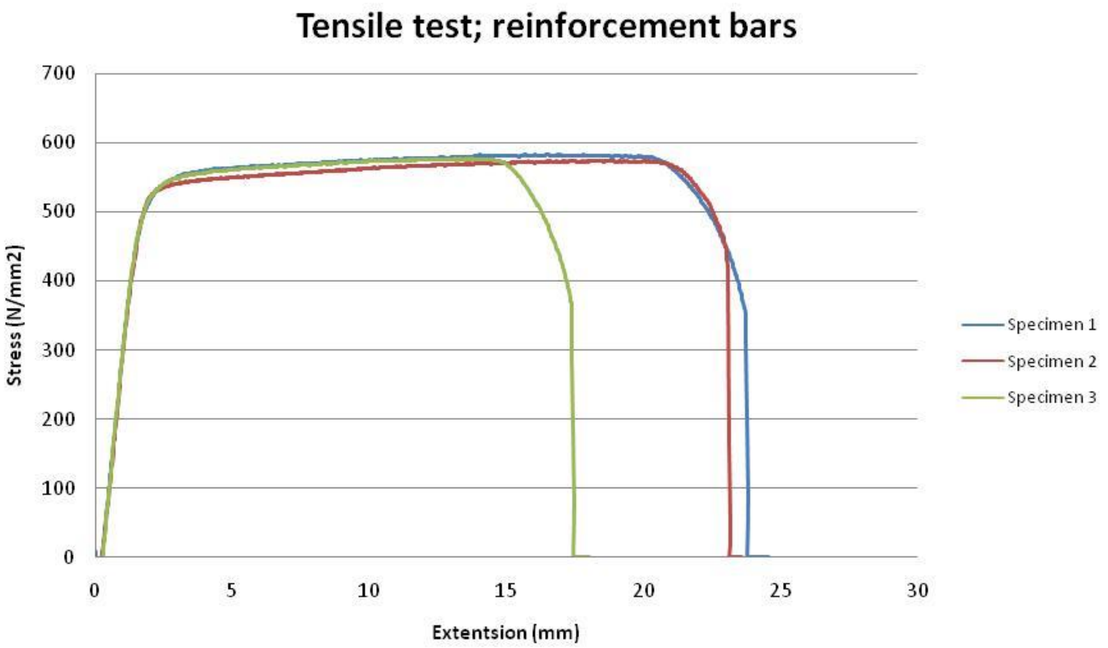


Figure D-2 Tensile tests reinforcement steel.

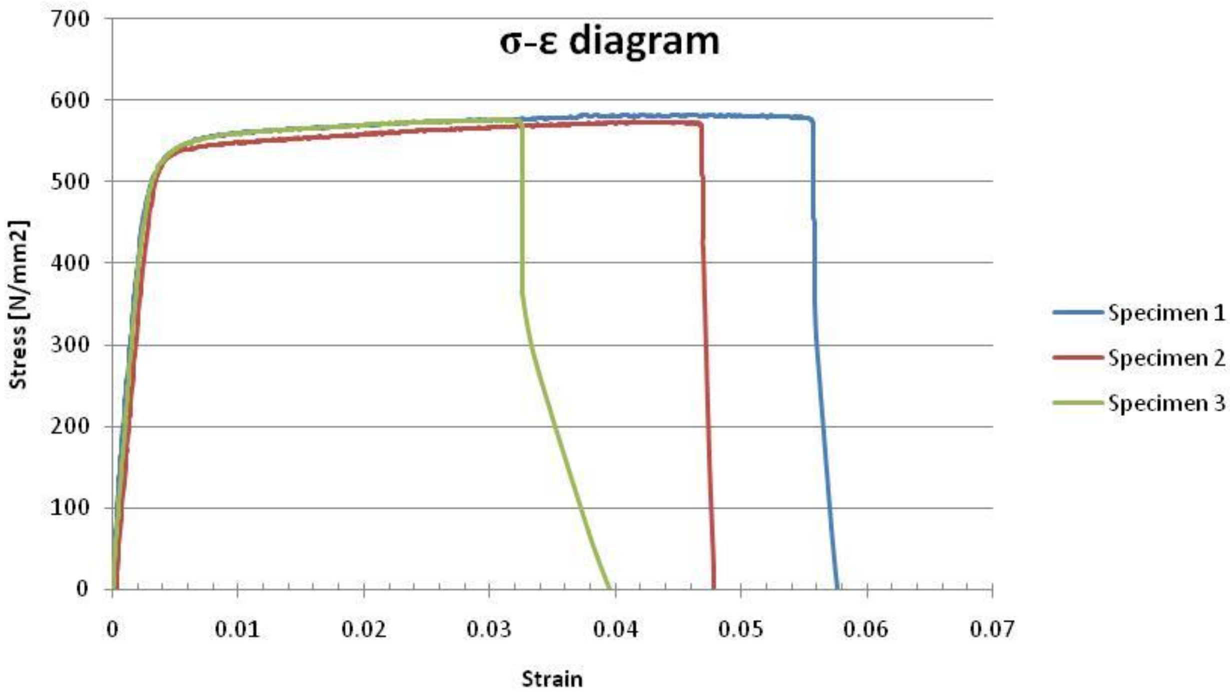


Figure D-3 Stress - strain relation, reinforcement steel.

All three bars follow the same path and start yielding at the same moment. The difference in maximum stress is due to a different moment of failure as the ductility is different for each bar. Up to 11 mm strain (based on the total extension of the specimen), all three bars follow the same path, "specimen 3" fails earlier as the first two specimens.

Table 30 Maximum force on each reinforcement bar.

Name	Max Force [kN]	0,2% yield stress [N/mm^2]	Max Stress [N/mm²]
Test 1	29.28	541	582.50
Test 2	28.87	536	574.30
Test 3	29.02	543	577.29
Average	29	540	578

Table 30shows the maximum force applied on each reinforcement bar at the moment of failure. In practice the specimen will fail once the first bar has broken. As the difference between the maximum values is small, the lowest value will be used to determine the negative bending moment as well as the vertical shear resistances of the specimens.
This implies a $f_u = 574\ N/mm^2$.

D.2. Testing of the concrete cubes

The concrete cubes have been tested right before or after the test. A total of 15 cubes (150*150 mm) have been casted. Below in Figure D-4 some pictures of the testing have been added.

a) Test cubes right after casting

*b) Window of the fourth cube tested, showing the maximum force and stress based on 150*150 mm area.*

c) Concrete cube about to be crushed, tested.

Figure D-4 Test cubes and test rig for defining the compressive strength of the concrete

Before each test three concrete cubes were tested in the concrete laboratory to determine the exact compressive strength on the day of testing. The test specimen are poured with C30/37 in order to provide sufficient compressive strength in the small concrete ribs needed to avoid crushing of the concrete and not passing the maximum compression zone height “Xu”. Below an overview is given of the results:

Table 31 Compressive strength of the concrete compression tests.

	Date / Time	Day	kN	N/mm²	mm²	Average (N/mm²)
Cube 1	04-10-16	21	949	42.18	22500	
Cube 2	04-10-16	21	863.5	38.38	22500	39.85
Cube 3	04-10-16	21	877.6	39.00	22500	
Cube 4	06-10-16	23	934.7	41.54	22500	41.54
Cube 5	12-10-16	29	835.3	41.24	20250	
Cube 6	13-10-16	29	956.8	42.52	22500	41.76
Cube 7	13-10-16	29	917.7	41.52	22100	
Cube 8	19-10-16	35	961.3	42.72	22500	42.093
Cube 9	19-10-16	35	932.9	41.46	22500	
Cube 10	20-10-16	36	981.4	43.61	22500	
Cube 11	20-10-16	36	964.2	42.85	22500	43.60
Cube 12	20-10-16	36	997.8	44.34	22500	

Using the data obtained in Table 31, a scatter has been plotted with the obtained test results. This can be seen in Figure D-5below:

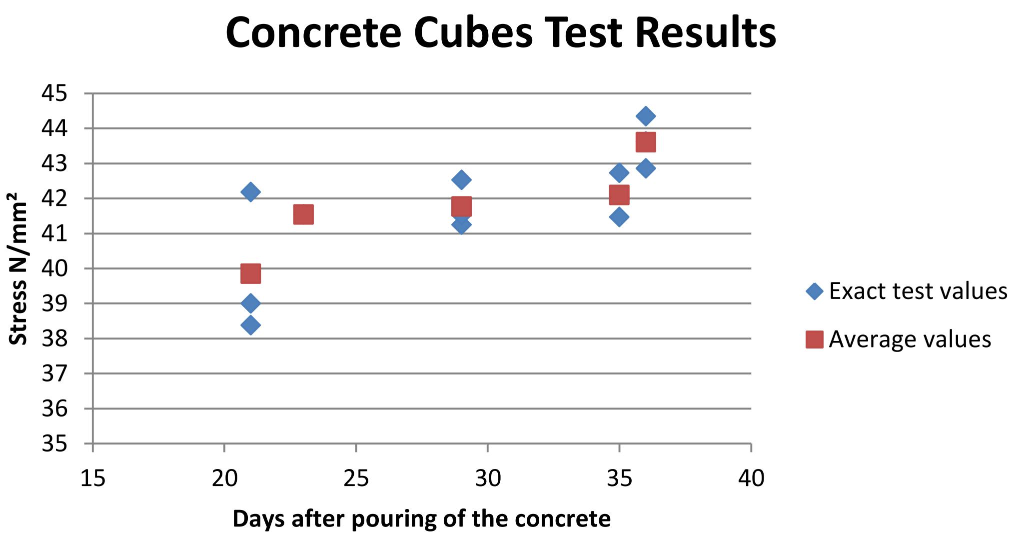
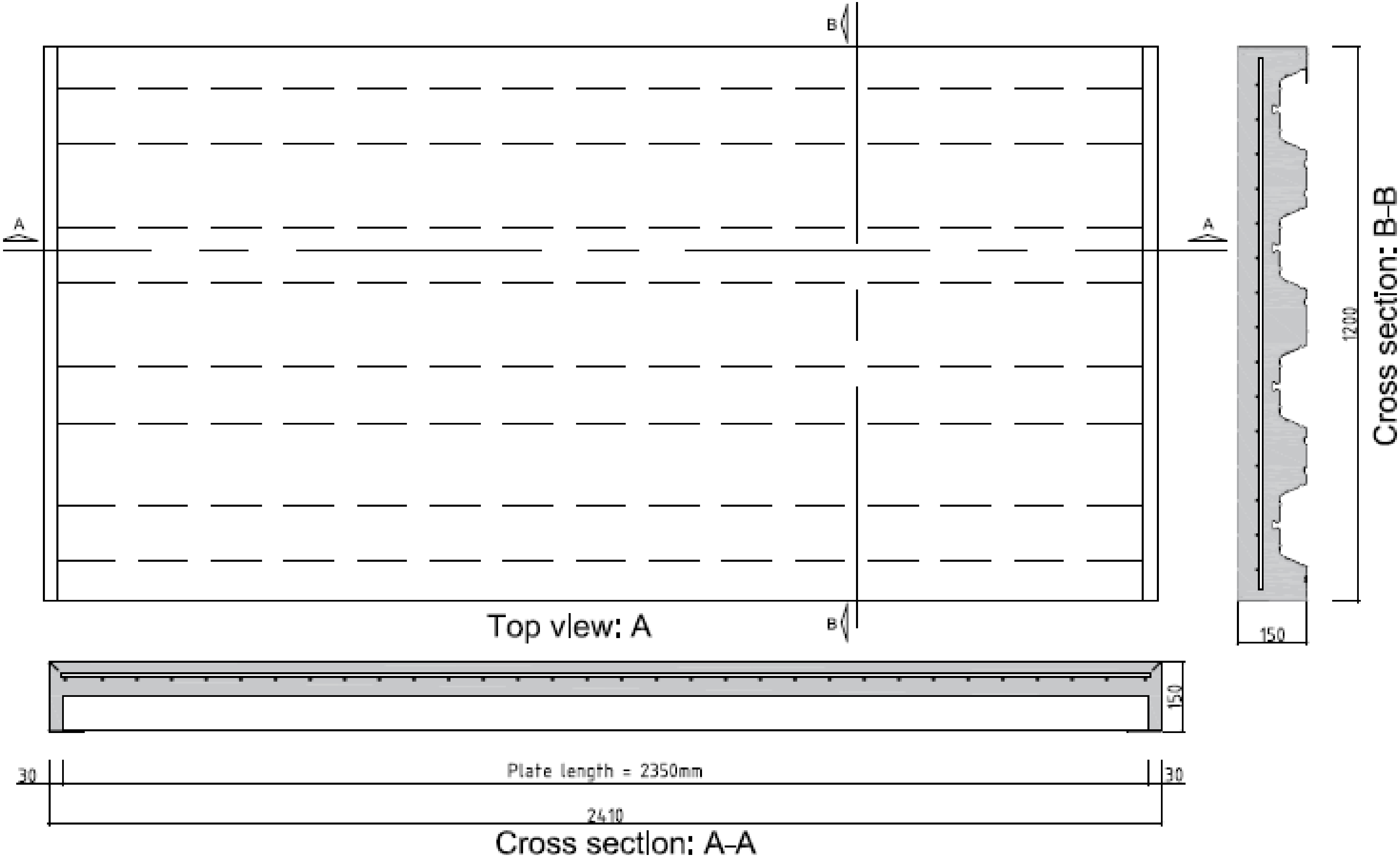


Figure D-5 Concrete cubes test results. Stress [N/mm2] at the days of testing.

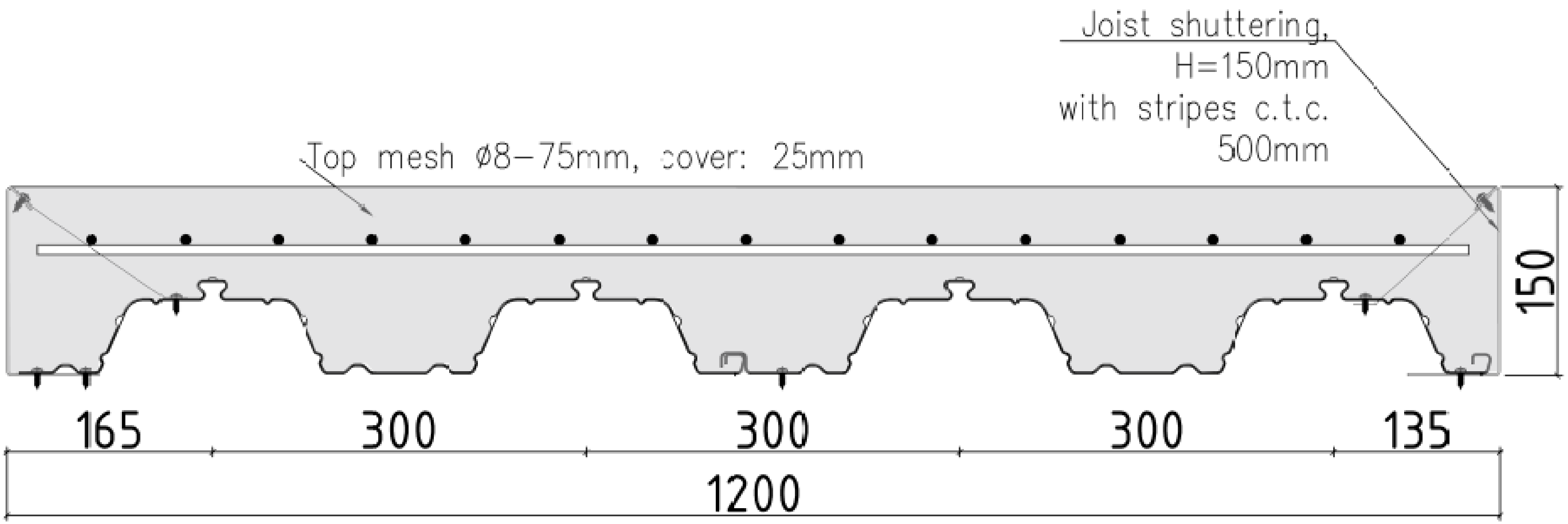
Due to incorrect pouring of the concrete into some of the molds, only 12 cubes were available in total, an average of two concrete cubes per test specimen. The compressive strength did not vary a lot, so this did not have influence on the calculation of the resistances.

APPENDIX E. Test Specimens

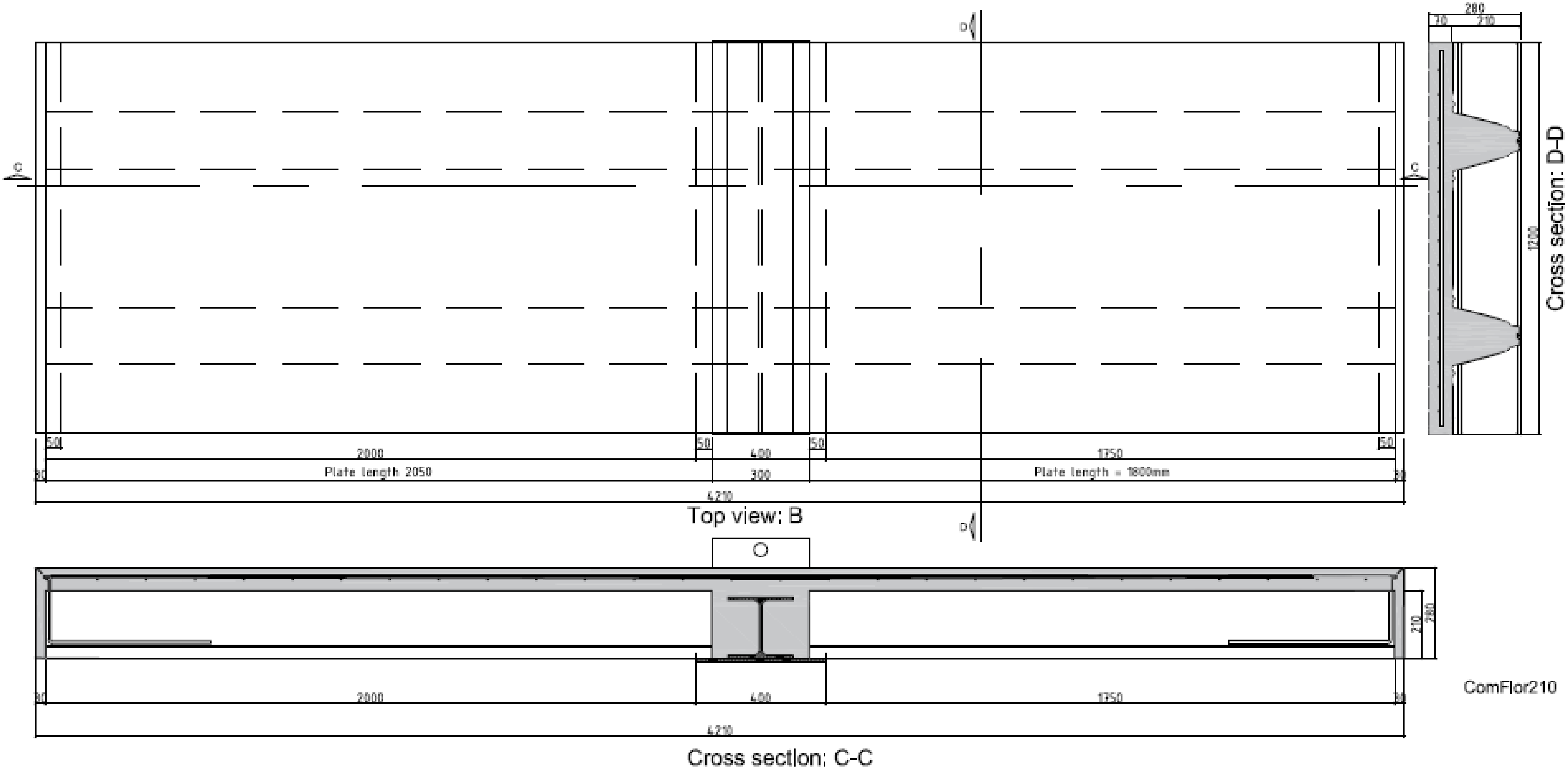
E.1. Drawings



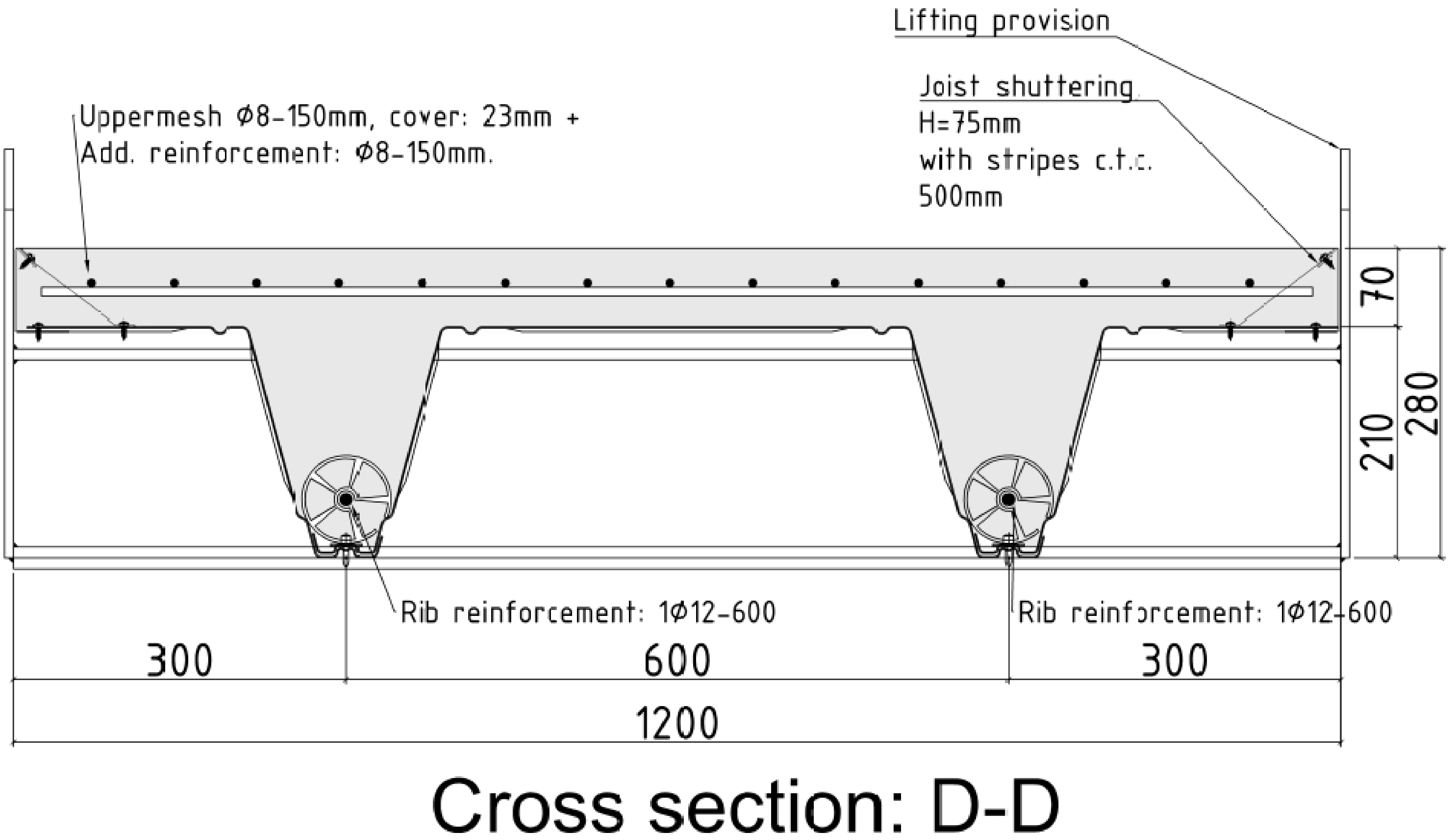
E.1.1. Overview ComFlor75 specimens



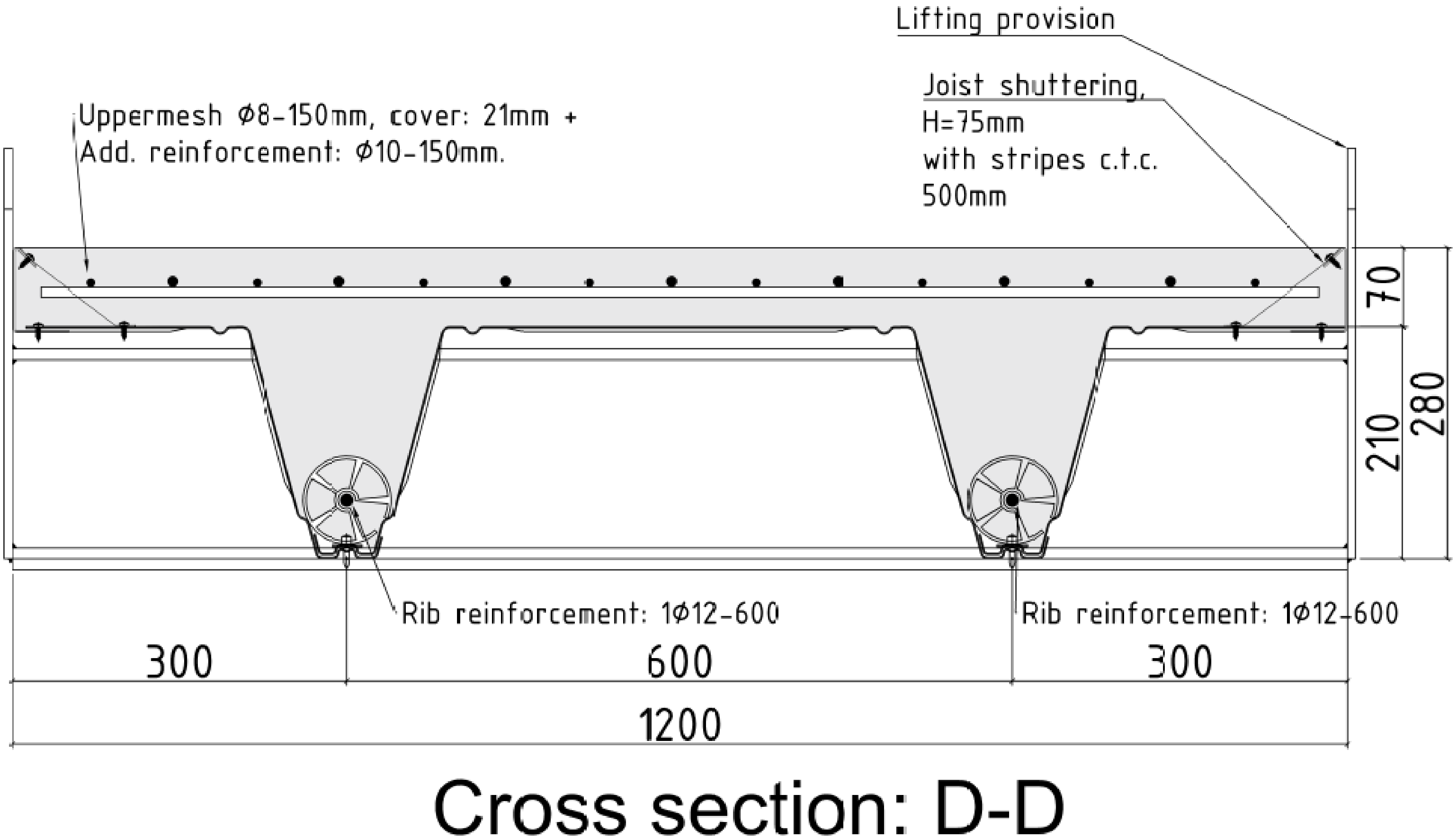
E.1.2. ComFlor75 Cross Section B-B



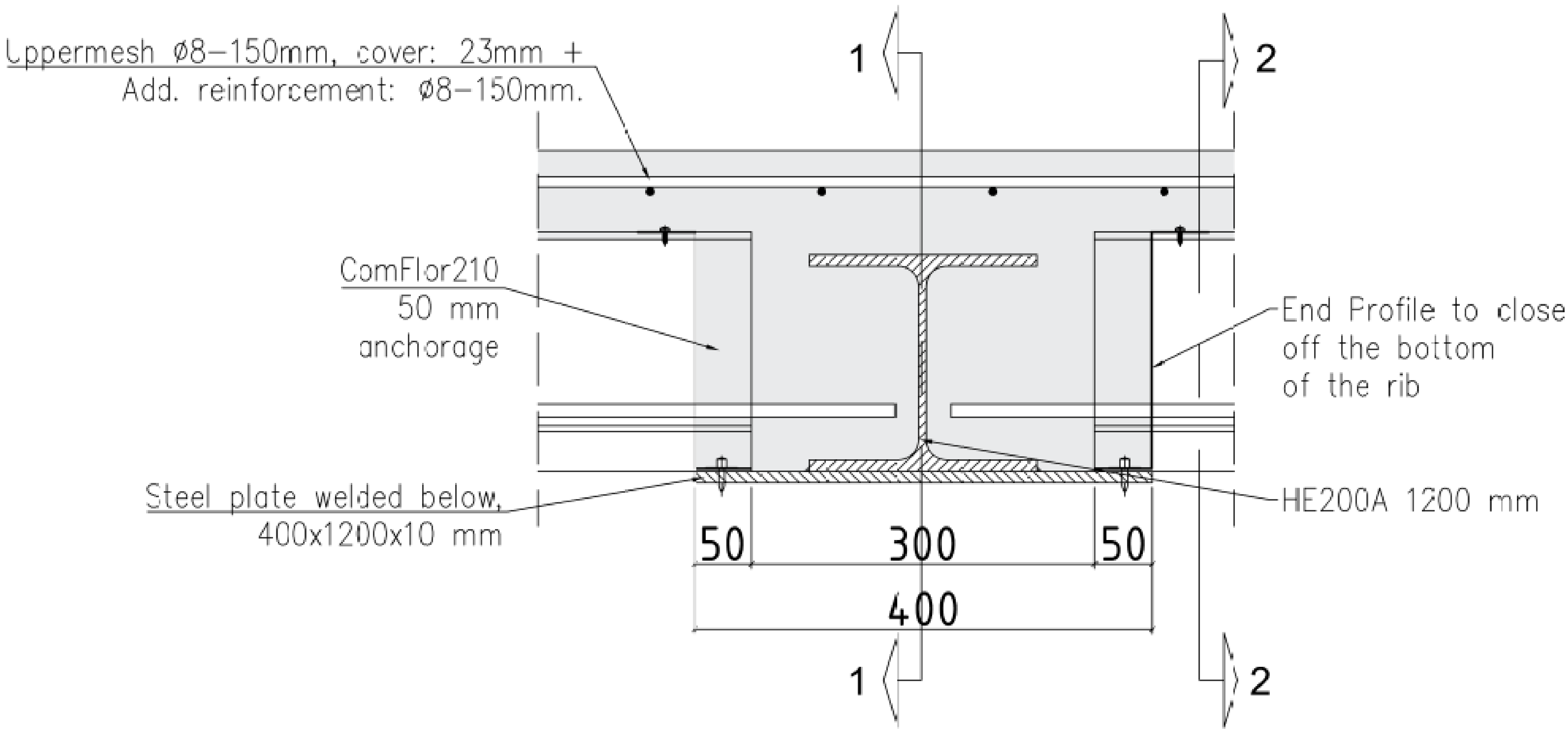
E.1.3. Overview ComFlor210 Specimens



E.1.4. ComFlor210 Cross Section D-D, Specimen 1 & 2



E.1.5. ComFlor210 Cross Section D-D, Specimen 3



E.1.6. Support detail ComFlor210, reinforcement conform test specimens 1 & 2.

E.2. Calculations

E.2.1. Test specimen 1 & 2: ComFlor210 Ø8-75

A detailed calculation is made for the resistances of test specimen 1&2 in this chapter. All ComFlor210 are conducted with an integrated HE200A steel beam. This means two different cross sections near the support can be critical. These cross sections can be found in Figure E 1. A more detailed cross section is found under chapter E.1 Drawings in the appendix.

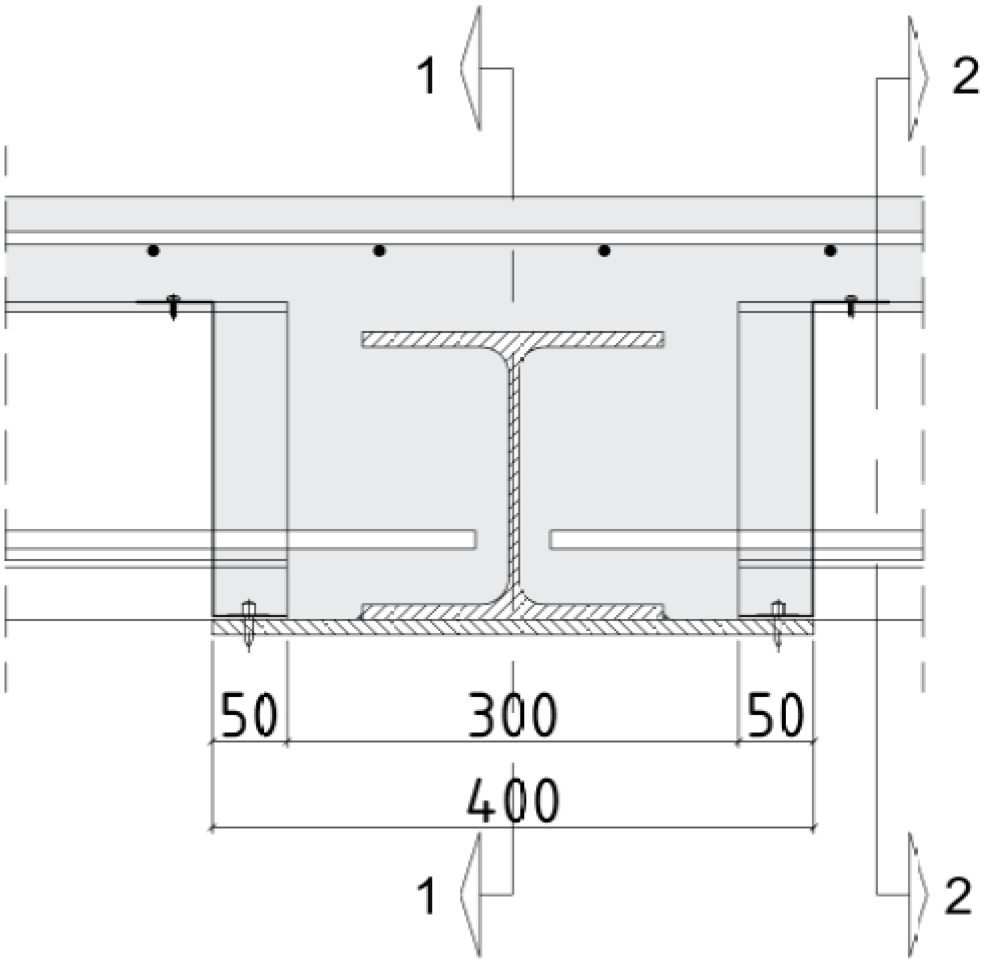


Figure E 1 Support detail test specimens 1,2 & 3

E.2.1.1. Negative bending moment resistance (cross section 1)

Cross section 1 is located at the centre line of the HE200A beam. This is also the location where the highest negative bending moment will occur. The difference is the size of the compressive region in the concrete. This calculation will calculate the resistance of the entire specimen. (Width of 1200 mm)

$$d = h - c - \frac{Ø}{2} = 280 - 23 - \frac{8}{2} = 253mm$$

$X_u = \frac{500 * d}{500 + f_u} = \frac{500 * 253}{500 + 574} = 117.8mm$

$A_{sl} = \frac{1}{4} * \pi * d^2 * 15 = 754\ mm^2 = 628mm^2/m$ (a total of 15 bars were placed over a width of 1200mm)

$N_s = A_{sl} * f_u = 432.7kN$

N_s is the maximum force the reinforcement can withstand, this will make equilibrium with the concrete poured around the HE200A steel beam.

$N_s = 0.85 * f_{cm} * b * y$

$432.7 * 10^3 = 0.85 * 41.5 * 1200 * y$

$y = 10.22mm$

$z = d - \frac{y}{2} = 253 - \frac{10.22}{2} = 247.9mm$

$M_{Rd}{}^{-} = N_s * z = 432.79 * 239.9 = 107.29kNm$ (for a width of 1200mm)

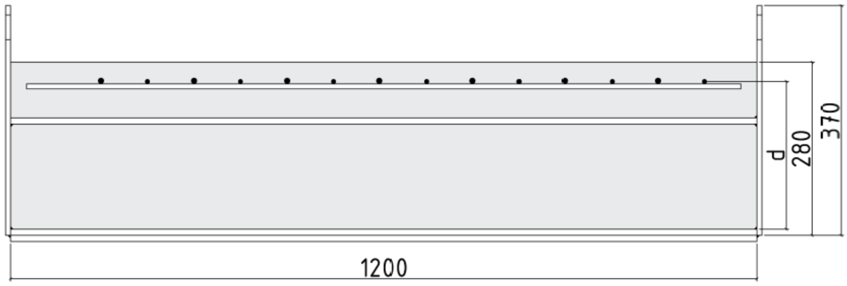


Figure E 2 Cross Section 1

This is the negative bending moment resistance at the location right above the support and at the centerline of the HE200A beam.

E.2.1.2. *Negative bending moment resistance (cross section 2)*

As shown in Figure E-2 the negative bending moment resistance is an internal force generated by the reinforcement steel under tension [N_s] and the concrete ribs under compression [N_c], separated by an internal lever arm [z]. The negative bending moment resistance follows from:

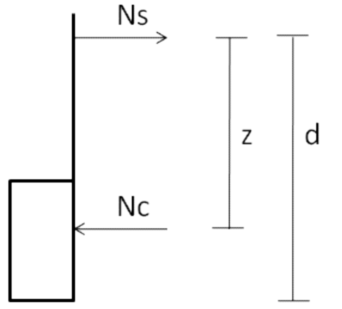


Figure E-2 Internal Forces making equilibrium

$$d = h - c - \frac{Ø}{2} = 280 - 23 - \frac{8}{2} = 253\,mm$$

$$X_u = \frac{500*d}{500+f_u} = \frac{500*253}{500+574} = 117.8\,mm$$

X_u is the maximum height of the compression zone (see Figure E-3). This is used later on to calculate the maximum compressive capacity of the concrete ribs.

$A_{sl} = \frac{1}{4} * \pi * d^2 * 15 = 754\,mm^2$ (a total of 15 bars were placed over a width of 1200mm)

$N_s = A_{sl} * f_u = 754 * 574 = 432.8kN$

N_s is the maximum force the reinforcement can withstand, this must be lower than the maximum compressive resistance of the concrete ribs: $N_{c,max}$ (2 ribs)

$N_{c,max} = (b_{rib} + x_{xu}) * X_u * f_{cm} * 0.85 * 2 \;\; = (56 + 32.3) * 114 * 41.5 * 0.85 * 2$

$N_{c,max} = 710\,kN > N_s = 432.8\,kN$

This means the tensile force N_s will make equilibrium with a concrete area in the ribs. This can be determined using the geometry. The height of the compressive area (y) is based on an area of $(b_{rib} + k)$as shown in Figure E-3. 'k' can be related to the slope of the ComFlor210, which equals: $\frac{(b_{rib,top}-b_{rib})}{2} \Big/ h_{sheet} = \frac{175-56}{2} \Big/ 210 = \frac{17}{60}$

$$k = \frac{17}{60} * y$$

$N_s = 0.85 * f_{cm} * (b_{rib} + k) * y$

$432785 = 0.85 * 41.5 * \left(56 + \frac{17}{60} * y\right) * y *2$

$y = \; 78.4\,mm$

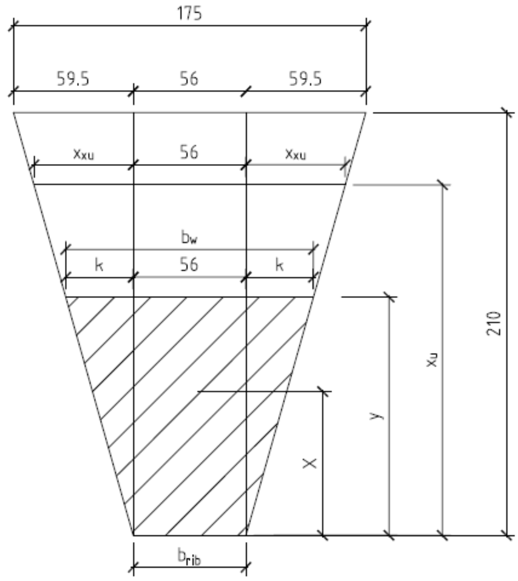


Figure E-3 Dimensions compression zone

The point of gravity (X) of an equilateral trapezium determines the final internal lever arm [z], see Figure E-3, the general formula to determine the point of gravity (starting from the base) equals:

$X = \frac{y}{3} * \frac{2*b_w+b_{rib}}{b_w+b_{rib}} = \frac{78.4}{3} * \frac{2*100.4+56}{100.4+56} = \; 42.9mm$ (With 'b_w' = 2*k + b_{rib})

$$z = d - X = 253 - 42.9 = 210.1mm$$

$M_{Rd}{}^- = N_s * z = 432.8 * 210.1 = 90.9\,kNm$ (for a width of 1200mm)

$M_{Rd}{}^- = 75.8\, {}^{kNm}/_{m}$ (Per meter slab). Located right next to the steel plate that is welded below the HE200A steel beam. (Cross section 2), steel ComFlor sheet is not included.

E.2.1.3. *Vertical shear resistance*

The vertical shear resistance has multiple contributing parts. First of all: the steel sheet, which in practice is used as a working floor and formwork for the composite slab, secondly the concrete which due to the acting moment has a compressive zone and a tensile zone. The compressive zone has an increased shear resistance, while the tensile region has a lower resistance. The applied tensile reinforcement keeps the concrete sides in contact and thereby increasing the friction between the concrete sliding planes. Calculation is done for a width of 1200mm according to the calculation determined in chapter 4.1.1, assuming the total resistance is a summation of different contributing parts.

The ribs will provide the following resistance:

A negative bending moment of: ${M_{Rk}}^{-} = 90.9\ kNm$, the height of the compression zone is equal to $y = \ 78.4\ mm$. The width [b_w] corresponding with this height follows below:

$$b_w = \frac{59.5}{210} * y * 2 + 56 = 100.4\ mm\ per\ rib = 200.85mm\ for\ a\ width\ of\ 1200\ mm$$

This is used to complete the formula given in EN 1992-1-1 [10] as shown below:

$V_{Rd,c} = \ [C_{Rd,c}k\left(100\ \rho_1 f_{cm}\right)^{\frac{1}{3}}]\ b_w d.$

The variables in this formula are determined below:

$\rho = \min\left(\frac{A_{sl}}{b_w * d}; 0.02\right) = \min\left(\frac{754}{200.85 * 245}; 0.02\right) = 0.015$

$k = \min\left(1 + \sqrt{\frac{200}{d}}; 2\right) = \ 1.90$

$v_{min} = 0.035 * k^{3/2} * {f_{cm}}^{1/2} = 0.59\ N/mm^2$ (v_{min} is the shear strength of the concrete itself, without reinforcement steel)

$$v = c_{Rd,c} * k * (100 * \rho * f_{cm})^{\frac{1}{3}} = 1.14 N/mm^2$$

($c_{Rd,c} = 0{,}15\ as\ discussed\ in\ chapter\ \mathrm{B.1.3}$ [14].

This results in a total shear resistance of the concrete ribs (2 ribs) of:

$V_{rib} = v * b_w * d = 1.14 * 200.85 * 253 = 57.9\ kN$

The steel sheet used is a 1mm thick ComFlor210 sheet in Z350.

The V_U of the steel sheet used is equal to $V_{plate} = 44.98\ kN/m$. [11] or APPENDIX B.1.3.

This gives a total V_U of the slab of 93.25 kN/m which equals 111.9 kN for the total slab. [Width 1,2m]

E.2.2. Test specimen 3: ComFlor210 Ø8-150 + Ø10-150

Test specimen 3 has the same dimensions as 1&2, the only difference is found in the reinforcement. An additional Ø10-150 is applied instead of the Ø8-150 which was applied in test specimens 1 & 2, hereby increasing the hogging bending moment resistance. This also reduces the concrete cover by 2 mm to a total of 21mm. A shorter calculation is given below.

E.2.2.1. Negative bending moment resistance (cross section 1)

Cross section 1 is located at the centre line of the HE200A beam. This is also the location where the highest negative bending moment will occur. The difference is the size of the compressive region in the concrete. This calculation will calculate the resistance of the entire specimen. (Width of 1200 mm)

$$d = h - c - \frac{Ø}{2} = 280 - 21 - \frac{10}{2} = 252\,mm$$

$X_u = \dfrac{500 * d}{500 + f_u} = \dfrac{500 * 252}{500 + 574} = 117.3\,mm$

$A_{sl} = \frac{1}{4} * \pi * d^2 * 7 + \frac{1}{4} * \pi * d^2 * 8 = 952\,mm^2 = 793 mm^2/m$ (a total of 8 Ø8 and 7 Ø10 bars were placed over a width of 1200mm)

$N_s = A_{sl} * f_u = 546.4 kN$

N_s is the maximum force the reinforcement can withstand, this will make equilibrium with the concrete poured around the HE200A steel beam.

$N_s = 0.85 * f_{cm} * b * x$

$546.4 * 10^3 = 0.85 * 42 * 1200 * x$

$x = 12.75 mm$

$z = d - \dfrac{x}{2} = 252 - \dfrac{12.75}{2} = 245.6 mm$

$M_{Rd}^{\ -} = N_s * z = 546.4 * 245.6 = 134.2\ kNm$ (for a width of 1200mm)

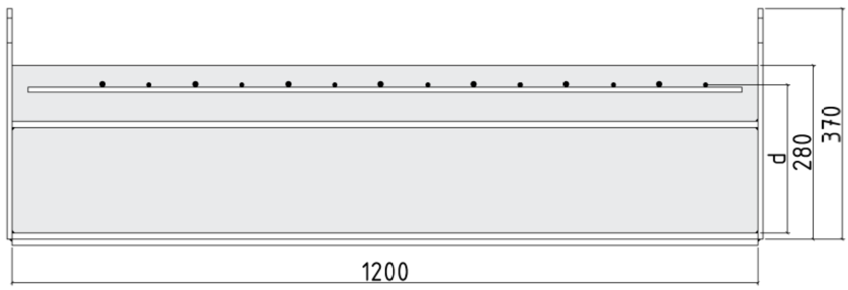


Figure E 4 Cross section 1.

This is the negative bending moment resistance at the location right above the support and at the centerline of the HE200A beam.

E.2.2.2. Negative bending moment resistance (cross section 2)

As shown in Figure E-5 the negative bending moment resistance is an internal force generated by the reinforcement steel under tension [N_s] and the concrete ribs under compression [N_c], separated by an internal lever arm [z]. The negative bending moment resistance follows from:

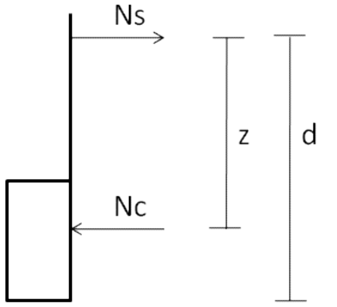


Figure E-5 Internal Forces making equilibrium

$$d = h - c - \frac{\text{Ø}}{2} = 280 - 21 - \frac{10}{2} = 252mm$$

$$X_u = \frac{500*d}{500+f_u} = \frac{500*252}{500+574} = 117.3\ mm$$

X_u is the maximum height of the compression zone (see Figure E-6). This is used later on to calculate the maximum compressive capacity of the concrete ribs.

$A_{sl} = \frac{1}{4} * \pi * d^2 * 15 = 951\ mm^2$ (a total of 8 Ø8 and 7 Ø10 bars over a width of 1200mm)

$N_s = A_{sl} * f_u = 951 * 574 = 546.4\ kN$

N_s is the maximum force the reinforcement can withstand, this must be lower than the maximum compressive resistance of the concrete ribs: $N_{c,max}$ (2 ribs)

$N_{c,max} = (b_{rib} + x_{xu}) * X_u * f_{cm} * 0.85 * 2 \ = (56 + 32.3) * 113.6 * 42 * 0.85 * 2$

$N_{c,max} = 716\ kN > N_s = 546.4\ kN$

This means the tensile force N_s will make equilibrium with a concrete area in the ribs. This can be determined using the geometry. The height of the compressive area (y) is based on an area of $(b_{rib} + k)$as shown in Figure E-6. 'k' can be related to the slope of the ComFlor210, which equals: $\frac{(b_{rib,top} - b_{rib})}{2} \Big/ h_{sheet} = \frac{175-56}{2} \Big/ 210 = \frac{17}{60}$

$$k = \frac{17}{60} * y$$

$N_s = 0.85 * f_{cm} * (b_{rib} + k) * y$

$546392 = 0.85 * 42 * \left(56 + \frac{17}{60} * y\right) * y * 2$

$y = \ 92.9\ mm$

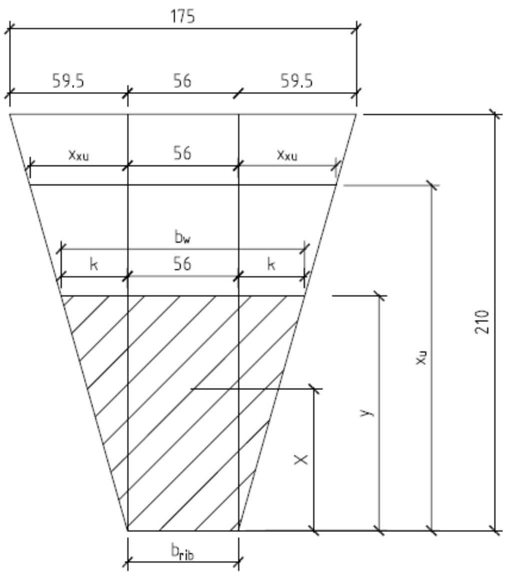


Figure E-6 Dimensions compression zone

The point of gravity (X) of an equilateral trapezium determines the final internal lever arm [z], see Figure E-6, the general formula to determine the point of gravity (starting from the base) equals:

$X = \frac{y}{3} * \frac{2*b_w + b_{rib}}{b_w + b_{rib}} = \frac{92.9}{3} * \frac{2*108.64+56}{108.64+56} = \ 51.4mm$ (With 'b_w' = 2*k + b_{rib})

$$z = d - X = 252 - 51.4 = 200.6\ mm$$

$M_{Rd}^{\ -} = N_s * z = 546 * 200.6 = 109.6\ kNm$ (for a width of 1200mm)

$M_{Rd}^{\ -} = 91.3\ {kNm}/{m}$ (Per meter slab). Located right next to the steel plate that is welded below the HE200A steel beam. (Cross section 2)

E.2.2.3. Vertical shear resistance

Same approach is used as for specimen 1 & 2.

The ribs will provide the following resistance:

A negative bending moment of: $M_{Rk}^{-} = 109.6\ kNm$, the height of the compression zone is equal to $y = \ 92.9\ mm$. The width $[b_w]$ corresponding with this height follows below:

$$b_w = \frac{59.5}{210} * y * 2 + 56 = 108.64\ mm\ per\ rib = 217.28mm\ for\ a\ width\ of\ 1200\ mm$$

This is used to complete the formula given in EN 1992-1-1 [10] as shown below:

$V_{Rd,c} = \ [C_{Rd,c}k(100\ \rho_1 f_{cm})^{\frac{1}{3}}]\ b_w d.$

The variables in this formula are determined below:

$\rho = \min\left(\frac{A_{sl}}{b_w * d}; 0.02\right) = \min\left(\frac{951}{217.28 * 244}; 0.02\right) = 0.018$

$k = \min\left(1 + \sqrt{\frac{200}{d}}; 2\right) = \ 1.90$

$v_{min} = 0.035 * k^{3/2} * f_{cm}{}^{1/2} = 0.60\ N/mm^2$ (v_{min} is the shear strength of the concrete itself, without reinforcement steel)

$$v = c_{Rd,c} * k * (100 * \rho * f_{cm})^{\frac{1}{3}} = 1.21N/mm^2$$

($c_{Rd,c} = 0{,}15\ as\ discussed\ in\ chapter\ \mathrm{B.1.3}$ [14].

This results in a total shear resistance of the concrete ribs (2 ribs) of:

$V_{rib} = v * b_w * d = \ 1.21 * 217.28 * 252 = 66.3\ kN$

The steel sheet used is a 1mm thick ComFlor210 sheet in Z350.

The V_U of the steel sheet used is equal to $V_{plate} = 44.98\ kN/m$. [11] or APPENDIX B.1.3.

This gives a total V_U of the slab of 100.2 kN/m which equals 120.23 kN for the total slab. [Width 1,2m]

E.2.3. Test specimens 4 & 5 ComFlor75 Ø8-75

E.2.3.1. Negative bending moment resistance ComFlor75 Ø8-75

In contrast to the ComFlor210, the joist shuttering does continue over the support in this experiment. To avoid unwanted uncertain extra resistances, the joist shuttering is cut for ¾ of its height in the specimen. It therefore does not contribute to the negative bending moment resistance.

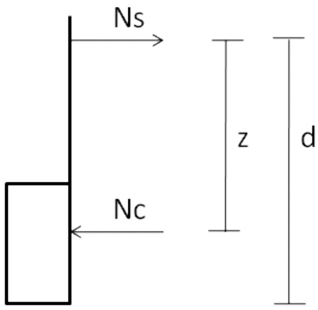


Figure E 7 Internal Forces making equilibrium

As shown in Figure E 7 the negative bending moment resistance is an internal force generated by the reinforcement steel under tension [N_S] and the concrete ribs under compression [N_C], separated by an internal lever arm [z]. Below the calculation is shown to calculate the negative bending moment resistance:

$$d = h - c - \frac{Ø}{2} = 150 - 37 - \frac{8}{2} = 109\ mm$$

The cover of 37 mm is due to an error in ordering concrete bricks to keep the distance between the ComFlor75 and the reinforcement.

$$X_u = \frac{500 * d}{500 + f_u} = \frac{500 * 109}{500 + 574} = 50.7mm$$

X_u is the maximum height of the compression zone (see Figure E-3). This is used later on to calculate the maximum compressive capacity of the concrete ribs.

$A_{sl} = \frac{1}{4} * \pi * d^2 * 15 = 754\ mm^2$ (a total of 15 bars were placed over a width of 1200mm)

$N_s = A_{sl} * f_u = 754 * 574 = 432.8kN$

N_s is the maximum force the reinforcement can withstand, this must be lower than the maximum compressive resistance of the concrete ribs: $N_{c,max}$ (4 ribs)

$N_{c,max} = (b_{rib} + x_{xu}) * X_u * f_{cm} * 0.85 * 4$

$= (120 + 24.4) * 56.3 * 43 * 0.85 * 4$

$$N_{c,max} = 1188\ kN > N_s = 432.8\ kN$$

This means the tensile force N_s will make equilibrium with a concrete area in the ribs. This can be determined using the geometry. The height of the compressive area (y) is based on an area of $(b_{rib} + k) * y$ as shown in Figure E-8. 'k' can be related to the slope of the ComFlor210, which equals: $\frac{(b_{rib,top} - b_{rib})}{2} \Big/ h_{sheet} = \frac{250-120}{2} \Big/ 150 = \frac{13}{30}$

$$N_s = 0.85 * f_{cm} * (b_{rib} + k) * y$$
$$432785 = 0.85 * 43 * \left(120 + \frac{13}{30} * y\right) * y * 4$$
$$y = 22.8\, mm$$

The point of gravity (X) of an equilateral trapezium determines the final internal lever arm [z], see Figure E-8, the general formula to determine the point of gravity (starting from the base) equals:

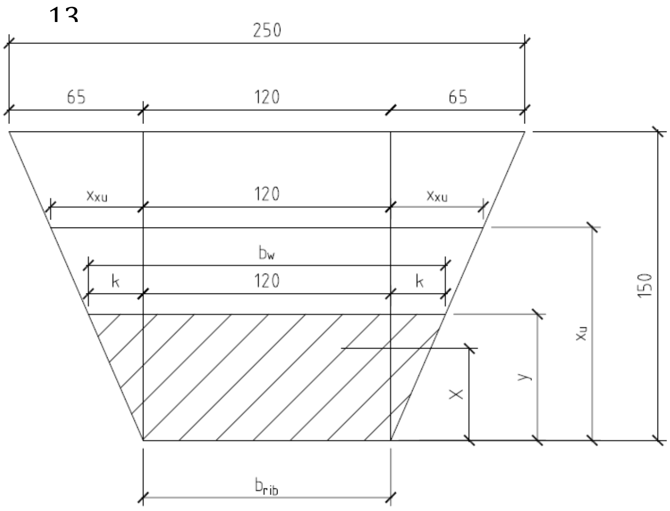


Figure E 8 Overview compression zone ComFlor75

$$X = \frac{y}{3} * \frac{2*b_w + b_{rib}}{b_w + b_{rib}} = \frac{22.8}{3} * \frac{2*139.76+120}{139.76+120} = 11.7mm$$ (With 'b_w' = 2*k +b_{rib})

$$z = d - X = 109 - 11.7 = 97.32mm$$

$M_{Rd}^{-} = N_s * z = 432.8 * 97.32 = 42.12\ kNm$ (for a width of 1200mm)

$M_{Rd}^{-} = 35.1\ {}^{kNm}/_{m}$ (Per meter slab).

This however neglects the steel ComFlor75 sheet. To include this the steel ComFlor75 sheet has been divided into smaller parts (see figure below), the calculation is shown on next page.

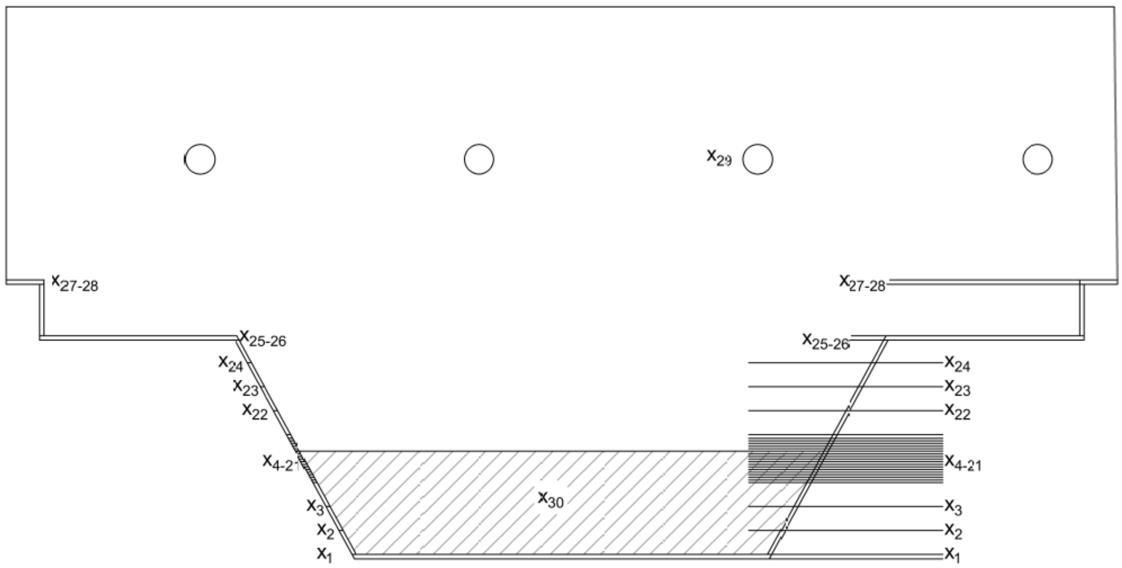


Figure E 9 Division ComFlor75 into smaller parts to calculate the Mpl.

Table 32 Calculation Mpl ComFlor75 including the steel sheet.

Contributing part	Tag (see figure)	Height (mm)	Point of engagement (mm)	Area per rib (mm^2)	Area per 1,2 m (mm^2)	Stress (N/mm^2)	embossements	Force [N]	Moment [kNm]
ComFlor75	x1	1.2	0.6	141	564	402	1	-226728	-5.87
	x2	7.64	4.42	13.4	53.6	402	1	-21547.2	-0.48
	x3	14.08	10.86	13.4	53.6	402	0.8	-17237.8	-0.27
	x4	20.52	17.3	13.4	53.6	402	0.8	-17237.8	-0.16
	x5	20.92	20.72	0.83	3.32919	402	0.8	-1070.67	-0.01
	x6	21.32	21.12	0.83	3.32919	402	0.8	-1070.67	-0.01
	x7	21.72	21.52	0.83	3.32919	402	0.8	-1070.67	-0.01
	x8	22.12	21.92	0.83	3.32919	402	0.8	-1070.67	0.00
	x9	22.52	22.32	0.83	3.32919	402	0.8	-1070.67	0.00
	x10	22.92	21.72	0.83	3.32919	402	0.8	-1070.67	-0.01
	x11	23.32	23.12	0.83	3.32919	402	0.8	-1070.67	0.00
	x12	23.72	23.52	0.83	3.32919	402	0.8	-1070.67	0.00
	x13	24.12	23.92	0.83	3.32919	402	0.8	-1070.67	0.00
	x14	26.96	25.54	13.4	53.6	402	0.8	-17237.8	-0.02
	x15	33.4	30.18	13.4	53.6	402	0.8	17237.76	-0.06
	x16	39.84	36.62	13.4	53.6	402	0.8	17237.76	-0.17
	x17	46.28	43.06	13.4	53.6	402	0.8	17237.76	-0.29
	x18	52.72	49.5	13.4	53.6	402	0.8	17237.76	-0.40
	x19	59.16	55.94	13.4	53.6	402	1	21547.2	-0.63
	x20	60.36	59.76	128	512	402	1	205824	-6.84
	x21	74.16	67.26	32	128	402	1	51456	-2.10
	x22	75.36	74.76	24	96	402	1	38592	-1.86
Reinforcement	x23	109	109.00	188.50	753.98	574	1	432785.8	-35.70
Concrete	x24		13.64	3485.17	13940.67	43	1	-509532	-6.56
								0.000737	**-61.45**

Effectivity	0.80
brib	120.00
k	11.89
bw	143.78
n.a.	27.43
M [kNm]	**-61.51**

Table 33 Data used

A horizontal equilibrium was sought by changing the position of the neutral axis. By plastic calculation this led to a hogging bending moment capacity of 61.52 kNm. (for a width of 1200mm)

E.2.3.2. Vertical shear resistance ComFlor75 Ø8-75

As with the ComFlor210 the ComFlor75 consist of a steel sheet and a concrete top layer, this layer consist of ribs (functioning as beams) and small slabs to connect each rib. In order to calculate the vertical shear resistance first the ribs are considered and finally the steel ComFlor75 sheet. Comparable to the ComFlor210 specimens, it is assumed the total resistance equals the summations of separate parts as no separate shear test is done to determine the resistance.

The ribs will provide the following resistance:

A negative bending moment of: $M_{Rk}{}^{-} = 61.51\ kNm$, the height of the compression zone is equal to $y = 27.43\ mm$. The width $[b_w]$ corresponding with this height follows below:

$$b_w = \frac{65}{150} * y * 2 + 120 = 143.78\ mm\ per\ rib = 575.12\ mm\ for\ a\ width\ of\ 1200\ mm$$

This is used to complete the formula given in EN 1992-1-1 [10] as shown below:

$V_{Rd,c} = [C_{Rd,c} k(100\ \rho_1 f_{cm})^{\frac{1}{3}}]\ b_w d.$

The variables in this formula are determined below:

$\rho = \min\left(\frac{A_{sl}}{b_w * d}; 0.02\right) = \min\left(\frac{754}{575.12 * 109}; 0.02\right) = 0.012$

$k = \min\left(1 + \sqrt{\frac{200}{d}}; 2\right) = 2$

$v_{min} = 0.035 * k^{3/2} * f_{cm}{}^{1/2} = 0.65\ N/mm^2$ (v_{min} is the shear strength of the concrete itself, without reinforcement steel)

$$v = c_{Rd,c} * k * (100 * \rho * f_{cm})^{\frac{1}{3}} = 1.118\ N/mm^2$$

($c_{Rd,c} = 0{,}15$ *as discussed in chapter* B.1.3 [14].

This results in a total shear resistance of the concrete ribs (2 ribs) of:

$V_{rib} = v * b_w * d = 1.118 * 575.12 * 109 = 70.07\ kN$

The steel sheet used is a 1mm thick ComFlor75 sheet in Z350.

The V_U of the steel sheet used is equal to $V_{plate} = 55\ kN/m$. (As explained in chapter B.2.3, based on the tests done in London.)

This gives a total V_U of the slab of 104.22 kN/m which equals 125.07 kN for the total slab.

APPENDIX F. Expectations

F.1. ComFlor210 test specimens 1 – 3.

The aim of the first test is to determine the negative bending moment resistance with a relatively small vertical shear present. This is done by using a cantilever with a length a/d = 6, which in test 1 was equal to 1620 mm from the center of the steel HE200A beam. The second and third test specimen will be loaded at a cantilever length of a/d = 3. There are three cross sections that could become critical (see Figure F 1), all have a different moment and shear resistance as discussed in5.2.2. This means the test specimen could fail at three different locations during the tests. Also due to the 200 mm distance between all cross sections, the occurring moment will be different, while the vertical shear force remains nearly equal. Apart from the negative bending moment due to the self weight and steel beams, the occurring difference in moment between all cross sections can be calculated as following:

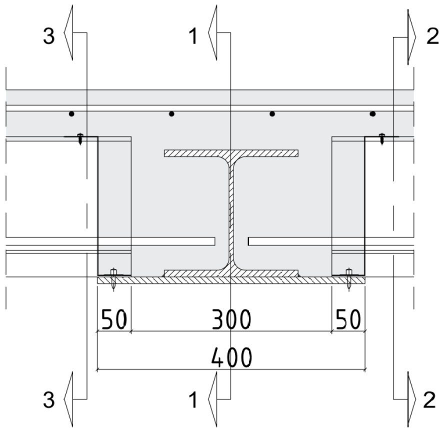


Figure F 1 Critical cross sections at the location of the steel HE200A support.

$$M^{-}[2] = \frac{1620 - 200}{1620} * M^{-}[1] = 87\% * M^{-}[1]$$
$$M^{-}[3] = \frac{1970-200}{1970} * M^{-}[1] = 90\% * M^{-}[1]$$

Here the factor used to calculate the relation between $M^{-}[1]\&M^{-}[2]/M^{-}[3]$ is the difference in distance to the applied load. In the case of the first specimen in cross section 2 is around 87% of the negative bending moment that occurs in cross section 1.

Figure F 2 Test setup, specimen 1

F.1.1. Mechanical scheme and predicted failure loads

The test setup for test specimen 1-3 are identical, the only difference is the location where the load is applied. In case of the first specimen a mechanical scheme is shown below:

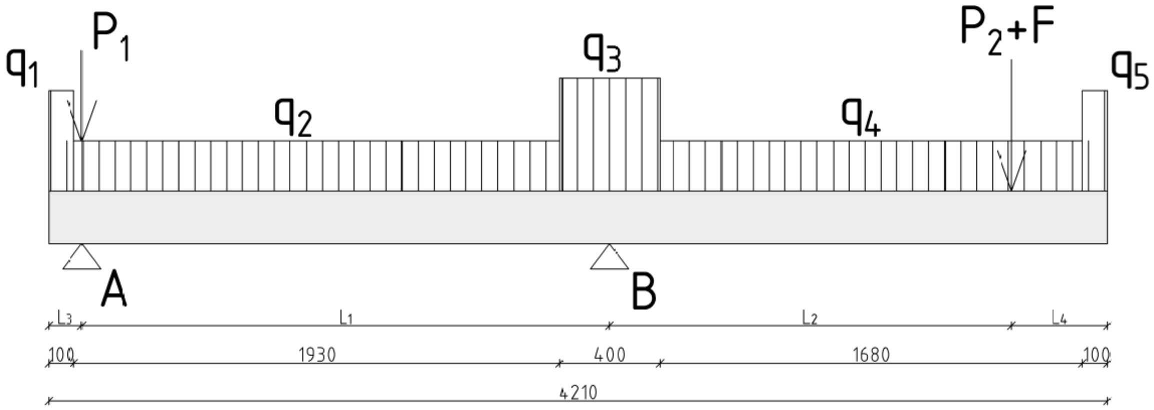


Figure F 3 Mechanical scheme for test specimen 1-3. Varying in point of engagement of the loads.

The different loads differ per specimen as the position of loading varies. The distances L_1&L_2 with the corresponding loads are shown in Table 34 for the ComFlor210 specimens.

Table 34 Actual values at the moment of testing, including prediction of the failure load [F]

Load tag	Test Specimen 1	Test Specimen 2	Test Specimen 3
P_1 [kN]	2.7	2.7	2.7
q_1 [kN/m]	9	9	9
q_2 [kN/m]	3.35	3.35	3.35
q_3 [kN/m]	12.65	12.65	12.65
P_2 [kN]	4.8	4.8	4.8
q_4 [kN/m]	9	9	9
q_5 [kN/m]	3.35	3.35	3.35
F [kN]	**56.9**	**104.4**	**133.8**
L_1 [m]	1.97	1.95	1.97
L_2 [m]	1.62	0.915	0.915

Table 35 Predicted relation between occurring hogging bending moment and vertical shear force, both related to the resistances of the test specimens.

	Test Specimen 1	Test Specimen 2	Test Specimen 3
M/M_U	100%	100%	100%
V/V_U	63%	106%	123%

The predicted loads and M-V relations at the moment of failure are used after the experiments to compare with the actual values.

F.1.2. M-V lines

For test specimen 1 a failure load [F] was determined based on chapter 5.4 and shown in Table 34 on the previous page, with F equal to 56,9 kN. Inserting the values found in Table 34 gives the following vertical shear and moment lines:

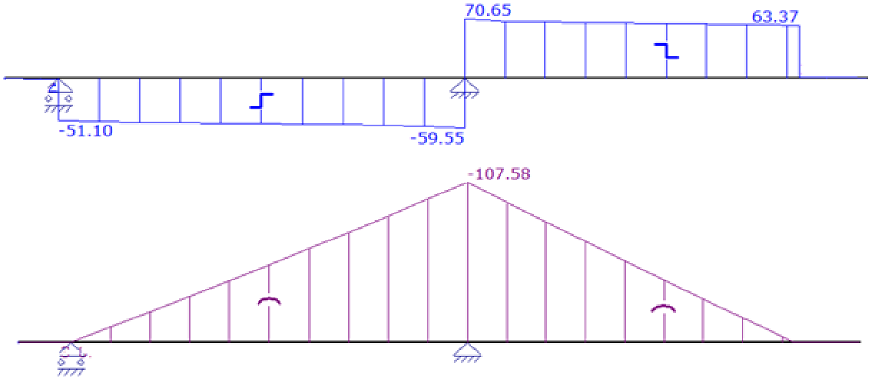


Figure F 4 Mechanical scheme; test specimen 1 at the moment of failure. Force applied at $L_2 = 1620$mm.

For the test specimen 2, a failure load has been predicted of F = 104.4 kN. Including the dead loads on the specimen gives the following M & V lines:

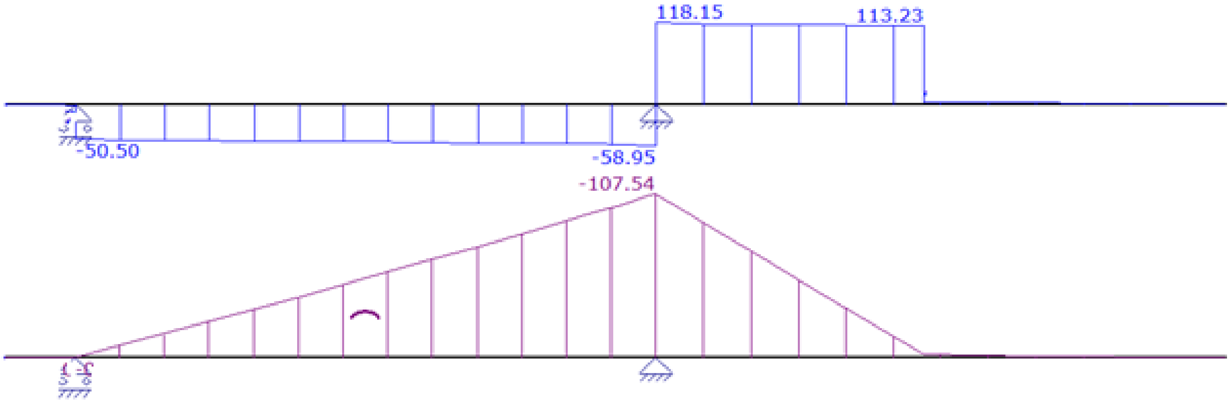


Figure F 5 Mechanical scheme; test specimen 2 at the moment of failure. Force applied at L_2=0.915 mm.

For the test specimen 3, a failure load has been predicted of F =133.8 kN. Including the dead loads on the specimen gives the following M & V lines:

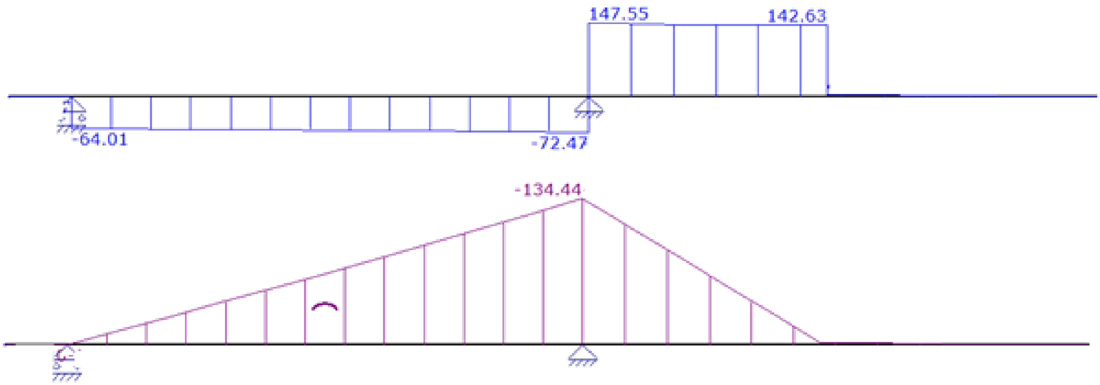


Figure F 6 Mechanical scheme; test specimen 3 at the moment of failure. Force applied at L_2=0.915 mm.

F.2. ComFlor75 test specimens 4-5.

The same procedure used on the ComFlor210 specimens will be used for the two ComFlor75 specimens. The first specimen (test specimen 4) will be used to determine the negative bending moment resistance using a cantilever equal to an a/d ratio of 6. The second test specimen (test specimen 5) will be tested on interaction at a distance of a/d equal to 3.

F.2.1. Mechanical scheme and predicted failure loads

The test setup for test specimen 4-5 are identical, the only difference is the location where the load is applied. In case of the first specimen a mechanical scheme is shown below:

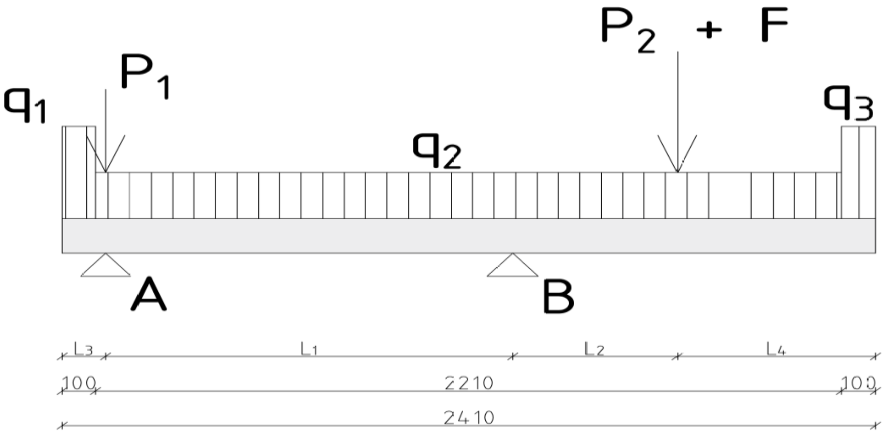


Figure F 7 Mechanical scheme for test specimen 4-5. Varying in point of engagement of the loads.

The different loads differ per specimen as the position of loading varies. The distances L_1&L_2 with the corresponding loads are shown in Table 36for the ComFlor210 specimens.

Table 36 Actual values at the moment of testing, including prediction of the failure load [F]

Load tag	Test Specimen 4	Test Specimen 5
P_1 [kN]	2.7	2.7
q_1 [kN/m]	4.5	4.5
q_2 [kN/m]	3.5	3.5
q_3 [kN/m]	4.5	4.5
P_2 [kN]	4.8	4.8
F [kN]	**84.8**	**174.75**
L_1 [m]	1.1	1.1
L_2 [m]	0.66	0.33

Table 37 Predicted relation between occurring hogging bending moment and vertical shear force, both related to the resistances of the test specimens.

	Test Specimen 4	Test Specimen 5
M/M_U	100%	100%
V/V_U	75%	147%

The predicted loads and M-V relations at the moment of failure are used after the experiments to compare with the actual values.

F.2.1. M-V lines

For test specimen 4 a failure load [F] was determined based on chapter 5.4, with F equal to 84.8 kN. Inserting the values found in Table 36gives the following vertical shear and moment lines:

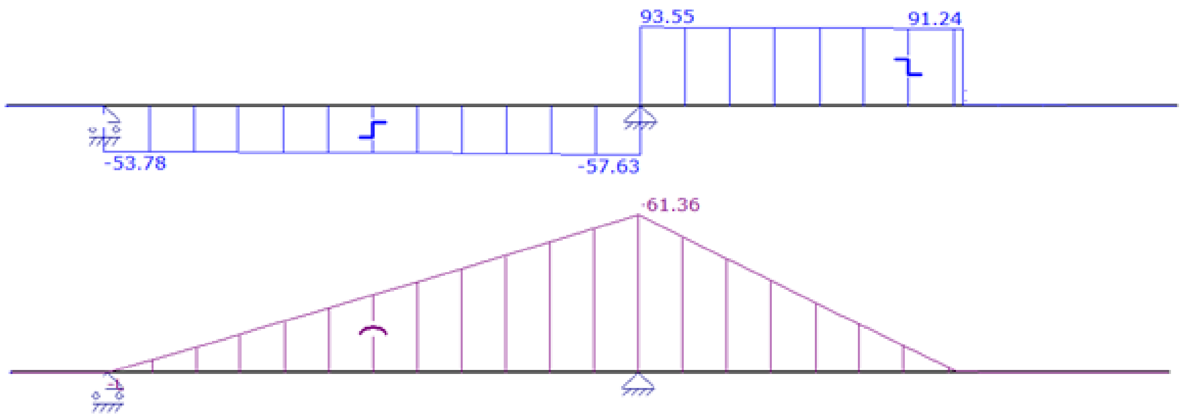


Figure F 8 Mechanical scheme; test specimen 4 at the moment of failure. Force applied at L_2=0.66 mm

For the test specimen 5, a failure load has been predicted of F = 174.75 kN. Including the dead loads on the specimen gives the following M & V lines:

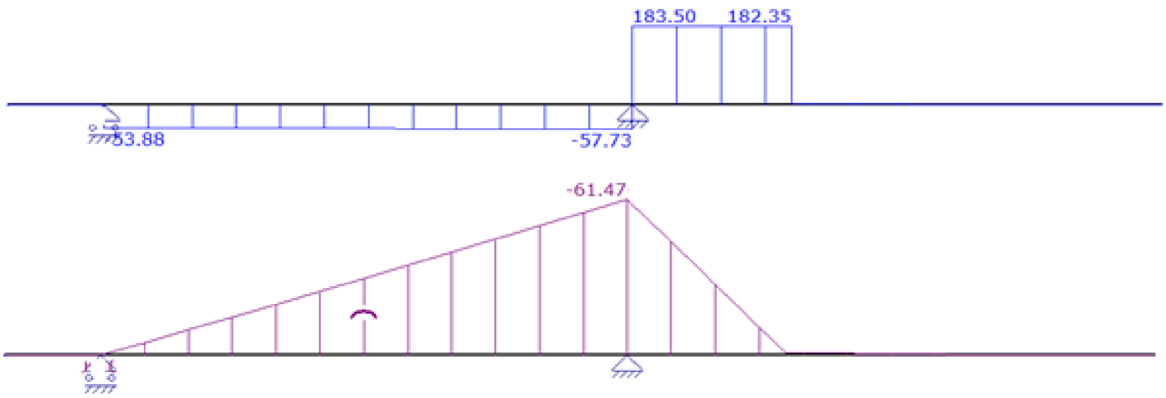


Figure F 9 Mechanical scheme; test specimen 5 at the moment of failure. Force applied at L_2=0.33 mm

APPENDIX G. Preparations test rig

G.1. Construction of the test rig

The supports are build up on top of 2 hot rolled sections, anchored to the 1200mm thick concrete floor of the Stevin II laboratory. The supports have been constructed with mechano steel parts to create stiff supports to transfer the occurring forces towards the floor.

a) Base parts of the test rig on top of the two main beams that transfer the forces to the floor.

b) *Anchorage of the two main beams to the 1200 mm thick concrete slab of the Stevin II laboratory*.

Figure G 1 Anchorage of the main parts of the test rig to the concrete floor below.

The supporting frame can move in longitudinal direction to change the distance between the supports and the applied load. In case of a change between floor types (ComFlor210 to ComFlor75) the distance between both supports has to be adapted.

G.1.1. End support (left side)

The end support first serves as a temporary support "A" to support the specimen before testing. After the placement of the top beam (Figure G 2a) it serves a vertical support "C".

a) *Beam that will be placed on top of the specimen, to serve as a support*

b) *Mechano parts to serve as a temporary support at the right height before testing, connected to the underlying steel beams.*

Figure G 2 Separate mechano parts that later form the end support on the left side.

The end support will be loaded by a force that will attempt to lift the test rig upwards. It is therefore important that the mechano parts have sufficient bolts (Figure G 3) and is anchored (Figure G 1) to the concrete floor.

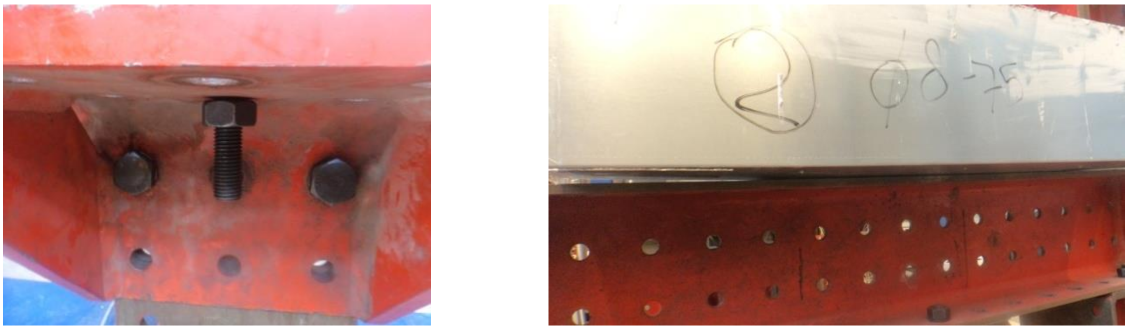
a) *Final end support detail with a ComFlor210 specimen in place.*

b) *Final end support detail with a ComFlor75 specimen in place.*

Figure G 3 End support, left side, preventing vertical movement but allowing rotation

Figure G 3 gives an overview of the end support with both a ComFlor210 and a ComFlor75 in place.

In order to avoid any resistance in rotation, the specimen must lose contact with the temporary support “A”. To realize this, a nut (Figure G 4) will be twisted 3 full rotations to allow some initial vertical displacement. Below the mechano beam, on top and the test specimen, a rubber strip will be place to allow rotations between the beam and the specimen.



a) *Nut below the support. To be twisted downwards.*

b) *Specimen at the start of the test. It has been lifted a little bit to avoid resistance to rotations of the specimen.*

Figure G 4 Nut below the support. To be twisted 3 full rotations to allow some initial vertical displacement to allow rotation of the specimen and avoid clamping of the specimen.

G.1.2. Mid support (support “B”)

The mid support serves as a compressive support. Once the specimen loses contact with the temporary support “A” it must withstand a force equal to:

$$R_B = R_C + F + dead\ load$$

To measure the reaction force R_B either two or five load cells have been applied, Figure G 5.

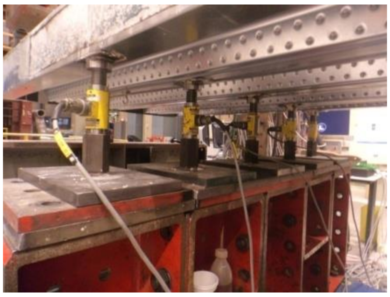
a) *Support B, ComFlor210 setup. Two load cells to support both ribs.*

b) *Support B, ComFlor75 setup. Five load cells to support 5 ribs.*

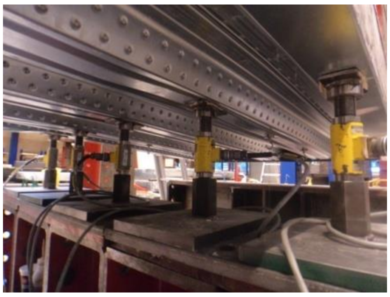
Figure G 5 Support B before placement of the specimens.

ComFlor210 specimens have 2 ribs, therefore two load cells have been placed underneath each rib. (Underneath the integrated HE200A steel beam)
ComFlor75 specimens have 3 ribs and 2x ½ ribs, leading to five load cells over the full length.

a) *Final support B, ComFlor210, underneath the HE200A steel beam.*



b) *Final support B, ComFlor75 (specimen 4) underneath each rib.*



c) *Punching shear occurring under certain ribs at the end of the fourth test.*

Figure G 6 Final support B of specimen 1 - 4. The fourth specimen resulted in some punching shear failure below the ribs.

At failure of the fourth specimen, punching shear failure occurred underneath each rib as shown in Figure G 6c. In order to avoid unwanted influence of this failure mode at the fifth specimen, where the occurring reaction force will be twice as high, a steel strip has been applied to simulate a support used in practice (a steel member).

a) The steel strip to avoid punching shear failure underneath the ribs.

b) Final mid support of the ComFlor75, test specimen 5, in place.

Figure G 7 Adjusted mid support for the last specimen to avoid punching shear failure.

G.1.3. Location of applying the load (right side, cantilever side)

The force is applied by a hydraulic press. It must push the specimen downward with a constant displacement over the width of the specimen. This means the portal used to apply the load must be stiff compared to the specimen and the spreader beam to redistribute the point load applied by the hydraulic jack to an equally distributed line load / displacement on the specimen.

a) *The hydraulic cylinder used to apply the force.*

b) *The spreader beams used to redistribute the applied load.*

c) *The convex point of the hydraulic cylinder will enter the concave bowl on top of the steel beam.*

Figure G 8 The hydraulic cylinder used to apply the force.

Underneath the spreader beams a hinge is create by a steel rod/strip. This allows a specific point of engagement of the load, avoiding the spreader beam from touching the specimen after the specimen starts to deform.

a) *Steel strip with a rod on top to create a hinge.*

b) *Horizontal displacement restrictions.*

c) *Steel strip plus rod in between the test specimen and the steel beams.*

Figure G 9 Hinge created at the point of applying the line load on top of the test specimen.

G.2. Instrumentation

G.2.1. Load cells

Load cells consist of two types. Tensile load cells located at support C and compressive load cells located at support B.

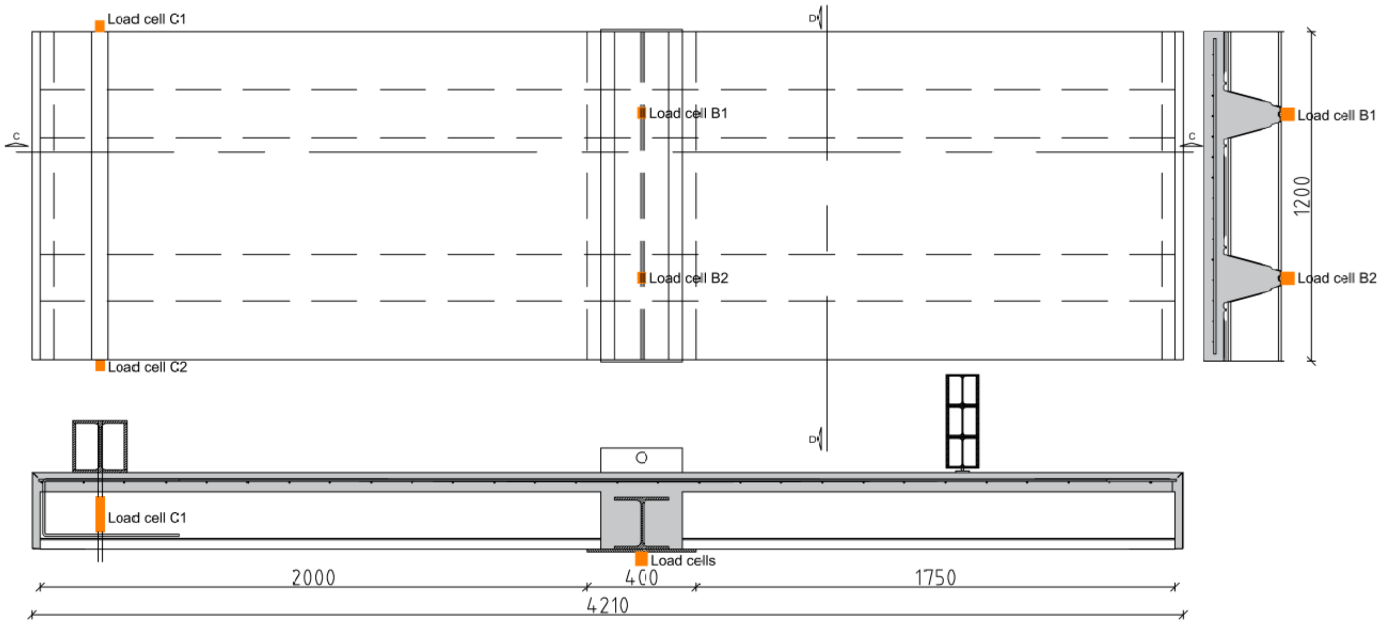


Figure G 10 Overview load cells ComFlor210 specimens.

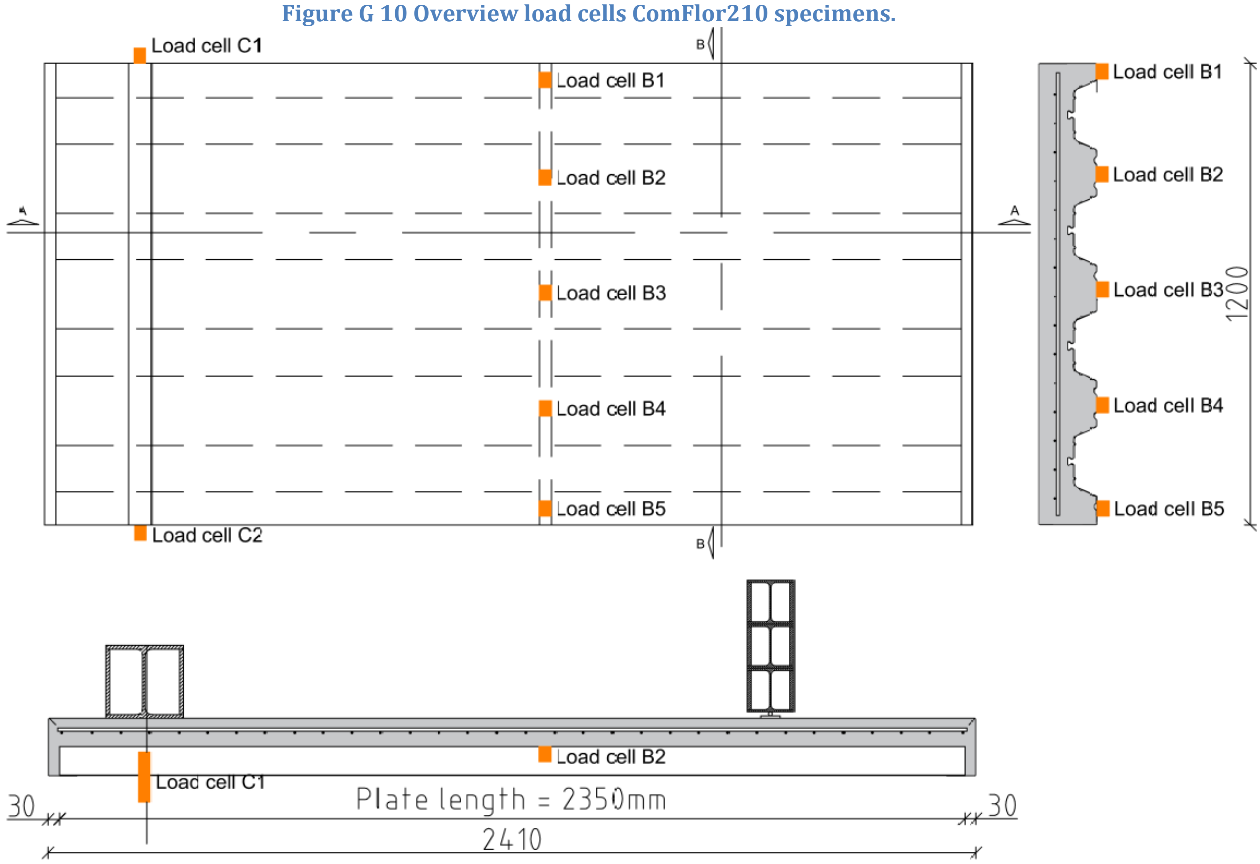
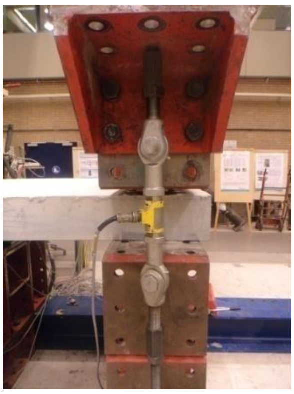


Figure G 11 Overview load cells ComFlor75 specimens.

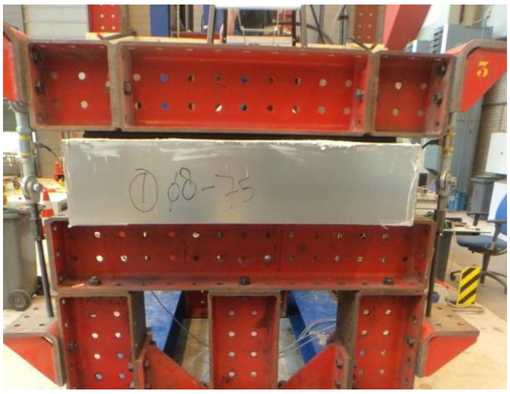
G.2.1.1. Load cells, support "C"

At the end of the specimen, a mechano beam of 271 kg has been placed to serve as a vertical support. Restricting vertical movement, but allowing rotation.

a) *Load cell in tension*



b) *Load cell in tension ComFlor75*



c) *Two load cells on both sides of the specimen.(Load cell 1 & 2)*

Figure G 12 Tensile load cells from difference angles / specimens designed to measure vertical tensile forces.

In Figure G 12the load cells at the end support are shown. The mechano beam on top of the specimen is hold at each side by a threaded stud with a load cell in between. This way the total vertical shear force can be measured. This support detail can be used identically for both the ComFlor210 as for the ComFlor75 specimens.

The total reaction force R_C is as following:

$$R_C = \sum_{i=1}^{2} R_{ci} = R_{C1} + R_{C2}$$

G.2.2. Load cells, compressive support

For support two variations exist, one for the ComFlor210 and the other for the ComFlor75. Both make use of the same type of load cells. The ComFlor210 already includes an integrated steel beam that simulates the support in practice and the ComFlor75 does not. The load cells used to measure the reaction forces in compression. On the top there is a thread with a rounded top that fits in the hollow bowl of the top side on the support. This allows free rotation.

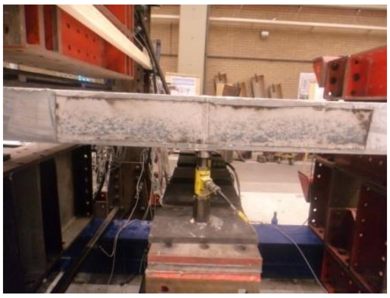
a) Load cell, connected to the computer and placed underneath the specimen

a) *Load cell before placement of the specimen.*

b) *Overview of both load cells underneath the integrated steel HE200A beam.*

Figure G 13 Load cells under the ComFlor210 specimens (1-3)

In Figure G 13both of the load cells used underneath the ComFlor210 specimens are shown. In Figure G 14the load cells used for the last two specimens are shown.



a) *The load cell underneath the ComFlor75 specimen.*

b) *All five load cells before placement of the specimen on top.*

c) *All five load cells including the steel strip underneath the last specimen.*

Figure G 14 Load cells under the ComFlor75 specimens (4-5)

In Figure G 14a a side view of the support is given. It can be seen that a part of the joist shuttering has been grinded out. This is done to prevent any contribution of the joist shuttering to the negative bending moment or the vertical shear resistance. This also allows insight in the crack pattern in the concrete. (Even though this crack pattern is on the side of the specimen and therefore is not necessarily representative for the entire cross section). As the fourth specimen almost resulted in local punching shear, the fifth specimen included a steel strip as shown in Figure G 14c.

R_B is calculated as follows:

$$R_B = \sum_{i=1}^{2-5} R_{Bi} \;= R_{B1} + R_{B2} + R_{B3} + R_{B4} + R_{B5}$$

G.2.3. Cylinder, applied force

The portal including the hydraulic cylinder has already been discussed; an overview of the most important aspects will be given.
The hydraulic cylinder has a capacity of 1000 kN in compression and a maximum displacement of +/- 140 mm (- 70 mm and + 70 mm).

a) *Overview portal with the hydraulic cylinder.*

b) *Point of engagement on the specimen, allowing rotation.*

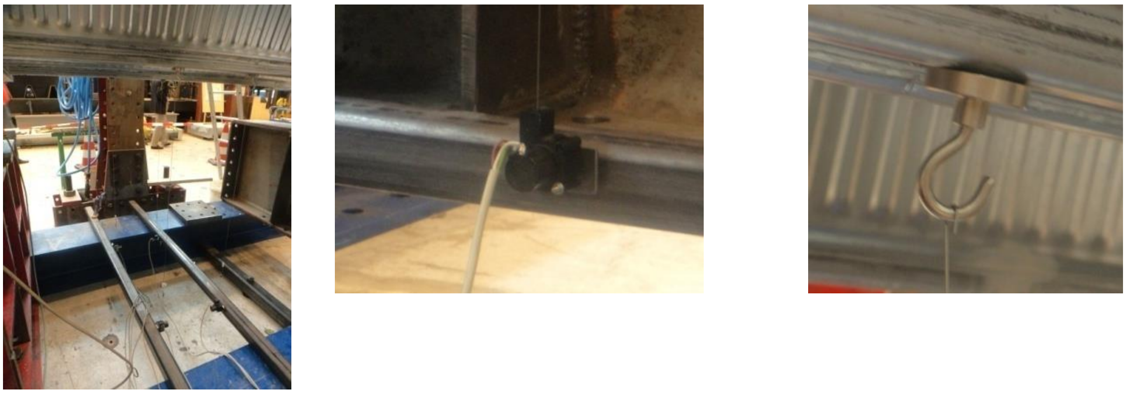
c) *Displacement regulator to assure smooth displacement during the yielding phase.*

Figure G 15 Main aspects of the hydraulic cylinder that enables the application of the vertical force.

The measurement of the applied force is the governing factor to answer the main question. Combined with the measured distance between the applied force and support B, this gives insight in the occurring hogging bending moment and the vertical shear.

G.2.4. Thread LVDT’s

The application of the four thread LVDT’s gives insight in the behavior of the slab during the tests. The data is not needed to answer the main question, but it can explain certain crack patterns and differences between load cells. Figure G 16gives an overview of the devices used:



a) *Overview of the four applied thread-LVDT’s under the specimen.*

b) *Measure device at the bottom.*

c) *Magnet applied at one of the ribs at a certain distance from the support.*

Figure G 16 Thread LVDT's to measure the deflection of the specimens.

G.2.4.1. Overview thread LVDT’s used for the ComFlor210 specimens.

Table 38 Location of the Thread LVDT's across the different test specimens.

	Location Thread LVDT’s, measured from support B			
	Thread LVDT 1 [mm]	Thread LVDT 2 [mm]	Thread LVDT 3 [mm]	Thread LVDT 4 [mm]
Test Specimen 1	1680	1680	600	600
Test Specimen 2	855	855	600	600
Test Specimen 3	855	855	600	600

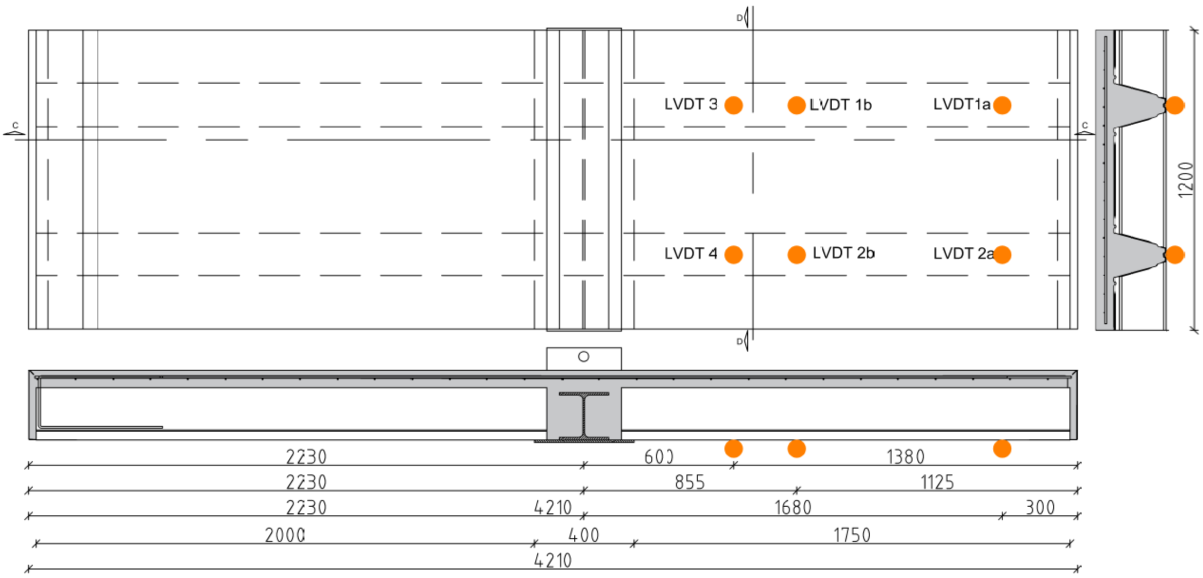


Figure G 17 Overview thread LVDT's used to measure the displacement. LVDT's tag with an 'a' are used for specimen 1. The LVDT's tagged with a 'b' are used for specimens 2 & 3.

G.2.4.2. Overview thread LVDT's used for the ComFlor75 specimens.

Table 39 Location of the Thread LVDT's across the different test specimens.

	Location Thread LVDT's, measured from support B			
	Thread LVDT 1 [mm]	Thread LVDT 2 [mm]	Thread LVDT 3 [mm]	Thread LVDT 4 [mm]
Test Specimen 4	660	660	335	335
Test Specimen 5	660	660	335	335

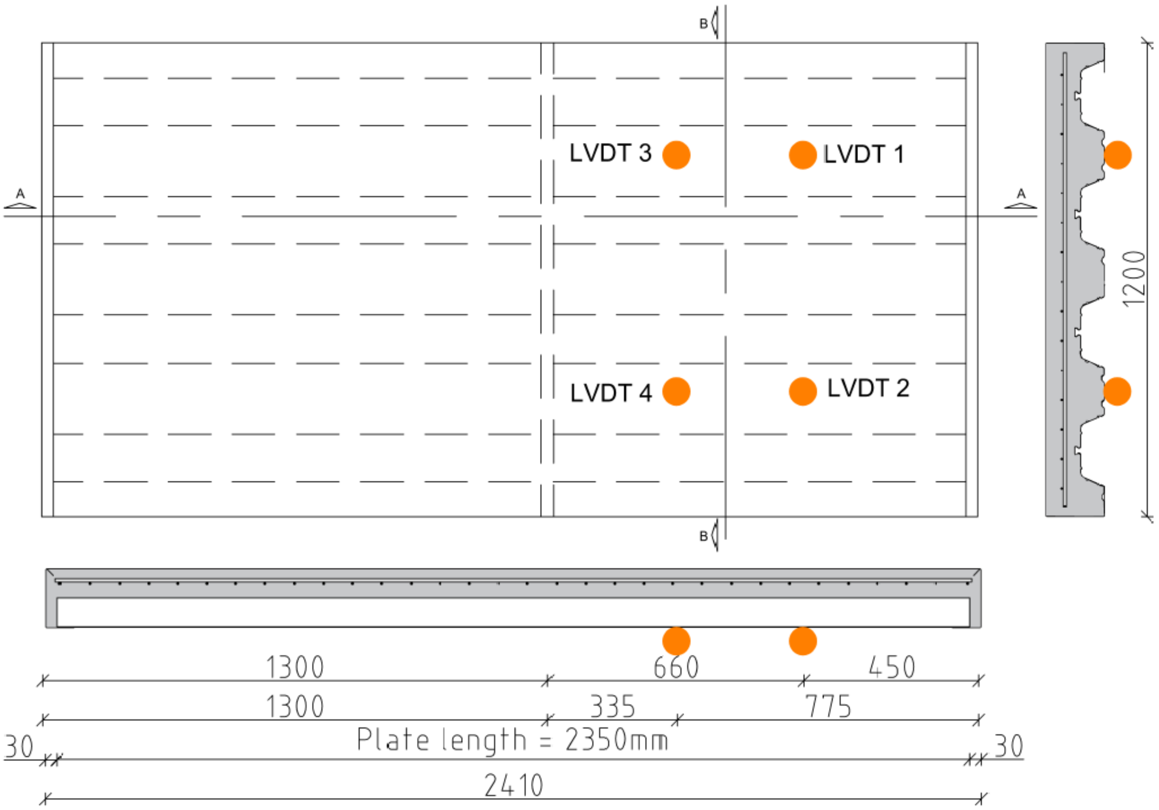


Figure G 18 Overview thread LVDT's for the ComFlor75 specimens 4 & 5.

APPENDIX H. Experiments

H.1. Results Test 1

Test specimen 1 has the aim of applying a low shear force and letting the specimen fail in bending. This means a long cantilever compared to the height of the slab. In Figure H 1the test rig is shown with the first specimen in place.

Figure H 1 Test specimen 1 right before testing

In this chapter a quick overview is given of the most important measurements, starting with the force applied in relation to the deflection shown in Figure H-2.

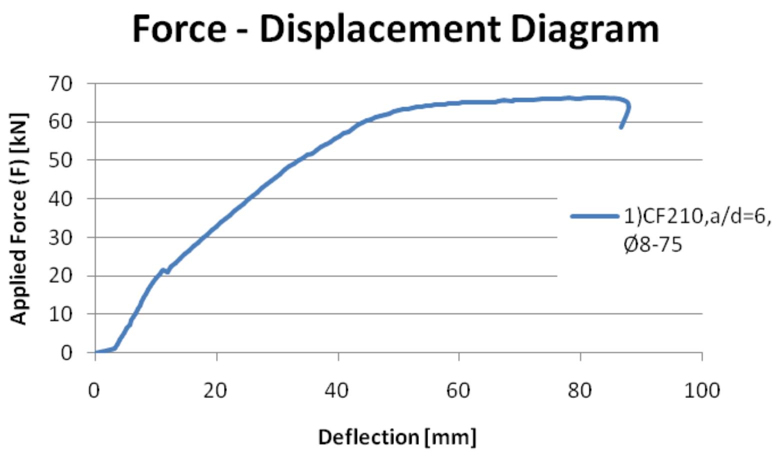


Figure H 2 P-δ Diagram. Test specimen 1. Final force: 66,21 kN, Final deflection: 87mm.

The specimen failed at an applied load of 66,21 kN at a deflection of 87 mm. The specimen failed at cross section 1 (see E.1.6), where the hogging bending moment was the biggest. The specimen failed in negative bending, no shear cracks were to be found.

Figure H 3 Crack pattern specimen 1, final failure at the middle of the steel beam

The measurements lead to the following reactions forces and applied load at the moment of failure:

Table 40 Maximum values at the moment of failure, test specimen 1.

Maximum values		
Total Support Reaction (R_c) [kN]	Total Support Reaction (R_B) [kN]	Total Force Applied (F) [kN]
55.11	155.97	66.21

In Figure H 4 the moment is a summation of the applied load multiplied by the lever arm and the already present self weight. This equals the moment occurring at cross section 1 at support B. (see E.1.6)

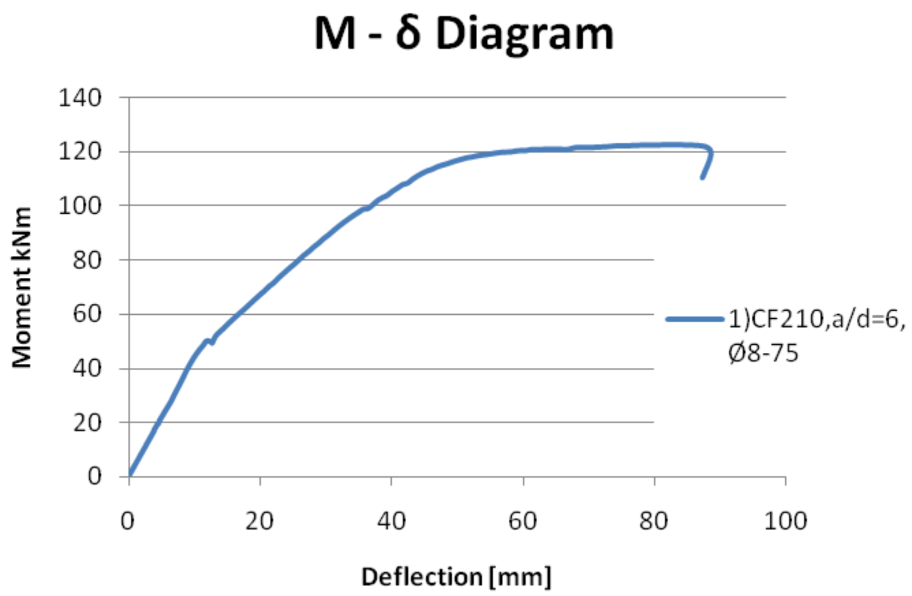


Figure H 4 M-δ Diagram test specimen 1, occurring moment, support B,cross section 1.

$$M_{cr} \approx 50\ kNm\ ; M_y \approx 100\ kNm\ ;\ M_U \approx 122\ kNm\ ;$$

The applied force of 66,21 kN results in an applied moment at cross section 1 at support B: (see E.1.6)

$$M = F * a = 66.21 * 1.62 = 107.26\ kNm$$

Including the already present moment from self weight and test rig elements of 15.12 kNm, these result in a total occurring moment of:

$$M = 107.26 + 15.12 = 122.38\ kNm$$

The data also shows a maximum in reaction forces as shown in Table 40. In order to check the maximum values with realistic reaction forces, the maximum applied force and the self weight of all elements has been put in Matrix Frame to get a feeling of these values. This can be seen in Figure H 5 on the next page:

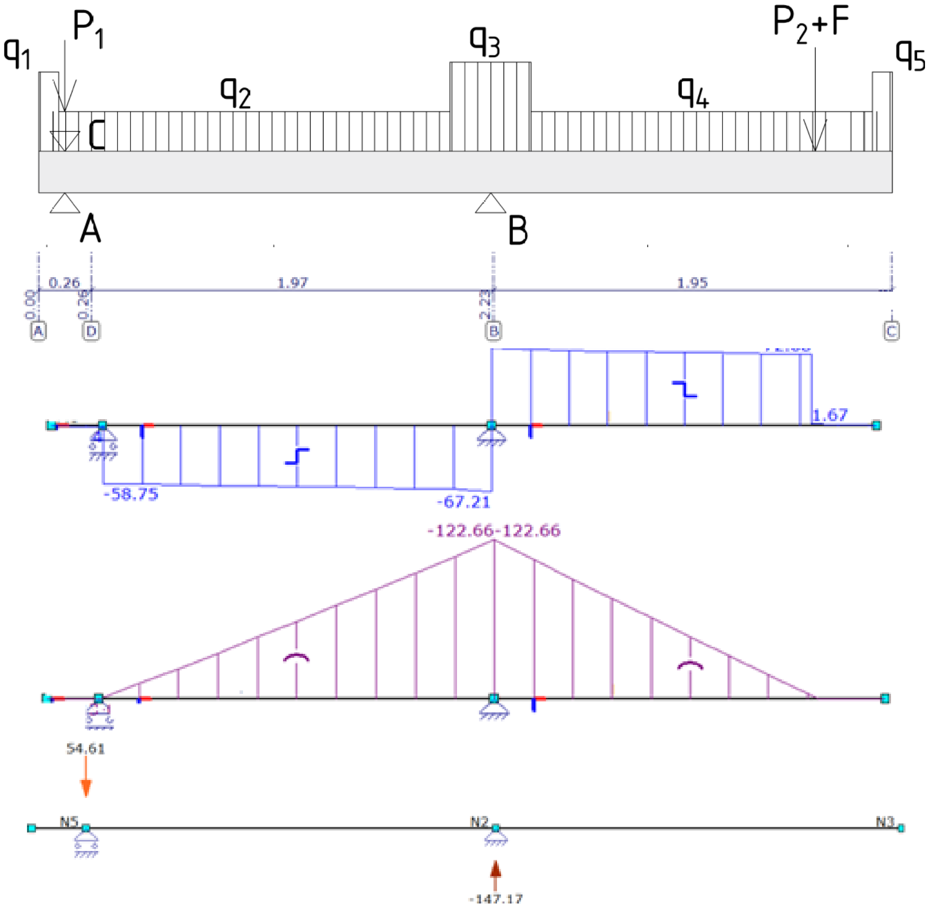


Figure H 5 Moment and Shear occurring due to self weight and the applied force, test specimen 1

Table 41 Reaction Forces Test Specimen 1: Difference between model and measurements.

	R_C [kN]	R_B [kN]
Measured	55.11	-155.97
Matrix Frame	54.61	-147.17
Difference	0.5	-8.8

The occurring moment at cross section 1 (M_B) is equal to 122 kNm.
The calculated resistance of the test specimen above the steel beam is 107.3 kNm (Table 7). This means there is a difference of **15.08 kNm**. This could be due to parts that have not been taken into account, but do contribute.

These could be:

-The ComFlor210 sheet (assumed to be simply supported as it does not continue and therefore does not have any negative bending moment resistance.)
-Concrete compressive strength (cubes are not confined, while the concrete in the test specimen is
-Stress diagram concrete in compression. Simplification has been used of 0,85 * fcd over the full compressive area.
-General spread in resistance of the specimen. (Deviation from the calculated mean value)
-The Additional profiles / joist shuttering as discussed earlier.
-Steel plate acting as a support (modeled in one point 'B', while it has a width of 400 mm)

These differences will be further addressed in 7 Results.

In order to verify the measured reaction forces, two graphs have been set up. The first one (Figure H 6) shows the total of the supports and to determine the difference in the applied forces (which should correspond with the calculated self weight in Table 8) and the second (Figure H 7) to check if the specimen is equally distributed over the width.

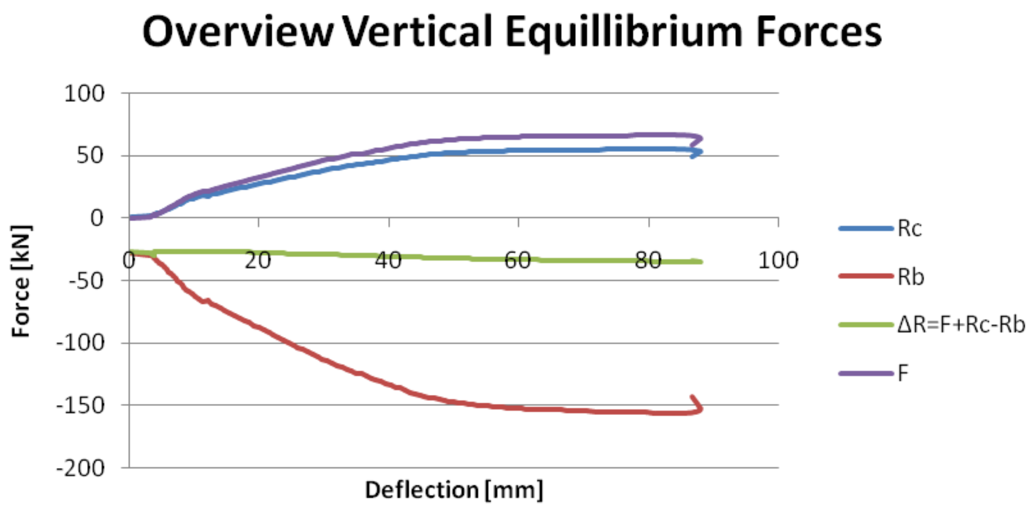


Figure H 6 Overview of the vertical equilibrium of the forces applied. The difference shown in green corresponds with the self weight.

In Figure H 6the difference in all applied forces should remain constant during the test and have the same magnitude as calculated in Table 8. ΔR=26,8 kN at the beginning and 35,5 kN at the end of the test. A difference of 8,7 kN, equal to the difference between Matrix Frame and measured values.

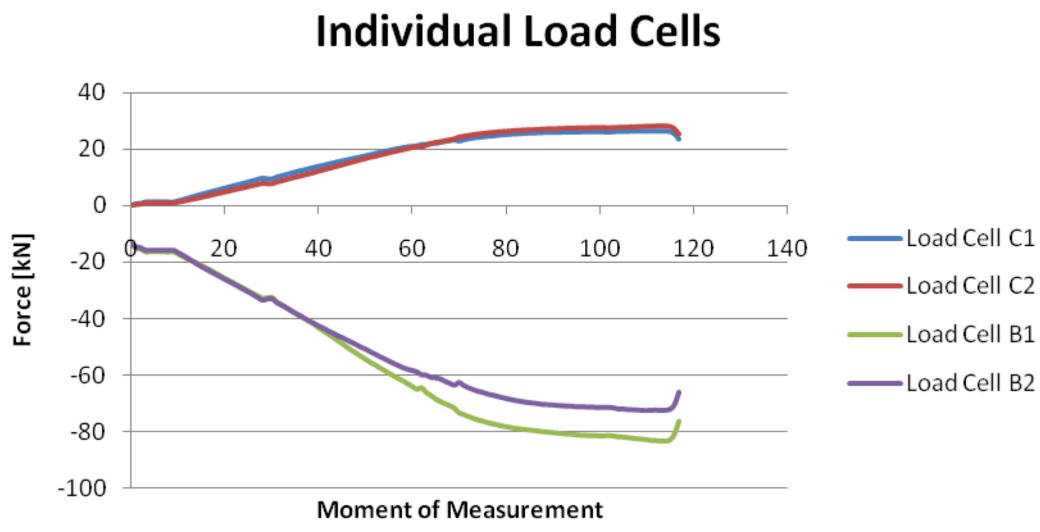


Figure H 7 Individual load cells to show possible applied torsion or deviation from the spirit level.

The individual load cells at each support should increase simultaneously during the test. It appears load cell B1 starts to increase faster than load cell B2.

H.2. Results Test 2

Test specimen 2 has the aim of applying a high shear force and exposing the specimen to both a high negative bending moment and a high vertical shear force. This means a shorter cantilever, which will result in a failure on negative bending, vertical shear or a combination (interaction) of both. In Figure H 8the test rig is shown with the second specimen in place.

Figure H 8 Test specimen 2 before testing

In Figure H 9the applied force with the corresponding displacement is shown.

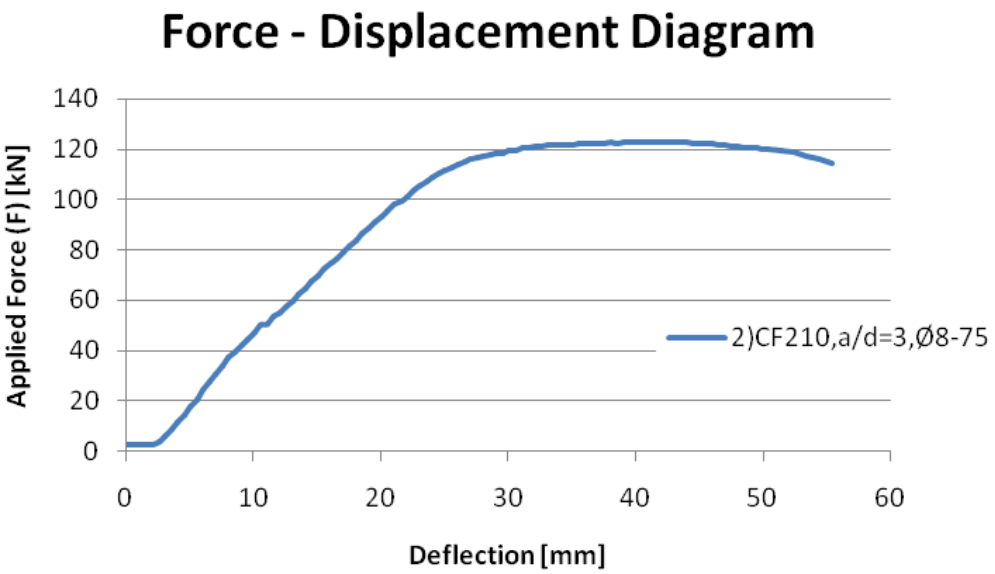


Figure H 9 P-δ Diagram. Test specimen 2. Final force: 122,91 kN, Final deflection: 55mm

The specimen failed at an applied force of 122.91 kN at a deflection of 55 mm. The specimen failed 53 mm left of cross section 1. The crack initiated at the transition between the steel beam and the ribs (cross section 3).

The measurements lead to the following reactions forces and applied load at the moment of failure:

Table 42 Maximum values at the moment of failure, test specimen 2

Maximum values		
Total Support Reaction (R_c) [kN]	Total Support Reaction (R_B) [kN]	Total Force Applied (F) [kN]
55.77	199.36	122.91

The maximum force applied by the hydraulic jack, plus the elements from the test rig and self weight cause a hogging bending moment at cross section 1 at support B. (see E.1.6) This is shown in Figure H 10.

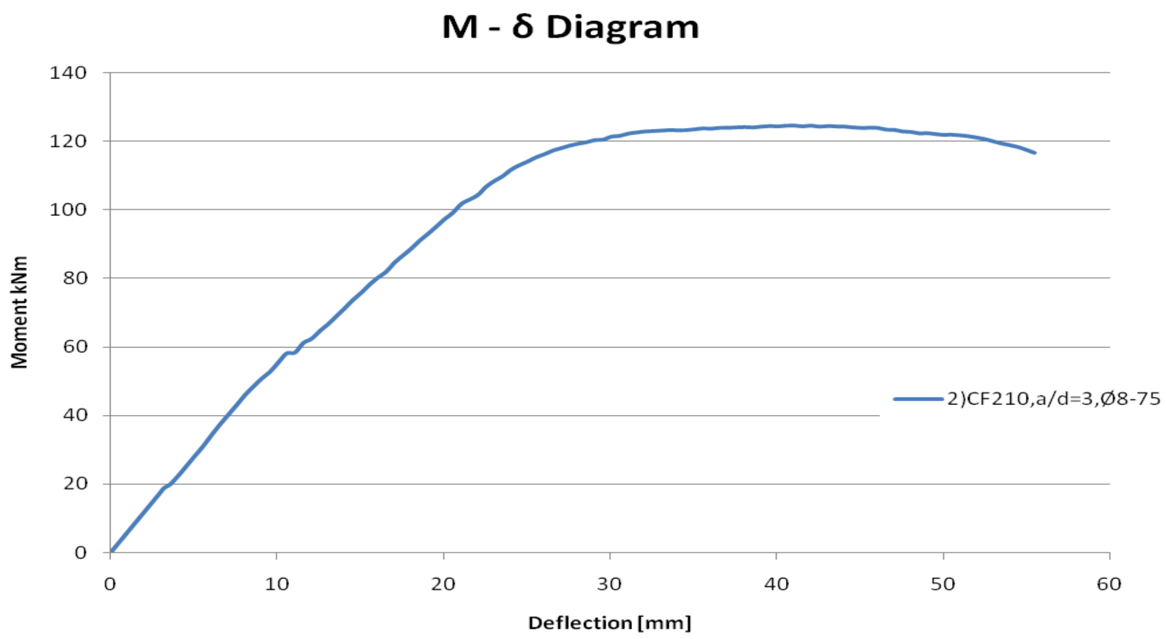


Figure H 10 M-δ Diagram test specimen 2, occurring moment at support B including self weight and test rig elements

$$M_{cr} \approx 57\ kNm\ ; M_y \approx 100\ kNm\ ;\ M_U \approx 124\ kNm\ ;$$

The applied force of 122.91 kN results in an applied moment at the middle of the beam of:

$$M = F * a = 122.91 * 0.915 = 112.5\ kNm$$

Including the already present moment from self weight and test rig elements of 11.73 kNm. this results in a total occurring moment of:

$$M = 112.5 + 11.73 = 124.19\ kNm$$

The data also show a maximum in reaction forces as shown in Table 42. As with the first specimen, the self weight and the failure load has been put into Matrix Frame to compare the reaction forces with one another.

This can be seen in Figure H 11below:

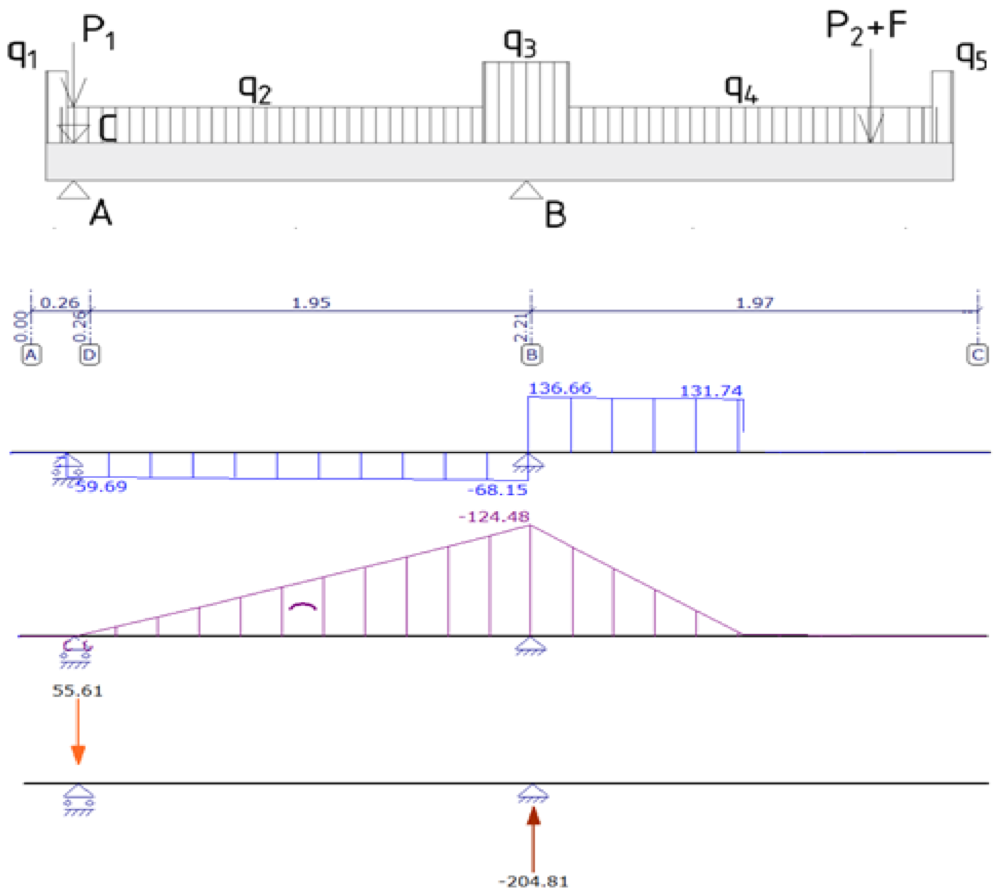


Figure H 11 Moment and Shear occurring due to self weight and the applied force, test specimen 2.

The calculated resistance of the test specimen above the steel beam is 107,3 kNm as shown in Table 7. This means there is a difference of **16.89 kNm**. This could be due to the same as discussed in the previous chapter.

Table 43 Reaction Forces Test Specimen 2: Difference between model and measurements.

	R_C [kN]	R_B [kN]
Measured	55.77	-199.36
Matrix Frame	55.61	-204.81
Difference	0.15	-5.45

The support reaction R_B based on the self weight and final applied load should be 204 kN, measured is 199 kN (Table 42). A difference of 5,5 kN, this will be looked into in a separate chapter.

As in the first test, each load cell is measured and used to verify the measured reaction forces.

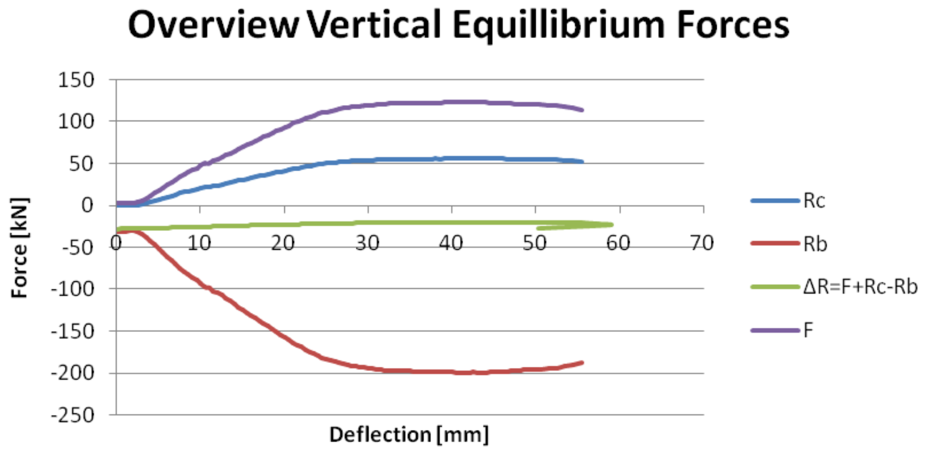


Figure H 12 Overview of the vertical equilibrium of the forces applied. The difference shown in green corresponds with the self weight.

As Figure H 12 shows, there is a slight increase in difference between the applied loads. ΔR=26,8 kN at the beginning and 33.6 kN at the end A difference of 6.8 kN. This is comparable to the 5.45 kN found between Matrix Frame and the measuredR_B.

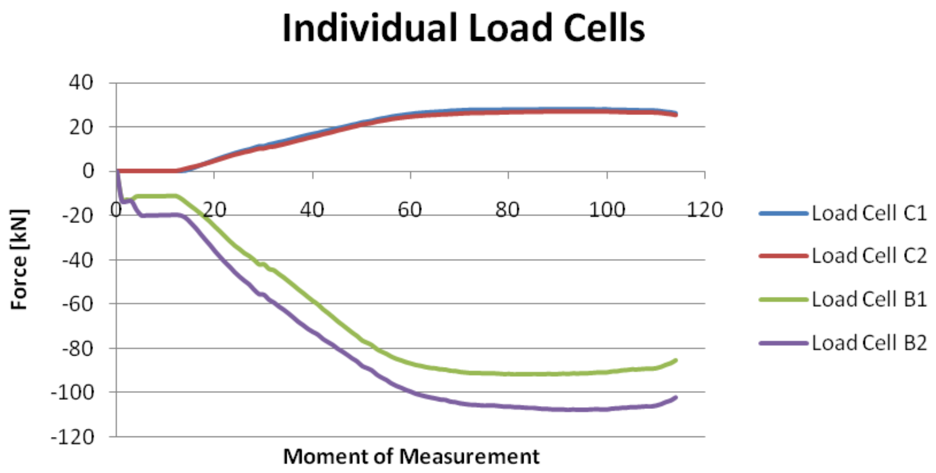


Figure H 13 Individual load cells to show possible applied torsion or deviation from the spirit level.

The individual load cells on each side, C1/C2 and B1/B2 should increase simultaneously during the test. Load cell B1/B2 do not increase simultaneously, this could be due to the failure of the flange shown in Figure H 14.

Figure H 14 Test Specimen 2, failure of the flange due to uneven loading due to imperfections top concrete layer.

H.3. Results Test 3

Test specimen 3 has the aim of duplicating the second test, but with a higher hogging bending moment resistance while the vertical shear resistance remains nearly equal. This way it is more likely to fail in shear or interaction. In Figure H 15the test rig is shown with the third specimen in place.

Figure H 15 Test specimen 3at the beginning of the test

In this chapter a short overview is given of the forces applied, moment of failure and verification of the measured reaction forces.

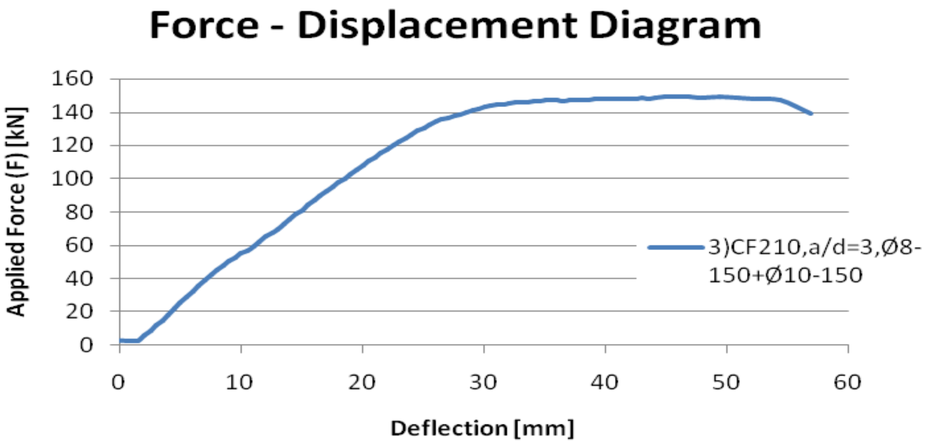


Figure H 16 P-δ Diagram. Test specimen 3. Final force: 149.2 kN, Final deflection: 57mm

The specimen failed at an applied force of 149.2 kN at a deflection of 57 mm. The specimen failed at support B, cross section 1. This is also the point where the hogging bending moment is at its maximum. The shape of the graph in Figure H 16has the same shape as test specimen 2.



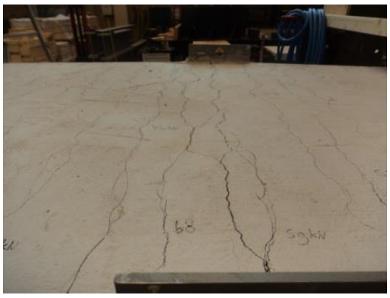


Figure H 17 Crack pattern of specimen 3, failure at the mid support, crack at 960mm from point of engagement.

The measurements lead to the following reactions forces and applied load at the moment of failure:

Table 44 Maximum values at the moment of failure, test specimen 3

Maximum values		
Total Support Reaction (R_c) [kN]	Total Support Reaction (R_B) [kN]	Total Force Applied (F) [kN]
69.14	251.16	149.20

The maximum force applied by the hydraulic jack, plus the elements from the test rig and self weight cause a hogging bending moment at the centre of the specimen. The increase of the hogging bending moment at cross section 1 at support B is shown in Figure H 18.

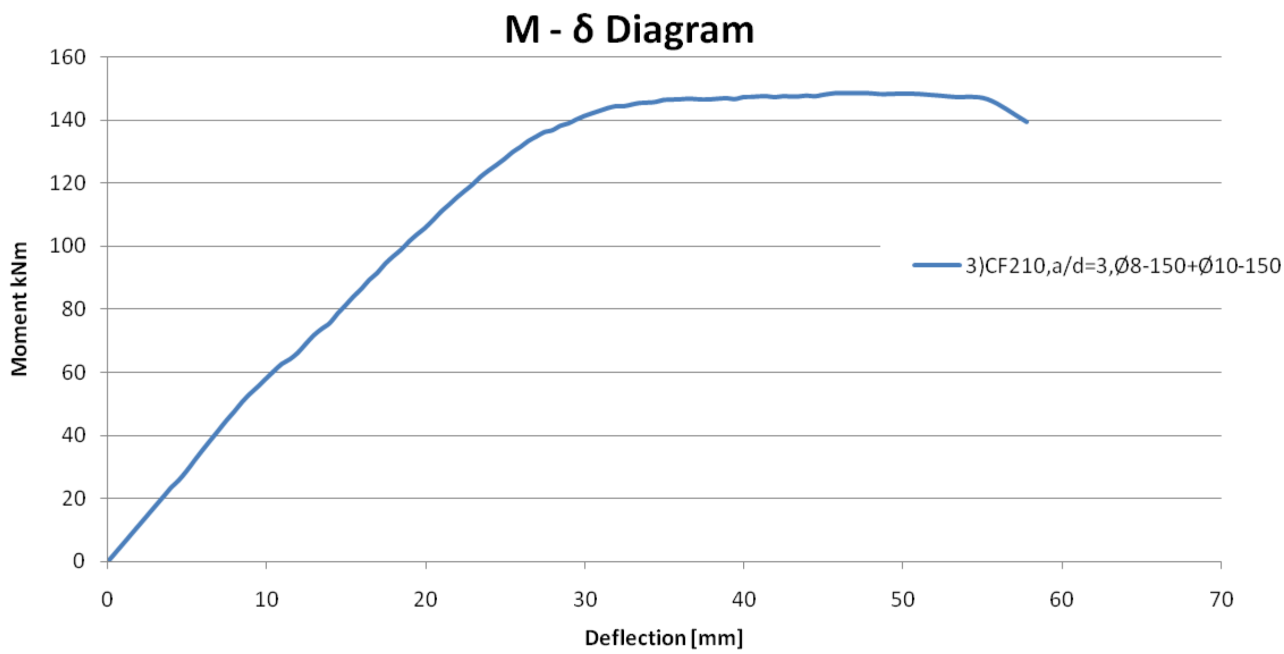


Figure H 18 M-δ Diagram test specimen 3, occurring moment at support B, cross section 1, including self weight and test rig elements

$$M_{cr} \approx 62\ kNm\ ; M_y \approx 130\ kNm\ ;\ M_U \approx 148.25\ kNm\ ;$$

The applied force of 149.20 kN results in an applied moment at support B, cross section 1:

$$M = F * a = 149.2 * 0.915 = 136.5\ kNm$$

Including the already present moment from self weight and test rig elements of 11.73 kNm. this results in a total occurring moment of:

$$M = 136.5 + 11.73 = 148.25\ kNm$$

The data also show a maximum in reaction forces as shown in Table 44. As with the first two specimens, the self weight and the failure load has been put into Matrix Frame to compare the reaction forces with one another.

This can be seen in Figure H 19below:

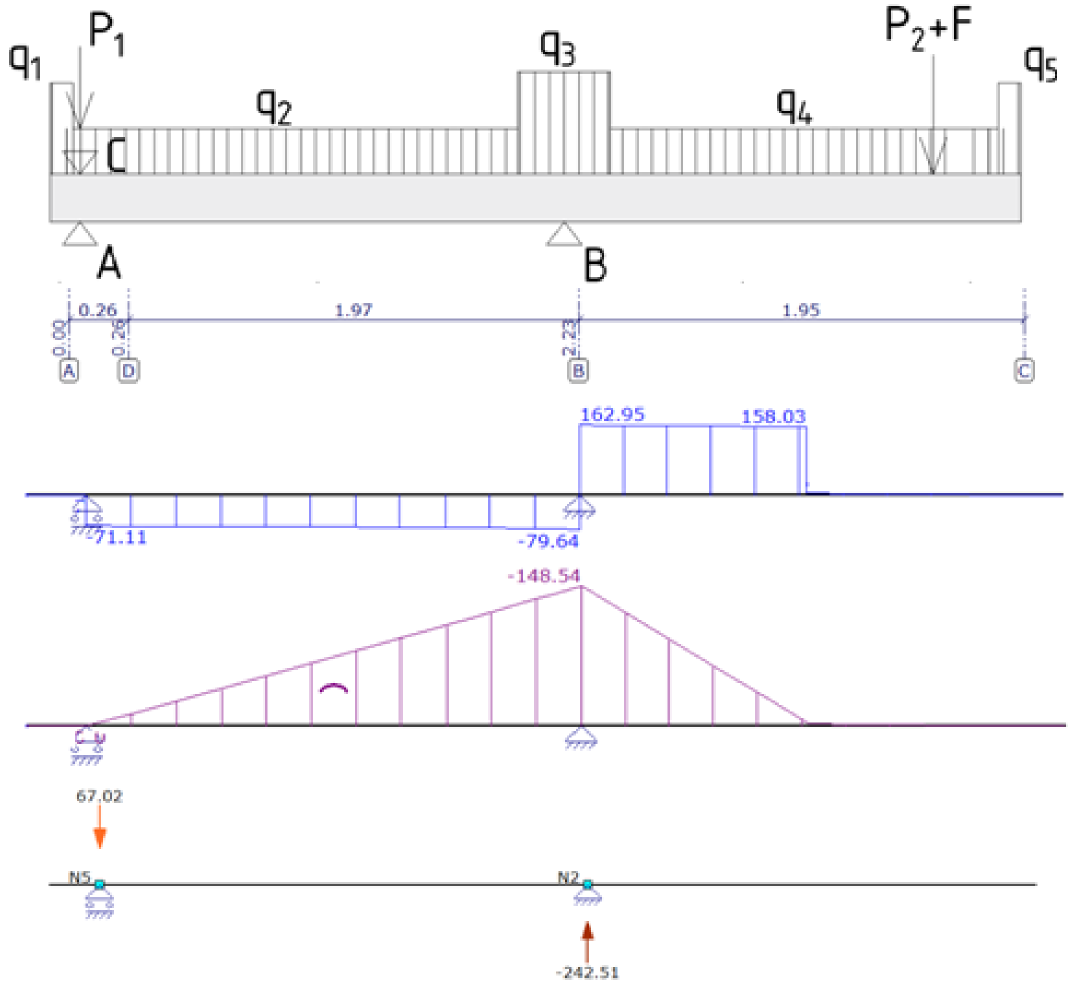


Figure H 19 Moment and Shear occurring due to self weight and the applied force, test specimen 3.

Table 45 Reaction Forces Test Specimen 3: Difference between model and measurements.

	R_C [kN]	R_B [kN]
Measured	69.14	-251.61
Matrix Frame	67.02	-242.51
Difference	2.12	9.1

The calculated resistance of the test specimen at support B is134.2 kNm due to the additional Ø10-150 instead of the Ø8-150 reinforcement. This means there is a difference of **14.05 kNm**. This could be due to the same reason as discussed in the previous chapter and up to now the difference is of the same magnitude, meaning it could be a constant contribution, independent of the test setup or applied force. This will be further discussed in 7 Analyzing of the Results.

As in the first test, each load cell is measured and used to verify the measured reaction forces. Below the difference in vertical forces to determine the self weight and the vertical equilibrium:

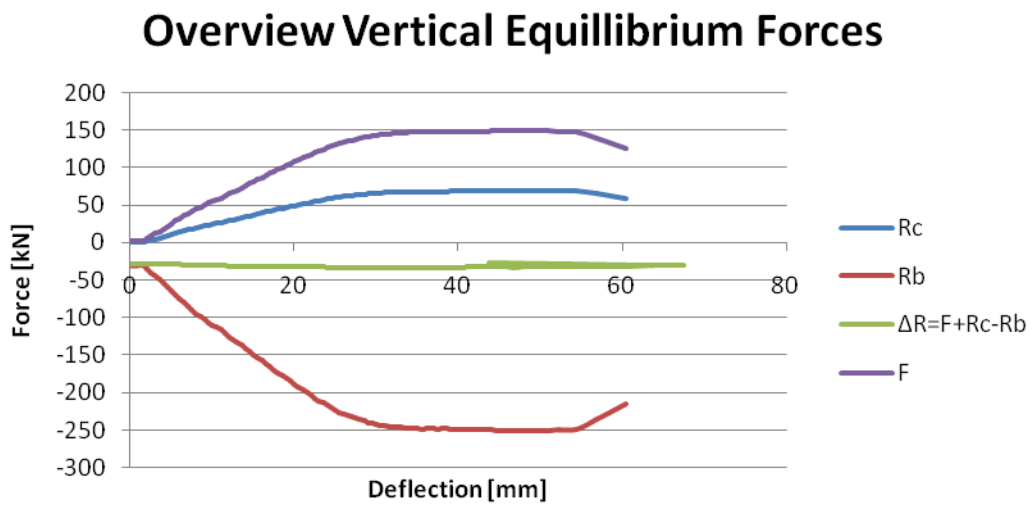


Figure H 20 Test Specimen 3: Overview of the vertical equilibrium of the forces applied. The difference shown in green corresponds with the self weight.

As Figure H 20 shows, ΔR=26,01 kN at the beginning and 32.82 kN at the end A difference of 6.81 kN.

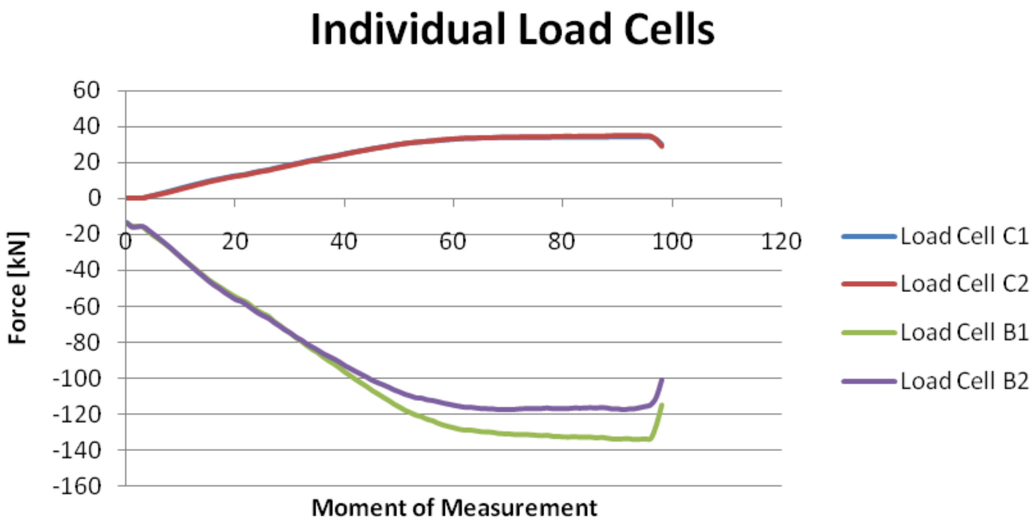


Figure H 21 Individual load cells to show possible applied torsion or deviation from the spirit level.

Same as the first two specimens. Load cell C1 and C2 increase simultaneously, while load cells B1 and B2 deviate. It is therefore possible that the load cells at B did not work properly or maybe had influence from horizontal forces. $R_B$however does not contribute to the hogging bending moment (M_B) so it does not influence the main research question.

H.4. Results Test 4

Test specimen 4 is the first specimen of the second series of tests. It now concerns the shallow deck (ComFlor75). The aim of test 4 is equal to the first specimen, namely obtaining the negative bending moment resistance with a low shear force present. It should therefore fail in bending. In Figure H 22the test rig is shown with the fourth specimen in place.

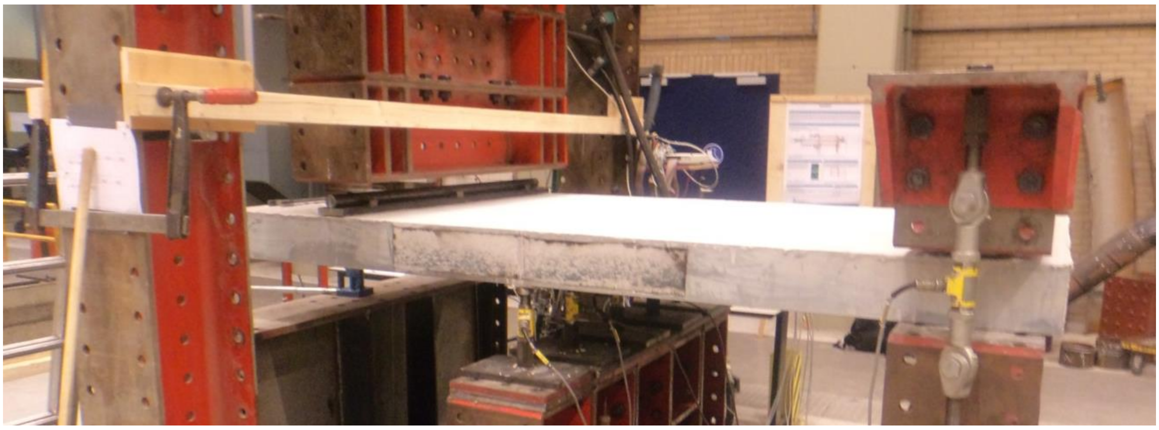


Figure H 22 Test specimen 4 right before testing

In this chapter the same measurements are shown as for the ComFlor210 series, with the difference in way of supporting at the support B. For the ComFlor75 specimens, 5 load cells have been used underneath each rib. This allows us to measure each rib individually.

In Figure H 23the force – displacement diagram is given:

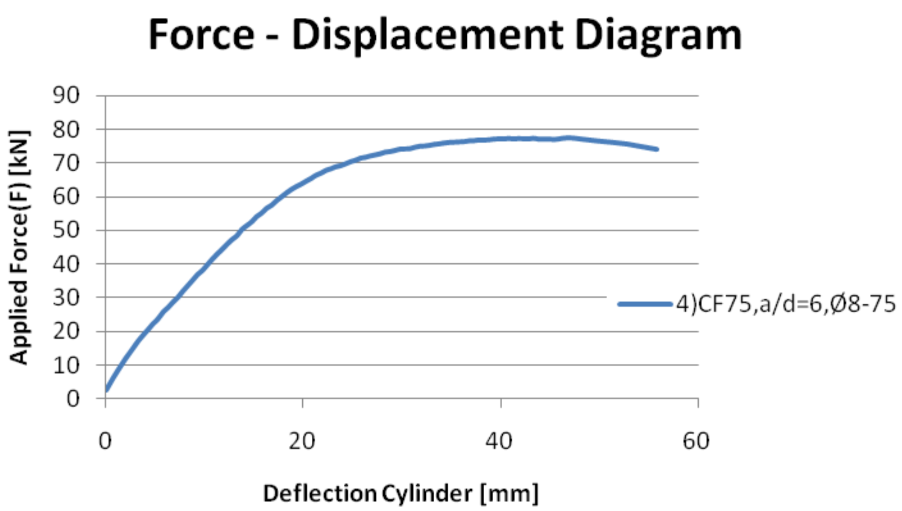


Figure H 23 P-δ Diagram. Test specimen 4. Final force: 77.63 kN, Final deflection: 61mm

The specimen failed at an applied load of 77,63 kN at a deflection of 61 mm. The force was applied at a distance of 660 mm or a/d = 6 from support B.

The measurements lead to the following reaction forces and applied load at the moment of failure: (Note that the total support reaction mid is an approximated value (5/2*measured) as only 2 load cells functioned)

Table 46 Maximum values at the moment of failure, test specimen 4.

Maximum values		
Total Support Reaction (R_c) [kN]	Total Support Reaction (R_B) [kN]	Total Force Applied (F) [kN]
61.52	133.65	77.63

In Figure H 24 the growth of moment above support B is shown. The deflection is measured at a distance of 330 mm from the support.

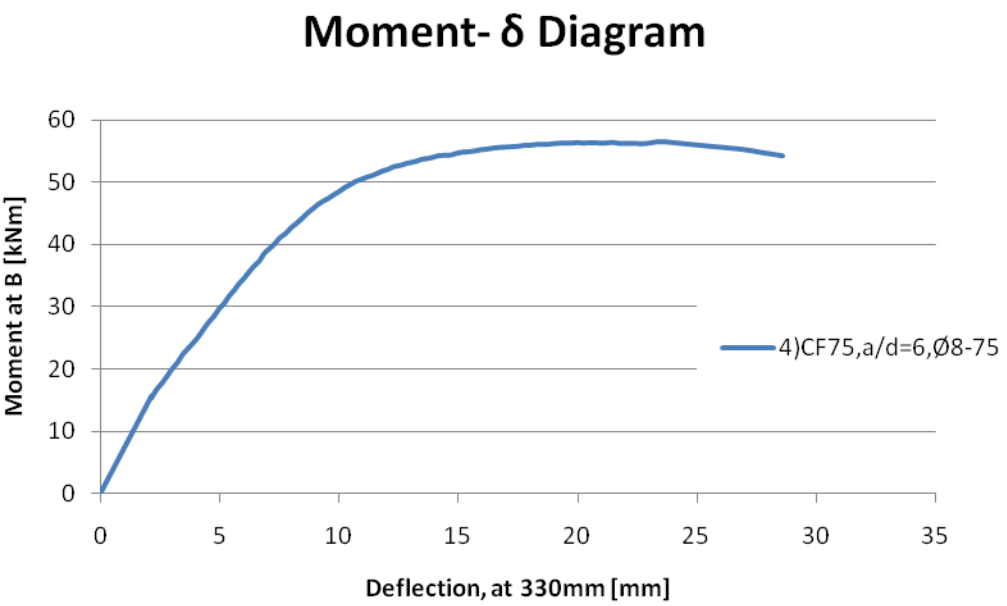


Figure H 24 M-δ Diagram test specimen 4, occurring moment at support B, including self weight and test rig elements, maximum moment at failure 56,6 kNm.

$$M_{cr} \approx 23\ kNm\,;M_y \approx 50\ kNm\,;\ M_U \approx 56.67\ kNm\,;$$

The applied force of 77.63 kN results in an applied moment at B of:

$$M = F * a = 77.63 * 0.66 = 51.24\ kNm$$

Including the already present moment from self weight and test rig elements of 5.43 kNm. this results in a total occurring moment of:

$$M = 51.24 + 5.43 = 56.67\ kNm$$

This moment will be checked by putting the self weight and the failure load into MatrixFrame to get insight in the occurring moment and shear forces.

This can be seen in Figure H 25 below:

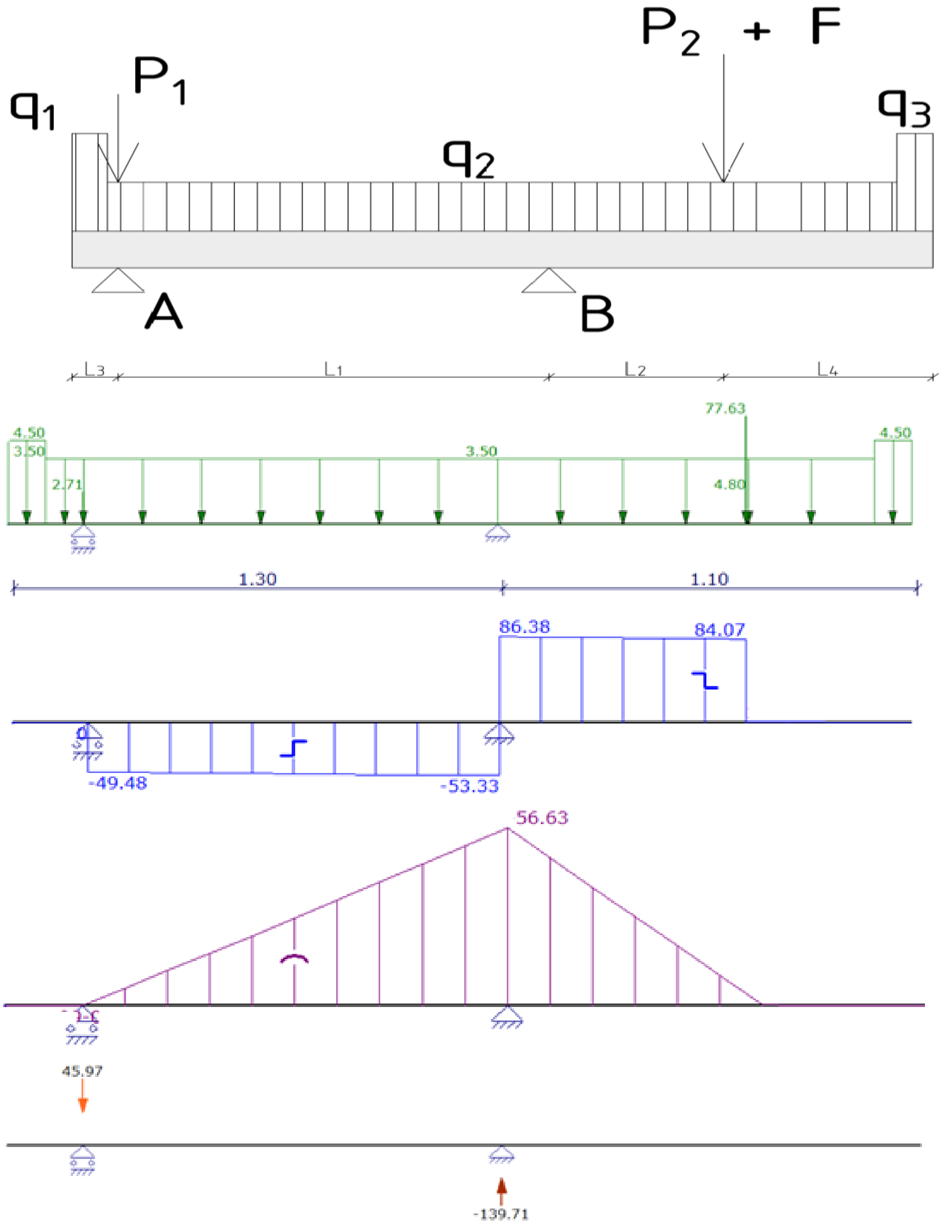
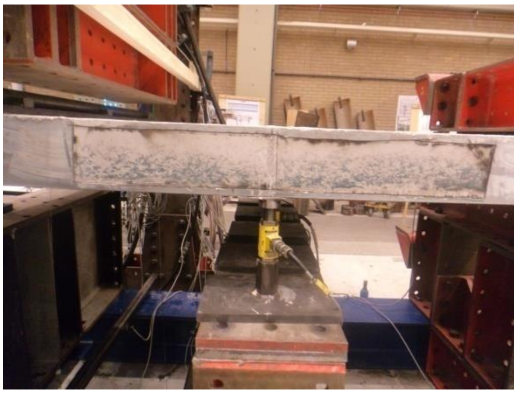


Figure H 25 Moment and Shear occurring due to self weight, test rig elements and the applied force, test specimen 4.

Table 47 Reaction forces test specimen 4, difference between model and measurements.

	R_C [kN]	R_B [kN]
Measured	47.36	-133.65
Matrix Frame	45.97	-139.71
Difference	1.39	-6.06

The occurring moment at B is equal to 56.67 kNm.
The calculated resistance of the test specimen is 61,51 kNm. This means there is a difference of -4.84 kNm. The specimen failed before reaching its capacity. Some ribs failed due to punching shear, this means a reduction in internal lever arm and could explain the early failure. On the next page some pictures have been added about this failure mode.



a) *Joist shutterings partly grinded out to avoid unwanted contribution. (before testing)*

b) *Moment of failure, the joist shutterings at the bottom could contribute slightly as well as the strips on top that connect the joist shuttering to the ComFlor75 sheet.*

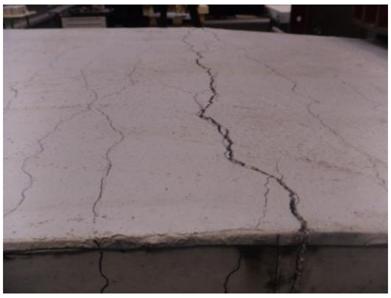
Figure H 26 Detail above the support, joist shuttering has been partly grinded out to avoid contribution to the hogging bending moment. However some slight contribution can be present from aspects discussed in chapter 5.2.2.

During the test, some of the ribs failed in punching shear, the ComFlor75 buckled inwards. This can be seen in Figure H 27.

a) *Punching shear failure below the second rib.*

b) *Punching shear failure overview from below.*

Figure H 27 Due to the small area of the load cells and possible weak spots in the ribs, punching shear occurred. This introduced a deviation in displacements and loading pattern of individual load cells.



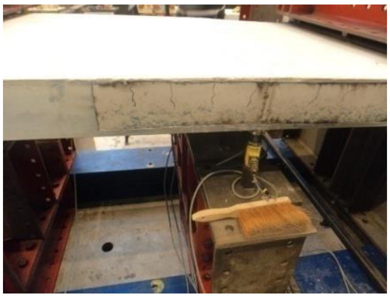


Figure H 28 Crack pattern of the fourth specimen. Failure at 665 mm away from the point of engagement of the load, right above the support B. Crack pattern can be seen on the side of the specimen.

Each individual load cell has been measured in order to check unequal loading and or distribution of loading over the ribs. However during the fourth test the connection of load cell B3-B7 did not connect well with the computer. Only four load cells were able to send data. This still provides us with information, but not the complete picture. Due to the lack of information from load cell B3-B7 the vertical equilibrium cannot be checked. However Figure H 29gives some interesting information about the functioning of the individual load cells.

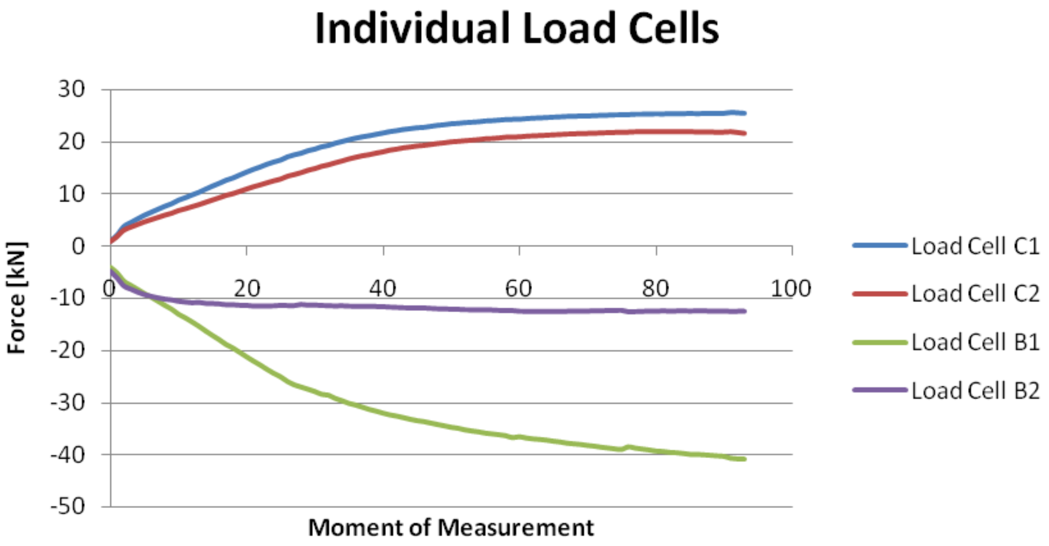


Figure H 29 Individual load cells C1,C2,B1 & B2 to show possible applied torsion or deviation from the spirit level.

At a force of only 10 kN on load cell B2 (second rib) stopped increasing. This effect kept intact until the end of the test, when the specimen failed in bending. This could mean malfunctioning of one or more of the load cells and / or local failure at one of the ribs as discussed before.

Figure H 30 Due to failing on punching shear the test specimen inclined both in longitudinal and horizontal direction, causing unequal support reaction at the far end.

H.5. Results Test 5

Test specimen 5 is comparable to test specimen 2. The cantilever is short and the specimen should fail in negative bending moment, vertical shear or a combination of both. In Figure H 31the test rig is shown with the fifth specimen in place.

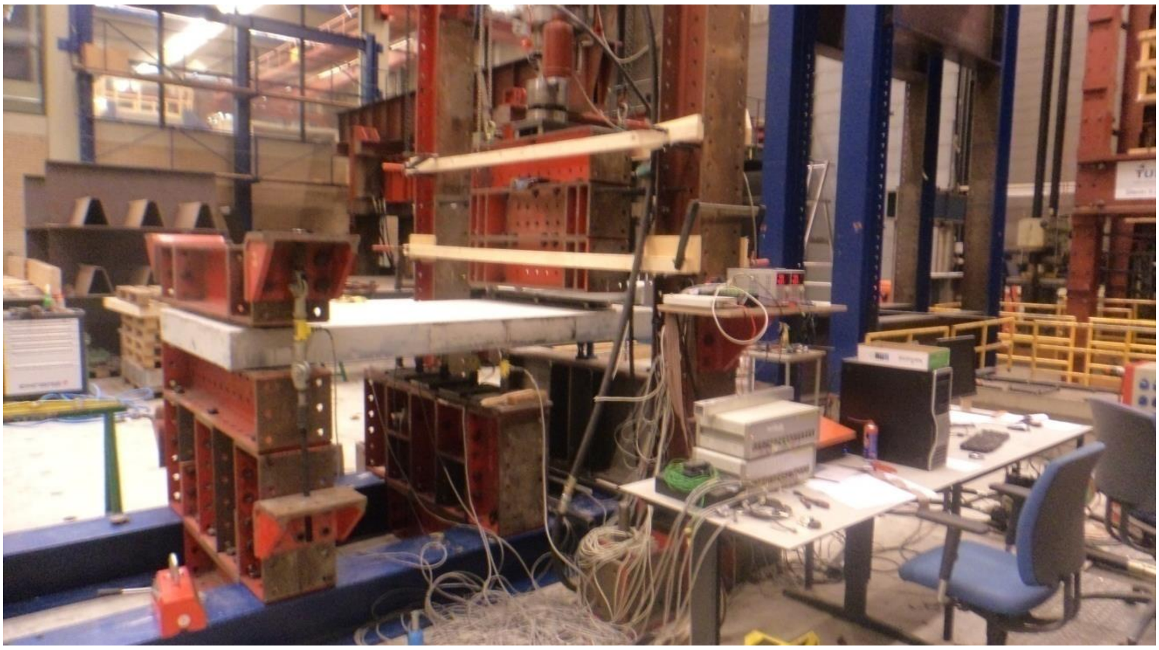


Figure H 31 Test specimen 5 right before testing

For the fifth specimen all 7 load cells were connected correctly. The load was applied at 330 mm away from the support or at an a/d ratio of 3.

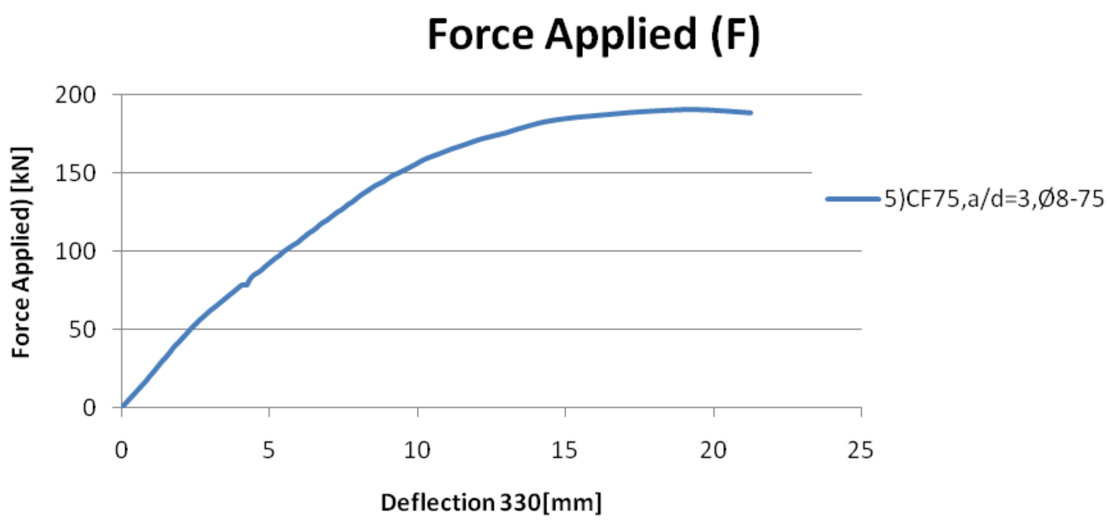


Figure H 32 P-δ Diagram. Test specimen 5. Final force: 181.72 kN, Final deflection: 36 mm

The load cells show a different pattern from the start of the test. Afterwards it appeared that the steel plate used to apply the load, did not touch the specimen at all places. This may cause the differences between each load cell. This also means certain ribs receive a higher shear force compared to other ribs. Taking the sum of all load cells and the applied load, gives the following table:

Table 48 Maximum values at the moment of failure, test specimen 5.

Maximum values		
Total Support Reaction (R_{c}) [kN]	Total Support Reaction (R_B) [kN]	Total Force Applied (F) [kN]
52.93	-256.69	181.72

The total support reaction at support C and the total force (F) applied by the hydraulic jack can be used to create a graph of the increasing moment, set out against the displacement. This is the moment right above support B:

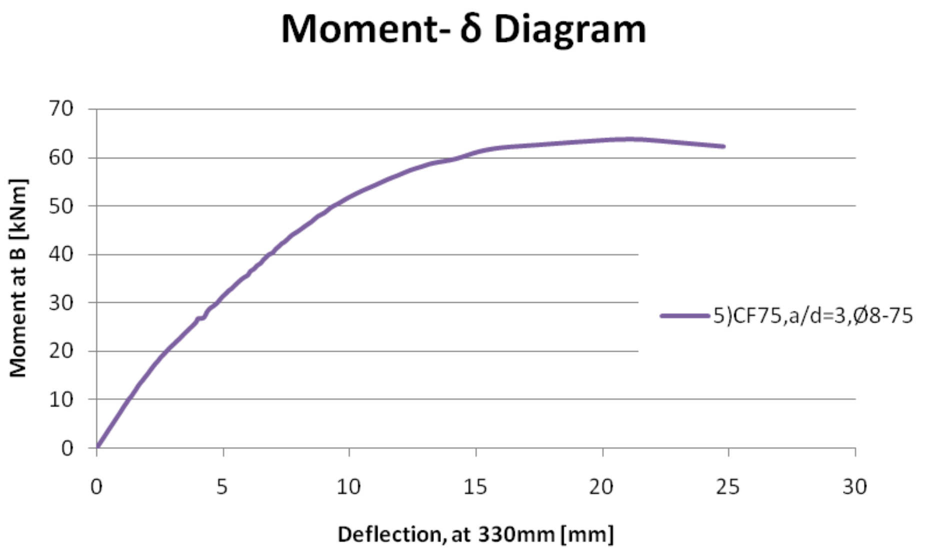


Figure H 33 M-δ Diagram test specimen 5, occurring moment right above support B, including self weight and test rig elements, maximum moment at failure 63.81 kNm.

$$M_{cr} \approx 26\ kNm\ ; M_y \approx 50\ kNm\ ;\ M_U \approx 63\ kNm\ ;$$

The applied force of 181.72 kN results in an applied moment at the middle of the beam of:

$$M = F * a = 181.72 * 0.33 = 59.97\ kNm$$

Including the already present moment from self weight and test rig elements of 3.84 kNm. this results in a total occurring moment of:

$$M = 59.97 + 3.84 = 63.81\ kNm$$

The reaction forces and occurring maximum values are now compared with a Matrix Frame input. This can be seen on the next page.

This can be seen in Figure H 34 below:

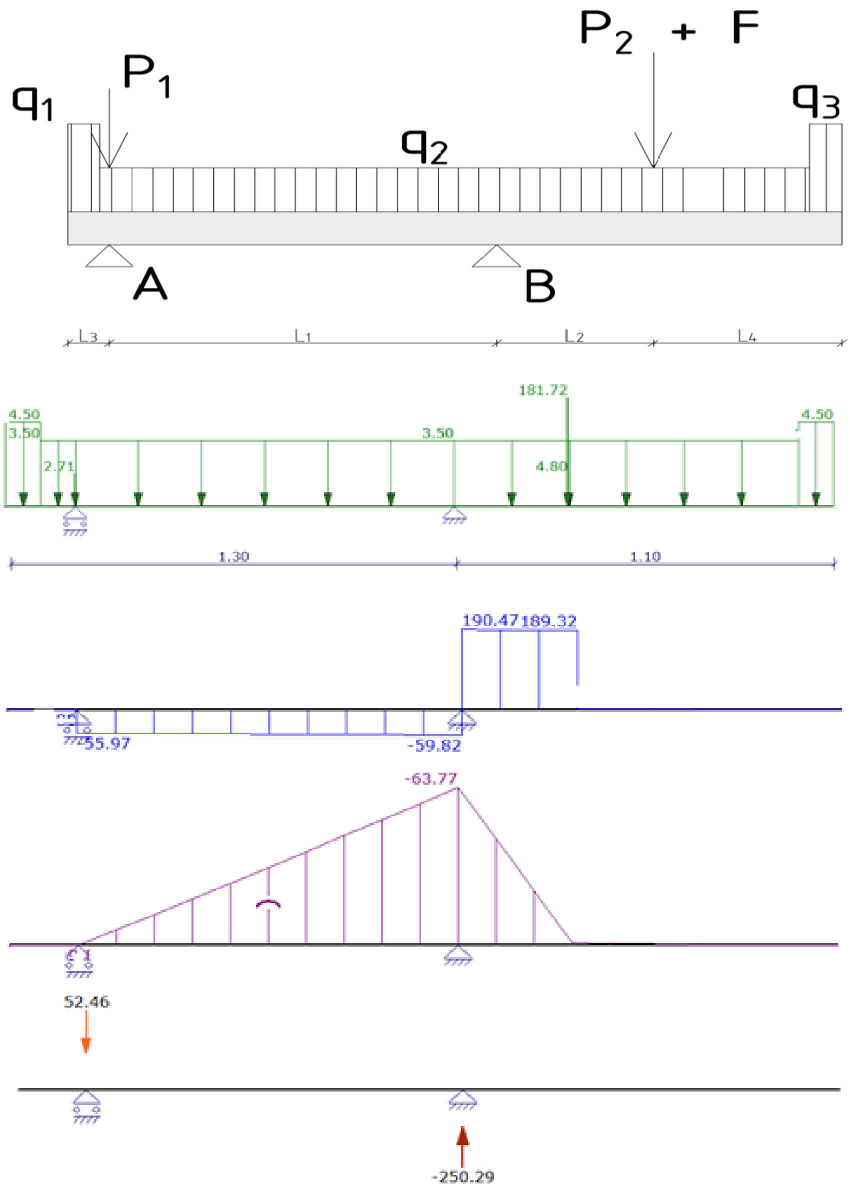


Figure H 34 Moment and Shear occurring due to self weight, test rig elements and the applied force, test specimen 5.

Table 49 Reaction forces test specimen 5, difference between model and measurements.

	R_C [kN]	R_B [kN]
Measured	52.93	-256.69
Matrix Frame	52.46	-250.29
Difference	0.47	-6.4

The occurring moment at B is equal to 63.81 kNm.
The calculated resistance of the test specimen is 61,51 kNm. This means there is a difference of -2.3 kNm. This means the calculated resistance is equal to the occurring hogging bending moment. Even with the high vertical shear force, the resistance of the specimen has been reached and no reduction was present.

The specimen failed at an applied load (F) of 181,72 kN and reached a maximum deflection of 36 mm. The final location of failure was at support B at 345 mm away from the applied load.

This time all load cells were measured and gave some interesting information. Figure H 35shows the equilibrium of all vertical forces.

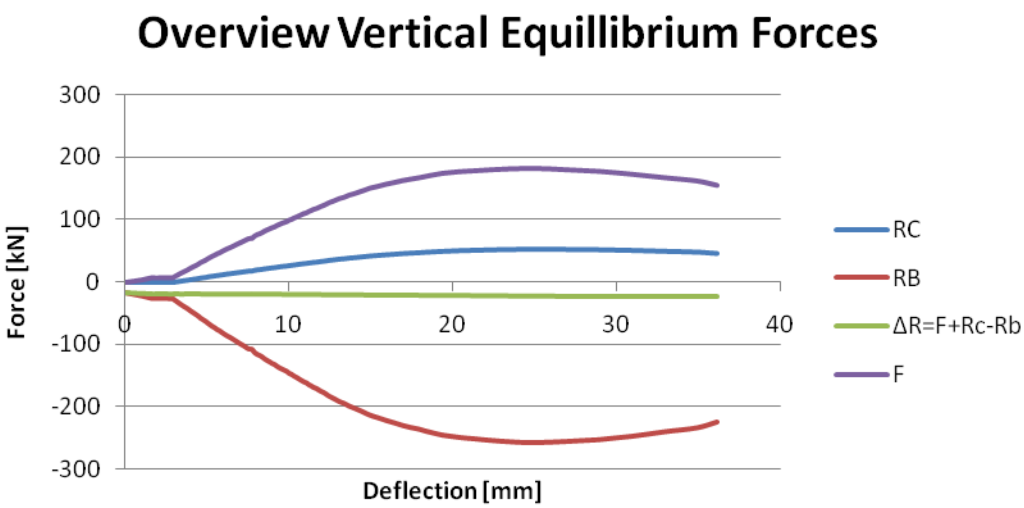


Figure H 35 Test specimen 5: Overview of the vertical equilibrium of the forces applied. The difference shown in green corresponds with the self weight.

The green line (difference in applied forces) should remain horizontal during the test. This is more or less the case as the value at the beginning of the test is 18.1 kN and the maximum value during the test touched 22,04 kN. This means a difference of 3.94 kN.

This time the test setup was provided with a steel strip that avoids punching shear above the load cells. It spreads the point load, providing a higher area to reduce the stress occurring in the ribs.

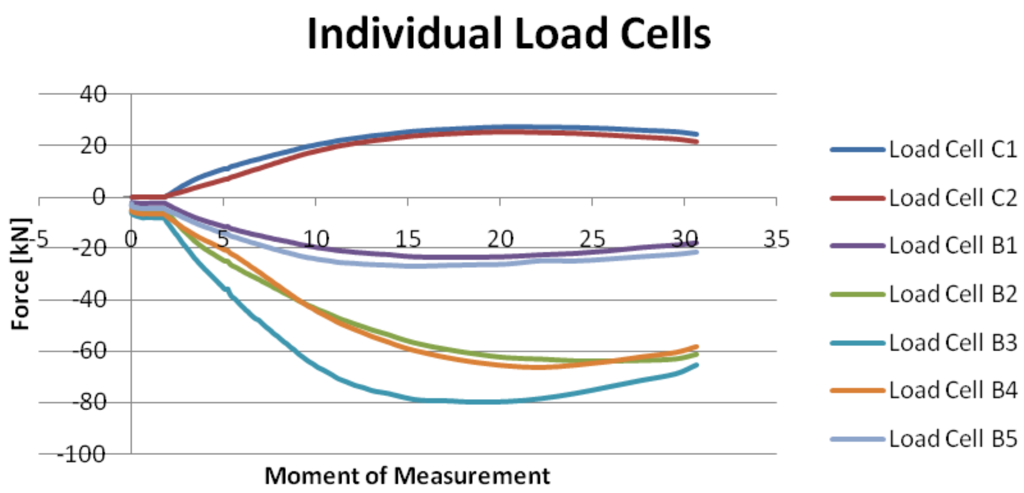


Figure H 36 Individual load cells to show possible applied torsion or distribution between different ribs.

H.6. Differences in load cells

During the test each separate load cell has been measured, giving insight in the reaction forces and the vertical equilibrium.

Table 50 Difference between Matrix Frame input and measured values.

Test Specimen	Difference R_C [kN]	Difference R_B [kN]	$\Delta\sum V$
1	0.5	8.8	7.85
2	0.15	-5.45	-5.52
3	2.12	9.1	6.81
4	1.39	-6.06	-7.44
5	0.47	-6.4	3.94

The increase in difference between the measured support reactions B & C and the applied force F ($\Delta\sum V$) is mostly due to an increase or reduction of R_B. In order to check if the applied force F and reaction force R_C increase linearly a graph is shown in Figure H 37. (same is done around F and C)

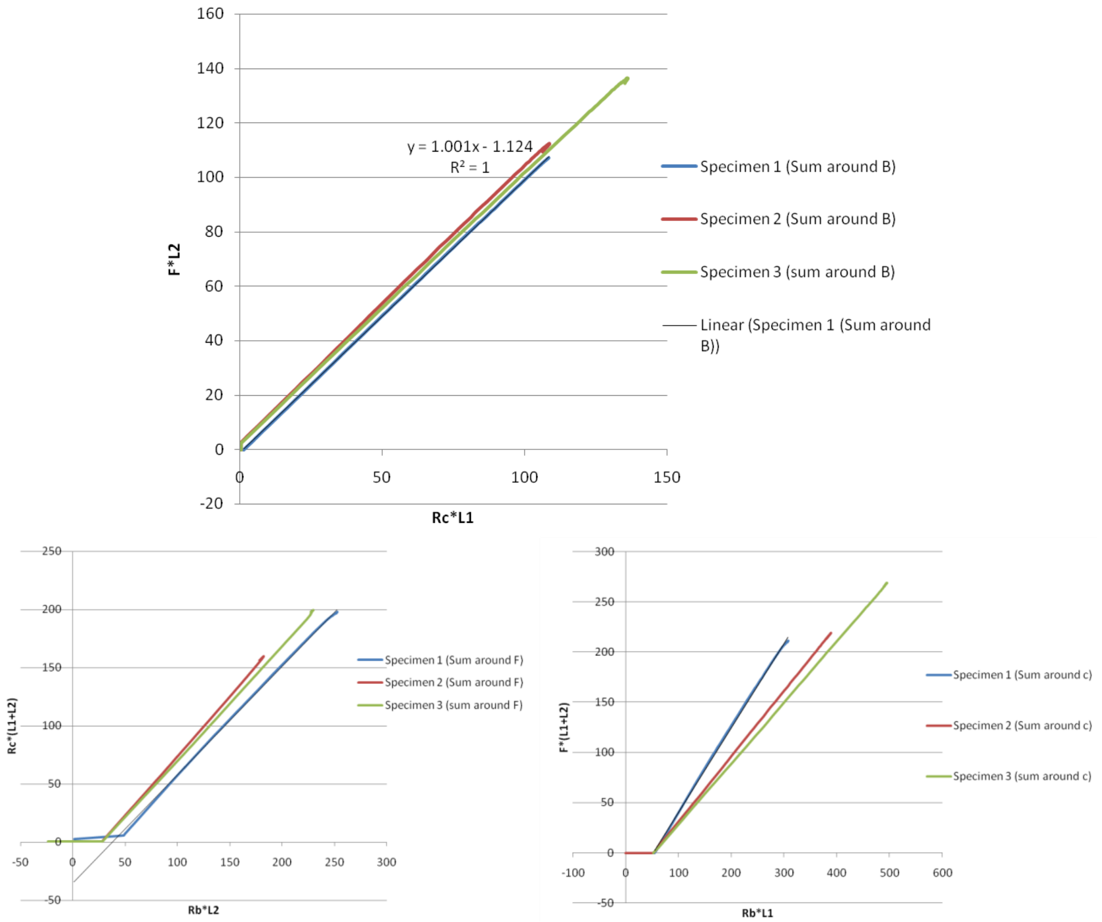


Figure H 37 Summation of moments around B.

In the first graph there is no influence of R_B. And all slopes are equal, in the other two graphs the slopes differ. As the first graph seems to be the most accurate, it might be the case that one or more of the load cell did not function properly. R_B does not influence the hogging bending moment, it therefore should not influence the results used to answer the main question.